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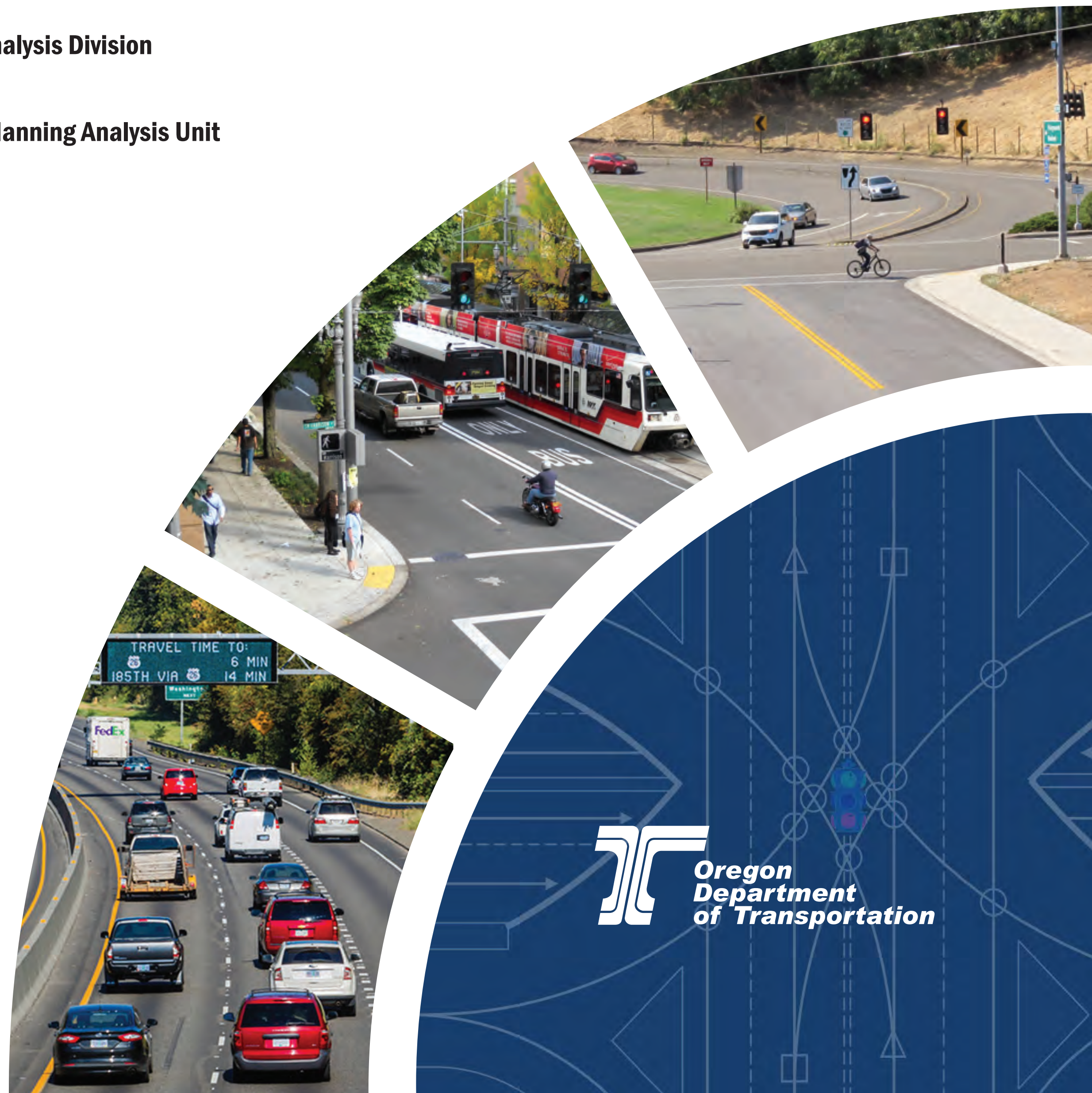
Analysis Procedures Manual

VERSION 2

Policy, Data & Analysis Division
Planning Section

Transportation Planning Analysis Unit
Salem, Oregon



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PREFACE/FOREWORD

Overview of Manual Purpose

The Analysis Procedures Manual (APM) was created to provide a comprehensive source of information regarding current methodologies, practices and procedures for conducting analysis of Oregon Department of Transportation (ODOT) plans and projects. Although this information is extensive, it is not intended to be exhaustive. For example, this manual does not fully address detailed topics such as Region safety investigations, the traffic signal approval process, or development review policies which are covered elsewhere.

The APM shall be utilized by ODOT staff as well as external consultants and contractors conducting and reviewing plans, projects and/or studies for ODOT. It also applies to work performed under ODOT Grants.

The procedures addressed in this manual have been generally organized to follow the progression of analysis conducted for a typical transportation plan or project. It begins with project scoping and data collection, proceeds through the analysis and concludes with the production of the final report. There are examples provided to "walk" the user through a process.

The APM is generally based on methodologies found in the Highway Capacity Manual (HCM). However, there are many locations in the APM, either because of limitations in the HCM or because of ODOT policies, where the APM recommends different methodologies to address these issues. Traffic analyses shall use the current edition of the HCM in effect at the start of the analysis unless otherwise specified in the APM.

While the direction provided represents recommended practices for producing consistent and accurate results, it should be recognized that every project analysis presents a unique set of opportunities and constraints. Persons applying the APM should consider relevant situation-specific factors including, but not limited to, cost, funding availability, environmental impacts, sustainability, economic development needs, community support, community vision, land use planning context, practical design, and other similar considerations as appropriate. The best alternative from a traffic analysis standpoint, may not be the best alternative for the project.

While working on various types of projects, a number of situations may arise requiring analysis methodologies not discussed in this manual. If ODOT does not have a preferred analysis methodology to offer, there are a number of technical resources available for consultation. Non-standard analysis proposals shall include thorough documentation of assumptions, methods and calculations in a methodology memorandum. Alternative methodologies must be approved by ODOT prior to analysis.

This manual is not intended to replace the need for sound engineering judgment, which must continue to be a vital part in the process of applying the methodologies to individual studies. Thorough documentation of key assumptions, decisions, and findings should be provided and archived with project files for future reference/use. Further, early and frequent collaboration between jurisdictional staff, consultants, contractors, and stakeholders involved in the analyses will help ensure that the project goals are met.

Note: All references in this manual to the Department refer to ODOT, and all references to Regions relate to ODOT Regions.

Manual Structure

Acronyms are shown in parenthesis after a term, phrase or reference is listed the first time. The acronym is used thereafter in the text.

Manuals, papers and other publication titles are italicized.

There are a number of references to web sites, web pages and web accessed documents. Many of these references are links within the document shown with blue, underlined text.

Examples are identified with a solid bar the width of the page at the beginning and end of each example.

Points that are critical for the analysis process are displayed as a box in italics with the stop sign icon.

Additional information to consider is displayed as a box in italics with the yield icon.

Manual Updates

Analysis techniques and project requirements change over time. The ability to immediately incorporate new information into this manual is essential to providing users with the most current resource possible. To accommodate expedient updating, the APM has been designed as an on-line tool, and the on-line version is the official document.

As this is an on-line document, and will not be published and distributed as a traditional publication, there is no user list for update notifications. It is the user's responsibility to verify they are using the most current version of information as their reference. Updated pages will have the change date as part of the document footer.

Please send your questions, comments and feedback to: APM@odot.oregon.gov.

Manual Website

The on-line version of this document is available at:
https://www.oregon.gov/ODOT/Planning/Pages/APM.aspx

Detailed document updates are available on the web page so that users can identify what has changed and when. Supporting information, data and tools used in this manual are available on the Planning Section Technical Analysis and Tools webpages at:
https://www.oregon.gov/ODOT/Planning/Pages/Technical-Tools.aspx

Periodic Training

TPAU offers some training related to this manual, generally related to new topics or methodologies. The ODOT APM User Group is a quarterly venue for training and discussion on APM procedures. Contact APM@odot.oregon.gov to inquire about user group meetings or other training opportunities.

TABLE OF CONTENTS

1 ODOT INFORMATION

1.1 Purpose

The Oregon Department of Transportation (ODOT), through its various Divisions, is responsible for developing Oregon's:

- System of highways and bridges
- Bicycle and pedestrian paths
- Public transportation services
- Rail passenger and freight systems
- Driver licensing and vehicle registration programs
- Motor carrier operations, and
- Transportation safety programs

This chapter is an overview of the ODOT, how it is organized and describe various divisions, sections, and units within ODOT that are commonly involved in decision making regarding transportation system planning, design, and operations. One or more of these units may need to be contacted for input or to discuss problems and possible solutions regarding a specific project. It is preferable to begin with staff at the Region or District level whose contact information may be found on the ODOT's Directory webpage.

1.2 Policy, Data & Analysis Division

The Policy, Data & Analysis Division (PDAD) is the part of ODOT that:

- Facilitates long and short-term transportation planning
- Keeps statistics about transportation
- Considers transportation policy
- Conducts research to help engineers, planners, and project designers
- Helps local governments with transportation through a variety of programs and services.

PDAD is comprised of four sections:

1. Climate Office
2. Statewide Policy and Planning
3. Statewide Research
4. Transportation Data

1.2.1 Climate Office

The Climate Office is responsible for integrating climate considerations into ODOT business and transportation systems. The Office works across ODOT Divisions, with other state agencies, local jurisdictions, and the public to lessen and prepare for climate change impacts. Mitigation work focuses on reducing greenhouse gas emissions from transportation, including implementing State directives (e.g. tracking progress and building on the Statewide Transportation Strategy, Executive Order 20-04, Oregon

Transportation Plan, ODOT Strategic Action Plan) and transportation electrification. Adaptation work focuses on preparing for and responding to the impacts of climate change to transportation infrastructure, such as wildfires, extreme precipitation, and sea level rise (i.e. embodied in the Climate Adaptation and Resilience Roadmap, with associated risk maps in TransGIS). The Office's Sustainability Program conserves resources, such as water and energy in ODOT business and operations, and includes efforts like the Oregon Solar Highways Program, and low carbon construction materials (e.g. implementation of HB4139). The Office also supports legislative and Governor's Office directives related to climate change mitigation, adaptation, or sustainability.

1.2.2 Statewide Policy and Planning Section

ODOT's Statewide Policy and Planning section is comprised of four units:

A. Transportation and Growth Management program
B. Statewide Transportation Planning Unit
C. Multi-Modal Freight Planning Unit
D. Transportation Planning and Analysis Unit.

This section provides direction for long term management and improvement of Oregon's transportation system and to promote the cost-effective use of public funds through effective research, development, and technology transfer.

A. The **Transportation and Growth Management** (TGM) program is a jointly managed program between ODOT and the Department of Land Conservation & Development (DLCD). The TGM program offers a competitive grant program to local jurisdictions for the creation and updating of Transportation System Plans (TSP), TSP refinement plans, transit plans, etc. There are also TGM planners based in the five ODOT Regions to lead and coordinate the TGM program. DLCD staff provide project management for the smaller community assistance programs (e.g. Code Assistance, Quick Response, Outreach, Parking and TSP Assessments.)

B. The Statewide **Transportation Planning Unit** (TPU) is responsible for statewide long-range planning and policy development, including creation and maintenance of the Oregon Transportation Plan and other modal plans, such as the Oregon Highway Plan. In addition, TPU is responsible for implementation of plans and policies through the preparation and dissemination of multi-modal, modal, and topic plans, policies, guidelines, and administrative rules. Their primary role is to assist and provide guidance to achieve statewide consistency for transportation planning products and activities. In the development of planning and implementation materials, TPU actively works with stakeholder committees and works to balance needs. They also respond to legislative requirements and initiatives related to statewide planning and policy.

C. The **Multi-modal Freight Planning Unit** (MMFPU) coordinates public-private, state-local, and state-federal freight transportation investment decisions and activities on a state-wide and state-to-state basis to support goods movement and the Oregon economy. MMFPU is responsible for developing and implementing the Oregon Freight Plan, and supporting the integration of freight issues into the State's modal plans, corridor plans, and other planning documents and activities. They manage freight transportation studies and intermodal planning programs and projects, including highway, rail, marine, pipeline, and air transportation.

D. The **Transportation Planning Analysis Unit** (TPAU) provides essential analysis and technical support for ODOT across a full spectrum of transportation planning, project development, and system operation activities. This includes analytical support in the development of statewide policy and plans, transportation system plans, corridor plans, analysis to aid project selection/prioritization, and various project-level analyses. TPAU develops and maintains a suite of analytical and modeling tools, procedures, and guidance. TPAU often acts as a resource to Region Traffic Units requesting technical assistance. TPAU performs analysis, modeling, and technical review for Regions 2-5. TPAU assists Region 1 Traffic, Major Projects, and the Urban Mobility Office as requested. TPAU is made up of three primary programs:

i. The **Planning Analysis Program** conducts, reviews, and provides technical support on a wide variety of studies, including transportation system plans (TSPs), refinement plans, project development, and management plans. The team also maintains the Analysis Procedures Manual.

ii. The **Oregon Modeling Improvement Program** works closely with jurisdictions throughout the state to develop, maintain, and apply state-of-the-art travel demand models for small urban areas, metropolitan areas, regions, and statewide coverage. The team also has developed and is maintaining a statewide travel demand model that integrates transportation, land use and economics to provide a reliable way to forecast and evaluate statewide policies and plans.

iii. The **Data Analytics and Performance Reporting** Program conducts—and develops analytical tools and guidance to support—mobility related, statewide transportation analysis and system performance reporting. The team manages the Regional Integrated Transportation Information System (RITIS) platform and conducts system planning analysis using the Highway Economic Reporting System (HERS-ST) model.

1.2.3 Statewide Research Program

The Statewide Research Program oversees the state's federally funded research, development and technology transfer program with particular emphasis on new technology intended to enhance the performance of Oregon's transportation systems. Major current research focuses on safety, infrastructure repair and preservation, maintenance practices, innovative contracting and project delivery, sustainable environmental practices, and the land-use, transportation connection.

1.2.4 Transportation Data Section

The Transportation Data Section (TDS) collects substantial data for system inventory, volumes and crash information that can be used for the purpose of conducting traffic studies. Because much of these data are collected and processed by different units within the Department, clear and frequent communication between units regarding what is desired and what is available is critical for ensuring these resources are readily accessible. Furthermore, good communication between units will help to obtain the right data in a timely manner, which is important for maintaining project schedules. Coordinate with the appropriate ODOT department or staff as noted below.

- The **Crash Analysis and Reporting (CAR) Unit** generates motor vehicle traffic crash data and statistics derived from Law Enforcement (LE) and Driver submitted crash report forms. These forms are administered by the Driver and Motor Vehicle Services Division (DMV).

  The CAR Unit analyzes reportable motor vehicle traffic crash reports to make the best determination of the crash event details and encodes the resulting information into a Crash Data System (CDS) where it is linked to ODOT public roadway inventory data elements. This data generation phase results in a crash dataset composed of approximately 180 data elements and 1,200 attributes related to characteristics about the crash, vehicles, and participant levels. CAR provides crash data to a broad range of internal and external customers such as engineers, traffic investigators, safety advocates, planners, economists, law enforcement, local governments, legislators, private investigators, law firms, academia, media, the public, etc. This data goes back to 1985 and the overall unit priority is developing, maintaining, and distributing detailed high quality crash data.

- The **Geographic Information Services** (GIS) Unit is responsible for Geographic Information System products and system management. Work products include preparation of Oregon Transportation Map Bases and Enlargement Area maps, official State Highway Maps, web and mobile GIS solutions and custom mapping and geospatial data products as requested. The unit provides geospatial data administration, bringing geospatial data layers into a single environment, available to the entire agency. This includes the development and support of the Oregon Transportation network (OR-Trans) and Emergency Mapping programs. GIS provides geospatial data support to many disciplines including environmental related requests such as natural resource support that includes threatened and endangered species protection under National Environmental Policy Act (NEPA) and water quality protection through Clean Water Act (CWA) compliance. GIS activities support programs reducing wildlife vehicle collision and cultural resources protection including both historic and archaeological resource management. GIS works closely with Geometronics to provide professional land surveyors with access to high accuracy local area network map projections and aid their resource-grade GPS community with GIS mobile solutions. Additional GIS efforts extend to support planning, social equity, environmental engineering, geology/geotechnical engineering, and hydrology through mapping, analysis and application development.

- **Road Inventory and Classification Services** (RICS) Unit collects and maintains road information necessary to classify and monitor the highways, roads, and streets within Oregon; provides mileage statistics; develops, maintains, and enhances ODOT's corporate data base known as TransInfo; maintains the Public Road Inventory which is a compilation of information about the status and condition of the road system in Oregon; and films and maintains the State Highway Digital Video Log. From data gathered, reports such as the Federal Highway Performance Maintenance System (HPMS) submittal and the Oregon Mileage Report are written.

- The **Transparency Accountability and Performance** (TAP) Unit provides leadership and application development to fulfill House Bill 2017's Sections 11 (Uniform Standards and Condition Reporting), and Section 12 (TAP Website Components). The TAP program works closely with Office of the Director staff to ensure uniform standards for condition reporting tools meets the intent of the legislation while delivering value to a diverse stakeholder group that includes City and Counties across the state. Statutory requirements of Section's 11 and 12 include: local government reporting of bridge & pavement condition reporting, AOC/LOC spending reports, on-time/on-budget information, links to local and Connect Oregon project website, and audit reports. The TAP managed Transportation Project Tracker Map application satisfies project-related Section 12 requirements. This application shows how and where Oregon's state and federal transportation funds are spent by local, state, and federal agencies—including projects in the STIP, ODOT's four-year capital improvement program (CIP).

- The **Transportation Systems Monitoring** (TSM) Unit develops and maintains a system to collect and process traffic related data on Oregon's Highways. TSM provides traffic volumes, flow maps, trends, turning movement counts and vehicle class on state highways to Federal, State, Local, private, and public constituents.

1.3 Delivery & Operations Division

The Delivery & Operations Division consists of a wide array of disciplines involved in the operations, construction and maintenance of the state's highways, bridges, and other parts of the transportation system.

1.3.1 Traffic Engineering Section

This Traffic Engineering section prepares specifications, maintains standards for traffic devices and related facilities and provides design expertise in materials, operations, and construction support services. It advises[1] the State Traffic Engineer who has delegated authority to approve the installation of all traffic control devices on state highways, including installation of new signals and major modifications to existing signals. The section consists of two units: Traffic Standards and Traffic Services.

- The **Traffic Services Unit** provides guidance and expertise in the areas of traffic & signal operations, traffic investigations, and traffic safety. Primary work includes administering the federal Highway Safety Improvement Program (HSIP), managing speed zoning for all public roads, applying, and supporting new signal software and technology, and providing engineering support for the State Traffic Engineer's delegated authorities and approvals.

- The **Traffic Standards Unit** oversees the development of standards and guidance for the placement and use of temporary and permanent traffic control devices including signs, pavement markings, signals, illumination, traffic structures and work zones. It provides training in traffic engineering design and construction inspection, performs asset management of signs, signals, and traffic structures, and reviews contract plans.

1.3.2 Roadway Engineering Section

The central Roadway Engineering section prepares specifications, maintains standards, maintains roadway related asset information and provides design expertise in roadway design. It advises the State Roadway Engineer who has delegated authority to approve design exceptions on state highways. The section consists of two units:

- The **Roadway Standards Unit** provides guidance and expertise in roadway design including, interchange design, rural design, roadside barriers, urban design, bicycle and pedestrian design, and the American with Disabilities Act (ADA) compliance. Primary work includes maintaining the Highway Design Manual, Standard Drawings, the ODOT CAD (Computer-Aided Drafting) Manual, Roadway CAD Manual, and providing engineering support to Regions and the State Roadway Engineer's delegated authorities and approvals.

- The **Roadway Asset Unit** is responsible for the inventory, maintenance and reporting of roadway related assets such as traffic barriers, impact attenuators, bike lanes, sidewalk, ADA curb ramps and pedestrian push buttons. It also provides guidance and expertise in inspection practices for ADA curb ramps and pedestrian push buttons.

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[1] All delegated authority requests for State Traffic Engineer approval should follow the process outlined in the ODOT Traffic Manual.

1.3.3 ADA Delivery Program

The American with Disabilities Act (ADA) Program is headquartered within the Delivery and Operations Division. The unit consists of both ODOT ADA staff and contracted consultant partners that are responsible for the delivery of STIP ADA Curb Ramp Projects. Under an ADA agreement, the ADA Program works to mitigate or remediate non-compliant curb ramps and pedestrian-activated signals, champion the use of temporary pedestrian access routes at construction sites, assist the public with comments, questions, concerns, and requests on pedestrian ADA facilities, and provide education for ADA issues on our highway system.

1.3.4 Environmental Section

The Environmental Section is committed to the protection and preservation of our state's unique environment and to the safety of our highway system. They are responsible for coordinating environmental regulatory compliance and tribal government to government consultation for all transportation improvement programs in the state and work closely with Hydraulic Engineering, Bridge and Geotechnical and Engineering Geology Sections. Environmental Section is comprised of the following teams and provide statewide leadership/direction and direct STIP/Maintenance project support for their areas of expertise:

- **Environmental Resources** which encompass wetlands, aquatic biology, administration of Fix-It fish passage program, terrestrial biology, wildlife passage, water quality (in support of Hydraulic Engineering Section), wildlife passage, botany, and regulatory agency liaisons.

- **National Environmental Policy Act (NEPA) and Environmental Policy** which encompasses project classification, leading NEPA review for Class 1 and 3 projects (Environmental Impact Statement and Environmental Assessment), planning and environmental linkages (PEL), environmental justice, LWCF section 6(f), visual impact assessments, and state and national legislation and policy.

- **Cultural Resources** which encompass archaeology, tribal consultation, historic resources, Section 4(f) of USDOT Act of 1969, and regulatory agency liaisons.

- **Environmental Engineering** which encompasses water quality, air quality, acoustics, erosion and sediment control, and roadside development.

1.3.5 Hydraulic Engineering Section

The Hydraulic Engineering section (HES) is committed to the protection and preservation of our state's diverse waterways and to the safety of our highway system. The HES primary focus areas include Hydraulic Engineering, Stormwater Engineering, Stormwater clean-up sites and program funding. HES employees provide statewide leadership, expertise, and direction in a variety of topics included in these technical areas. HES works closely with the Environmental Section, Bridge Section, and Maintenance Operation Branch. HES primary focus areas include:

- **Hydraulic Engineering** encompasses conveyance and stability of rivers and natural channels, bridge hydraulics, flood plains, culverts, fish passage, and asset management condition assessment and inventory of culverts.

- **Stormwater Engineering** encompasses stormwater conveyance and control systems, permitting, asset management and condition assessment and inventory of stormwater features, and supporting the agency MS-4 and UIC permits.

- **Stormwater clean-up sites** encompasses monitoring, controlling, and rehabilitating, EPA & DEQ designated sites (including Portland Harbor).

- **Program Funding** encompasses administering funds for Culvert Fix-It Program, Major Culvert Maintenance Program, Stormwater Retrofit Program, and the Protect Program.

1.3.6 Geotechnical Engineering, Engineering Geology, and Hazardous Materials Section

The Geotechnical Engineering, Engineering Geology, and Hazardous Materials Section (GEEGH) is comprised of technical experts in the following areas: foundations, retaining walls, embankments, landslides, rock slopes, material sources and hazardous materials. The GEEGH Section develops and maintains standard: drawings, design (Geotechnical Design Manual and HazMat Program Manual), and specifications and special provisions for construction. GEEGH provides review of contract plans, specification concurrence, statewide technical training, asset management of unstable slopes, and material sources. Develops standard drawings, special provisions, and specialized work as needed. The Section provides design and construction support for Regions and the State Geotechnical Engineer has the delegated authority to approve design deviations for these disciplines.

1.3.7 Right of Way

ODOT's central Right of Way (ROW) Section provides expertise in real estate and other right of way matters to the Department. The ROW Unit is responsible for:

- The appraisal, acquisition, and management of property acquired for public projects.

- Assisting people and businesses in relocating from the acquired rights of way.

- Administering, directing, and supervising the various programs that reside in the ROW Section.

- Oversees the **Access Management Unit** (AMU) that is part of the Technical Services Branch of the Highway Division. The AMU is responsible for statewide development and administration of the Department's access management program statutes, rules, and policies. It also supports the Regions' Access Management programs by coordinating the rules, policies, and procedures for permitting, locating, and operating driveways that access the state highway system. They manage and coordinate use of the statewide access permitting database AMES (Access Management Enterprise System).

Additionally, each of the five ODOT Regions each have their own ROW and Access Management units. Region ROW facilitates with the acquisition and improvement of real property necessary for the construction and maintenance of Oregon's transportation system and are tasked to maximize the return on the Highway Trust Fund's real property investment through efficient management and sale of surplus property. Region Access Management staff implements the policies and guidelines developed by the Access Management Unit. They report to the Region Traffic Manager, the Region Planning Manager or Region Right of Way Manager. The Region Access Management Engineer (RAME) is the Region technical expert for access management issues. Tasks include:

- Fielding Access Management applications from private interests;

- Coordinating the Access Management Process between ODOT technical staff, the applicant, and their representatives (i.e. engineering consultants, lawyers);

- Reviewing technical data (i.e. traffic studies) submitted by the applicant, or the applicant's representative;

- Approving or denying access application requests and issuing approach permits.

1.3.8 Construction Section

The Construction Section under the Statewide Project Delivery Branch administers statewide construction policies, procedures and processes and provides construction expertise and training programs. The traffic analyst may provide axle-based volume data to the Pavement Services Unit for use in pavement design on projects.

1.3.9 District Maintenance Offices

The District Maintenance Offices are responsible for the on-going preservation and operation of state transportation facilities and the permitting of all activities (utility, access, miscellaneous) within the highway right of way. They are familiar with local issues and the operational and maintenance history of individual highways and can offer valuable input during the identification of needs and alternatives, in addition to tracking the status of existing permits. Because they are ultimately responsible for maintaining any proposed improvements, they should be consulted during the selection and design.

1.3.10 Mobility Program

The Statewide Mobility Program ensures that traffic delays and freight restriction are minimized while Work Zone Safety is emphasized at all levels of planning and implementation. The Highway Mobility Operational Notice (PD-16) provides guidance on implementing key ODOT mobility policies, processes, roles, and responsibilities related to project delivery. This operational notice is consistent with the policies and procedures contained within the ODOT's Mobility Procedures Manual, which is the accepted authority for mobility policy for the Agency. This manual sets project standards and minimum requirements regarding communication and coordination, vertical and horizontal clearance, bridge weight restrictions, delays, detours, staging, and design.

1.4 Other Key ODOT Divisions

1.4.1 Public Transportation Division

The Public Transportation Division is the grantee for Federal Transit Administration (FTA) funds and is responsible for state-level transit program development and management. The division consists of several programs that can be divided into two units that focus on Passenger Rail and Pedestrian & Bike. These programs are responsible for assuring the compliance requirements associated with FTA and state funds are met, even when compliance is primarily the obligation of its recipients. The division provides:

- Technical assistance to transit agencies,
- Grant management, and
- Oversight of projects utilizing state or federal transit funds.

House Bill 2017 established the Statewide Transportation Improvement Fund (STIF) program that is a dedicated source of funding for improving, maintaining, and expanding public transportation for all users.

1.4.2 Rail Safety Section

The Rail Safety Section is under the commerce and Compliance Division of ODOT and has jurisdiction over railroad crossings and traffic control devices used within crossing areas. The section consists of four teams:

- The **Safety Inspection Team** inspects for compliance with state statutes that ensures the safety of rail employees.

- The **Federal Railroad Administration (FRA) Inspection Team** inspects for compliance with federal laws and regulation.

- The **State Safety Oversight Team** oversees safety and security of transit agencies such as light rail, streetcars add trolleys.

- The **Crossing Safety Team** coordinates with project development teams to define responsibilities and liabilities and navigate the crossing order process.

The Rail Safety Section should be contacted any time a project may have an impact directly to or within 500 feet of a railroad or rail crossing at CCDRailCrossing@odot.oregon.gov.

1.4.3 Transportation Safety Office

The Transportation Safety Office (TSO) plans and implements Oregon's Highway Safety Plan and its related safety programs for improving transportation safety for Oregonians and visitors to our beautiful State. ODOT TSO's main responsibility is to improve the safety of all roadway users and all modes of travel in reducing risky driving behaviors through education and outreach, such as impaired and distracted driving behaviors. They provide coordination and technical assistance to help meet ODOT safety improvement goals. TSO awards grants and contracts to partner agencies and non-profit organizations with the goal of eliminating fatalities and serious injuries on Oregon's roadways.

1.5 Other Groups

1.5.1. Urban Mobility Office

The Urban Mobility Office (UMO) oversees, aligns, and implements the Urban Mobility Strategy to reduce congestion and modernize the Region 1's infrastructure. The Urban Mobility Strategy includes once-in-a-generation projects that aim to reduce congestion, update bridges and roads to withstand seismic events. These projects include the following:

- I-205 Abernethy Bridge Project
- I-5 Boone Bridge Replacement Project
- I-5 Rose Quarter Improvement Project
- Westside Multimodal Improvements Study

1.5.2 Department of Aviation

The Department of Aviation establishes**,** coordinates and reviews airport planning rules as per Federal Aviation Administration (FAA) guidelines, funds airport infrastructure improvements, and coordinates efforts to improve airport and airplane safety. When developing projects adjacent to a public or private airport, the airport's runway flight triangles must be taken into consideration, to avoid conflicts. Also, it is a mode of transportation, which should be considered by ODOT staff and consultants when developing Transportation Impact Analyses (TIA), Transportation System Plans (TSP) and other plans.

Appendix 1A – ODOT Traffic Engineering Authority

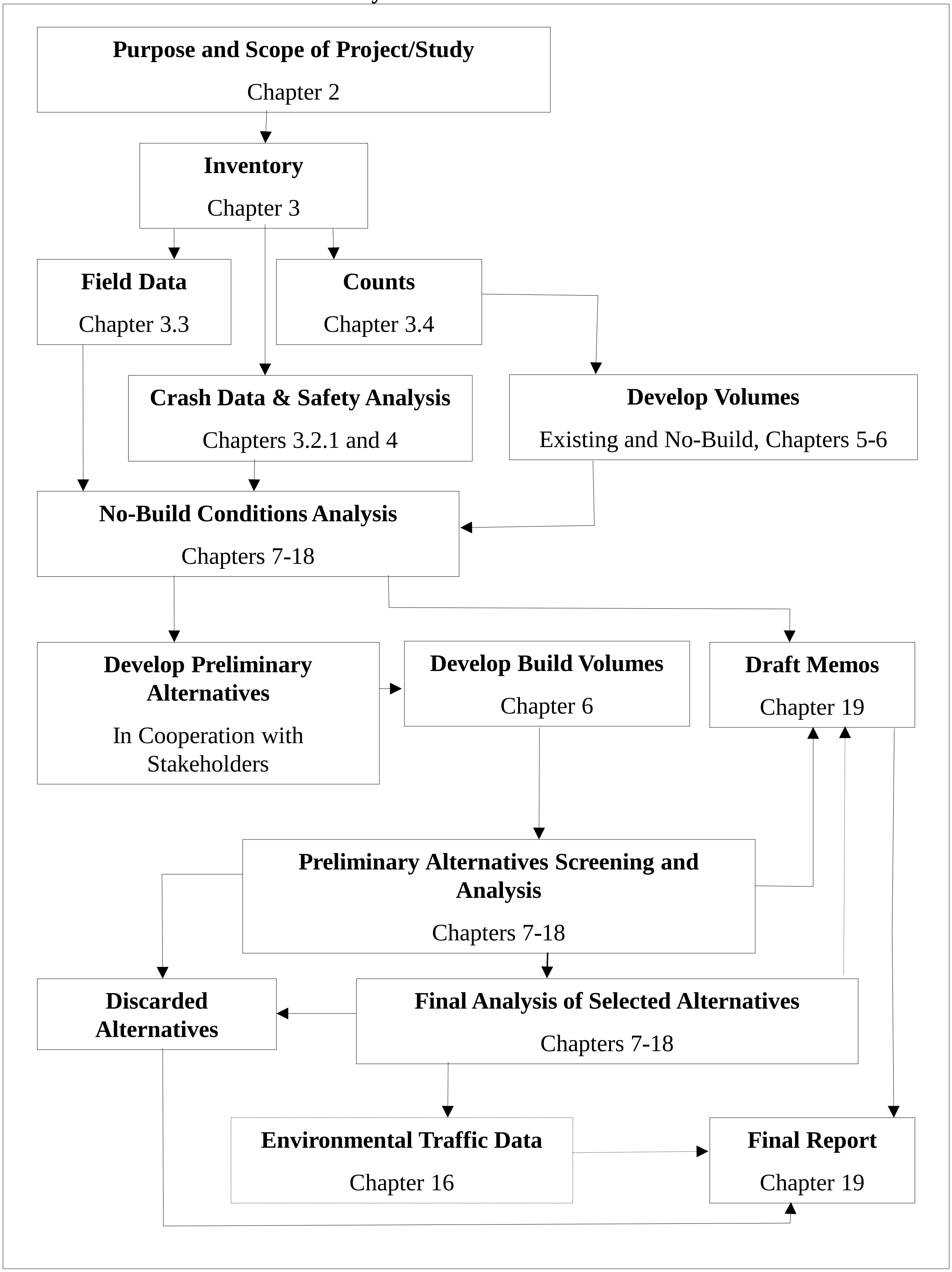
2 SCOPING PROJECTS

2.1 Purpose

The purpose of this chapter is to provide guidance to identify the various steps for scoping the analysis of a transportation study or project. The general flow of traffic analysis steps and the corresponding APM chapters are shown in Exhibit 2-1 below.

The first step is to have a thorough understanding of the work that is identified in a problem statement. The next step is to identify the appropriate level of detail and tools. The last step is to then create a scope of work for the study or project.

Exhibit 2-1: Process of Traffic Analysis



2.2 Problem Statement

2.2.1 Project Understanding

One of the most important steps in conducting analysis work is to clearly define the scope and purpose for the work. Every plan and project is unique with its own set of assumptions and applicable methodologies. There needs to be a clear understanding of what needs to be done by when and at what cost. The understanding should be conveyed through a problem statement that clearly defines the purpose and need for the work. The problem statement is the basis for either creating an analysis work plan for in-house work or for creating a scope of work for contracted work. A detailed scope of work or work plan helps to limit scope creep and lays out expectations for all parties. A problem statement template is available in Appendix 2A.

Information on geometrics, safety, volumes, past studies, prior projects, other analysis performed along with standards, guidelines, and procedures is available and should be used to gain general knowledge of the study area. There may be a project prospectus or initial planning/environmental documents available as well. The analyst should consult or coordinate with the project team to complete the problem statement. There are many useful tools and resources available on the ODOT website. Major sources are explained in Chapter 3.

2.2.2 Project Constraints

Various constraints need to be considered when the work is scoped. If any of these change during the project duration, the problem statement and scope of work should be reassessed and consequences determined. Most constraints fall into two categories. The first is project specific, given the details of what is needed. The second is the project delivery constraints related to delivery /completion date and budget for the work.

The analysis work is controlled by various project factors, issues and concerns. The following questions can focus the problem statement:

- What is the Purpose and Need for the work?
- What questions need to be answered?
- What key issues should be considered?
- What are the Goals and Objectives of the work?
- Who is the audience?
- At what level will the work need to be analyzed and evaluated?
- What types of alternatives need to be evaluated?
- What evaluation measures will be used?
- What is the overall and traffic analysis study area, if different?
- What types of useable information and tools are available and practical?

The purpose and need, goals and objectives and questions and issues typically are determined by a project team, however direction can come from statutes (ORS), rules (OAR), legislation, the Oregon Transportation Commission (OTC), ODOT management, or local jurisdiction. The level of work, types of alternatives and the evaluation measures comes from a process or project/study management team. The study area, tools and information available influence the work. For example, a study in the Portland Metro area can rely on data from "PORTAL" which has very detailed volume information on the freeways. The rest of the state must rely on physical counts or nearby data recorders to obtain volume information. This difference can be a constraint on the project. Similar to data, the choice of performance /evaluation measures can also be a constraint. Chapter 10 provides more guidance on performance measures and their data needs.

2.2.3 Schedule, Resource, and Budget Constraints

The work is driven by a need to deliver an answer in an identified time frame with an identified resource. Questions to identify these factors include:

- What is the timeframe for the analysis work?
- What are the impacts from changes to Purpose and Need?
- What are the risks from outside sources such as other jurisdictions, stakeholders, and private citizens? For example, local concerns/issues/ politics can easily add time to a projected schedule.
- Are there outside factors or time constraints that may dictate delivery of work items? For example, crash information is needed but cannot be obtained in the specified time frame.
- What resources are available? Are they internal or external?
- Are tasks dependent on resources not within analyst's control?
- Does the project funding require certain analysis tools and procedures?
- Is the budget adequate to perform the desired analysis and data collection?
- What is the availability and quality of existing data?
- Can the work be divided? Are tasks independent of each other? Are tasks sequential or concurrent?

2.2.4 Additional Details

After the problem, schedule and budget constraints are completed, additional thought needs to be given to what likely performance measures and tools will be used in the project. A project objective will have specific evaluation criteria/measures that will require a particular performance measure which then will require a certain tool to be used. Level of detail (see Section 2.3), and constraints will determine which tools (see Section 2.4) are practical for the effort. This can be somewhat iterative, so the problem statement may need to be modified as the scope of work or internal work plan is constructed.

For example, under a project goal or objective of mobility, the evaluation measure may be travel time. This might be measured by the buffer index which would require either a travel demand model or a micro-simulation depending on the level of detail needed at a particular step in the process.

Once the steps in this section and the previous sections are completed, this will give the analyst the basis to create the scope of work analysis tasks or an internal analysis work plan.

Example 2-2-1 Problem Statement

The below is an example project statement which includes a summary of the field scoping conditions and the defining statement questions and constraints.

OR193 is an older congested regional highway (not a freight route) in an urban arterial corridor within the city of River City. The highway is mostly four lanes undivided (no center turn lane) with a 35 mph posted speed and an average daily traffic load that exceeds 25,000. The corridor has dense retail/service commercial development with limited right-of-way along its length. There are numerous driveways, many closely spaced, because of past uneven growth patterns. Parking and bike lanes are spotty along the length. Sidewalks are curb-tight with many poor slopes in driveway areas and sub-standard curb ramps. In addition, a number of the intersections are high-crash locations. Congestion extends through the peak hour as a significant bottleneck exists in the study area at a bridge over a local small river. The bridge is still structurally acceptable but functionally obsolete with only two lanes, no bike lanes, narrow lanes and sidewalks. The city wants revitalize the corridor with urban renewal and thus wants to improve it for all users, not just vehicles. Efforts to improve access management in the past have met with resistance from property owners. An improvement project has been listed for this location in the City's Transportation System Plan.

The project has been scoped and meets the needs for an Environmental Assessment (EA) because of the potential impacts. A draft project Purpose and Need (P&N) has been developed: "The purpose of this project is to improve multimodal mobility, increase safety, and enhance the economic development potential of the corridor." Some draft Goals and Objectives have been formed aligning with the P&N on mobility, safety, environmental, limiting natural and built-up impacts, and economic development.

The project needs to conform to practical design objectives so that the alternatives can stay within a 25-50 million dollar range. This would keep the project reasonably likely so it could be included in the next available State Transportation Improvement Plan. A travel demand model is available for the area.

Problem Constraints

- What is the Purpose and Need for the work?
    - From the draft P&N: "The purpose of this project is to improve multimodal mobility, increase safety, and enhance the economic development potential of the corridor."
- What questions need to be answered?
    - What are the base conditions to help define the project need? What are the actual congestion and safety problems? What are the future conditions? What is the impact of access management on congestion? What are the pedestrian, bike, and transit needs in the corridor? Are there cost-effective alternatives available that can address most of the issues?
- What key issues should be considered?
    - Whether to widen or to build a new vehicle and/or ped/bike bridge
    - Balancing different modal needs
    - Access management and parking impacts to properties

- What are the Goals and Objectives (G&O) of the work?
    - From draft G&O document: Mobility, safety, environmental, limiting natural and built-up impacts, and economic development.
- Who is the audience?
    - The audience is multilevel with state and local staff and stakeholders (i.e. business groups, bike community, neighborhood associations, freight users) and the general public.
- At what level will the work need to be analyzed and evaluated?
    - Alternative concepts will need to be evaluated on a screening basis.
    - Once concepts are more developed into alternatives, then the alternatives can be analyzed at key locations using deterministic tools.
    - The final set of build alternatives that go into the EA (including the no-build) are analyzed in full detail including simulation, multimodal and predictive safety tools. Air & Noise traffic data will be required for the final alternatives.
- What types of alternatives need to be evaluated?
    - No-build (this is what other alternatives are measured against)
    - Possible future land use alternatives as this drives the future improvement needs (this would need to be settled first before getting into detailed analysis).
    - Couplet (new) alignments
    - Bridge replacement
    - Road diet to better accommodate other modes
    - Wider or better utilized right-of-way to improve multimodal, parking and mobility needs including bridge widening.
    - Transportation System Management (TSM)/Safety improvements including painted/raised medians, center turn lanes, improved crosswalks, and improved access management.
    - Alternative strategies such as a new ped/bike bridge, off-street parking, transportation demand management (TDM), or TSM.
- What evaluation measures will be used?
    - Ones that are applicable to this project area include: These can include: volume-to-capacity ratio (v/c), level of service (LOS), queuing, travel time, delay, emergency response time, multimodal level of service (MMLOS), ped/bike system connectivity, accessibility, duration of congestion, percent crash reduction, expected crashes, access spacing, and conflict points.
- What is the overall and traffic analysis study area, if different?
    - Land use and bridge scenarios will require a city-wide look initially to determine if any significant negative impacts exist on areas potentially outside the project limits. If these occur, and the alternatives will still be pursued, then the project limits and/or the analysis study area may need to be modified.
    - Because this is a congested area and traffic simulation will be necessary, the study area will need to go out two signalized intersections outside of the project limits (area of potential impact). The project will at least need to go a couple of block on each side of the highway to accommodate the couplet alternatives.

- What types of useable information and tools are available and practical?
    - Inventory data, some counts, analysis and modeling work done for TSP
    - A travel demand model and local modeling staff time is available.
    - Mesoscopic methods (i.e. windowing out of a model area)
    - Highway Capacity Manual (HCM) and Highway Safety Manual (HSM) methods
    - Micro-simulation

Schedule, Resource, and Budget Constraints

- What is the timeframe for the analysis work?
    - This project is going to kickoff in July, so counts will need to be obtained immediately to capture the 30th highest hour conditions.
    - The project is one of the Region's top priorities
    - Since this is an EA, the end date is somewhat unknown but is expected to last at least 36 months.
- What are the impacts from changes to Purpose and Need?
    - Likely impacts could be additional alternatives, rework of analysis which will lead to more time required to do the work.
- What are the risks from other sources such as other jurisdictions, stakeholders, and private citizens? For example, local concerns/issues/ politics can easily add time to a projected schedule.
    - Access management and parking concerns will likely cause delay or create issues with the city council/planning commission.
    - Potential issues with internal and/or outside business groups
    - Environmental/Riparian issues along the river
    - Potential additional bridge routes/alignments
- Are there outside factors or time constraints that may dictate delivery of work items? For example, crash information is needed but cannot be obtained in the specified time frame.
    - Counts will need to be obtained between July and September in order to stay on schedule.
    - Staff time to perform analysis is limited because of other legislative high-priority work. Will likely need to have help from consultant to stay on schedule.
- What resources are available? Are they internal or external?
    - Internally, there is the project lead (manager) and at least one analyst available. The EA consultant has additional analysis staff available to help out on contingency.
- Are tasks dependent on resources not within analyst's control?
    - Yes, model applications are dependent on current workload; alternatives are dependent on region designers, alternative development process is dependent on feedback from stakeholders, public open houses and the teams themselves.
- Does the project funding require certain analysis tools and procedures?
    - The project falls under NEPA requirements so full counts and inventory data will be necessary to stay consistent with land use requirements and to support the environmental analysis. In addition, the travel demand model is required to be used.
- Is the budget adequate to perform the desired analysis and data collection?
    - The budget is adequate for the data collection, and up to three detailed alternatives

in the EA document.

- What is the availability and quality of existing data?
    - State inventory data is current. The TSP analysis is still usable but more counts and data will be needed for the project analysis.
- Can the work be divided? Are tasks independent of each other? Are tasks sequential or concurrent?
    - Most work can be divided into concurrent tasks, but simulation work must be done sequentially from a single source to avoid inconsistencies in assumptions. Generally, no-build work must precede alternative work.

Additional Details

- Given the above mentioned evaluation measures and other issues what are the likely performance measures that will be needed?
    - Volume to capacity ratio for the state highway and level of service for the local system will be required to compare with established performance targets and standards. In addition, travel time and queuing will be needed to test the overall efficiency of alternative operations. Multimodal level of service will be used to gauge the impact of mobility alternatives on the pedestrian/bike/transit operations and vice versa.
- Likely tools to be used?
    - Analysis of the no-build and the alternatives will require the use of the travel demand model to help develop the volumes and to create high-level screening measures to test any model-usable concepts (i.e. road diet, couplet and other network changes), Tools such as Highway Capacity Manual-based software (i.e. HCS and Synchro) and the multimodal tools will be needed to develop the detailed analysis. Micro-simulation will also be needed to create the travel time measures for the detailed analysis and to create the queuing data.

2.3 Level of Detail

There are many types of analysis work done for transportation related issues. The analysis ranges from high-level policy and procedures, through subject or facility plans to specific issues, locations or improvements projects. For example, a 10-mile long facility plan should have a different level of analysis than a single intersection realignment. The analysis process can have multiple levels such as single-issue or fatal flaw screening through detailed reporting like micro-simulation.

2.3.1 Types of Work

Planning Studies

These studies are generally limited to a specific geographic area or corridor or can cover multiple topics/issues. All of these can be either rural or urban and can overlap into different elements as well as varying level of detail. In some regions, the terms “corridor”, “facility”, “refinement” all mean the same level of effort. Expressway plans are similar but deal with a certain facility type and condition reports only deal with existing or future no-build needs yet the plans may not look different from a typical refinement plan.

The key difference in plans will be what are the specific questions needing answers and the detail level needed to answer them regardless of what the plan is named. For example the I-205 corridor in a dense urban area would involve use of a regional travel demand model, dynamic traffic assignment (DTA), and micro-simulation while the comparatively rural US395 corridor would only have an urban model in a couple locations and use a higher segment-based analysis.

Examples of each can include:

- Statewide Policy / Plans - This work is generally of a statewide nature but can be topic specific; typically guidance or an overriding policy such as:
  - Greenhouse Gases (GHG)
  - Least Cost Planning (LCP)
  - Legislative Studies / ORS / OAR
  - Performance Measures like Alternative Mobility Standards
  - OTP and Modal Plans (OHP, Freight Plan, Rail Plan, Bike/Ped Plan)
- System Plans – these focus on:
  - Metropolitan (urbanized) areas –Regional Transportation Plan (RTP)
  - County/multiple jurisdictional – County TSP, Regional Transportation Plan.
  - City – City Transportation System Plans
- Corridor/Facility Plans – these focus on:
  - Specific highway corridor
  - Land Use (TPR-type zone change, UGB expansion)
- Refinement Plans – these focus on:
  - Topic (Road Diet or Conversion of Two-Way to One-Way)
  - Feature (IAMP),
  - Mode (Safe Routes to Schools)
  - Highway segment
  - Sub-Area

Projects

This operational analysis is limited to specific locations often with specific guidelines or parameters that influence the work. They include

- Modernization – this work covers large issues / needs that must be measured against specific data needs or standards. The intent is to bring the facility/project up to standards and/or formalize exceptions.
- Safety / Operations / Preservation – this work is specific in nature and may allow deviation from standards. For example on a facility with a high crash rate fitting a minimal left turn lane within the given right-of way may not have standard widths and tapers.
- Traffic Impact Analysis (TIA)/Traffic Impact Studies (TIS) – this work is driven by the development itself. The analysis is to determine facility adequacy, significant effects and required mitigation.
  - Transportation Planning Rule (TPR)/Zone Change –When a proposed

    development requires a change in land use and has a significant effect to the state facility, certain criteria must be met.
  - Conforming Use Development- When a proposed development does not fall in the TPR/Zone change, the analysis is dependent on local land use code and approach permitting
- Approach Permitting – These are for developments that do not require a TIA. This driveway type and location considerations such as sight distance, conflict points, influence areas.

Other Procedural / Research Studies

This is analysis work in support of a specific topic or tool, such as but not limited to trip generation studies, determination of analysis factors, procedures or calculators. These studies should conform to appropriate national methods and accepted analytical processes. This work needs to be detailed out with specific deliverables and agreed to by the parties requesting and accepting the work.

2.3.2 Level of Analysis

The questions being asked and the data available can determine the level of analysis needed. This can range from policy and systemic reporting to a very detailed specific need. Resources are limited, so the level of analysis needs to be tailored to the questions.

- Rules of Thumb – Generalization based on common conditions with very little detail. Can be in tabular or checklist format or "canned." For example, when AADT exceeds 60,000, then six lanes are needed. These need to be taken into context as a general ballpark only. They may be used to help determine a starting point. Actual decisions need to be based on more detailed analysis.
- Broad Brush (a.k.a. "30,000- or 10,000-foot view") – Big picture generalized planning analysis or a preliminary estimate of a more detailed later task. This could include using daily traffic volumes (AADT's), assumed roadway geometrics, or default values. This is typically at the system or corridor level (not intersection level). Data is typically at an aggregated level.
- Screening – A mid-level analysis where some plan/project specific data is available but likely not fully developed. This process is used to whittle down large number of scenarios/concepts/alternatives/options to a more reasonable number. This can include both qualitative and quantitative elements. Measures can include items such as policy/standard compliance (a.k.a. fatal flaw analysis), natural and built environmental constraints, segment demand-to-capacity ratios, and key intersection volume to capacity ratios. Context sensitivity / practical design considerations are important at this level and below. Typically in a plan or project, multiple levels of screening are used in increasing detail.
- Detailed – This is a comprehensive look a study area or topic. This uses study-specific data where the use of defaults is limited. This analysis typically furnishes detailed numbers for establishment of standards or policies or details for a specific design (i.e. storage lengths, signal progression). Actual plan or project decisions can be made from information derived from this level.

2.4 Tools

There are many tools used for transportation and planning analysis. They are a mixture of commercially developed and internally created programs serving anything from analyzing the effect of a specific policy to a specific project detail. Often a particular process becomes identified as a tool and is included for discussion purposes.

2.4.1 Individual Tool Descriptions

The following tool descriptions are not exhaustive list and mainly concentrate on the ODOT-used ones. Some of these tools are discussed in more detail in following chapters. Some of these tools could be considered a process, i.e. sub-area modeling or zonal cumulative analysis. Some tools may package together multiple tools (i.e. HCM analysis and simulation) in a suite format. Other tools and methodologies not listed here or later in the APM may be acceptable if explained and documented in a methodology memorandum and approved by ODOT. For more information or availability of these tools, please contact TPAU staff.

Activity Based Models (ABM) - Compared to traditional trip-based models, the activity-based model system has more detailed and accurate representation of space, time, travel patterns, and significantly more person and context-based explanatory variables by individually modeling persons and households. These improve the modeling of non-motorized travel, time-of-day, ride sharing, non-home-based travel, accessibility effects, and provide a flexible household travel survey-like database for custom summaries. This does come at the cost of greater data complexity, data needs, and run time. This modeling system was also developed as the eventual framework for exploring new policy issues: new vehicle types and emissions, parking and different pricing scenarios, connected and automated vehicles, vehicle ownership moving to service, light-weight vehicle infrastructure, telecommuting, and others.

ATR Characteristic Map - The ATR Characteristic Map is web-based mapping tool that displays ATR's based on their characteristic trend. Detailed information can be displayed on the map by clicking on ATR symbols. The ATR Characteristic Table and map are updated on a yearly basis, typically in September.

ATR Characteristic Table – The Automatic Traffic Recorder, or ATR, Characteristic Table can be used to estimate seasonal traffic count adjustments based on common characteristics such as number of lanes, weekly trends, and rural/urban area types.

Crash Decoder Tool – This macro-based spreadsheet tool converts the extensive numerical codes in the typical "long-form" comprehensive PRC crash listing to words. This eliminates the need for the analyst to be familiar with the crash coding manual. The tool will also summarize and graph the crash characteristics.

Crash Graph Tool – This Excel macro-based spreadsheet uses input from the Vehicle Direction report to create a standard set of crash graphs and table by year, severity, collision type, time of day, day of week, surface condition, light condition, on and off-roadway crashes, and milepoint.

DANA – The Database for Air Quality and Noise Analysis (DANA) tool from FHWA provides probe traffic data-based inputs to the Motor Vehicle Emission Simulator (MOVES) vehicle emissions model and the Traffic Noise Model Aide (TNMAide). This allows for real-world measurements of traffic conditions instead of using transportation model base year traffic data (e.g. vehicle-miles traveled, speed) for environmental studies.

Dynamic Traffic Assignment (DTA) - DTA is an alternative mesoscopic method that assigns traffic across time periods so that adjacent time periods and congestion affect each other. DTA is useful when the typical peak hour traffic spreads across multiple hours. The assignment is more detailed as it incorporates basic signal timing and platoons of vehicles which allow for queuing to be modeled. DTA is a further level of effort and refinement of the model assignment between the typical travel demand model and micro-simulation. Common DTA software tools are Dynus-T and Dynameq but is also included in VISUM and VISSIM.

FIXiT - The Future Improvement Examination Technique (FIXiT) sketch planning tool from the Texas A&M Transportation Institute (TTI) is used to assist in the proper allocation of funds in regard to choosing the right set of congestion mitigation and mobility strategies. The FIXiT tool can be used as an early screening tool in conjunction with a series of factors when allocating funds and prioritizing projects for an area.

Future Volume Tables – These show the future annual average daily traffic volumes for state highway segments based on either 20 years of historical traffic counts or travel demand model-based growth trends depending on the location. These are updated annually.

HCS (HCM) – Highway Capacity Software is a faithful implementation of the Highway Capacity Manual methodology. These methods are point/segment based and are isolated so adjacent sections do not affect each other. This is the only source of deterministic analysis tools for freeway operations (segment, merge, diverge, and weave). HCS/HCM also considers pedestrian, bike and transit multimodal analyses. The HCS/HCM is mostly operational (high detail level) based but there are some planning-level methodologies available using mostly default values and average daily traffic volumes.

HCM Pedestrian Crosswalk LOS & Delay tool – This tool implements the HCM 2010 (and later) MMLOS methodology for analyzing pedestrian crosswalk delay.

HSM Spreadsheets – These implement the Highway Safety Manual Part C predictive safety analysis methods for rural and urban facilities. These include current available Oregon calibration data and factors.

HSM Part B Screening Tools – These characterize observed crash data from a large study area using a minimum of extra data. The results are used to identify a smaller set of locations that can then be analyzed with the HSM Part C predictive methods. These include the Critical Crash Rate and the Excess Proportion of Specific Crash Types explained in Chapter 4.

IHSDM – The IHSDM (Interactive Highway Safety Design Model) implements the HSM Part C predictive method for rural and urban highway segments. The IHSDM evaluates the effects of geometric changes on safety. The IHSDM has a very high input detail level as it requires full geometric design data (curves, grader, stationing, super-elevation, etc). This is best suited for evaluating final design alternatives.

ISATe –HSM-based Part C predictive analysis spreadsheet-based tool for free-flow facilities, interchanges, ramp terminals and ramp roadways.

Level of Traffic Stress (Bicycle & Pedestrian) – These index-based methodologies quantify the perceived “traffic stress” (i.e. comfort) safety issue of an user being in close proximity to vehicles on a spacing, speed, or and/or volume basis for rural or urban facilities.

MMLOS Intersection – These are index-based analysis tools based on intersection elements (e.g. curb ramp quality, speed) that affect pedestrian and bicyclist safety and comfort using an expert judgment/index basis rating rather than using the high data and time requirements of the full HCM MMLOS method.

MMLOS Segment – These are simplified HCM MMLOS methodologies for pedestrian and bicycle facilities and transit operations along roadway segments.

NCHRP 562 Crossing Treatment tool – This is a spreadsheet tool that can be used as a guide to select or screen potential pedestrian crossing treatments for plans and projects ranging from signing/marking to full mid-block traffic signals.

OTSDE - the Oregon Transportation Safety Data Explorer, is a web based GIS tool, that supports ODOT work in safety and multi modal areas, helping users see connections to leverage efforts across programs. OTSDE has three main areas: identifying corridors, filtering and extracting crash data, and multi modal active transportation work. Users can view and filter crash data, network screening data, and active transportation data to work on crash data analysis, traffic safety investigations, multi-modal analysis, and TSP (Transportation System Plan) review.

Qualitative Multimodal Assessment –This methodology generally follows the principles of the HCM multimodal level of service (MMLOS) but uses general roadway characteristics and applies a context-based subjective “Excellent/Good/Fair/Poor” rating for a relative high-level analysis where the HCM MMLOS does not apply, in small cities, or where data and resources are limited.

RITIS Analytics - The Regional Integrated Transportation Information System (RITIS) is a data integration tool that has an automated data sharing, dissemination, and archiving system that includes many performance measures, dashboards, and visual analytics tools that can be used to gain situational awareness, measure system performance, and communicate information between agencies and the general public. The major sources of data in RITIS are probe speeds, incidents and events, sensors and detectors, dynamic message signs, and weather. In transportation system analysis, RITIS allows users to calibrate models to existing conditions, identify/map existing system performance, and perform before-after analysis.

Seasonal Trend Table - The Seasonal Trend Table can also be used to estimate seasonal traffic count adjustments using seasonal factors organized into different characteristic trend types. The Seasonal Factor Table is updated on a yearly basis, typically in September. Season Trend Table factors are calculated using the previous full year of ATR data.

SIDRA – SIDRA is a lane-based mesoscopic analysis tool that can analyze urban facilities for the full capacity, delay and queuing impacts in the system without the need for micro-simulation. SIDRA has been typically used for roundabouts, but also is necessary for any intersection/segment configurations that cannot be handled in standard HCM methodologies such as three-way stops, complex interchanges (e.g. single-point and diverging diamonds). SIDRA can also analyze signalized pedestrian crossings, ramp meters, or bus-only, light-rail or truck lanes on segments.

SimTraffic – SimTraffic is a micro-simulation based on the arterial portion of CORSIM and thus is best suited for simulating arterial networks. This allows system-level analysis of a study area network for queuing impacts, travel time, delay of individual vehicles as well as network wide.

Subarea modeling – This is a mesoscopic technique by cutting out a portion of a travel demand model (also called windowing) to be used separately or adding detail to a portion of a model (also called focusing). Both methods involve adding detail to the model (e.g. signal timing, smaller zones, additional centroid connectors, better refined capacities). This additional detail will improve calibration between the model and field counts, possibly to the point that post-processing is very limited. Typical software for the windowing method would be VISUM.

SWIM (StateWide Integrated Model) – is a forecast model designed to represent the Oregon economy with respect to land-use and transportation by simulating the activity and market exchanges made by people and businesses. Household and business location decisions are simulated, as well as the travel generated by activities - such as commuting to work, purchasing commodities for industrial production and transporting final goods to markets within Oregon and outside of the state. It is designed for statewide and regional long range transportation planning and policy analysis. Information from SWIM can be used to inform other modeling tools, answer freight flow questions, determine re-routing impacts (from emergencies, landslides, seismic events, construction projects, etc.), discuss answers to questions from the Oregon Transportation Plan around economics and land use, and to understand traffic impacts in locations where no other modeling tools exist.

Synchro – Synchro is mainly for the operation and system optimization including timing and progression of signalized study area networks. Synchro is mainly focused on the analysis of signalized intersections and arterials following current HCM methodologies.

Trip Generation - This process implements the ITE Trip Generation Manual procedures based on land uses and other site size and type characteristics.

Urban Models (4-Step Travel Demand Models) – Travel Demand Models are generally built as system-level tools so detail is limited at a facility level; including basic characteristics such as number of lanes and speed limits. These models can be used as a screening tool using district travel patterns (origin-destination), demand-to-capacity ratios, percent volume differences, and high level estimates of travel times. Travel Demand Models are also used to develop future volumes for plans and projects through post-processing methods. See the available TPAU and

MPO urban model map on TPAU's website at:
https://www.oregon.gov/odot/Planning/Pages/Technical-Tools.aspx#travelDemandModel

VisionEval – This model can be described as a "disaggregate demand/aggregate supply" model. It combines demographic and socioeconomic details from a synthetic population with aggregate treatments of travel (multi-modal vehicle-miles traveled and congestion without explicit trips, or transport networks). The implications of the "aggregate supply" model is that VisionEval cannot be used to evaluate performances of specific projects or corridors. This model is used for large-scale regional emissions planning as part of an area's Greenhouse Gas (GHG) reduction efforts. The model is used to help establish a transportation strategy for meeting greenhouse gas targets, it can also be used for asking a very broad range of "what if" questions about how the transportation system might perform, and how its benefits and costs might be distributed over the community. It can efficiently process hundreds of scenarios looking at many different types of interventions, alternative policies, and hypothetical future conditions and travel behaviors.

Vissim – Vissim is a complex but comprehensive micro-simulation which can be used to model virtually any road network. Vissim also has the capability to handle DTA and multiple modes. The software is extremely customizable but requires significant effort to calibrate and report out useable data. Vissim is a preferred tool for simulating roundabouts, complex intersections, interchanges, and freeway operations.

Vistro- Vistro software analyzes and optimizes traffic operations for intersections, corridors, networks, or transportation impact analyses. Vistro allows use of the multi-resolution modeling process as Visum files can be imported and detailed for a sub-area analysis or eventually passed into Vissim for micro-simulation.

Zonal Cumulative – The Zonal Cumulative volume development process is mainly a manual three-step (generation, distribution and assignment) analysis. This involves creation of zones, uses ITE Trip Generation methodologies, external trip origin-destinations, and gravity-based distribution. Calibration is also required. The Visum modeling software is used to streamline the assignment process in the Enhanced Zonal Cumulative method.

Other Planning Analysis Tools – There are a number of less complex tools that can be used to evaluate a system at a relatively high level. These also can be used as part of a more detailed plan or project analysis as an alternative screening tool. Most of these are based on average daily traffic. Some examples are preliminary signal warrants (PSW) and other tools and methods in the Planning and Preliminary Engineering Application Guide (companion to the HCM)

2.4.2 Tool Evaluation/Selection

The second step for scoping is to determine what tools are available and can be used at particular stages in the analysis effort. Selection of the appropriate analysis procedures from this manual will often be determined by project-specific characteristics such as the type of project, the surrounding environment and land uses, availability of data and the type of traffic controls present in the field. Generally, similar types of projects will use the same analysis procedures to varying degrees. Depending on the study and the project's purpose and need, additional data may be required. For further information on project scoping and selection of traffic analysis procedures, refer to the Federal Highway Administration's (FHWA) website for the Traffic Analysis Toolbox: https://ops.fhwa.dot.gov/trafficanalysistools/

In this manual, tools are broken into two basic categories: deterministic and stochastic and two sub-categories: systemic and specific. Considering the project needs and constraints, tool selection can greatly impact the resources and time necessary to do the analysis.

- Deterministic – These tools use a set of fixed inputs and result in a single set of outputs. These include, for example, Highway Capacity Manual procedures and the three or four-step travel demand models.
- Stochastic – These tools use a set of inputs and create multiple sets of outputs. These include, for example, micro-simulation and dynamic traffic assignment, activity-based models, and the statewide model. These tools are much more time-intensive for inputs, process, and analysis of outputs.
- Systemic – These tools often cover multiple points, corridors, or networks that interact / influence with each other (e.g. SIDRA Intersection network analysis).
- Specific – These tools analyze a single location (e.g. HCM segment-based analysis).

In the sections that follow, the tools are grouped first by study type and then by analysis type, however, the tables are not exhaustive. Studies likely require more than one type and related tools over their duration. Even within these tools there is a range of uses from high-level planning to project specific. Some tools can be used for widely varying efforts depending on the questions asked. An "X" indicates the tool type in the following exhibits.

The following discussion assumes that these tools have been created, tested, and operational. If a tool is not fully updated and useable, additional time or special considerations may be needed in tool selection.

For each tool, the general data needs, staffing requirements, and time required are shown. Many tools may have a range depending on the task at hand which may be "high" for a simulation model, to "medium" for facility analysis, and to "low" for individual intersection analysis. The following exhibits attempt to reflect a relative difference between the tools and may not capture every specific circumstance.

- "High" (H) generally means obtaining large quantities of data, confidential data, data not readily available, or in the correct format. Long times to collect data or the cost of collection is also considered. These tools require a special separate project effort to collect and clean the data or to customize the tool to the location. Use of default values is very limited. Some tools may require many staff members such as updating a travel demand model or may require many hours from a single person, such as micro-simulation. Processes may take months or years in some cases.
- "Medium" (M) generally means data obtainable in the field or from available information/databases although it might still need conversion or cleaning. Staffing requirements will be a single to a few staff members. Time requirements are generally on the order of weeks.
- "Low" (L) generally means published data or available data in correct formats and can be immediately used. The tool may have a high use of default values. Overall data

requirements are limited in total amount or elements. These tools can be used by a single staff member and time requirements are on the order of days.

Tools Grouped by Study Type

Policy Tools

Used for answering policy level questions such as for the Oregon Transportation Plan or Oregon Highway Plan. Exhibit 2-2 shows this group. These questions can be from the legislature or upper management. Questions can be very complex, to determine the effects of potential policy decisions involving many diverse factors in broad areas including economics, land use and transportation. SWIM is the primary tool used for this purpose. VisionEval has been used to determine strategic direction for greenhouse gas (GHG) emission impacts.

Exhibit 2-2: Policy and Statewide Planning Tools

Tools	Deterministic	Stochastic	Systemic	Specific	Data Needs	Staffing	Time
	Category				Applications		
SWIM		X	X		H	H	H
VisionEval		X	X		H	M	M

Statewide Planning Tools

Used for statewide or multiregional analysis studies such as the Bridge Limitation Strategy. These studies usually involve the highway network or elements such as freight movement, congestion, economic impacts of delay/detours, etc. SWIM is used for general flows including freight movements and economic impacts.

System Planning Tools

Used for a wide range of uses in regional, county or city planning studies such as Regional Transportation Plans (RTP) and Transportation System Plans (TSP). Exhibit 2-3 shows the tools for this group. VisionEval is used for establishing regional strategies for greenhouse gas emissions in urbanized areas. For example, SWIM has been used to create growth rates for rural projects, separate truck growth rates, external station growth for models, and distributions for supplementing cumulative analyses. Both activity-based and traditional urbanized and small city travel demand models are used to determine system travel patterns or behavior, to compare multiple land use and network scenarios, or can be used to develop post-processed volumes for more detailed analysis for specific locations.

The DANA tool can be used to quantify emissions (including GHG) across a system or a specific route and create outputs for use in more-detailed tools such as for traffic noise. RITIS Analytics modules can be used to define system-level performance. If no travel demand models exist, then a zonal cumulative or a traditional historic volume growth process may be used.

Other tools cover specific routes, segments, or locations. Multimodal analysis would be typically done on at least a collector-level and higher facilities across all applicable modes using generally qualitative or low-data requirement quantitative methods. The HSM screening tools are used to flag safety issue areas across an entire area. Highway Capacity Manual operational or planning analysis techniques can be used to determine operational results of roadway segments for all modes. Other planning analysis tools include daily-traffic based intersection tools or service volume tables for determining needs or general traffic control types.

Exhibit 2-3: System Planning Tools

Tools	**Deterministic**	**Stochastic**	**Systemic**	**Specific**	**Data Needs**	**Staffing**	**Time**
	Category				**Applications**		
SWIM		X	X		H	H	H
VisionEval		X	X		H	H	H
Urban Models (MPO & small urban)	X		X		M	M	M
Activity-Based Models		X	X		H	M	M
DANA	X		X		L	L	L
RITIS Analytics	X		X	X	L	L	L
HCS (HCM)	X			X	M	L	L
HCM Planning Analysis	X			X	L	L	L
HSM Part B Screening Analyses	X		X		L	L	L
Multimodal Analysis (Qualitative & LTS)	X		X	X	L	L	L
Other Planning Analysis	X			X	L	L	L

Corridor & Refinement Planning Tools

Used for specific highway segments, either over an entire highway or a smaller section within or between cities. Exhibit 2-4 shows this group. Corridor plans have more detail than system plans yet may lack the specific details of project or facility refinement plans. Corridor plans may deal with single or multiple parallel facilities or may be dealing with the potential of multiple potential new alignments. Refinement/facility plans are generally limited to the specific roadway in question. The tools used form the full range from the systemic travel demand model-based to intersection operational analysis to micro-simulation. Travel demand model-based tools are used to compare scenarios or help with screening alternatives. Some tools like dynamic traffic assignment (DTA) are used to incorporate shifting demand into the model assignments to better match the field conditions and limit the amount of post-processing needed. If no travel demand models exist, then a zonal cumulative or a traditional historic volume growth process may be used.

Historic crash analysis is needed at the corridor planning level and crash screening could be done to supplement if there are enough intersections or segments. Predictive crash analysis may be used on the final solutions for urban arterials and rural non-freeways. The NCHRP HSM spreadsheets are used to do the predictive analysis while the Crash Decoder Tool is used to simplify the historic crash analysis.

Analysis would be typically done across all applicable modes at varying levels of detail from HCM-based analysis to micro-simulation. HCM planning analysis can be used for determining operational results of state highway segments along the corridor. RITIS Analytics can be used to help identify bottlenecks, travel times, incident effects and other measures across a corridor. If intersection analysis is needed then any of the HCM-based methods and tools (such as HCS or Synchro) can be used. Typically, micro-simulation will not be needed in corridor plans other than in metropolitan areas. Vissim is typically used for freeway operations, complex configurations, roundabouts, and/or including multimodal issues and SimTraffic is used for signalized arterial corridors.

Exhibit 2-4: Corridor & Refinement Planning Tools

Tools	**Deterministic**	**Stochastic**	**Systemic**	**Specific**	**Data Needs**	**Staffing**	**Time**
	Category				**Applications**		
Vissim		X	X	X	H	H	H
Urban Models (MPO & small urban)	X		X		M	M	M
Activity-based Models		X	X		H	M	M
Dynamic Traffic Assignment		X	X		M	M	M
Subarea modeling	X		X	X	M	M	M
SimTraffic		X	X		M	M	M
HSM Part B Screening Analyses	X		X		L	L	L

Tools	Deterministic	Stochastic	Systemic	Specific	Data Needs	Staffing	Time
	Category				Applications		
NCHRP HSM Spreadsheets	X			X	M	L	M
ISATe	X			X	M	L	M
RITIS Analytics	X		X	X	L	L	L
Synchro	X		X	X	M	M	L
Vistro	X		X	X	M	L	L
SIDRA Intersection	X		X	X	M	M	L
HCS (HCM)	X			X	M	L	L
Crash Decoder Tool	X			X	L	L	L
HCM Planning Analysis	X			X	L	L	L
Multimodal Analyses (LTS & MMLOS)	X		X	X	M	L	L

Project Tools

Project analysis is the most detailed of all analysis types. Project analysis can involve the full range of tools from the travel demand model to micro-simulation and may involve specialized areas such as environmental analysis. Travel demand model-based tools are used to compare scenarios or help screen alternatives and are used to create post-processed design hour volumes to be used by other tools. Some tools like dynamic traffic assignment (DTA) are used to incorporate shifting demand into the model assignments to better match the field conditions and limit the amount of post-processing needed. The smaller scope of these plans allows sub-area (window) modeling to be done, which will further reduce the post-processing need. If no travel demand models exist, then a zonal cumulative or a traditional historic volume growth process may be used.

Detailed historic and/or predictive crash analysis is needed at the project level. Tools like IHSDM and the NCHRP HSM spreadsheets, and ISATe are used to do the predictive analysis on urban and rural arterials and freeways while the Crash Decoder Tool is used to simplify the historic crash analysis.

Analysis would be typically done across all applicable modes at varying levels of detail from HCM-based analysis to micro-simulation. HCM/other planning analysis tools are mainly used for alternative scoping or screening. RITIS Analytics can be used to help identify bottlenecks, travel times, incident effects, calibration data, and other measures across a corridor. An intersection-level analysis is usually needed so any of the HCM-based methods and tools (such as HCS or Synchro) can be used. In congested areas or where the HCM methods no longer apply then micro-simulation is required. Vissim is typically used for freeway operations, complex configurations, and/or including multimodal issues, and SimTraffic is used for signalized arterial corridors. Some projects may involve environmental traffic analysis for noise, air quality and energy impacts which normally requires the use of intersection analysis tools and spreadsheets.

Exhibit 2-5: Project Tools

Tools	Deterministic	Stochastic	Systemic	Specific	Data Needs	Staffing	Time
	Category				Applications		
Vissim		X		X	H	H	H
IHSDM	X			X	H	M	H
Urban Models (MPO & small urban)	X		X		M	M	M
Activity-Based Models		X	X		H	M	M
Dynamic Traffic Assignment		X	X		M	M	M
Subarea modeling	X			X	M	M	M
SimTraffic		X	X	X	M	M	M
RITIS Analytics	X			X	L	L	L
Synchro	X		X	X	M	M	L
Vistro	X		X	X	M	L	L
SIDRA Intersection	X		X	X	M	M	L
NCHRP HSM Spreadsheets	X			X	M	L	M
HSM Part B Screening Analyses	X			X	L	L	L
ISATe	X			X	M	L	M
HCS (HCM)	X			X	M	L	L
Crash Decoder Tool	X			X	L	L	L
Trip Generation	X			X	L	L	L
HCM Planning Analysis	X			X	L	L	L
Multimodal Analyses (LTS & MMLOS)	X		X	X	M	L	L
Other Planning Analysis	X			X	L	L	L

Specific Analysis Elements

Volume Development Tools

These tools are used to help develop volumes to be used in the analysis at various levels of detail as warranted by the individual study. Where a travel demand model is available, model assignments can be created to help in creating growth rates for post-processing or modeling specific scenarios Dynamic traffic assignment and subarea modeling techniques can be used to refine the assignment. Zonal cumulative analyses are used where a travel demand model is not available but may involve use of model assignment software. At the very least, the Future Volume Tables can be used to create forecasts, and the ATR Characteristic and Seasonal Trend Tables are used to adjust for the base conditions. Trip Generation (manual lookup or software-based) is used in cumulative analysis methodologies using rates from the ITE Trip Generation manual.

Exhibit 2-6: Volume Development Tools

Tools	**Deterministic**	**Stochastic**	**Systemic**	**Specific**	**Data Needs**	**Staffing**	**Time**
	Category				**Applications**		
Zonal Cumulative methods	X			X	H	M	M
SWIM		X	X		H	H	H
Urban Models	X		X		M	M	M
Activity-Based Models		X	X		H	M	M
Dynamic Traffic Assignment		X	X		M	M	M
Subarea modeling	X			X	M	M	M
Future Volume Tables	X			X	L	L	L
ATR Characteristic & Seasonal Trend Tables	X			X	L	L	L
Trip Generation	X			X	L	L	L

Screening Analysis Tools

Screening analysis is typically a high-level analysis used to reduce the number of scenarios/concepts/alternatives to a manageable level. Most studies will have multiple levels of screening at increasing detail levels. At the top end, travel demand model-based tools are typically used for the system impacts. The Planning & Preliminary Engineering Application Guide, a companion to the HCM, offers numerous levels of screening methods and tools from intersection analysis to service volume table lookups. Middle and lower screening levels are more detailed at the intersection level, which may be using key intersection volume-to-capacity ratios, or multimodal level of services. HCS/HCM methods can use defaults to simplify operational analyses for a quick assessment including reliability. Screening analysis in a project –level analysis will be more complex than in a system plan. Other analysis type tools such as RITIS, FIXiT (TTI), and FHWA's DANA tool, to name some examples, all use probe data, which can be used to assess different performance metrics at a screening level.

Exhibit 2-7: Screening Analysis Tools

Tools	Deterministic	Stochastic	Systemic	Specific	Data Needs	Staffing	Time
	Category				Applications		
Urban Models	X		X		M	M	M
Dynamic Traffic Assignment		X	X		M	M	M
Subarea modeling	X			X	M	M	M
HCS (HCM)	X			X	M	L	L
HCM PPEAG methods	X			X	L	L	L
Preliminary Signal Warrant Analysis Worksheet	X			X	L	L	L
Other Planning Analysis	X			X	L	L	L

Multimodal Analysis Tools

For a complete analysis of a system or a project area, a multimodal analysis is needed. Urban MPO (Metropolitan Planning Organization) models and the newer subset of activity-based models can be used to determine mode share for the base case as well as alternatives. Bicycle and Pedestrian Level of Traffic Stress methodologies are used to assess network connectivity, quality, and accessibility and can be also used to prioritize improvements based on the size of the resulting low-stress network. Multimodal level of service (MMLOS) is used to create qualitative and quantitiave assessments for the pedestrian, bicycle, and transit modes across different facility types for segment and intersection analysis.

Common analysis tools such as Synchro and HCS use HCM methods that ODOT does not use (other than a few specific cases) because of the large amount of data required, so these are not in the tables. In congested areas or in areas where non-auto modes are very common mixing with vehicular traffic such as light rail, streetcar, and buses, SIDRA mesoscopic analysis or Vissim micro-simulation will be needed to judge the impacts of these modes on each other. Vissim is also needed where heavy rail modes cross roadways. Analysis of off-roadway operations (e.g. heavy/light rail, bus rapid transit guideways) requires different resources, such as the Transit Capacity Quality of Service Manual (TCQSM), which will be a future addition to this manual.

Exhibit 2-8: Multimodal Analysis Tools

Tools	Deterministic	Stochastic	Systemic	Specific	Data Needs	Staffing	Time
	Category				Applications		
Vissim		X		X	H	H	H
Urban Models	X		X		M	M	M
Activity-Based Models		X	X		H	M	M
SIDRA Intersection	X		X	X	M	M	L
Qualitative Multimodal Assessment	X		X		L	L	L
Bicycle & Pedestrian Level of Traffic Stress	X		X	X	M	L	L
MMLOS methods – Segment	X			X	L	L	L
MMLOS methods - Intersection	X			X	M	L	L
NCHRP 562 Crossing Treatments	X			X	L	L	L
HCM Pedestrian Crosswalk LOS & Delay	X			X	L	L	L

Safety Tools

Most studies will require some sort of safety analysis. At a minimum, a historic crash analysis is needed. The Crash Graphing and Crash Decoder tools simplify the processing and analysis of individual crash records. The Oregon Transportation Safety Data Explorer (OTSDE) GIS tool can expedite safety screenings or capture project-level historic crash data and also allows simultaneous viewing of geographic, safety, and multimodal data. Historic crash data are also needed to support more-detailed predictive analysis. Plans and projects can use Highway Safety Manual (HSM)-based screening methods for establishing critical crash rates within a community or predicting the number of crashes associated with changes to roadway segments or intersections. Tools for screening include the HSM Part B screening method critical rate and excess proportion of crash types calculators. Additional HSM-based spreadsheets tools are available to conduct segment and intersection predictive analyses or ISATe for freeways and interchanges. Interactive Highway Safety Design Manual (IHSDM) software allows for very detailed evaluation of safety effects of geometric design decisions and requires a project-level data collection effort to be used appropriately. See Chapter 4 for more information on safety analysis tools and methods.

Exhibit 2-9: Safety Tools

Tools	Deterministic	Stochastic	Systemic	Specific	Data Needs	Staffing	Time
	Category				Applications		
IHSDM	X			X	H	M	H
ISATe	X			X	M	M	M
HSM Spreadsheets	X			X	M	L	M
Excess Proportion of Crash Types Calculator	X		X		L	L	L
Critical Rate Calculator	X		X		M	L	L
OTSDE GIS Tool			X	X	L	L	L
Crash Decoder Tool	X			X	L	L	L
Crash Graphing Tool	X			X	L	L	L

Environmental Traffic Data Tools

Sometimes, plans and projects will require some sort of environmental analysis. This can range from high-level travel demand model-based efforts to very detailed traffic volume development. VisionEval is a strategic-level tool that estimates greenhouse gas emissions (GHG) across a metropolitan region and thus will be mainly used in the development of regional transportation plans (RTP) and related scenario planning/strategic assessments. Development of RTP's requires an air quality conformity determination. Also some metropolitan and smaller urban areas have been (none currently are) non-attainment areas for air quality. The urban models feed data into the air quality model (MOVES) which is the tool for determining air quality conformance. Tools such as FHWA's DANA package allow data for MOVES to be created from probe data automatically, saving time. Project analyses will generally involve at least a noise study with many larger projects needing air quality or GHG emission studies. Traffic data for these studies is typically done using spreadsheets.

Exhibit 2-10: Environmental Traffic Data Tools

Tools	Deterministic	Stochastic	Systemic	Specific	Data Needs	Staffing	Time
	Category				Applications		
VisionEval		X	X		H	H	H
DANA	X		X		L	L	L
Urban Models	X		X		M	M	M
Spreadsheet-based analyses	X		X	X	M	M	M

2.5 Creating a Scope of Work (SOW)

The third step for scoping is to determine the specifics for the scope of work (SOW). SOWs can be written for TSPs, Corridor/Facility Plans, Refinement Plans, IAMPs and projects. Traffic Impact Analyses/Statements (TIAs/TISs) also have a SOW, but specific criteria should be consistent with Chapter 3 of ODOT's Development Review Guidelines.

The purpose of establishing a SOW for a transportation study is to define critical parameters such as the study area boundaries, analysis requirements, data needs and the identification of specific concerns to be addressed. An effective SOW should always produce a completed study that satisfies the needs of the corresponding project.

Common elements for most types of transportation studies include:

- Background or Purpose Statement
- Objectives of the Study
- List of Work Tasks
- Identification of Deliverables
- Project Schedule
- Project Budget

It is important that the work tasks and corresponding deliverables be clearly defined and that the party responsible for completing them is identified. The distribution list for deliverables should generally include all the pertinent teams/groups and specific ODOT sections needed for review.

All ODOT analyses must have a discussion on methodologies and assumptions used. For SOWs there must be a requirement for a Methodology Memorandum clearly shown. This memorandum details out the methodologies and assumptions that are to be used in the existing and future volume development and analysis. It should include the range of analysis methodologies from identifying count locations through simulation, including any safety and multimodal analysis. This memorandum should be provided to and approved by ODOT before any analysis work is conducted. This helps to significantly reduce the amount of review by ODOT and potential re-work by the Contractor.

The SOW must require that the methodology memorandum is completed by the Contractor and reviewed by and agreed to by ODOT prior to the Contractor starting any volume development and/or analysis tasks. In the absence of a SOW requirement, the APM requires use of the same methodology memorandum. Appendix 19A contains an annotated example methodology memorandum. This example does not necessarily include all methodologies that may be applicable in a given context.

2.5.1 Traffic Scoping Considerations

Each SOW likely has specific individual issues; however, there are many common needs such as professional engineer licensing requirements and specific requirements for the state highway system. Some typical SOW traffic analysis statements are shown below.

The following lists are typical of suggested statements used in SOW's for the traffic analysis tasks and are not exhaustive. Not all of these statements will apply to a single study. These tasks may be modified as desired to fit a study's particular needs.

General

- Final versions of the Contractor's transportation analysis must be stamped by an Oregon-registered Professional Engineer (P.E.) with license being current and in good standing, with expertise in civil or traffic engineering.
- Traffic analysis must follow the Highway Capacity Manual (HCM) [named current version] procedures and comply with ODOT's Analysis Procedures Manual available at https://www.oregon.gov/ODOT/Planning/Pages/APM.aspx. Signalized intersection v/c's shall be computed manually unless software-calculated.
- Contractor shall coordinate all traffic analysis with ODOT's Transportation Planning Analysis Unit (TPAU) and Region [1-5] Traffic Section. [Coordination with local jurisdictions or groups such as MPO's, may be necessary.]
- Consultant shall obtain approval of existing and future analysis methodology from TPAU and Region [1-5] Traffic via a methodology memorandum prior to beginning analysis.
- All documents will be readable and usable in black and white [Exceptions can be specified for certain deliverables.]
- All documents must be written in plain language and use an easily understood format.
- Contractor shall review all applicable plan/past project documents to the study area
- Contractor shall allow two weeks for review of written and analysis deliverables or as agreed to by the contract administrator.
- Contractor shall furnish written and electronic documentation for all assumptions, data, calculations, and results. This includes paper and computer files (i.e. spreadsheets and analysis software files).

Inventory

- Counts should be broken down by type and duration as suggested in the Analysis Procedure Manual Chapter 3. For clarity, the count locations, types, durations should be

identified on a map as follows:
  - Contractor (or ODOT/Agency) will provide 16-hr intersection classification traffic counts with 15-minute intervals at the morning and afternoon peak hours at the following locations:
  - Specific locations in list
  - [Note: Similar language is needed for any peak period counts or any road tube counts]
  - All counts must have at least 15-minute breakdowns from 2 - 6 PM. All counts must include bicycles, pedestrians, and turning movements.
  - State highway volumes and classification information is available here: https://highway.odot.state.or.us/cf/highwayreports/traffic_parms.cfm
- Field inventory information – lane configurations, traffic controls, speeds, operational issues (queuing, unique driving behaviors, etc.) will need to be obtained by the Contractor.
- If micro-simulation is desired, then Contractor shall obtain necessary calibration data.
- Note: Five (5) years of crash data shall be obtained from ODOT's Crash Analysis & Reporting Unit for state highways and any local roadways desired in the study area.

Volumes

- All traffic volumes must be adjusted to reflect the 30th highest hour [Note: use the alternative standard (i.e. average volumes) if it applies to the study area].
- Areas that are covered by a travel demand model must use the model to develop future no-build and build alternative volumes.
- Contractor must submit a model request to ODOT's Transportation Planning Analysis Unit (TPAU) at least three weeks before the data are needed. The model request form is available at: https://www.oregon.gov/odot/Planning/Pages/Technical-Tools.aspx#travelDemandModel [Note: This is only for travel demand models that TPAU has developed.]
- All raw model numbers must be post-processed or used only in relative (percentage) comparisons.

Analysis

- Analysis locations will include at least the traffic count locations. Exact intersection analysis locations will be determined during negotiations.
- Traffic analysis must follow methods in the Highway Capacity Manual (HCM) [named current version] published by the Transportation Research Board (TRB) or as agreed to by ODOT. All traffic analysis software programs used must follow Highway Capacity Manual [named current version] procedures. Signalized intersection v/c's shall be computed manually unless software-calculated.
- Contractor shall prepare and submit a Methodology Memorandum documenting methodology and assumptions to be used for existing conditions (i.e. seasonal factors used), future conditions (i.e. volume development/post-processing methodology), and alternative analysis (i.e. peak hour factors, analysis parameters, calibration, etc.) to TPAU and Region [1-5] Traffic Section. Consultant shall obtain approval of methodology from TPAU and Region [1-5] Traffic Section prior to beginning analysis. Consultant shall obtain approval of analysis and conclusions from TPAU and Region [1-5] Traffic Section

prior to submitting Draft Tech Memos.

- [Note: for existing conditions] Contractor shall obtain the past five years of crash data from Agency's Crash Data & Reporting Unit for both state and non-state roadways and perform crash analysis. Contractor's data for state highways shall include locations of Safety Priority Index System (SPIS). Contractor shall use the Highway Safety Manual Part B Network Screening Critical Crash Rate and Excess Proportions of Specific Crash Types methods for intersections. Each reference population used in either method must have at least 5 (five) sites. In addition, at least two sites must have at least 2 (two) crashes of the target crash type to be applicable for the excess proportion method. If this is not met, intersection crash rates need to be compared with the published 90th percentile rates (See ODOT Analysis Procedure Manual Chapter 4).
- If the local agency or region has established a critical crash rate for segments or if each segment type within the study area includes a minimum of five study segments, Contractor shall use the Highway Safety Manual Part B Network Screening Critical Crash Rate method for segments to compare segment crash rates to a critical rate. If fewer than five segments exist for a segment type, segment data must be compared with the official published crash rates (ODOT Crash Tables – Table II) for similar facilities [Does not apply to urban areas]. Segments must be homogenous in number of lanes, area type, and volume.
- For intersections that exceed the identified critical crash rate, the excess proportion of specific crash types, and/or the published 90th percentile rate, crash patterns, evaluation of causes and potential countermeasures must be identified for each site. Consultant shall map locations of these safety issue areas and the Safety Priority Index System sites. Consultant shall utilize the Crash Data and MMLOS/LTS to identify potential countermeasures and safety improvement alternatives.
- For segments that exceed the identified critical crash rate or published crash rate and intersections that exceed the identified critical crash rate, analysis of crash patterns, identification of contributing factors, and potential countermeasures need to be completed. Contractor shall map locations of these safety issue areas and the SPIS sites.
- [Add to the above before the last sentence for projects and detailed refinement plans/IAMPs]. Contractor shall perform HSM Part C predictive analysis for all screened locations exceeding segment or critical rates for the existing conditions.
- [Note: Future safety for TSP's, similar detail level plan, and project development screening] For each alternative developed to specifically address a safety concern, Contractor shall summarize safety impacts of each design. Contractor shall use the All-Roads Transportation Safety (ARTS) Crash Reduction Factor Appendix as the initial source of countermeasures. If the ARTS Appendix/List is not sufficient, then Contractor shall use the Crash Modification Factors (CMF) in the HSM Part D and/or the FHWA CMF Clearinghouse to indicate the potential crash reduction for each safety alternative or countermeasure. The ODOT CMF standard is to only use CMF's with a quality rating of three stars or better.
- [Note: Future safety for projects or detailed refinement plans/IAMP's] For each alternative developed to specifically address a safety concern, Contractor shall summarize safety impacts of each design using HSM Part C predictive methods. Full predictive analysis should only be completed on and reported out on the final alternatives (including the future no-build) to be included in the [plan/project].

- Consultant shall conduct a qualitative ("Good, Fair, Poor") multimodal assessment for the Project Area collectors and above as per APM Chapter 14. The assessment analysis must include bicycle, pedestrian, and transit (if applicable) operations.
- Consultant shall conduct Level of Traffic Stress (LTS) analysis for all roadways in the Project Area as per APM Chapter 14. As much as possible, data should be obtained from current aerial photography and (TSP) roadway inventories before field data collection. Bicycle & Pedestrian LTS will be evaluated and results graphically displayed for the existing conditions.
- Contractor shall conduct a high level MMLOS analysis for [indicate roadways to be studied] in the study area. MMLOS analysis must include vehicle, transit, bicycle, and pedestrian operations. Pedestrian analysis should not include the effective width of sidewalk data and calculations but rather assume a clear sidewalk width for each side of each segment. Bike analysis should use the link-level analysis only. Transit analysis should use as much general or average data from available transit district information as possible.
- Traffic analysis at ODOT intersections must be consistent with ODOT's Analysis Procedures Manual (APM) available on the internet at https://www.oregon.gov/odot/Planning/Pages/APM.aspx . Traffic analysis at non-state intersections needs to be compatible with ODOT procedures and must follow standard engineering procedures and practices.
- ODOT may approve a different or additional intersection analysis method prior to use when the different method can be justified for local and ODOT facilities. Contractor must provide documentation fully explaining the issue and the reasons for the proposed change. Contractor must obtain approval before use.
- Operational targets for state facilities should include the volume to capacity (V/C) ratio. Existing conditions and future no-build must be compared to the Oregon Highway Plan (OHP) v/c targets. Build alternative v/c's are to be compared with ODOT's Highway Design Manual v/c's. Standards for non-state facilities can be v/c and level-of-service (LOS) or a combination of v/c and LOS, depending on the local standards.
- Other performance measures may be required which can include queuing, MMLOS, simulation-based MOE's, etc. All secondary performance measures shall be included in the methodology memorandum.
- Simulation to determine queues or other measures of effectiveness should be used if v/c's exceed 0.70 and simulation shall be used if v/c's are equal to or exceed 0.90. Simulation shall also be used if existing conditions show congested conditions, (i.e. intersection queuing backs into adjacent intersections/connections) or if Agency requires it.
- If simulation is desired to obtain 95th percentile queue lengths or other measures of effectiveness, then the simulation shall be calibrated following procedures in Chapter 15 of the Analysis Procedure Manual.
- Areas with complex freeway geometry/operations, transit operations (bus, light rail, etc), railroad pre-emption, require simulation and shall follow the VISSIM Protocol available at: https://www.oregon.gov/ODOT/Planning/Pages/APM.aspx

Traffic Data to Support HB 2017 Benefit Cost Analysis

- Following procedures in the APM, traffic data shall be prepared to support HB 2017 Benefit Cost Analysis (BCA). The analysis shall be coordinated with and prepared using assumptions and parameters provided by the Region and the economists in the ODOT Program Implementation and Analysis Unit (PIAU). Traffic data shall be provided for the No Build and Build Alternative for both the Base Year and Horizon Year.

Air/Noise/Energy Traffic Data

- Air/Noise/Energy traffic data must be obtained and reported as in the Analysis Procedure Manual Chapter 16 or as agreed upon by Agency.
- Traffic Data for Noise Analysis (Contingency)
    - Consultant shall prepare traffic data needed for noise analysis and noise technical report. This analysis shall include:
        - Existing, future build (design year), and future no-build traffic data for each roadway link in the project area, including collector and higher functionally classified cross streets, for the peak hour and the peak truck hour and in an MS Office-compatible spreadsheet in the form of:
            - Link volumes for each traffic direction
            - Percentages of the following vehicles on each link:
                - Automobiles (FHWA vehicle classes 1-3)
                - Medium trucks (FHWA vehicle classes 4-5)
                - Heavy trucks (FHWA vehicle classes 6-13)
        - Existing and future posted speeds
        - Existing 85th percentile speeds (if available)
        - For each traffic signal in the project area, the percentage of vehicles affected (expected to come to a stop).
        - Land use zoning information for properties within the project area in the form of:
            - Existing zoning
            - Future zoning or predicted changes in land use from existing use
        - Please note that the peak truck hour is typically not in the same period of the day as the peak hour, so longer duration vehicle classification counts (ideally 16+ hour) are necessary. Please refer to ODOT's Analysis Procedures Manual Chapter 16 for details on roadway link creation, vehicle classification, and required factors and their calculation.
        - If peak hour or the peak truck hour link volumes exceed the maximum LOS C volumes (LOS C/D threshold) then the link volumes shall be capped at the maximum LOS C volume. LOS C comparative volumes for can be obtained from current Highway Capacity Manual methods. LOS C volumes for intersection approaches also require an iterative process to obtain the target LOS C value.
    - The methodology for creating the noise traffic data shall be documented in a methods and assumptions memorandum to be reviewed and approved by Agency Transportation Planning Analysis Unit and Region Traffic Engineer before work on creating the noise traffic data starts.
    - The completed draft noise traffic data and related documentation (calculations, notes, etc.) shall be reviewed and reviewed and approved by the Agency Transportation Planning Analysis Unit and Region Traffic Engineer. The noise

traffic data shall be provided as appendix material in the draft and final Noise Technical Report.

Coordination with other Work Areas

- If rail facilities are within the study area, coordination with the ODOT Rail Division is required.
- If the study area is adjacent to an airport or includes any overlay zones then coordination with the Department of Aviation is necessary.
- If studied facilities are formally recognized freight routes, then compliance with ORS 366.215 "Reduction in Capacity" may be needed if alternative concepts could potentially restrict the roadway width (i.e. curb extensions, medians, etc.).

2.5.2 Developing Work Plans

Once a SOW has been determined, a detailed internal workplan that outlines the analysis tasks and timelines should be developed. This will help the analyst determine the analysis process necessary to achieve the SOW deliverables. For internal ODOT-only studies, where SOW's are not developed, then a detailed workplan is required. This is typically developed by the project manager but also could be developed all or in part by the by the project analyst. Exhibit 2-1 shows the typical traffic analysis work flow.

The work plan must include the project title, highway name and number, and a purpose statement to identify the project objectives. This work plan must be consistent with the overall project schedule and should be submitted to the project leader for review of timelines. This document will help explain to the project leader/contract administrator analysis and staffing/resource needs and timelines. At the very least, this will start conversation about project expectations. Delays or additional work requests will extend the timelines. The work plan should be updated as the study proceeds especially if major changes occur. A revision date should be on the first page of the work plan. A typical analysis will include most, but not necessarily all, of the following tasks. The tasks are not necessary linear, some may be concurrent or overlap while others provide feedback to other tasks.

The following example tasks assume a typical plan or modernization/safety project. This would need to be modified to fit any specific study.

- **Task 1 – Project Identification and Understanding**
  The purpose of this task is for the analyst to have an understanding of purpose and need of the study, the parameters and constraints that influence it, the questions that need to be answered, and to define the level of analysis and tools to be used.

  The methodology of this task should include the following which impact the analysis:
    - Study purpose and need and goals and objectives
    - Identify any prior relevant plans (i.e. TSP's, refinement plans), analysis (i.e. prior/adjacent projects, traffic impact analyses) that need to be considered in the study area including any usable inventory data
    - Identify any concurrent project or planning efforts in or adjacent to study area. This will require coordination (i.e. data, timelines, project progress etc.) between

these efforts.
  - Identify any new data needs (such as need to update a travel demand model)
  - Identify any constraints or issues that may affect the project (i.e. funding, natural or built environment, politics).

  Task deliverables includes a summary of any prior or current studies in or adjacent to the study area with issues, constraints, and impacts to the analysis discussed. A traffic analysis methodology memorandum may be appropriate if the work needs to be coordinated with multiple staff and or studies.

- **Task 2A – Transportation System Inventory**
  The purpose of this task is to review existing data and collect additional inventory data for the study area. Note: Allow 6-8 weeks from date of request for counts requested from ODOT.

  The methodology for this task should specify the following:
  - Count Request – This should include count locations, types and durations and any other special considerations. Refer to Chapter 3.
  - Field inventory data needed including, but not limited to the suggestions in Chapter 3.
  - Office inventory data needed including, but not limited to:
    - Five years of crash data for roadways in the study area
    - Available inventory data
    - Pertinent map/aerial photograph of area for figures
    - Roadway functional class, designations and planning information
    - State and local performance measures
  - If simulation, or reliability, multimodal, or predictive crash analysis will be needed then more extensive inventory data will be required.
  - Other optional inventory data that may be needed, depending on project.

  Task deliverables include inventory information, project area map and photo for use in the following tasks.

- **Task 2B – Methodology & Assumptions Memorandum**

The purpose of this task is to create the methodology and assumptions memorandum (see Chapter 19) which needs to be done before any volume development and analysis is performed.

All methodologies and assumptions going into the volume development and analysis for the existing conditions, future no-build and any build alternatives needs to be described. This covers items including, but not limited to:

  - Study area coverage
  - Count locations/types/durations
  - Seasonal/growth factors
  - Analysis years
  - Peak hours/periods
  - Travel demand models/scenarios to be used

- Trip generation/distribution assumptions
- Performance measures to be used
- Analysis tools to be used – both at screening and detailed levels
- Major analysis parameters (e.g. saturation flows, peak hour factors)

All proposed analysis processes for historic/predictive crash analysis, segment/intersection operational analyses, multimodal, reliability, and micro-simulation should be covered in the memorandum. Calibration documentation tasks for micro-simulation or other analyses should be part of this memorandum or as an appendix instead of in a separate process to improve efficiency. The timeline needs to allow two weeks for review by ODOT Region Traffic, other ODOT units, and local jurisdictions as necessary.

Task deliverables include the draft and final versions of the methodology and assumptions memorandum to be used in the following tasks.

- **Task 3 – Develop No-Build (Existing and Future) Design Hour Volumes**
  The purpose of this task is to develop base year, build year and future year no-build design hour volumes (30th highest or other alternative standard). Occasionally, other interim years may be needed such as air quality or project phasing (short-term fixes) issues. The base year is the year of the study, or when most of the data was gathered. The build year is the year that has the day of opening of the project. Generally, the build year is one year (for small projects like intersections) to two or more years (for large projects like interchanges) from the let date shown on the project prospectus. The future (design) year is typically 20 years from the build year. For example, a 2015 interchange project with a let date in 2019 could have a base year of 2012, a build/opening year of 2021 and a future/design year of 2041.

  The methodology for this task is to use the manual count data to obtain the existing/base 30th highest hour volumes (30HV) using historical and seasonal adjustments in Chapter 5. A single system peak hour must be used with volume balancing as appropriate.

  The future volume development methodology should be described, whether by historical trends, cumulative analysis, or with a transportation model (see Chapter 6).

  Task deliverables include the figures/worksheets showing the traffic volume development process and the balanced base 30 HV and future no-build volumes on figures in the technical memoranda for the existing and the future no-build conditions.

- **Task 4 – Analysis of No-Build Transportation System**
  The purpose of this task is to evaluate no-build system conditions for the base, build and future years. This may help identify any deficiencies related to the study purpose and need.

  The methodology for this task should use the base year, build year and future year data developed in Task 2 along with current Highway Capacity Manual (HCM)-based analysis software to evaluate the system by performing the following:

- Use crash data from Task 1 and perform a safety analysis following procedures in Chapter 4. This needs to include a Highway Safety Manual Part B network screening process of locations before doing more detailed safety analysis. This should cover intersections and segments and use historical and predictive methodologies as appropriate.
- Operational analysis including preliminary signal warrants, turn lane criteria, access/street spacing, and signal progression should be covered.
- Evaluate the volume to capacity (v/c) and level of service (LOS) or other performance measures as appropriate by jurisdiction for the study area for intersections, merge/diverge/weaving sections, freeway mainlines and highway segments.
- Perform a multimodal evaluation depending on study type and desired detail level following procedures in Chapter 14.
- Micro-simulation modeling may be needed if there are multiple signals involved or congested conditions exist. All simulations shall be calibrated according to APM methods. Simulation outputs should at least include 95th percentile queues, travel times, speeds, and overall hours of delay. Remember to allow the additional time (least a month) to calibrate the existing condition simulation model.
- Turn bay storage lengths will be compared to the 95th percentile no-build queues. Blocking of turn bays and upstream intersections must be noted if microsimulation is performed.

The task deliverable is a technical memorandum which includes the safety and operational analysis including the various performance measures. If simulation is used, then a calibration report on data gathered, calibration locations and overall calibration results (i.e. methodology memorandum appendix) will be required.

- **Task 5 – Evaluate Preliminary Alternatives (Screening)**

  The purpose of this task is to work with the project team to develop and screen the preliminary alternatives.

  The methodology for this task is to review goals and objectives with the team considering identified needs, and evaluate each preliminary alternative by comparing operations, safety, multimodal, or other higher-level performance measures at major intersections or other agreed upon key location(s) for the appropriate years. Travel demand models can also be used to screen alternatives effectively if the alternatives have the potential to change traffic patterns beyond the local area. Travel demand model screening can include relative volume comparisons and link demand-to-capacity (d/c) ratios. The future no-build alternative needs to be included in the analysis as the baseline that the preliminary alternatives are compared against.

  Any comparisons using HCM-based v/c's need to use the current Highway Design Manual (HDM) v/c's. Travel demand model-based link d/c's have different methodology and cannot be directly compared to the HDM v/c's. Travel demand model-based screening criteria should be based on relative comparisons. Comparisons may also need to be made to local jurisdiction operational standards which could be LOS, v/c or delay based.

  The task deliverable is a technical memorandum, with the screening criteria and results shown for each alternative. A summary comparison table that shows how the alternatives and the future no-build alternative perform against the screening-level criteria and each other must be included. Recommendations for keeping or dropping alternatives based on traffic analysis considerations should be included.

  Note: Post-memo project team decisions on alternatives should be documented in the next technical memorandum on build alternatives and need to be documented in the alternatives considered but dismissed section of the final memo/narrative.

- **Task 6 – Evaluate Build Alternatives**

  The purpose of this task is to work with the project team to develop and completely evaluate the detailed alternatives that satisfy the future transportation needs for this project. All tasks need consider timelines per alternative. Ideally, no more than three detailed alternative analyses are performed. Too many full–detail alternatives will result in excessive time or budget requirements.

  The methodology for this task is:

  - Develop build and future year Design Hour Volumes (DHVs) for each of the final alternatives. Either distribute the no-build volumes on the build alternatives or create new build volumes for each alternative if currently diverting traffic (i.e. latent demand; see Chapter 6) that would return with the new alternative or new induced demand that could be created is sufficient to invalidate the use of the no-build volumes.
  - Operational considerations such as preliminary signal warrants, functional area, and turn lanes will be evaluated.

- Evaluate the v/c and LOS and other performance measures for the study area for intersections, merge/diverge/weaving sections, freeway mainlines, and highway segments.
- Perform a predictive safety analysis if appropriate following procedures in Chapter 4.
- Perform multimodal evaluations across the pedestrian, bicycle and transit modes.
- Microsimulation of build conditions including determining turn bay storage lengths using the 95th percentile build queues and blocking of turn bays and upstream intersections.
- The output build v/c's must be compared with the HDM design v/c's for state facilities. Non-state v/c or LOS need to be compared with the appropriate local operational performance measure standard.
- Work with ODOT Traffic-Roadway Section and Region Traffic if new signals/roundabouts or changes to existing signals are involved. A progression analysis for the study area is needed if more than one signal is included in the alternative.
- Determine if access, intersection and interchange spacing meets or improves over current conditions in the OHP spacing requirements.

The task deliverables include a technical memorandum with volumes, v/c, LOS, queues, safety, multimodal, and other measures of efficiency shown on diagrams for each alternative, a summary comparison of the alternatives (including a table) and how they fared against the evaluation criteria and each other. Also, the design storage lengths and other geometric details need to be forwarded onto the appropriate design staff, either ODOT or consultant.

- **Task 7 – Environmental (Air/Noise/GHG) Traffic Data (If Needed)**
The purpose of this task is to produce environmental traffic data for the no-build and build alternatives. The base year and no-build future year environmental traffic data can be started immediately after the Task 3 volumes are completed. The build alternative environmental traffic data should not be started until the final build alternatives are selected. Note: need to make sure that the data collection efforts properly support this task (i.e. ADT -capable classification counts at most intersections; 12+ hours).

This includes the base year, build year and future volumes. Larger projects may also require the creation of short-term future year (10 year) data. The analyst should contact the air/noise specialist(s) who will be using this information before beginning this task to ensure the correct information is provided. There may be some differences in data requirements from project to project, depending on the needs of the user of the data.

The methodology for this task uses the balanced hourly volumes as the basis to develop the average daily traffic (ADT), LOS C volumes and VMT (vehicle-miles travelled). This volume data, in addition to truck classifications, speeds, and segment lengths, is needed for the base, build and future years to compute the environmental traffic data. The LOS C volumes (representational of the noise hour which is the maximum number of vehicles moving at the maximum speed) are only needed for noise analysis. Work with the noise consultant to confirm years and data results requirements. In congested areas, differing levels of air quality data might be needed depending on the specific pollutant and environmental study type reporting needs. Energy (burden) data may be needed on large projects of regional significance (environmental impact statements) which involves vehicles miles traveled.

The task deliverables include the environmental traffic data for the base, build and future years delivered to the air/noise/energy consultant(s). Diagrams are also required as the identification/location key for the data.

- **Task 8 –Traffic Analysis Narrative/Final Technical Memorandum**
  The purpose of this task is to prepare the written documentation of the project analysis. This will assist the project team in making the preferred alternative selection. For large or complex projects a full narrative report is required. For smaller projects such as single intersection safety or operational projects can be documented with a final technical memorandum.

  The narrative/memo should draw from, summarize, and discuss information from all of the previous technical memoranda and any other analyses into a stand-alone document. Draft and final versions will need to be produced. In addition, the final narrative/memo needs to have an engineer's stamp as it is a document of record for the traffic analysis. The narrative/memo will document all of the selected and dismissed alternatives and may or may not make any recommendations.

2.5.3 Typical Task Times

Every project is different so actual task times can be quite different depending on project complexity, actual staff time available or overall project schedule (i.e. fast-tracked) to name some of the factors involved. Timelines should be defined by number of weeks required to complete each task. Target completion dates for each task should be established but should allow for overall analyst workload. For example, the timeline may show three weeks to complete the task if 100% of time was available, but if only 25% of time is available then the target date should be 12 weeks out. The final task timelines are negotiated with the project leader depending on the resources involved. Actual task timeline totals will depend on the overall size of the project, how many different years are analyzed, and number of alternatives/options to be analyzed.

Here are some general guidelines for typical studies:

Project Understanding/Scoping

- Allow at least two weeks to develop work plan, identify issues, constraints and impacts to analysis in review of existing/current plans and projects.

Counts/Inventory

- Allow six to eight weeks for counts to be completed and processed
- Allow at least two weeks if crash data are to be requested from the Crash Analysis and Reporting Unit
- Count timing – can add many months if the data collection window is passed

Volume Development

- Allow a month to create the existing 30th highest hour volumes
- Time to update a travel demand model if needed (6 -12 months or more)
- Allow at least three weeks for model application requests to be completed (activity-based model requests are more complex so these need at least four weeks)
- Allow a month to create the future no-build volumes (including all interim years)
- Allow about two weeks to create volumes for each alternative

Analysis

- Allow at least four weeks to perform and report on alternative screening analysis. This will depend on number of alternatives and how many levels of screening to do (i.e. travel demand model relative comparisons followed by key intersection v/c's).
- Allow two weeks for analysis of each build alternative

Simulation

- Allow one to six weeks for simulation model construction/error checking (depends on software)
- Simulation calibration – SimTraffic (two weeks to a month); Vissim (one month +)?
- Allow one week per alternative for simulation results

Environmental Traffic Data

- Allow one to two weeks to setup link diagrams, spreadsheets, etc.
- For smaller projects allow one to two weeks per existing, no-build future and alternatives (three to six weeks total)
- For larger projects allow four weeks per existing, no-build future and alternatives (12 weeks total).
- Allow one to two weeks for review of data (dependent on reviewer's schedule).

Documentation/Review

- Allow three weeks to write and review tech memos
- Allow a week for a reviewer to review the draft tech memos/ narrative (dependent on reviewer's schedule).
- Allow four to six weeks to write and review draft narrative; two to three weeks for final publication
- Allow one to two weeks for completed deliverable review and comment

Appendix 2A – Problem Statement Worksheet

3 TRANSPORTATION SYSTEM INVENTORY

3.1 Purpose

Before any analysis can begin, data for the study area must be collected from the field or other available in-office sources. This chapter provides guidance in the selection criteria and collection methods of appropriate inventory data types for use in transportation analysis. Inventory data are the foundation that all other later decisions are based on, so it is important that the scope of the data collection is appropriate and adequate to support the needs/outcomes of the plan or project.

3.2 Office Data Resources

There is a wide range of data sources that can be obtained prior to field investigation. Gathering this information gives the analyst a "feel" of the study area and what level of data are available, what may need to be verified versus gathered.

3.2.1 Crash Data

Crash data can come from a variety of sources and are useful for identifying problem areas of the highway experiencing an above-average frequency of crashes or reoccurring crash patterns. The analysis procedures that use crash data are described in following chapters, while the data are described below.

Sources of Crash Data

The following describes sources of crash related data and information. Tools available for the analyst to use in crash analysis are discussed in Chapter 4.

Oregon Motor Vehicle Traffic Crash Database

ODOT's Crash Analysis and Reporting (CAR) Unit provides the official motor vehicle crash data through database creation, maintenance and quality assurance, information and reports and limited database access. Crash data since 1985 is maintained at all times. Vehicle crashes include those coded for city streets, county roads and state highways. The CAR Unit website offers a variety of publications containing information on monthly and annual crash summaries.

Although there are other sources of information, such as police departments, local groups that collect information, and anecdotal information, the ODOT CAR Unit data are the standard source. Oftentimes these other sources include reporting calls, not the actual investigation/report, and groups often include near-misses and/or don't have all the facts, so the validity of reports from other sources may be questionable. The CAR data has the checks and balances built into the data collection that matches "both sides of the reports" for a crash, treating all areas the same so comparisons can be made. This is also the database used to calculate all the comparison rates published in the crash rate books.

The CAR Unit obtains documentation of reported crashes from DMV and other sources which they use to code the crash database. It should be noted that not all crashes are reportable, not all those that should be reported are reported, self-reporting of crashes has limitations on data accuracy, and even those reported by law enforcement may have issues with accuracy. The documentation provided to the CAR Unit can range from being very thorough to very limited and can include conflicting or incorrect information. The documentation is carefully checked and may be corrected before being coded. Crash codes are defined in the ODOT Traffic Crash Analysis and Code manual, available on the TransData Crash Data webpage. For detailed information about the coding process contact the CAR Unit.

Depending on the level of analysis needed, there are various reports that can be obtained. It should be noted that even though this database has the most current fully quality controlled (QC'd) data available, data for a given year may not be available until months later.

Detailed information for individual crashes can also be obtained by contacting the CAR Unit and specifying the segment of highway (roadway) and time period of interest. Because crashes are reported by specific points, an issue may be located just outside of the analysis area. Therefore, the area of requested crash data should always be greater than the area of analysis. The additional distance should be enough to catch the character of the road such as the adjacent blocks in an urban setting and a quarter to half mile in a rural setting.

Crash data is always available from the CAR unit and should be obtained through them for complex, controversial or legal situations since they are the official reporting source. For most analysis efforts, using the on-line reports is sufficient. Crash data are available on both state highways and local roads on the ODOT internal Crash Reports website or the external Crash Statistics & Reports website.

Crash data are requested by state highway number and milepoint for specific years. Local road crashes are available at the above websites by clicking on the Local Roads tab.

Crash information is categorized first by county, then city and finally by street name. Selecting the summary detail will bring up the entire length of the roadway. The summary is only useful when the entire roadway is within the study area. If only a portion of the street is desired, the data needs to be obtained from the comprehensive report (PRC). The analyst will need to review the report and select the appropriate crashes to summarize for use in the analysis.

While several years of data may be available for any given roadway segment, it is common practice to analyze only the most recent, complete three to five (3-5) years of data as factors such as traffic volumes, environmental conditions and roadway characteristics may change with time. Three years of data gives a minimal picture and is the minimum analysis period for a safety analysis, while the five-year dataset is preferred. Roadways with a small number of crashes should be looked at through a five-year period to get a better representative sample. In some circumstances even a longer period may be reviewed. Also, remember that the crash databases are regularly updated to include more recent data, so care should be taken to select the correct timeframe(s). For more information on pulling crash data, refer to Appendix 4A.

Crash Data Viewer (CDV)

The Crash Data Viewer allows users to view, filter and query the last 10 years of crash data in a spatial (i.e. GIS application) environment. The application has numerous filters so different crash attributes can be easily visualized.

Oregon Transportation Safety Data Explorer (OTSDE)

The Oregon Transportation Safety Data Explorer (OTSDE), has the five most recent fully quality controlled years of crash data in a GIS-type application. Filters can be used to view data for specific needs. Reporting allows the user to draw an area on the map and download data from that specific area in one data pull.

TransGIS Mapping Tool

TransGIS is a web mapping tool designed for users of every skill level. It presents many data levels in an interactive map format in multi-level views. This mapping tool provides information on many types of safety, volume and crash data on a state map. The user can choose which information layers are displayed and can zoom in on the map to examine a location in detail as well as display city and county boundaries along with other data. Extensive work with individual layers may be required in the generic TransGIS application, to obtain all necessary information about a study segment.

The ODOT GIS Unit may be able to produce custom maps or applications in some cases, depending on the nature of the request and work priorities. GIS maps or web applications may be possible, as well as additions to TransGIS. A web application is a custom TransGIS website with pre-defined layers. GIS software is not required. The user can zoom in/out and turn layers on/off as desired. The application may be permanent or temporary, as needed for the duration of the project. Examples are located on the Maps and GIS webpage under the GIS Mapping Applications dropdown.

All requests for mapping products are submitted to the ODOT GIS Unit (GISU) by ODOT staff. Consultants may initiate a request through ODOT staff. Requests should be clearly defined prior to submittal to avoid re-work. Contact the GIS Unit for further information. Custom mapping requests need to use the GIS Project Request form which is available by contacting the GISU.

Crash (Collision) Diagrams

Crash diagrams are typically used to evaluate operational and safety projects, not analytical ones. The typical project and planning analysis would not require this level of detail. The CAR Unit can create a crash diagram that depicts the crashes on a given roadway section. Depending on the size of area requested and the number of crashes in the requested area, this can be time consuming. The ODOT Regions also have access to Crash Magic software to create collision diagrams, contact the CAR unit to get access to the tool (internal ODOT access only). Region 1 has a user's guide that includes the required data disclaimer that should be included on any collision diagrams produced by the Regions. For more information, refer to the ODOT Safety

Investigation Manual available on the ODOT Highway Safety Manual webpage.

Crash Rate Tables

Crash Rate Tables have been published annually by the CAR Unit since 1948. Tables in the front of the book list statewide crash rates for several categories of the State Highway System. More tables list the crash rates for selected sections of each state highway, as well as a rural/urban break out. Additional tables list intersection crash data and fatal crash data. These tables are discussed further in Chapter 4 (Safety) and are available on the Crash Statistics and Reports webpage.

3.2.2 Roadway Data

ODOT maintains a wide variety of roadway characteristic and features data available on the ODOT State Highway Reports. TransGIS includes numerous roadway, roadside, and traffic attributes.

These sites are also available on the Transportation Data Portal under the Data & Maps section on the ODOT Internet site.

These data include and are not limited to:

- Roadway alignment – horizontal and vertical (grades)
- Roadway cross-sectional data – lane, shoulder and median widths
- Roadway features – pavement type, number of lanes, speed limits
- Roadway details – structures, connections, mile points and equations
- Segment Vehicle Classifications and Characteristics
- Summarized Traffic Information

Other data sources are also available through the above links. They include:

- Digital Video log
- Traffic Count Information
- Functional Classifications
- Highway Classifications including freight routes, scenic byways
- Operations and Performance Standards including spacing and mobility
- Design Standards
- ODOT generated maps

Additional Data that should be located includes:

Transit Information

The following information is likely needed from the local transit authority

- Transit vehicle characteristics
- Route specifics such as paths, schedules, dwell time, headway
- Passenger boarding/alighting and occupancy data
- Transit specific control and timings

Traffic Signal Timing Sheets

Traffic signal timing sheets are needed when creating analysis files. The best source of current information for state highways is through the various Region Traffic units. Occasionally, the local jurisdiction will be responsible for the signal timing, so they are the source of that data along with any local signalized intersections of interest.

Traffic Plan Sheets

An analyst should obtain related information about the roadway and traffic control for the intersections of interest. Specific details such as the detector layouts and striping sheets may be available for locations from Region Traffic units.

Other useful information

Information such as zoning and local classification maps needs to come from local jurisdictions. Private entities may have aerial photos also available. Although on-line aerial and ground-level mapping data sources such as Google and Bing may yield useful (scalable) information, they can also contain information that is not very current and may not be consistent in years between overhead and street/driver views. Street names often differ from the official names. Information from these sources needs to be verified.

3.2.3 Reports and Tools

Transportation Planning On-line Database (TPOD)
This is a map-based, graphical tool for locating planning studies within a specific area. This may include but is not limited to Transportation System Plans (TSP), refinement plans, Interchange Area Management Plans (IAMP) and others that may have factors that significantly impact the study. For example, a TSP may include alternative mobility standards, local jurisdiction's operational standards, fiscally constrained project list and many more. Because this tool is only updated periodically, the most recent plans may not be included in the collection. Contact the region planning manager to verify.

To use: click on "Search" and draw box. Plans within that area will be listed, with links to them. Click on "Navigate" to go back to moving the map without highlighting to show plans.

Features Attributes and Conditions – Statewide Transportation Improvement Program (FACS-STIP)
This is an ODOT Asset Management database in both listing and map-based forms. Information includes data such as bike/ped facilities, ADA accommodations, access locations, and traffic features similar to what is available in TransGIS. This is an evolving database that is being continually expanded as data are collected.

In addition to gathering data from TPOD and other on-line databases, the analyst should determine / locate other studies that may have bearing within the study area, including previous (related) studies, Safety Investigations, and Traffic Impact Studies. The analyst should contact local jurisdictions, the Region Traffic Manager, the Region Access Management Engineer (RAME) and the Region Planning Manager to locate any of this work.

3.3 <u>Field Inventory</u>

There is no substitute for a field visit as an analyst cannot get a good feel for the project area otherwise. Specific data related to field conditions that may affect traffic safety and operations shall be collected directly during a visit to the area. In addition, inventory data collected through other sources such as previously conducted studies, databases maintained by the road authority, or aerial photos should be field verified. Notes, photographs and/or video should be taken of the project area in addition to the inventory data to reference and possibly included as graphical elements in the final report. The most common types of field data needed are discussed below.

3.3.1 Geometric Data

Geometric data are the physical characteristics of the facility, typically including:

- Street names
- Segment lengths
- Lane/shoulder/median widths
- Lane configurations
- Storage bay and ramp taper lengths
- Acceleration and deceleration lane lengths and tapers
- Storage bay lengths (from stop-bar to start of taper)
- Bike/multi-use facility locations and dimensions
- Parking locations and dimensions
- Raised medians/pedestrian refuges/islands locations and dimensions
- Location of stop/yield bar, intersection guidelines (dashes)
- Pedestrian facilities (type, condition, location) including crosswalk dimensions (length and width)
- Intersection and access spacing
- Access type, location, width and land use served

3.3.2 Operational Data

Operational data describes the characteristics of traffic control and flow, and typically includes:

- Speed limits
- Saturation flow, speed and travel time studies
- Intersection controls (signalized, stop-controlled, yield, merge, etc.)
- Signal characteristics (timed, actuated, split-phased, protected left turns, etc.)
- Type of pedestrian signal head (countdown or not)
- Signing (especially turn prohibitions)
- Parking signing, striping, and maneuver frequency
- Pedestrian crossings including at crosswalks and improper (jay-walking);

frequency of use
- Transit stop locations and amenities
- Rail crossing locations, train frequency and duration of blockages
- Intersection sight distance

3.3.3 Field Observation Data

Observations related to travel patterns and driver behavior. This can include:

- Perceived or actual operational problems
- Length and duration of queues
- Driver behavior for lane choice, turn paths, upstream and downstream movements or maneuvers, left turn "sneaker" (left turn drivers that turn enter and depart the intersection during the yellow or red intervals)
- Pedestrian, Bicycle or other user behavior including actual travel paths
- Evidence of crashes such as glass on the shoulder, tire marks on curbs or medians
- Tire wear on pavement
- Damaged or worn physical features

3.3.4 Simulation-Specific Data

In addition to the geometric and operational field data, additional simulation specific data are needed if a project requires simulation. Simulation-specific data typically may include:

- Number of detectors, length, and distances from stop bar or crosswalk
- Turning speeds and radii (if unusual geometrics or conditions exist)
- Free-Flow speeds on ramps, highway segments, or on intersection discharge legs approaches (can be collected using private-sourced data, road tubes or speed guns)
- Existing travel times and average speeds
- Important travel patterns (OD Data), Example: ~75% of traffic exiting the mall makes left at Main Street
- Lane Use
    - Details of user specific lanes (e.g. HOV, truck/bike/bus)
    - Use of the shoulder or median to move around blockage points or due to driver confusion/disregard
    - Lane-change positioning lengths or landmarks where vehicles tend to position themselves in a lane to make an upcoming turn or merge
    - Freeway/arterial guide sign locations (for lane change distances)
    - Noticeable lane imbalance issues (some lanes used more heavily than others) and causes (i.e. lane drops, multiple turn lanes, closely spaced intersections/accesses, etc.)
    - If more than one lane is available to turn into from an approach, indicate the lane that turning vehicles align into (left or right alignment) regardless if the movement is legally proper or not.
- List the approximate average and maximum queues for each lane
- If upstream intersections or bays are blocked by congestion from the observed

intersection, what percentage of the site visit hour did blocking occur <5%, 10%, 25%, 50%?
- Arrival type for each approach (i.e. are the vehicles arriving in a platoon, if so, are they arriving on red or green?)

3.3.5 Safety-Specific Data

Additional data beyond the general geometric, operational, and observation data may be needed to support Highway Safety Manual techniques (See Section 4.4.4). Safety-specific data may include:

- Fixed object density and average distance offset from travel lanes
- Intersection skew angle
- Presence/absence of centerline rumble strips, lighting, red-light cameras, and automatic speed enforcement
- Left turn phasing: permissive or protected.
- Number of schools, bus stops and alcohol-selling establishments within 1000’ of an intersection.

3.3.6 Field Data Collection Requirements

For studies not needing simulation, field inventory data are recommended to help with the analysis work.

For projects requiring simulation, the simulation specific field data and counts MUST be collected at a time representing the analysis time period (the 30th highest hour) as closely as possible. The simulation calibration data should be obtained by the analyst conducting/ overseeing the simulation work. This effort should occur during the vehicle count collection if feasible. See Chapter 15 for guidance and instruction on calibrating simulations.

With some project areas, it is impractical to obtain all counts and inventory on a single day representing the 30th highest hour. When this happens, the counts and field inventory at the primary locations should be on days that closely represent the 30th highest hour. If it is not possible, then short sample counts should occur during the field inventory collection to factor the off-peak counts to the day the study area was visited. Factor the counts for seasonal influences (described in Chapter 5). If this is the case and simulation is required for the study, use the seasonal factor methodology to determine if the vehicle count day is representative of the 30th highest hour.

If the study’s primary counts occurred at a time that is more than 10% different than the 30th highest hour period (considering seasonal trend type) then short duration sample counts need to occur during the field collection time to calibrate the “existing conditions” model. The counts need to be at critical locations (high volumes, bottlenecks, unusual conditions) to assist with representing the traffic flows. These rules are established to help ensure that calibration volumes 1) are near the 30th highest hour and 2) represent conditions that have been witnessed in the field.

3.3.7 Field Inventory Worksheet

The Field Inventory Worksheet has been designed by the Transportation Planning Analysis Unit (TPAU) to be generic enough to aid in the collection of field data for all studies. The [Field Inventory Worksheet](), Exhibit 3-1, assists the field data collection process. The worksheet is an example of how to organize/document the information. It shows much of the suggested information, but it should be customized according the study's needs. The worksheet can be used for projects where just geometry and observational data are required or for projects requiring simulation where all the data listed above should be addressed. Exhibit 3-1 shows a completed worksheet for a simulation project. Note that the worksheet may be printed multiple times for a given project area. The collection of worksheets can be placed in a three-ring binder providing a hard writing surface. A worksheet can be used for each intersection or area of interest in the study and all copies can be neatly organized in a single project binder.

Exhibit 3-1 Field Inventory Worksheet - Intended Setup

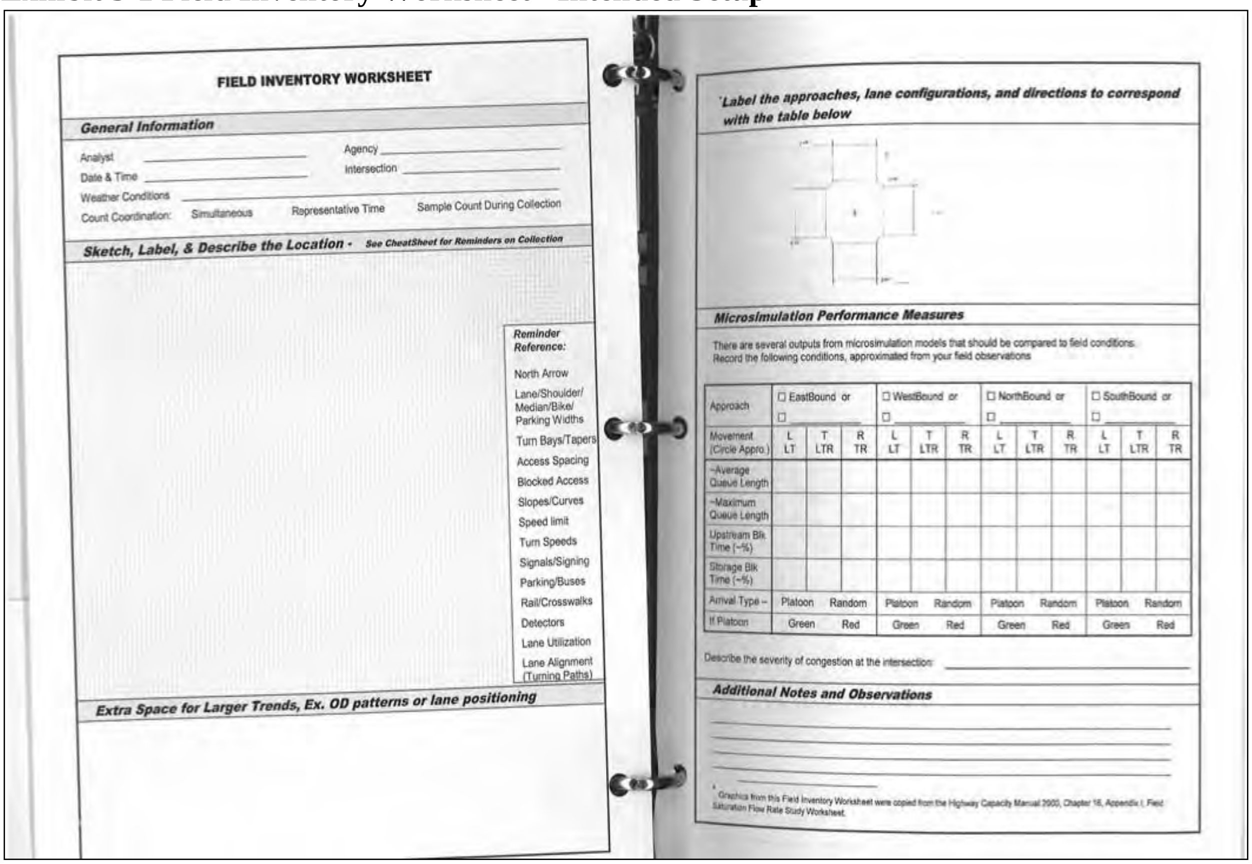
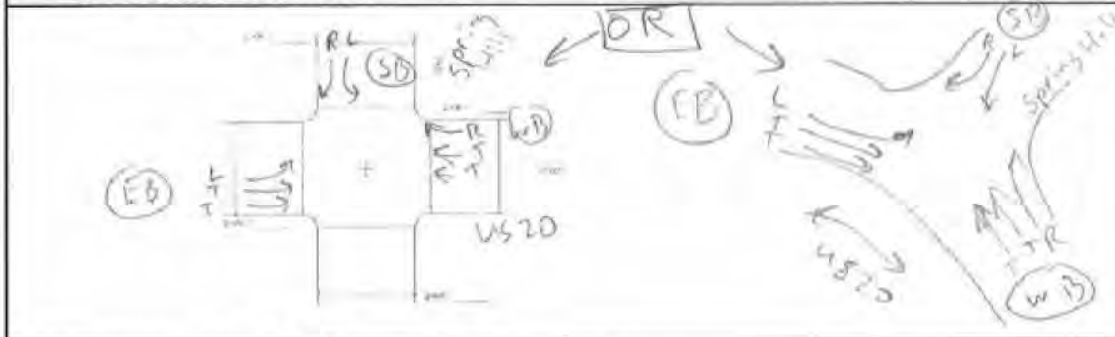


Exhibit 3-2 Completed Example Field Inventory Worksheet

***Label the approaches, lane configurations, and directions to correspond with the table below**



Microsimulation Performance Measures

There are several outputs from microsimulation models that should be compared to field conditions. Record the following conditions, approximated from your field observations

Approach	☑ EastBound or ☐			☑ WestBound or ☐			☐ NorthBound or ☐			☑ SouthBound or ☐		
Movement (Circle Appro.)	L LT	T LTR	R TR	L LT	T LTR	R TR	L LT	T LTR	R TR	L LT	T LTR	R TR
~Average Queue Length	50	150			200	50				200		50
~Maximum Queue Length	150	400			Back to Bridge	400				300		300
Upstream Blk Time (~%)												
Storage Blk Time (~%)					5%					25%		<5%
Arrival Type –	Platoon		Random	Platoon		Random	Platoon		Random	Platoon		Random
If Platoon	Green		Red	Green		Red	Green		Red	Green		Red

Describe the severity of congestion at the intersection: Little to no congestion most of the hour, except pk 5 min

Additional Notes and Observations

This Collection seemed to represent a peak hour/season and commuter driving characteristics. This data should be applicable to counts taken on days representing the 30th highest hour for this area.

* Graphics from this Field Inventory Worksheet were copied from the Highway Capacity Manual 2000, Chapter 16, Appendix I, Field Saturation Flow Rate Study Worksheet.

3.4 Vehicle Count Surveys

Disruptive events such as the 2020 COVID-19 pandemic can cause major changes in traffic patterns for extended periods of time. Under these conditions taking new traffic counts for the project will often not be advised and state and local traffic count programs will likely have been suspended. Refer to Appendix 3E for guidance on traffic counts in planning and project development when disruptive conditions are present.

The data collected from vehicle count surveys is used in nearly all types of analysis procedures, and can include information regarding volumes of vehicles, types of vehicles, vehicle speeds and directions of vehicle flow. When such information is needed, the analyst must determine the appropriate time and method of data collection to obtain the desired results.

How many counts to take, of what type, depends on the context of the plan or project goals and objectives. For outsourced projects and plans, a draft scope/work plan with a completed objective section is critical for efficient use of time, money and data for all involved parties. The level of count detail required will be dictated by the level of detail in the plan or project. For example, Transportation System Plans (TSP) will be less detailed than a TSP Refinement Plan.

3.4.1 Vehicle Count Types and Durations

Intersection Classification Counts

Intersection classification counts provide vital information for project development. They provide peak hourly volumes (PHV), Average Daily Traffic (ADT) and vehicle classifications such as cars, pickups, buses and trucks for each approach and movement. Additionally, the K-factor (percent of ADT in the peak hour) and the D-factor (percent of traffic in a single direction) can be derived from the intersection count data. These are then used to convert PHV to ADT. For further explanation of traffic volume characteristics, refer to the HCM – Part I: Overview.

Intersection classification counts are typically 16 to 24-hours in duration, so average daily traffic (ADT) and other relationships can be created. These counts are used at signalized intersections, intersections that may become signalized, and other important major intersections, such as interchange ramp terminals. A 16-hour count is needed when requirements exist such as multiple peak periods, truck classifications, signal warrants, pavement design, air quality and/or noise studies in environmental documents. Sixteen–hour counts can also be easily used for other purposes such as pavement design (see Chapter 6) or other plans or projects. The average cost of an ODOT-performed 16-hour Full Federal Manual Classification Count is approximately $800(2021 costs), depending on the specific ODOT region. This cost is dependent on the complexity of the intersection, whether or not it is a high-volume intersection, and other special requirements including travel.

Intersection classification counts group 13 different types of vehicles, pedestrians and bicycles. Refer to the FHWA vehicle classification descriptions in Chapter 6 and in the Environmental

Traffic Data Chapter 16. Classification counts can either be done manually in the field or by use of video cameras. ODOT typically uses video cameras as it does not require the presence of a field technician throughout the duration of the count, may have less influence on driver behavior in some situations, allows for more flexibility in scheduling and processing counts, and provides a database that can be easily revisited if more information is desired at a later time or if an error in the count is detected. Data are recorded in the field and then sent to the Transportation Systems Monitoring (TSM) Unit for processing.

Passenger and other two-axle vehicles are tabulated both with and without trailers. The number of axles for single-unit trucks and for all single, double and triple trailer trucks is recorded along with buses and motorcycles.

An identification number will be given to each count so that it can be accessed easily. Counts are sent to the requestor either electronically as a spreadsheet file or hardcopy by mail. The first page of the ODOT intersection count provides a sketch of the intersection counted, the date, location, count number and the ADT for each movement. The second page provides a summary of movements broken down into 1-hour increments. Some intersection counts will break the peak periods into 15-minute intervals instead of 1-hour intervals. The rest of the pages show individual turning movements with the vehicle classifications, a summary of the bicycle and pedestrian counts (if originally requested) and a summary of the movement volumes. Sample Count Request and Sample ODOT Counts have been included in Appendix 3B.

Peak Period Counts

Peak period counts capture the individual vehicle movements at a location. These counts are typically used to capture the in/out turning movements at driveway accesses or to count all movements at minor or unsignalized intersections that are not being considered for signalization. Generally, separate truck percentages are not available. Use of turning movement counts are limited to counting in a single peak period. Typical peak periods are morning (6:00 AM – 9:00 AM), mid-day (11:00 AM – 1:00 PM), and evening (3:00 PM – 6:00 PM). A three-hour count is a typical duration to capture the peak hour. A four-hour afternoon peak period count can be obtained to capture both school and commuter peaks. Truck peak hours could be any hour outside of the typical commuter peaks so are unlikely to be captured by a peak period count. For count durations of more than four hours or when more than one peak period is needed, it is more practical to collect a 16-hour count. Count durations less than three hours make it difficult to capture the peak hour and should be avoided unless previous counts clearly identify the system peak hour in which case shorter duration counts may be acceptable. Typical ODOT count costs are variable depending on travel, duration, and other specifics, but are in the $300 (2021 costs) range, depending on the specific ODOT region.

> *Peak period counts should not be used to create daily traffic volumes (ADT) in most cases. Exceptions would be volumes to support HSM analyses and preliminary signal warrants where sufficient 12+ hour counts are available to calculate K-factors to be applied to the peak period counts. Air/noise traffic data production must use longer duration counts. See Chapter 5 for more information.*

Road Tube Counts

> *Axle hit counts (volume only road tube counts) obtained from the Oregon Traffic Monitoring System (OTMS) program must be axle factored before use. For each volume only count, OTMS identifies a representative classification count to use for the axle factors.*

Road tube counts are often employed when the details provided by intersection counts are not needed, impractical given the data needs; or when certain additional data are needed that would best be collected by tubes, such as roadway speed (Section 3.5.2) or available gaps in traffic. These count axle hits only, or can be setup to capture vehicle classifications. These counts are used to capture mid-block volumes on streets and for segment volumes on most highways and interchange ramps. Road tubes are subject to vandalism or damage, and should not be done where vehicles may stop on the tube (in congested areas or near intersections) or cross the tube at an angle (near intersections or driveways) because under or over-counting may occur. Tubes are also susceptible to being damaged on roadways with speeds at or above 40 mph, and for employee safety reasons, cannot be placed on high-volume expressways and freeways. It is also not recommended to use road tubes during winter months (November 1st – April 1st), due to the use of studded tires, which have been shown to destroy hose tubes, even at slower city speeds. Road tube counts are typically done in a 48-hour format so an entire 24-hour period can be obtained.

A 7-day count can also be done if daily fluctuations over a week need to be captured and are only done on roadway segments. Typical ODOT road tube count costs are around \$300 (2021 costs), depending on the specific ODOT region.

3.4.2 Other Sources of Traffic Data Information

Frequently, existing or alternative count sources are overlooked so these should be reviewed before completing the initial count list. This can, in some cases, substantially reduce the number of new counts, save on data collection costs, and cut down the number of SOW review iterations. When searching for older traffic counts, generally the counts should only be a few years old (3-5). The longer period may be acceptable when there is little to no change in volumes or when developing a preliminary analysis. When counts are older than three years, growth rates may not reflect the growth when much change occurs. The further the count is being interpolated, the more likely for error to be introduced. See Chapter 6 for information on forecasting.

Transportation System Monitoring (TSM) Unit Data

- **Previously Collected Counts**

  Besides obtaining new counts there are some other sources of count information which may be used to reduce the overall new count requirement needs. ODOT has a large quantity of traffic volume data and previously collected counts. Before any new counts are ordered, the Transportation System Monitoring (TSM) Unit should be contacted to determine if any previous usable counts are available for the study area.

In general, counts in the study area should be three years old or less. Older counts between three and five years old can sometimes be used if they are the correct type and no significant changes, such as new roads or developments, have occurred to influence traffic flows.
A count may not accurately represent the traffic flows on a roadway section even if less than three years old if recent development has occurred within or near the study area since the count was taken.

Prior traffic counts can be accessed through the Oregon Traffic Monitoring System (OTMS) (https://ordot.ms2soft.com/tdms.ui/OneLogin/Home?target=%2Ftcds). During high data pulls during the year, reports may be switched to a link emailed to the user. There are three ways to obtain access:

- Any ODOT employee can have access by going to OTMS and clicking on the initial "ADFS" blue button for the single sign-on access. This will give the user access to the Traffic Count Database System (TCDS) module. If the user wants access to other modules, the user will need to contact TSM.
- Consultants/non-ODOT users requiring access will need to contact TSM supplying them with their email address and a password.
- Public access to obtain prior traffic counts can be found using the OTMS public link (https://ordot.public.ms2soft.com/). Reports and data views are limited compared to the login access. Other prior traffic counts may be obtained by vendors that will sell traffic data from a compiled database.

OTMS is unable to accept estimated values as previously created for Automatic Traffic Recorder (ATR). If MADT = MAWDT, then ATR month was estimated.

- **Transportation Volume Table (TVT) and Highway Performance Monitoring System (HPMS) Count Sites**

The 48-hour tube counts used for the development of the TVT and HPMS sample sites are available from TSM. These counts are collected in 15-minute intervals at a minimum. Some TVT and all HPMS counts have vehicle classification information as well. (Note: Volumes listed in the TVT are for a single point, not the entire segment).

- **State Highway Vehicle Classification Data**

State Highway vehicle classification information is available through the TSM Unit's Internet Traffic Volumes and Vehicle Classification webpage. Exhibit 3-3 shows an example excerpt of the State Highway Vehicle Classification Report. This report is also the source of information for highway segments AADT. With this information, the daily and hourly volumes can be obtained along with truck classifications which will substantially reduce the need for 48-hour road tube counts. Vehicle classes 4 through 13 are considered trucks for most applications, including Pavement Design and HCM analysis. However, class 3 vehicles may be considered light trucks in some software.

Note that volumes listed in the State Highway Vehicle Classification report are for segments, as shown in Exhibit 3-3. A beginning mile point is shown for the starting point of each segment. Volumes listed in the Transportation Volume Tables are for individual points, as shown in Exhibit 3-4. The mile point shown is where the traffic count was taken.

Exhibit 3-3 Excerpt from State Highway Vehicle Classification Report

Traffic Volumes and Vehicle Classification

Effective Date 12/31/2011 — *Highway #: 033 CORVALLIS-NEWPORT Hwy*

Beginning Mile Point	Roadway	Mileage Type	Overlap Code	Dir Code	AADT Volume	AADT 20 YR Volume	ATR Count	Design Hour Factor	Ton Mileage Factor
0.00	1			E	14900	0	N	11.0	6.4

	Class 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
Vehicle Class Percentage	0.66	50.35	43.04	0.73	1.90	0.75	0.21	0.27	1.12	0.82	0.02	0.04	0.09
Volume	98	7502	6413	109	283	112	31	40	167	122	3	6	13

Beginning Mile Point	Roadway	Mileage Type	Overlap Code	Dir Code	AADT Volume	AADT 20 YR Volume	ATR Count	Design Hour Factor	Ton Mileage Factor
0.05	1			E	13000	0	N	11.0	6.4

	Class 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
Vehicle Class Percentage	0.66	50.35	43.04	0.73	1.90	0.75	0.21	0.27	1.12	0.82	0.02	0.04	0.09
Volume	86	6546	5595	95	247	98	27	35	146	107	3	5	12

Exhibit 3-4 Excerpt from Transportation Volume Tables

CORVALLIS-NEWPORT HIGHWAY NO. 33

Milepoint indicates distance from Oregon Coast Highway (US101), in Newport

On Olive Street

0.04	*15400*	0.04 mile east of Oregon Coast Highway (US101)
0.07	*13400*	0.02 mile east of Avery Street
0.30	*16000*	0.02 mile west of Eads Street
0.34	*14000*	0.02 mile east of Eads Street

- **Automatic Traffic Recorders/Automatic Vehicle Classifiers (ATR/AVC) Sites**

ATR and AVC sites record bidirectional volumes on an ongoing basis and can be used as substitutes for classification and regular road tube counts. ATR sites only include bidirectional volumes, but to understand vehicle classifications, every ATR site is also counted with a 24-hour classification count every three years which is available from the

TSM Unit. AVCs continually classify data so classification data will be available throughout a given year at these locations. AVCs are gradually replacing ATRs so eventually all recorder sites will have classification abilities. ATR/AVC “Critical Hour” listings are also available which breakdown a year’s worth of data down to the hour level so a 30 HV can be easily obtained at that location.

- **Ramp Volume Diagrams**

While 16-hour counts at an interchange ramp terminal are preferable, the ramp volume diagrams in the Transportation Volume Tables and on the TSM Unit webpage can be used to substitute if a count is not available and intersection turn movements or intersection operations are not desired. Many of the interchange ramp volumes have 48-hour tube counts that were used to create these volumes, so an analyst should check for their availability. These counts are taken on a 3-year schedule. Free-flow ramp volumes (i.e. between two Interstate highways) can be obtained from the diagrams if a 48-hour tube count is not available or practical. The TVT ramp volumes are balanced and may not represent the actual count volumes. Contact the TSM Unit directly if the actual count is needed.

Other Jurisdiction’s Counting Programs
In addition, some counties and larger cities may have traffic counting programs in place. The TSM Unit webpage also has links to many of these jurisdiction’s Internet traffic data pages on the Traffic Counting Program webpage.

These counts are typically daily volumes and can be used to supplement the local system and can reduce the need for 48-hour road tube counts. Sometimes intersection counts are available, but differing classification breakdowns and durations from ODOT standards can make these difficult to use except for a source for local peak period counts.

Traffic Signal Controller Counts
The new Model-2070 and earlier Model-170 traffic signal controllers can store loop or video detection information that can be downloaded at a later date. These data are attractive to the end user as there are a large number of usable installations available. Controller count data are primarily used by Region signal timers, when preparing field refinements to signal timing plans. For traffic analysis, controller counts are useful in determining trends between weekday and weekend traffic, or in establishing relationships for side streets (i.e. seasonal adjustments). It is imperative to have a copy of the loop detector diagram for the specific intersection, when deciphering both Model-2070 and Model-170 controller count data. The Model-2070 controller stores the data in up to 32 columns or bins, depending on the complexity of the intersection, with two loops per detector phase (standard). However, field modifications may differ from the “as-built” plans, which is why a current detector diagram is necessary.
The most recent detector diagrams can be obtained from the Region Tech Center.

Special Vehicle Counts (short duration)
Frequently on projects, there is a need to collect additional peak hour data for driveways, for a check count, or other overlooked spot. Sometimes these counts are done for specific purposes such as capturing headways, weaving movements, or saturation flow rates for simulation

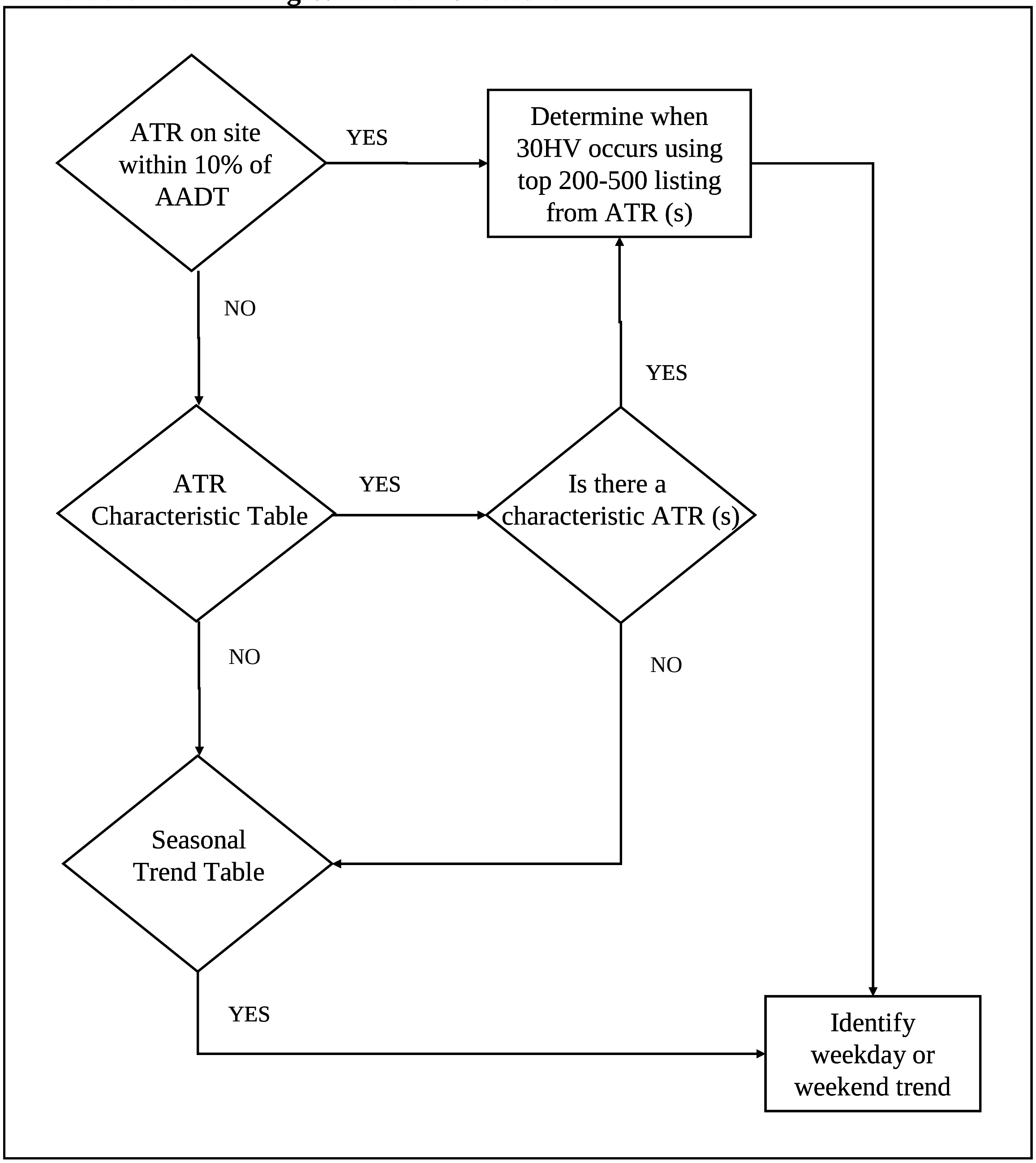
calibration. These counts typically are collected by the project analyst rather than region or consultant counting staff. Counts longer than an hour or in many locations should be done by region staff, TSM staff or traffic count contractors.

These counts can be manually tabulated in case of several small adjacent driveway counts or may be streamlined with use of a video camera or handheld electronic count board. Video cameras can be useful assuming that a good vantage point is available that will provide a clear view of all movements being counted. When using video cameras to collect count surveys, be sure to have an adequately charged battery and large enough media to collect the amount of data needed.

3.4.3 Vehicle Count Periods

For most traffic studies, the 30th highest hour volumes (30 HV) should be used to represent future volumes. It is recommended a top 200- to 500-hour count listing (Critical Hour) of the ATR(s) is obtained from the Transportation Systems Monitoring Unit. The 30 HV at the ATR(s) will be included in the list so that it will be possible to determine when the 30 HV occurs during the day and in the week. Manual counts can then be timed for the period when the 30 HV will likely occur, minimizing seasonal adjustments. Exhibit 3-5 illustrates the general process for identifying when the 30 HV occurs. Refer to Chapter 5 for detailed guidance on each of these steps.

Exhibit 3-5 Determining When 30 HV Occurs



To get a typical traffic mix of the 30 HV for the analysis, the counts should be taken as close to the likely 30th highest hour as possible. This typically requires collecting counts on a weekday afternoon in most larger urban areas, but may include weekends for high recreation areas (the coast or Central Oregon), or areas experiencing lunch hour peaks or high reverse direction flows during the day. Where capturing school trips is important, counts need to be taken when school is in session. In some cases two sets of counts may be needed, during months when school is in session as well as during the summer. In fully developed portions of Metropolitan Planning Organization (MPO) areas, the 30th highest hour is generally assumed to be represented by the typical weekday evening commuter peak hour. Outside of fully developed MPO areas, a seasonal adjustment will be required to convert the counts to 30 HV. For access management, the peak hour is defined as the highest one-hour volume during a typical or average week in urban areas, and the 30th highest hourly volume on rural roadways.

Seasonal adjustments should not be more than 30% because the traffic flow characteristics are most likely NOT represented by the count information. A seasonal adjustment greater than 30% indicates that the count was taken at the wrong time of year. Turn movement patterns may be so different they cannot be adequately represented by a seasonal adjustment. Count timing is critical especially if the project/plan SOW will not be complete until after October. Please refer to Existing Volume Development (Chapter 5) or contact the Transportation Planning Analysis Unit (TPAU) for advice.

Counting Considerations to Minimize Seasonal Adjustments

- Coastal or summer recreational areas should be counted during the traditional summer period (Memorial Day to Labor Day). Outside of coastal/recreational areas, most areas can be counted from March to October. Larger MPO areas or commuter-based corridors can be counted most months, but should generally avoid December to February as these are the lowest traveled months, have several holidays, and have the most weather-related problems. Winter recreation areas (i.e. Mt. Hood area) should be counted in the December to February timeframe to capture the peak periods. Recreational areas (or routes that travel to or between recreational areas) may require counting on the weekends.
- If alternate periods/volume thresholds other than the typical summer/ or 30 HV like what would be used to create an alternate mobility standard, then counts need to be taken in those alternate periods so resulting seasonal factors do not exceed 30%. This might mean for certain efforts, multiple sets of count data may be required. For example, a summer recreational area that uses a non-summer standard, counts should be obtained in the non-summer (Oct-Apr) period. For areas that use an annual average, counts should be obtained in Sep-Oct and/or Apr-May periods. Winter or winter-summer recreational areas will be a variation/combination of the methods.
- Road tube count placement is limited to the April to October period because of studded tire damage potential.
- In general, days potentially influenced by state or federal holidays or other significant events (such as local festivals, sporting events, hunting season, etc.) that may alter normal traffic patterns should be avoided.
- Counts that may be influenced by nearby construction projects may be affected and such counts should be thoroughly investigated and may require adjustments.
- It is also common to avoid Monday and Friday counts when weekday data are desired, as the trip characteristics on these days generally differ from the remainder of the week.
- Consideration should be given to the presence of generators such as schools/colleges/universities because the enrollment and events (such as spring break) can vary greatly through the year. Consideration should also be given to major employers or attractions such as regional shopping centers that experience significant peaks in generated trips that may or may not occur during the other peaks because of shift changes or event scheduling. Note: The City of Corvallis requires that counts occur when OSU is in session because of the large percentage of the population related to

school being in session.

- In agricultural areas, truck traffic may be highly seasonal and have a substantial impact on the system. Counts may have to be timed carefully to balance the overall peak months with the harvest periods.
- In the Portland Metro area, while infrequent, there may be times when additional data must be collected to capture the 2nd hour needed to evaluate the adopted 2-hour OHP mobility target. This is generally only necessary when the mobility target for the second hour of the peak period is lower than the mobility target for the first hour of the peak period and the analysis shows the first hour does not meet its target but satisfies the target for the second hour.

Counting Considerations for Congested Conditions

Counting under congested conditions (i.e. Portland Metro area) requires some additional considerations as there may be places that the actual demand is queued up and unable to flow smoothly through the system. Volumes are being reflected on the ground while demand is being reflected in the form of unserved queues (cannot pass through in a single signal cycle over a 15-minute period at least). In this case, a typical count would only measure the discharge flow through the intersection versus what would like to use the intersection. This could lead to an underestimation of demand, especially under a build alternative condition. Both volume and demand should be quantified in the analysis to help inform the analyst on volume development steps and following analyses.

If there is a suspicion that counts may not reflect true demand...

Step 1: Check existing tube counts or automatic traffic recorders for any peak hour “M” effect where the shoulder hours on each side of the peak hour/period are higher than the peak hour/period or the shoulder hours share the same capacity as the peak hour. If this is the case, demand has likely exceeded capacity.
Step 2: Check existing manual turning movement counts to see if the 15-minute shoulder intervals of the estimated peak hour/period are higher than the peak hour/intervals.
Step 3: If easily available, check private sector vehicle probe data such as RITIS to see if the corridor experiences speeds lower than the posted speed for more than one hour.
Step 4: Do a field visit or have video taken to verify whether vehicles can be served before ordering counts.
Step 5: Consider collecting counts upstream from the desired count location where the roadway is not oversaturated if congestion is observed in the field, video, and/or by counts. When counting upstream, consider counting side streets between the unsaturated count location and the desire count location that may have traffic leave or enter the roadway.

Other considerations:

- If traffic is heavily favoring a single lane more than others, lane by lane utilization counts or field observations are needed. For example, this can easily occur in one lane of a dual left turn lane because of an immediate lane drop or heavy turn movement downstream. Lane utilization counts should also be considered where a through lane is reduced downstream from a traffic signal.

- Locations need to be noted where queue spillback occurs from turn lanes blocking through lanes or vice versa. This may require additional counts upstream or field observations.

3.4.4 Vehicle Count Locations

Vehicle count locations should be identified in the project work plan/SOW, and should be determined based on the needs of the subject plan or project. For example, planning efforts that are expected to generate potential highway projects within three years will require more detailed counts than a standalone or long-range plans, such as TSPs.

For planning projects it is important to correspond with the local jurisdiction and TPAU/Region Traffic to make sure that count needs cover the system to be analyzed at the appropriate level of detail and address issues. The grant/project manager should meet with TPAU/Region Traffic staff to discuss traffic count requirements after the objective section of the SOW (or project prospectus) is completed as this section provides the context for the plan/project. For Transportation Growth Management (TGM) grants, it is usually more efficient to arrange a meeting with the appropriate TPAU/region staff to go over multiple studies at once. For construction projects, the project team or at least the region traffic engineer/manager, the environmental lead, and/or project leader should be consulted.

While differences of opinion may exist on the number and type of counts versus the available budget, remember that the ultimate goal will be to have enough data to analyze and answer the questions, address the needs, evaluate alternatives, and cover the level of detail in the plan/project as described by the project objectives and the local jurisdiction(s). Staff will need to come to an agreement whether the data collection budget and/or the number/type of counts need to change.

The following plan/project-specific count location guidelines do not cover every possibility or combination of elements but are intended to help generate a reasonable starting point for discussion. The location guidelines are generally laid out at an increasing level of detail.

County Transportation System Plan (TSP)

The arterial and major collector system needs to be documented (counted). It is generally unnecessary to count lower functional class roads as these usually carry very little traffic and possibly are unpaved unless the county government wants a specific roadway included because of operational issues. Analysis at the County level is more system-based with a higher emphasis on ADT rather than peak hour and many of the analysis tools require ADT as an input.

- Need to have at least ADT-level count coverage of the arterial and major collectors. Acceptable previously taken counts may exist at the state or local level.
- Major arterial intersections with other arterial and major collector intersections should be counted where operational issues exist. State highway segments (between major intersections) should use the TSM Unit’s vehicle classification data to capture volumes and truck classifications.
- The TSM Unit’s ramp volume diagrams (where available) should be used to capture any free-flow ramp connections.

- County arterials and major collectors should have at least a 48-hour classification tube count performed so truck traffic can be captured, and ADT can be calculated.
- Peak period counts should be obtained at signalized intersections, unsignalized highway to highway junctions, and county arterial – highway intersections. If this is a TSP Update, refer to the old TSP to help identify the critical intersections that should be counted.

City Transportation System Plan (TSP)

The arterial and collector system needs to be documented (counted). It is generally unnecessary to count lower functional classes unless the roadway is area-significant, provides an alternate path for trips to bypass congested areas (as in a parallel local street), or the local government has previously identified operational issues. Analysis at the City level is more centered on the peak periods and individual facilities/intersections which require more detail.

- Need to have at least ADT-level count coverage of the arterial and collectors. Acceptable previously taken counts may exist at the state or local level.
- Major cities (> 50,000 pop,) generally need to have at least the arterial system counted.
- Medium cities (10,000 – 49,999 pop.) generally need to have the arterial and representative/significant collectors counted.
- Small cities (<10,000 pop.) generally need to have the arterial and significant collectors counted.
- Major arterial intersections with other arterial and significant collector intersections should be counted. Peak period counts should be obtained at minor arterial/collector signalized intersections, unsignalized highway to highway junctions, city arterial – highway intersections, and major private development accesses (i.e. regional shopping mall). If this is a TSP Update, refer to the old TSP to help identify the critical intersections that may need to be re-evaluated.
- Significant collectors extend across the city for a considerable distance, are a direct route, or extend outside the city.
- If multiple signals exist, it may not be necessary to have a count at every one, but a reasonable representation of the system needs to be counted.
- Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- Sixteen-hour counts should be obtained at interchange ramp terminals and signalized major arterial intersections. If tube classification counts are available to provide ADT and truck volumes on each leg, then shorter duration counts can be used. The TSM Unit's ramp volume diagrams (where available) should be used to capture any free-flow ramp connections.
- State highway segments (between major intersections) should use the TSM Unit's vehicle classification data to capture volumes and truck classifications.
- City arterials and collectors should have at least a 48-hour tube count performed so ADT can be calculated. Larger cities may already have this count data.
- If detailed refinement plans and/or actual highway projects are expected out of the TSP within three years and plan to use the TSP data, then the counted major intersections should be 16-hour counts with the lesser unsignalized intersections or access points using peak period counts.

Interchange Area Management Plan (IAMP)

The roadway system needs to be counted within at least a half-mile radius of the interchange. Analysis at the IAMP level can be close to a project-level of detail (see project section) depending on whether it is standalone or not. If the IAMP is part of a project, then the IAMP should be using the project counts and volumes and no new counts should be necessary unless the counts are very old (greater than three to five years old) or development patterns in the area have changed. It may be necessary to obtain a few "check counts" to see if volumes are substantially different before replacing all or most of the counts. If the IAMP is a standalone plan but it is anticipated that a project may occur within three years, then the IAMP needs a project-level count request. If the IAMP is a standalone plan but no project is anticipated within three years:

- Major arterial intersections with other arterial and major collector/collector intersections should be counted.
- Sixteen-hour counts should be obtained at the ramp terminal intersections, other arterial/arterial intersections, or unsignalized intersections that may need to be signalized.
- Peak period counts should be obtained at other existing signalized and unsignalized intersections and unsignalized intersections.
- 48-hour road tube counts may be necessary to support HSM safety analyses.
- If multiple signals exist, it is unnecessary to have 16-hour counts at every one. Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- Most, if not all, driveway accesses should be counted with peak period counts as many of these will be rerouted to new connections.
- State highway segments (between major intersections) should use the TSM Unit's vehicle classification data to capture volumes and truck classifications.
- The TSM Unit's ramp volume diagrams (where available) should be used to capture any free-flow ramp connections.

Refinement, Management or Facility Plans

The arterial and collector system needs to be counted within the defined study area limits. It is generally not necessary to count lower functional classes unless the roadway is area-significant, provides an alternate path for trips to bypass congested areas (as in a parallel local street), or the local government has previously identified operational issues. If it is anticipated that a project may occur within three years, then a project-level count request is needed. If no project is anticipated within three years:

- Major arterial intersections with other arterial and major collector/collector intersections should be counted.
- Facilities parallel to the subject arterial should be counted.
- Longer roadway sections without intersections should use road tube counts.
- Sixteen-hour counts should be obtained at signalized intersections and major unsignalized intersections (i.e., ramp terminals, four-way stops) to capture truck traffic or where larger scale improvements may be needed.
- Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- Unsignalized intersections or major accesses should be counted with peak period counts.

- 48-hour road tube counts may be necessary to support HSM safety analyses.
- If an Interstate Highway or statewide expressway exists in the study area, the mainline shall be counted by direction between interchanges in addition to any interchange ramp terminals. Road tube counts may be necessary to capture movements on ramps or connections.

Local Street Network (LSN) or Downtown Plan

These kinds of plans are generally trying to identify new roadway or multimodal connections to control congestion on the state highway or make limited improvements in the downtown area. The arterial and collector systems need to be counted. It is generally not necessary to count lower functional classes unless the roadway is the only access to a neighborhood, provides an alternate path for trips to bypass congested areas (as in a parallel local street), or the local government has previously identified operational issues. Larger numbers of peak period counts may be necessary with a few 16-hour counts at major intersections.

- Major arterial intersections with other arterial and collector intersections should be counted.
- Sixteen-hour counts should be obtained at signalized intersections and major unsignalized intersections (i.e., ramp terminals, four-way stops) to capture truck traffic or where larger scale improvements may be needed.
- If multiple signals exist, it is unnecessary to have 16-hour counts at each one. Each major roadway should have truck traffic captured on it in at least one location.
- Unsignalized intersections or major accesses should be counted with peak period counts.
- 48-hour road tube counts may be necessary to support HSM safety analyses.
- Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- State highway segments (between major intersections) should use the TSM Unit's vehicle classification data to capture volumes and truck classifications.
- The TSM Unit's ramp volume diagrams should be used to capture any free-flow ramp connections.

Pedestrian or Trail Plans

Generally, counts are only needed if the state highway system will be affected by removing or narrowing through travel lanes or if new crossings are to be added. Count requirements in the lane reduction areas should follow the LSN/Downtown Plan recommendations above.

Plans with proposed mid-block trail crossings of state highways or local arterials should have a 48-hour classification road tube count performed at the crossing location. For plans with existing pedestrian crossings (formally defined or not) where the number of crossing pedestrians is desired, the crossing count should be replaced with a 16-hour video classification count with bike and pedestrians requested.

Pedestrian and/or bicycle counts are more adversely affected by weather conditions than vehicle counts, and are recommended to be taken when pedestrians and/or bikes are anticipated, such as not in the winter in many cases, or during school-in-session periods if near a school/college/university.

Traffic Impact Studies (TIS)
For TISs, the analysis area and study intersections are typically selected from estimates of anticipated impacts from added traffic based on site trip generation and distribution, and existing intersection operations. Count requests need to be developed with the guidance of the Region Access Management Engineer or appropriate region staff and the Development Review Guidelines.

- Sixteen-hour counts should be obtained at major unsignalized intersections (i.e., ramp terminals, four-way stops) to capture truck traffic; obtain the basis for signal warrants, or where larger scale improvements may be needed.
- Signalized intersections may use a 16-hour count or a peak period count depending on the particular study area.
- If multiple signals exist, it may not be necessary to have 16-hour counts at each one. Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- Unsignalized intersections and accesses should be counted with peak period counts.
- 48-hour road tube counts may be necessary to support HSM safety analyses.
- The Interstate/expressway/highway mainline shall be counted by direction in addition to any interchange ramp terminals. Road tube counts may be necessary to capture movements on ramps or connections.

Construction Projects
For most other project types (modernization, safety, operations, etc) the analysis area and study intersections are selected by considering the problem that is being addressed by the project and the information that is required to fully assess the problem and propose appropriate solutions. Project analysis is needed to support roadway and intersection control improvements, pavement and bridge design, air quality, and noise mitigation. Larger projects, especially those with required environmental studies (such as noise and air quality) may require multiple full 16-hour classification counts.

- Sixteen-hour classification counts should be obtained at signalized intersections and major unsignalized intersections (i.e., ramp terminals, four-way stops) to capture truck traffic or where larger scale improvements may be needed.
- Truck classification data must be captured on each roadway segment in the study area.
- Minor unsignalized intersections and accesses should be counted with peak period counts.
- 48-hour road tube counts may be necessary to support HSM safety analyses.
- Significant driveway accesses should be counted as many of these will be rerouted to new connections.
- If an Interstate Highway or grade-separated highway exists in the study area, the mainline must be counted by direction between interchanges in addition to any interchange ramp terminals. Road tube counts may be necessary to capture movements on ramps or connections.

3.4.5 ODOT Internal Count Request Process

The TSM unit has required traffic count requests to be submitted using TSM's Planner Traffic Count Request Template. Instructions for completing the request are included in the template. A sample completed request is available in Appendix 3B. As shown in the template, the name of the contact person (requestor), the person to whom the data will be sent, the locations, time periods, dates, types of counts and collection methods must be clearly communicated to those conducting the counts. Count requests should group different count types (classification, peak period, road tube) separately for clarity. The count request should also list any special requests, count intervals, count time windows (start and finish dates), and a charge number (Expenditure Account (EA) for internal counts). Examples of a special request would include counting only on a specific day or counting certain intersections or elements of intersections at the same time. Pedestrians and bike counts also need to be specifically requested. For further information on the count request template contact the TSM Unit.

The template provides space to include a map showing the count locations, durations and other special requirements.

When ordering intersection counts, be sure to specify the duration and type for each location. Fifteen-minute intervals must be specified for at least the standard morning, noon and evening peak periods in 16-hour counts. Peak period counts should be done in 15-minute intervals. It is not required, but very helpful if 48-hour road tube counts are counted in 15-minute intervals as well.

Specify the latest acceptable date by which the count is needed for analysis. Keep in mind, based on scheduling and staff limitations, that it can take at least five weeks from the date of the request date to get the count scheduled (not including weather restrictions) and then another three to four weeks to have the count processed, recorded and distributed. Therefore, counts need to be requested about nine weeks ahead (or more if weather is a factor) of when they will be needed for the analysis work.

All count requests should have copies sent to both the Region Traffic Manager and to the TSM unit to alert them that they are requested and need to be scheduled. Either TSM or the region staff will have the counts performed (in-house or with contractors) and should assure that the data are processed into an ODOT format before being released to the requestor. The TSM Unit needs know what counts are being requested so staff resources can be allocated. The TSM Unit coordinates the counting schedules of all Region traffic counting staff. Coordinating with the TSM Unit in the loop allows for count requests to be added to the count databases, which can avoid unnecessary duplication and limit counting needs by others and minimize delays. ADT – capable counts performed by third parties or consultants are also encouraged to be submitted to the TSM Unit to be added to the database.

3.4.6 Using the Oregon Traffic Monitoring System (OTMS) Program

The Oregon Traffic Monitoring System (OTMS) is an online program maintained by the contractor," MS2", with help from the TSM Unit and available for access by ODOT staff, consultants and the public. It contains recent counts (2008 forward) conducted throughout the state of Oregon. Counts performed by the TSM Unit and counts provided to the TSM Unit are entered into the program. It is important to contact the TSM Unit (also Region Traffic as some counts that are done never make it to TSM) if you are unable to find counts in your study area or to determine if there are existing counts that have not been processed.

There are many different count types and each requires OTMS to generate a different report. The Oregon Traffic Monitoring System (OTMS) Program Count Report Guide, included in Appendix 3C, provides step-by-step instructions on how to obtain traffic count information from OTMS in the correct format for the most common types of counts used by the analyst. This includes intersection counts, ATR/AVC sites, truck summary, and tube (machine) counts.

3.4.7 Count Validation

Once counts are completed and processed and are available to the analyst, the counts should be checked to make sure that everything is furnished as requested. This includes count days, time periods, 15-minute intervals, movements, and classification requirements. Missing data, intersection approaches, etc. should be reported back to the TSM Unit and the appropriate region for recounting (if possible) or reprocessing (in case of a video count) as soon as possible. Do not wait until all of the counts are completed to perform these checks.

Check the counts for any “red flags.” Do the values look okay? Counts have had approaches mislabeled, or wrong orientations (i.e. flipped east to west but also can be west to north).

As counts are being assembled for volume development, additional issues may arise between adjacent counts. Compare these counts with previous counts historical/seasonal growth factored in; they should be similar (within 10%) if nothing has changed in the field between the counts. If adjacent or a whole section of counts appear to be very low or high, then verify that no incidents (road closures on the subject or adjacent roadways, crashes, bad weather, or scheduled events nearby) occurred while the count was in progress. If the count is 18 months or older then crash data records can be checked. Recent counts may require more investigation (contact TSM Unit, Region traffic units, local maintenance districts, or traffic operation centers).

3.5 Travel Time, Speed and Other Data Collection

3.5.1 Travel Time

Travel time surveys measure the duration of time taken for a vehicle to travel from one point to another along a designated route, and are often used to quantify congestion over a corridor. The data collected from travel time surveys works well with statistical analysis, and the results are often more easily understood by the public than other methods used for measuring congestion. Probe data sources such as RITIS are the first source for obtaining current or historical travel times and speeds. RITIS has data going back to 2016 for all public roadways in Oregon and Calrk County, Washington.

Data Collection

One common method used for this data collection uses a "floating car." The elapsed time is measured from a car driven along the designated route maintaining an average travel speed relative to other cars on the road. Other methods include vehicle or license plate matching and the use of various intelligent transportation system technologies. Travel time data are collected at the beginning and end of a designated route, and can be collected between predetermined points along the route as well, depending on the level of information desired. While travel time can easily obtained from probe data sites such as RITIS, the floating car techniques may be still necessary if the desired locations do not have coverage such as local and low-volume roadways.

When collecting travel time data, all measurements should be taken under good weather conditions and during a time representative of the period of interest for the study. Except when collecting the data for simulation calibration, it is good to distribute the travel time runs over several days and over multiple weeks that are representative. To have an accurate representation of the field conditions for simulation calibration, travel times need to be taken at the same time as the operational field data collection. The floating car technique requires the driver to mimic or match the speed of the traffic stream for a given roadway. However, it may be difficult for test drivers to mirror the actions of the traffic stream as drivers often revert to their own driving habits instead of staying with the majority of traffic.

It is recommended that a minimum of 10 travel time runs be collected in each direction for each hour to be simulated (and each lane where lane imbalances occur) for both freeways and arterials. The 10 travel time runs should be collected during the same time as other data collection if possible but can be collected over multiple days if necessary especially if the route is long and all repetitions cannot be completed within the analysis period. The travel time runs can also be a combination of the floating car and Bluetooth data methods. See Chapter 15 for more information regarding microsimulation.

Floating Car Data Collection with GPS

Floating car travel time runs are conducted using a handheld GPS device that records vehicle location, speed, and direction of travel every 1 to 5 seconds. This method allows the actual roadway conditions to be analyzed as the data returned from the probe vehicles will reflect the periods of congestion and free-flow speeds experienced by other motorists. When designing a floating car study (regardless if the data capture is manual or by GPS), the goal is to have a large enough sample which is optimally spaced for the purpose of capturing variability within the traffic stream. Some things to consider:

1. Route selection
    a. Try to include as many key intersections, segments as necessary
2. The number of floating car "probes" and the area (route) covered per run
    a. In general, increasing the number of probes sampled during the run and/or reducing the area size will increase the resolution of the samples.
3. Lane position of probe car

    a. Results are provided in fine enough detail to determine speeds based on lanes occupied. It may be difficult for the driver to determine which lane most closely represents the average vehicle speed.
4. Weather, Tree cover and %trucks along route.
    a. High instances of either will lead to signal disruption and loss of data points.
5. Start the routes at least 15 minutes ahead of recording period
    a. Establishing the route prior to the recording time minimizes driver route errors
6. Driver Notes are helpful (see Exhibit 3-5 Determining When 30 HV Occurs).
    a. Record queue position at intersections
    b. Lane position

Exhibit 3-6 Example of Notes

I hit lots of reds on OR99E. There were only a couple instances where I hit greens along the cooridor.
I was in the inside lane on OR99E and the outside lane on I5. I used the inside lane going northeastbound on OR99E to position myself in the appropriate ramp lane, but also because the curbside lane delayed at driveways (Especially at Killdeer) as people turned into the shopping center. I used the inside lane going southwestbound on OR99E to position myself for a quick turnaround at Ermine Drive and to avoid delays due to right-turning traffic. I remained in the right hand lane on I5 for all but (2) loops. The ramps are closely spaced. As long as I was following multiple vehicles, I did not feel disadvantaged by sitting in the right hand lane. During (2) loops, I was behind a semi-truck and followed other vehicles around the slow moving truck.

The queues were no more than 10 cars-
The longest queues existed for the inside lane southwestbound OR99E at Albany/Airport Road and both lanes for northeastbound OR99E at Waverly

4:30-4:45 loop - I didn't turn left at the appropriate location- this will show in the GPS
5:10-5:20 loop - stuck behind a Hay truck turning left from Century Drive to Old Salem Road at the Murder Creek Interchange
- no reds on OR99E
5:20-5:33 - Used the left lane going NB on I5 to go around a semi
5:33-5:39 - Used left lane going SB on I5 to go around a semi

Upon completion of the floating car runs, the data are downloaded to an ArcGIS shapefile format for cleaning and analysis (see Exhibit 3-7). It is typical that some data cleaning will be required such as removing outliers and converting the speed to MPH. This is easily done in ArcGIS.

Exhibit 3-7 Data format

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W
1	LATITUDE	LONGITUDE	ALTITUDE	EASTING	NORTHING	UTCDATE	UTCTIME	SOG	COG	MAG_VAR	SATS_USED	HPE	VPE	EPE	HDOP	VDOP	PDOP	QUALITY	DIFF_AGE	DIFF_ID	DEPTH	DEPTH_OFF	WATERTEMP
2	44.9486862720	-123.0151426399	25.062	-123.015	44.949	3/31/2011	221953	51.7	95.3	298.0	4	0.0	0.0	0.0	2.4	5.1	5.7	1			0.0	0.0	0.0
3	44.9486749269	-123.0149701418	25.319	-123.015	44.949	3/31/2011	221954	47.1	95.2	302.2	5	0.0	0.0	0.0	2.4	5.1	5.7	1			0.0	0.0	0.0
4	44.9486585704	-123.0146570282	25.515	-123.015	44.949	3/31/2011	221956	40.7	92.8	328.3	5	0.0	0.0	6.4	2.4	5.1	5.7	1			0.0	0.0	0.0
5	44.9486549742	-123.0145250729	25.643	-123.015	44.949	3/31/2011	221957	34.6	91.7	340.9	5	0.0	0.0	6.4	2.4	5.1	5.7	1			0.0	0.0	0.0
6	44.9486544290	-123.0143104926	25.720	-123.014	44.949	3/31/2011	221959	25.2	89.0	359.1	5	0.0	0.0	6.4	2.4	5.1	5.7	1			0.0	0.0	0.0
7	44.9486555250	-123.0142312615	25.765	-123.014	44.949	3/31/2011	222000	19.9	88.6	0.0	5	0.0	0.0	6.4	2.4	5.1	5.7	1			0.0	0.0	0.0
8	44.9486582178	-123.0141097766	25.944	-123.014	44.949	3/31/2011	222002	14.5	88.5	358.5	5	0.0	0.0	6.4	2.4	5.1	5.7	1			0.0	0.0	0.0
9	44.9486774096	-123.0140094421	31.027	-123.014	44.949	3/31/2011	222004	12.8	74.7	32.7	5	0.0	0.0	6.4	1.8	3.3	3.8	1			0.0	0.0	0.0
10	44.9487257761	-123.0139409302	31.848	-123.014	44.949	3/31/2011	222005	7.0	94.3	346.9	6	0.0	0.0	6.4	4.3	3.3	5.4	1			0.0	0.0	0.0
11	44.9486807570	-123.0139490074	31.053	-123.014	44.949	3/31/2011	222007	2.1	41.2	2.4	5	0.0	0.0	7.4	1.8	3.3	3.8	1			0.0	0.0	0.0
12	44.9487274773	-123.0139084054	31.876	-123.014	44.949	3/31/2011	222008	0.6	174.7	350.5	6	0.0	0.0	7.4	4.3	3.3	5.4	1			0.0	0.0	0.0
13	44.9486788817	-123.0139464597	30.948	-123.014	44.949	3/31/2011	222010	0.1	89.7	354.2	6	0.0	0.0	7.4	1.8	3.3	3.8	1			0.0	0.0	0.0
14	44.9486784519	-123.0139470807	30.861	-123.014	44.949	3/31/2011	222012	0.0	172.9	353.5	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0
15	44.9486785752	-123.0139470839	30.843	-123.014	44.949	3/31/2011	222013	0.0	302.1	353.5	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0
16	44.9486787955	-123.0139472568	30.763	-123.014	44.949	3/31/2011	222015	0.1	41.1	354.5	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0
17	44.9486787737	-123.0139475311	30.690	-123.014	44.949	3/31/2011	222017	0.1	221.8	353.0	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0
18	44.9486787242	-123.0139477957	30.640	-123.014	44.949	3/31/2011	222018	0.0	348.0	353.9	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0
19	44.9486786977	-123.0139478861	30.656	-123.014	44.949	3/31/2011	222020	0.1	24.6	354.4	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0
20	44.9486784582	-123.0139481406	30.636	-123.014	44.949	3/31/2011	222021	0.1	253.1	353.0	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0
21	44.9486782224	-123.0139486691	30.538	-123.014	44.949	3/31/2011	222023	0.1	187.9	353.3	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0
22	44.9486782015	-123.0139487914	30.514	-123.014	44.949	3/31/2011	222025	0.1	261.9	352.4	6	0.0	0.0	6.1	1.8	3.3	3.8	1			0.0	0.0	0.0

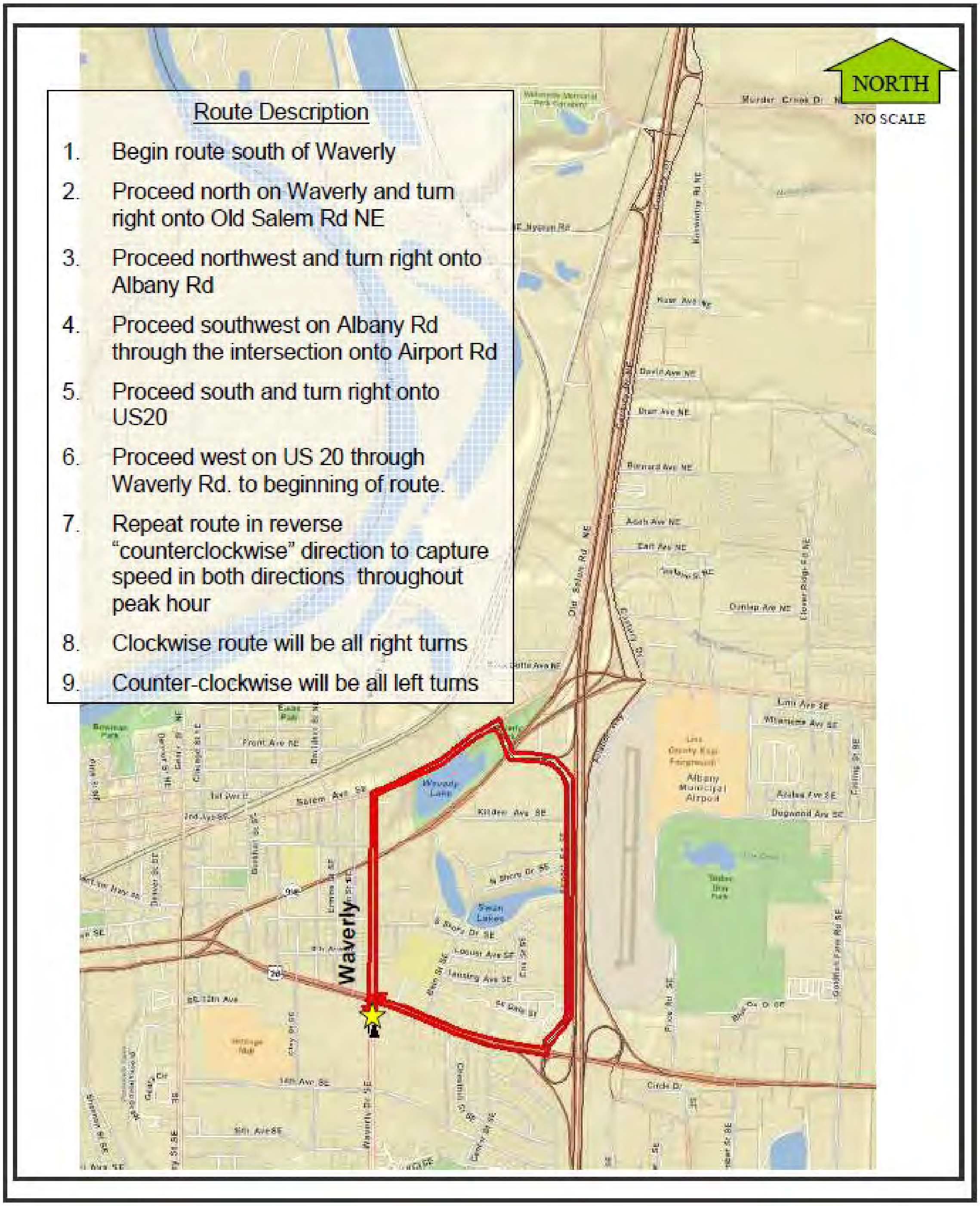
Example 3-1 Floating Car Data Collection with GPS

For a project in Albany, travel time data are needed for calibration of a simulation model for the Existing Year. See Chapter 15 for calibration procedures.

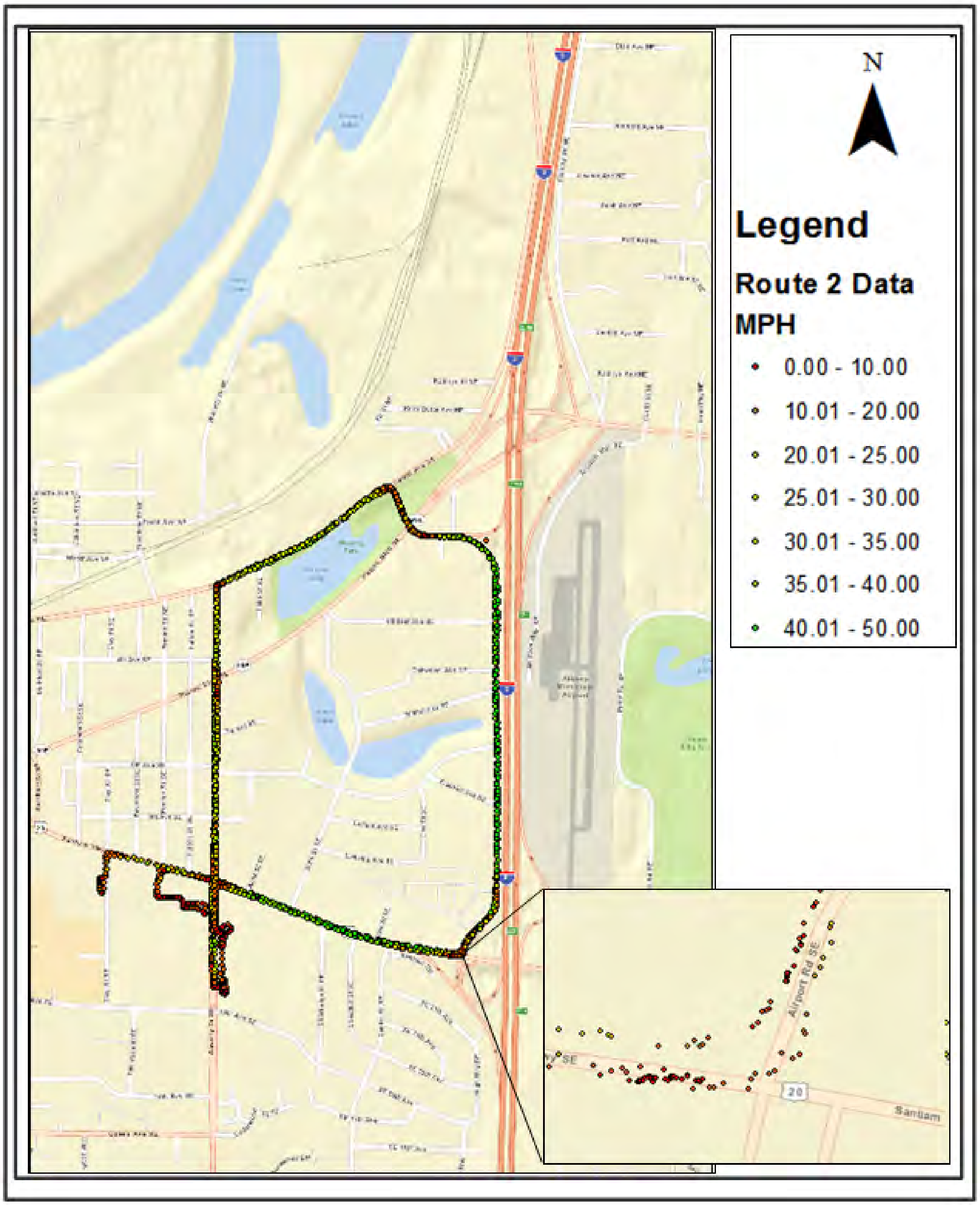
Floating car data were collected on March 31st 2010 in the peak hour between 4:45 PM and 5:45 PM. Three vehicles were equipped with GPS units and each assigned a separate travel route. In order to represent the most likely driving conditions drivers were asked to travel according to their best judgment of the traffic stream's speed and collect as many full routes as possible during the data collection period.

Average speeds along the routes were recorded in the GPS track log where time/location points set to record at 1 second intervals. A greater interval period could be used in other applications such as for a rural corridor. At each 1 second interval, the GPS recorded the coordinate location, the date and time the data point was recorded and the instantaneous speed of the vehicle. Instantaneous speed and travel times along each route were averaged over the total number of completed runs and summarized based on link segments from the SYNCHRO network. The figures below display the floating car data collection route and output results. The table below summarizes the results of the data collection.

Floating Car Data Collection, Route 2



Floating Car Data Output Results



Route 2 Floating Car and SimTraffic Data Comparison

Roadway	Section	Floating Car Data Collection		SimTraffic Calibration	
US20	BETWEEN SANTIAM INTERCHANGE RAMP TERMINALS (EB)	Speed (mph)	24	Speed (mph)	11
		Travel Time (s)	29	Travel Time (s)	59
US20	BETWEEN SANTIAM I5 NB RAMP TERMINAL / SPICER DR SE / US20 AND PRICE RD SE	Speed (mph)	39	Speed (mph)	27
		Travel Time (s)	14	Travel Time (s)	22
US20	BETWEEN PRICE RD SE AND TIMBER ST SE	Speed (mph)	46	Speed (mph)	37
		Travel Time (s)	13	Travel Time (s)	17
US20	BETWEEN TIMBER ST SE AND GOLDFISH FARM RD SE	Speed (mph)	35	Speed (mph)	32
		Travel Time (s)	28	Travel Time (s)	28
US20	BETWEEN GOLDFISH FARM RD SE AND TIMBER ST SE	Speed (mph)	45	Speed (mph)	36
		Travel Time (s)	22	Travel Time (s)	25
US20	BETWEEN TIMBER ST SE AND PRICE RD SE	Speed (mph)	46	Speed (mph)	36
		Travel Time (s)	14	Travel Time (s)	17
US20	BETWEEN PRICE RD SE AND SANTIAM I5 NB SLIP ON-RAMP	Speed (mph)	42	Speed (mph)	28
		Travel Time (s)	5	Travel Time (s)	5
US20	BETWEEN I5 NB SLIP ON-RAMP AND SANTIAM I5 NB RAMP TERMINAL / SPICER RD SE	Speed (mph)	12	Speed (mph)	10
		Travel Time (s)	27	Travel Time (s)	41
US20	BETWEEN SANTIAM INTERCHANGE RAMP TERMINALS (WB)	Speed (mph)	13	Speed (mph)	10
		Travel Time (s)	53	Travel Time (s)	69
US20	BETWEEN SANTIAM I5 SB RAMP TERMINAL AND CENTER ST SE	Speed (mph)	35	Speed (mph)	27
		Travel Time (s)	13	Travel Time (s)	20
US20	BETWEEN CENTER ST SE AND BAIN ST SE	Speed (mph)	38	Speed (mph)	40
		Travel Time (s)	19	Travel Time (s)	33
US20	BETWEEM BAIN ST SE AND WAVERLY DR SE	Speed (mph)	14	Speed (mph)	7
		Travel Time (s)	42	Travel Time (s)	69
US20	BETWEEN WAVERLY DR SE AND BAIN ST SE	Speed (mph)	32	Speed (mph)	29
		Travel Time (s)	17	Travel Time (s)	18
US20	BETWEEN BAIN ST SE AND CENTER ST SE	Speed (mph)	36	Speed (mph)	35
		Travel Time (s)	19	Travel Time (s)	20
US20	BETWEEN CENTER ST SE AND SANTIAM I5 SB SLIP ON-RAMP	Speed (mph)	19	Speed (mph)	25
		Travel Time (s)	9	Travel Time (s)	8
US20	BETWEEN SANTAIM I5 SB SLIP ON-RAMP AND AIRPORT RD SE / SANTIAM I5 SB RAMP TERMINAL	Speed (mph)	6	Speed (mph)	6
		Travel Time (s)	53	Travel Time (s)	49

3.5.2 Speed

Speed data are used in multiple areas of analysis. There are measured and calculated speeds. A measured speed is a direct "point" measurement obtained with the use of equipment such as road tubes, Radar, or Lidar. A calculated speed is derived from a combination of a time and distance information (i.e. travel time over a specific length segment).

Measured Speed

One of the easiest ways to obtain a measured speed is through the use of road tubes with vehicle classifying counter. This method can only be used for free-flow non-congested segments where tubes can be safely placed, outside the influence of any intersections causing platooning effects. Collection of tube count data is restrained by weather and installation issues and needs to be coordinated with count program staff. Another method to obtain a measured speed is through the use of hand-held speed devices such as Radar/Lidar. See the ODOT Speed Zone Manual for procedures for measuring speeds with these devices.

Measured speeds can be used to calculate segment speeds and headways. A measured speed may also be needed to determine/verify a specific segment speed such as a turning speed around a "non-standard" radius or corner for the purpose of simulation.

ODOT has a contract with Inrix probe data through the Regional Integrated Transportation Information System (RITIS) platform and has access to Inrix historical data going back to 2016 for the state of Oregon and Clark County in Washington. In rural or smaller urban areas, data coverage is generally limited to state highways (which have the most probe vehicles). Larger urban areas are more likely to include data on non-state facilities. These speeds are based on probe data (mobile, fleet including connected vehicles, in-pavement sensors, etc.) and is available in granularity as fine as one-minute and can be aggregated up to the desired analysis period. Travel time is calculated from the average speed and segment length.

Public agencies (all state agencies, local government agencies, and universities) within Oregon and Clark County in Washington can freely use the data for planning and analysis. ODOT can authorize contractors to have access to the data and conduct analysis on ODOT's behalf. Some limitations are that Inrix data cannot be merged or averaged with competitor's data (e.g.. HERE, Tom-Tom) and that Inrix must be given credit when the data are used. Contact ODOT TPAU for more information on data availability.

Calculated Speed

Calculated speeds are obtained from sources that furnish point or vehicle identification with a time stamp so data matching can yield a time between known points to determine an average speed. Data may be obtained from vehicle detection equipment, historic data or private sector sources. Common uses for calculated speeds are for calibration and validation of simulation models and creation of performance measures. Floating car travel time data can be converted into "running speed", segment or corridor travel speeds as long as distances are available.

3.5.3 Other Data Collection

Origin-Destination (O-D) Surveys
These surveys obtain route information that a vehicle takes between specific points. Project complexity can range from simple, such as a weave study, to medium focusing on select paths within a project/study area, to complex such as a city-wide network. The number of “interview” stations adds complexity to the process.

- Direct Interview - This process entails stopping a sample of vehicles to ask specific data collection such as point of beginning and ending, trip purpose, route choice, etc. Sometimes these are viewed as disruptive and invading privacy. These are very expensive (labor and traffic control) and require Oregon Transportation Commission (OTC) approval for state highways because of impacts to the traffic flows.
- License Plate Surveys – This process uses in-person or recorder images of vehicle identification (license plates) to obtain routing information without the stops associated with a direct interview process. This method can be expensive and labor intensive to match the data inputs. The more stations that are recorded, the more complex the process.
- Mail Survey – This process uses a sampling method to deliver/receive route information from driver input. This method requires a large distribution of requests to get a valid sampling of information. It can have accuracy (driver reporting), privacy and timeliness issues.
- MAC Address Reader (Bluetooth) - This process uses roadside detectors (portable or permanent) to record an identifying signature of an electronic device. Using multiple detectors, the device can be tracked to a specific route. To alleviate privacy issues, only a minimal amount of characters needed to uniquely specify a device is recorded. The electronic files of data can be matched using software programs, thus reducing labor costs.
- Probe data - Providers such as AirSage, RITIS, and Streetlight Data offer O-D tools and processed information which can be purchased by project or for a specific area and /or time period. These can speed up the path determination process through a complex set of origins and destinations like through a set of interchanges or across an urban area. It can take some time to understand and interpret what is being presented especially if other data sources and types are merged in.

Saturation Flow Rate Studies
The saturation flow rate is a critical component in signalized intersection analysis. It is defined as the flow in vehicles per hour accommodated by a lane group assuming that the green phase were displayed 100 percent of the time. Oregon saturation flow studies done to date show that the HCM 6 value of 1750 passenger cars per hour of green per lane is appropriate for small urban areas.

Except in larger urban areas, field conditions generally do not allow the HCM saturation flow study procedures in Chapter 31 of the 6 HCM to be met. A roadway approach may not have long enough queues during the study or intersection spacing may be so tight that long enough queues without gaps are not possible. In these cases, a default ideal unadjusted saturation flow is determined as follows:

- Outside of the Portland, Salem and Eugene MPO urban areas use 1750 passenger cars per hour of green per lane (pcphgl) for the unadjusted saturation flow rate.
- Inside the Portland, Salem and Eugene MPO urban growth boundaries an unadjusted saturation flow rate of 1900 pcphgl may be used, unless one or more of the following conditions is present, in which case 1750 pcphgl shall be used. Conditions indicating use of lower base saturation flow rate inside urban growth boundaries:

    - On-street parking
    - Greater than 5% of volume is trucks
    - Severe intersection skew angle (i.e., greater than 20 degrees off perpendicular)
    - One or more driveway approach(es) with a combined volume in excess of 5 vph, are present downstream of the intersection within the functional area (see Chapter 4) or upstream within the length of the standing queue
    - Poor signal spacing or observed queue spillbacks between signals during the peak hour
    - Travel lanes narrower than 12-foot in width.

The ideal (unadjusted) saturation flow rate is converted to an actual flow rate by applying adjustment factors to account for the influence of lane widths, heavy vehicles, approach grades, on-street parking, frequent bus stoppages (3-4 or more per hour per direction) in the intersection vicinity or roadway segment, area type, lane utilization, turning movements and bicycle and pedestrian conflicts. Theoretically, once adjusted, the result would be equivalent to the field measured value.

Field Measurements of Saturation Flow Rates

When an analyst desires a verification of the saturation flow (conditions are outside the default conditions above or a need for simulation), an investigation should be performed. The field measurement of the saturation flow rate shall be in accordance with methodology described in Chapter 31 of the 2010 Highway Capacity Manual (*HCM*) and submitted on the *HCM* Field Saturation Flow Rate Field Study Worksheet(s).

Data requirements:

- Measure for each signal cycle and each desired lane.
- Cycles must have a minimum of 8 vehicles are stopped in queue at the start of green to be used.
- Record values needed to calculate the average saturation flow headway per vehicle:
    - The number of vehicles in the stopped queue when the signal turns green.
    - The elapsed time elapsed between when the front axle of the fourth vehicle in queue crosses the stop line and when the front axle of the last vehicle in queue crosses the stop line.
    - Record time at end of green
- Discard cycles with events such as downstream queues blocking the flow of traffic or presence of emergency vehicles.
- A minimum of 15 signal cycles with a minimum of 8 vehicles in the stopped queue is needed to obtain a statistically significant value.

$$\text{Saturation Headway} = \frac{\text{(Time of last stopped vehicle - Time of 4th vehicle)}}{\text{(Vehicle position of last vehicle - 4)}}$$

$$\text{Saturation Flow} = \frac{3600 \text{ s/h}}{\text{saturation headway}}$$

In order to facilitate this process, a Saturation Flow Rate Data Collection Form and a Saturation Flow Rate Calculator tool have been developed. The data collection form is arranged as one row per cycle. For each cycle, the analyst records the time of the 4th vehicle, time of last stopped vehicle, number of stopped vehicles and time at end of green. Once collected, the data from the form are directly entered into the calculator office form tab. If more than 25 cycles of data have been collected, additional cycles can be added to the calculator by pressing the “Add 25 More Cycles” button.

Once the analyst has input the field data, click the “Calculate Sat Flow” button. The calculator automatically checks to ensure there are at least 15 cycles with 8 or more vehicles in queue. Cycles with less than 8 vehicles in queue are ignored. For more specifics on the tool see the Instructions tab.

Field measured saturation flow rates are preferred over estimation and do not require further modification. Using default values and adjustment factors will not produce more accurate results. If possible, saturation flow rates should be collected at no less than one major intersection on each main study area roadway. When using these values in analysis be sure to set all of the adjustment factors to 1.0.

Once the field saturated flow rate is obtained, the ideal (unadjusted) saturation flow rate should be back-calculated by applying adjustment factors to account for the influence of lane widths, heavy vehicles, approach grades, on-street parking, bus stops, area type, lane utilization, turning movements and bicycle and pedestrian conflicts. Heavy vehicles, parking maneuvers, turning movements, and bicycle and pedestrian conflicts must be collected during the same period as the field saturation flow study to be able to back- calculate an accurate value.

TPAU has a limited database of saturation flow rates. To find out if specific saturation flow rates are available for a specific site/area, contact TPAU analysts. Copies of saturation flow rate studies should be sent to TPAU for inclusion in appropriate studies such as verification of default values. Coordinate data collection and verification with TPAU so any acceptable saturation flow studies can be included in the database.

Appendix 3A – Field Inventory Worksheet

Appendix 3B – Sample Count Request

Appendix 3C – Oregon Traffic Monitoring System (OTMS) Count Report Guide

Appendix 3D – Saturation Flow Rate Data Collection Form

Appendix 3E – Project Traffic Counts during COVID-19 and Other Disruptive Events

4 SAFETY

4.1 Purpose and Overview

The purpose of this document is to provide guidance on safety analysis procedures for specific transportation planning and project development applications with a safety component. All planning and project development efforts need to be individually scoped as there are a number of different tools and techniques that can be applied. APM Section 4.1.2 identifies the recommended safety analysis procedures for common planning and project development applications.

The primary goal of any safety analysis presented in this chapter is to promote a proactive approach to reducing the frequency of fatal and serious injury (Injury-A) crashes. This is consistent with the Oregon Transportation Plan (OTP) that states “it is the policy of the State of Oregon to continually improve the safety and security of all modes and transportation facilities for system users including operators, passengers, pedestrians, recipients of goods and services, and property owners.” The Oregon Transportation Safety Action Plan implements the OTP policy.

The first edition of the Highway Safety Manual (HSM) provides the technical foundation for many of the procedures discussed in this chapter. However, this chapter does not replicate the entire guidance of the HSM, and the reader is encouraged to consult the HSM directly where appropriate. The HSM is published by the American Association of State Highway and Transportation Officials (AASHTO) with support from the Federal Highway Administration (FHWA), the Institute of Transportation Engineers (ITE), and the Transportation Research Board (TRB) Highway Safety Performance Committee (ANB25).

The HSM is a national guide—and the first of its kind—providing science-based methods, procedures, and measures that integrate quantitative estimates of crash frequency and severity into roadway planning, evaluation, and project development. Prior to the HSM, crash analysis for planning and project development was typically limited to simple evaluations of crash data and somewhat subjective analysis. Evaluations of future safety performance were primarily limited to meeting design standards, with few options for comparing alternatives. In contrast, the tools in the HSM allow safety to become a meaningful performance measure that can be implemented at any stage of the transportation decision-making process.

> *HSM methodologies are provided to assist agencies in their effort to integrate safety into their decision-making processes, but are not intended to be a substitute for the exercise of sound engineering judgment. No standard of conduct or any duty toward the public or any person shall be created or imposed by the publication and use or nonuse of the HSM. The HSM does not supersede publications such as the MUTCD, the AASHTO Green Book, or other AASHTO and agency guidelines, manuals and policies.*

As stated in the HSM, it is neither intended to be, nor does it establish, a legal standard of care for users or professionals.

HSM screening tools provide a robust methodology for objectively evaluating historical crash data based on frequency, severity, collision type, and other crash characteristics. The screening tools identify locations with the highest potential for reducing the frequency and severity of crashes and, by identifying factors contributing to the crashes, help choose effective potential countermeasures. Screening tools are discussed in APM Section 4.3.

The HSM Predictive Method is the first comprehensive model for estimating the frequency and severity of crashes based on traffic, roadway, and roadside characteristics. Predictive analysis can be applied to quantify the safety impact of design alternatives and forecast scenarios, using the understandable language of crash frequency and severity. Predictive analysis can also be applied in conjunction with historical data analysis to overcome statistical limitations inherent in historical data analysis. Predictive tools are discussed in APM Section 4.4.

The HSM framework also provides local agencies a methodology to expand on the foundation of the HSM by calibrating to local conditions and developing custom safety performance functions (SPFs). ODOT's webpage on the HSM includes information on completed and future research projects.

The APM does not address ODOT highway safety program procedures or traffic operations-level safety analysis. This includes road safety audits, collision diagrams, detailed safety investigations, and benefit-cost analyses. Contact the Traffic-Roadway Section for procedures related to those programs.

4.1.1 Statewide Crash Rate References

Statewide average crash rates are used in the critical crash rate analysis method and are useful resources for informal discussions of crash frequency.

The Oregon State Highway Crash Rate Tables are published annually by the ODOT CAR Unit. Crash Rate Table II shows statewide average crash rates for each of the last five years, by urban and rural area and by roadway classifications for state highways. These crash rates are based on overall crash frequency and total vehicle miles traveled on mainline state highways. Federal functional classifications can be found on the ODOT Federal Functional Classification (FC) webpage.

Exhibit 4-1 shows intersection crash rates by land type and traffic control, based on a 2011 assessment of data from 2003-2007. The crash rates here are based only on crashes that occurred at an intersection or because of an intersection and are given as a rate per million vehicles entering the intersection [million entering vehicles (MEV)].

Intersection crash rates also need to be compared to the published statewide 90th percentile intersection crash rates in Exhibit 4-1. Any rates close to or over the 90th percentile rates need to be flagged for further analysis. The intersection crash rate is calculated by the following formula:

$$Intersection\ Crash\ Rate\ per\ MEV = \frac{Annual\ Number\ of\ Crashes\ x\ 10^6}{(AADT)x\ (365\ days/year)}$$

The values shown in Exhibit 4-1 represent the 90th percentile crash rates from a study of 500 intersections in Oregon. The crash rates are grouped by rural/urban, signalized/unsignalized, and three-leg/four-leg intersections. Intersections with crash rates that exceed the 90th percentile values shown in the table should be flagged for further analysis. For more information on crash rates and using this table, see Section 4.3.4 Critical Crash Rate.

Exhibit 4-1: Intersection Crash Rates per MEV by Land Type and Traffic Control

	Rural				Urban			
	3SG	**3ST**	**4SG**	**4ST**	**3SG**	**3ST**	**4SG**	**4ST**
No. of Intersections	7	115	20	60	55	77	106	60
Mean Crash Rate	**0.226**	**0.196**	**0.324**	**0.434**	**0.275**	**0.131**	**0.477**	**0.198**
Median Crash Rate	0.163	0.092	0.320	0.267	0.252	0.105	0.420	0.145
Standard Deviation	0.185	0.314	0.223	0.534	0.155	0.121	0.273	0.176
Coefficient of Variation	0.819	1.602	0.688	1.230	0.564	0.924	0.572	0.889
90th Percentile Rate	**0.464**	**0.475**	**0.579**	**1.080**	**0.509**	**0.293**	**0.860**	**0.408**

Source: Assessment of Statewide Intersection Safety Performance, FHWA-OR-RD-18, Portland State University and Oregon State University, June 2011, Table 4.1, p. 47.

Note: Traffic control types include
3SG (three-leg signalized),
3ST (three-leg minor stop-control),
4SG (four-leg signalized),
4ST (four-leg minor stop-control).

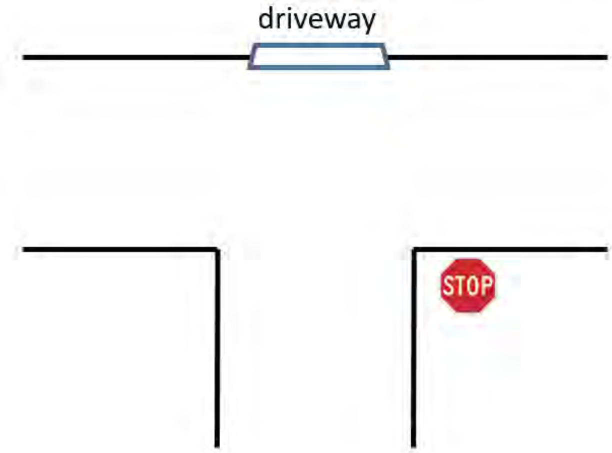
For intersections other than the configurations shown in Exhibit 4-1, there are usually too few locations with that intersection configuration to provide statewide statistics. There are some stop controlled intersection configurations that could be approximated as indicated in Exhibit 4-2 and Exhibit 4-3 below. Any other intersection configurations not in Exhibit 4-1, Exhibit 4-2, or Exhibit 4-3 should by default be flagged for further analysis, since the unusual configuration is likely to warrant a closer look at the crashes.

Exhibit 4-2: 3 Legged Stop Control, with Driveway(s) into Intersection

3 legged stop control, with driveway(s) into intersection
at what would be a fourth leg location.
Crash rate higher than the reference rate could indicate that the driveway volumes are affecting safe operation of the intersection.

If the driveway volume is low compared to the opposite minor leg, the 3ST reference crash rate could be applied.
Example:
Rogue River, E. Main at Broadway St.

If the driveway volume is high compared to the opposite minor leg, the 4ST reference crash rate could be applied.
Example:
Bend, Cooley Rd at NE Hennell Rd, Bend



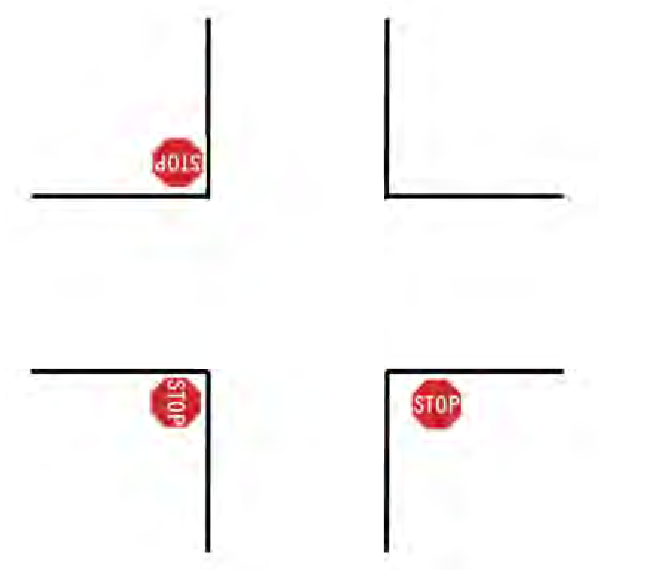
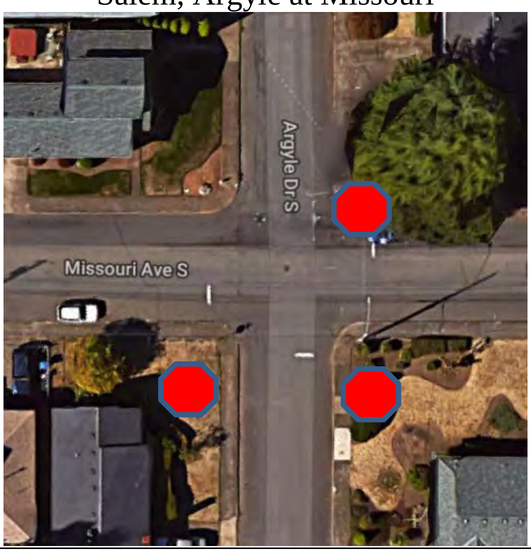
Rogue River, E. Main at Broadway St.

Bend, Cooley Rd at NE Hennell Rd

Exhibit 4-3: 4 Legged Intersection, 3 Way Stop

<table>
<tr><td>4 legged intersection 3 way stop

This configuration could apply the 4ST reference crash rate, since the minor legs have left turn conflicts similar to the 4ST.

Example:
Salem, Argyle at Missouri</td><td></td></tr>
<tr><td>Salem, Argyle at Missouri
</td><td>Street view, looking west.
</td></tr>
</table>

4.1.2 Tools and Procedures by Application Type

Safety analysis tools should be specified during the scoping process. Model scoping language is provided in APM Chapter 2. In order to facilitate scoping of tools, this section describes the recommended safety analysis tools and procedures for common application types as shown in Exhibit 4-4. Safety analysis tools are identified in order of increasing level of effort in the chart going from left to right. Plan or project level of detail is shown in increasing level of detail in the chart going from top to bottom. Best practice/recommended methods are shown with closed circles, while open circles identify optional or supplemental methods.

Exhibit 4-4: Applicability of Safety Analysis Tools by Plan or Project Type

Level of Effort +

Details +

Plan or Project Type	Other Methods			Highway Safety Manual (HSM) Methods					Other Tools
					Statistical Crash Data Screening		Predictive Methods		
	State Crash Rate	SPIS	Queue	CMFs	Critical Crash Rate	Excess Proportion of Specific Crash Types	Net Change in Predicted Crash Frequency or Predicted Crashes	Excess Expected Crash Frequency	PlanSafe
System Plans (TSP, RTP etc.)	○	●		●	●	●			○
MMA's	●	●	●	○	○	○			
Facility Plan	○	○	○	○	○	○	●	●	
Development Review	○	○	○	○	○	○	●	●	
NEPA / Project Development	○	○	○	○	○	○	●	●	

● Best practice or recommended method
○ Supplemental methods

The tools and procedures recommended here describe the crash analysis appropriate for the scope and scale of typical applications. A balanced approach to safety analysis should also consider queues, sight distances, safe and convenient crossing opportunities, and other safety-related techniques where appropriate for the application context.

- All Applications:
  - Safety Priority Index System (SPIS) – Identify top 5% or 10% locations from the most recent three (3) SPIS Site listings
- RTPs, Transportation System Plans (TSPs) and High-Level Corridor Plans:
  - Critical Crash Rate – required
  - Excess Proportion of Specific Crash Types – required
  - Crash rate comparison – minimum requirement when other methods can't be applied. Compare intersection crash rates to the 90th percentile crash rates (Exhibit 4-1) and segment crash rates to Table II in the CARS crash rates tables.
  - PLANSAFE – optional, system-wide predictive method. Recommended if there are regional transportation network changes proposed or if there is an expectation of significant demographic changes.
  - Crash Modification Factors (CMFs) – Optional. Use to estimate potential crash reduction of alternatives.

- Multimodal Mixed-Use Areas (MMAs)
  - MMAs are governed by OARs which require that public safety is not compromised. The primary safety analysis methods recommended for MMAs are the OAR requirements:
    - If the area has a crash rate above statewide averages. Crash rates are generally considered on the mainline and the crossroad.
    - If the area includes a top 10% SPIS site
    - If existing or future traffic queues will create a safety concern on the mainline highway exit
  - The new safety analysis options could provide additional value for MMA analysis.
  - The Critical Crash Rate could be a complement to crash rates as described in the OARs.
  - Excess Proportion of Specific Crash Types can be used to identify crash patterns and multimodal safety concerns.
  - The HSM Predictive Method can also be used to evaluate an MMA location and identify mitigation needs before an MMA is granted, or could be incorporated into guidelines for evaluating plan amendments within MMAs.
- Facility Plans/Refinement Plans:
  - HSM Predictive Method – required if valid model exists
    - Excess Expected Crash Frequency – use for existing conditions evaluation
    - Net Change in Predicted Crash Frequency – use for alternatives evaluation
  - If no valid model exists:
    - Use Critical Crash Rate, Excess Proportion of Specific Crash Types, and statewide crash rate comparisons for existing conditions
    - Use CMFs for alternatives evaluation
- Development Review:
  - HSM Predictive Method – recommended if valid model exists
    - Excess Expected Crash Frequency – use for existing conditions evaluation
    - Net Change in Predicted Crash Frequency – use for alternatives evaluation
  - If no valid model exists:
    - Use Critical Crash Rate, Excess Proportion of Specific Crash Types, and statewide crash rate comparisons for existing conditions
    - Use CMFs for alternatives evaluation
- Project Development / National Environmental Policy Act (NEPA) Work:
  - HSM Predictive Method – required if valid model exists
    - Excess Expected Crash Frequency – use for existing conditions evaluation
    - Net Change in Predicted Crash Frequency – use for alternatives evaluation
  - If no valid model exists:
    - Use Critical Crash Rate, Excess Proportion of Specific Crash Types, and statewide crash rate comparisons for existing conditions
    - Use CMFs for alternatives evaluation

- Countermeasure Development:
    - Using site characteristics and analysis results, identify contributing factors
    - Document Crash Modification Factor(s)
    - Perform additional geometric safety assessments, as applicable:
        - Intersection functional area
        - Sight distance
        - Conflict points
        - Access management

4.2 <u>Crash Data</u>

Crashes are used as the basis of safety analysis presented in this chapter. Crash frequency (number of crashes per year) and crash severity (rating based on most severe injury sustained in a crash) are fundamental indicators of the "safety" of a roadway.

Observed crash data provides information for describing and analyzing crash frequency and severity for historical time periods. Predictive methods proactively estimate crash frequency and severity for situations where observed crash data are not available, such as for future conditions. This chapter focuses on analysis of "objective" safety, which is based on quantitative measures that are independent of the observer. Objective safety is not directly experienced by a traveler, unless he or she is involved in a crash. In contrast, analysis of "subjective" safety involves the perception of how safe a person feels while using the transportation system. An assessment of subjective safety for the same location may vary between observers, and techniques for assessing the subjective safety of a location are not covered in this chapter. Subjective safety is an important component of many design and policy decisions. A person's choice of mode and route is strongly affected by how safe and comfortable the mode and route feels.

4.2.1 ODOT Crash Data Sources

The ODOT Crash Analysis and Reporting Unit (ODOT CAR) maintains a statewide crash record database that includes all reported crashes involving a motor vehicle on public roads. These data are collected by the Department of Motor Vehicles from police and driver reports then provided to ODOT CAR for quality assurance, standardization, and distribution. ODOT CAR produces a variety of publications annually that summarize crashes throughout the state of Oregon.

Crash data only include reported crashes to the Department of Motor Vehicles (DMV). Many crashes are not reported because they fall under the $2,500 reporting threshold or are just not reported (i.e., single vehicle incident, or on tribal lands). Errors can still occur in the coded crash records so it is important to carefully review and note any anomalies. Reporting errors on officer reports and DMV forms and officer reports such as crash location are common and while the crash reporting technicians attempt to reconcile them, sometimes data are not available or is incomplete. This is why sometimes perceived crash issues from local residents differ from available crash data. See Chapter 3 for more information.

The Crash Data System provides analysts access to detailed crash reports for a custom study area and time period. The Crash Data System can be accessed online on the Internet through ODOT's Unified Access Gateway and ODOT's Intranet site.

The Crash Data System provides tools for querying crash data by jurisdiction, location, and time. Crash data formatting and querying is different for state highways, city streets (non-state roads within city limits), and county roads (non-state roads outside city limits). Crash data can be downloaded in print-formatted or spreadsheet-formatted reports. Report download options include:

- **Summary by Year CDS150:** A general summary of crashes for the queried location, displayed by year, collision type, and generalized severity (fatal, nonfatal injury, property damage only).
- **Crash Location CDS390:** A detail report with a single line of data for each crash, including location, date, collision type, injury severity, and contributing factors.
- **Comprehensive (PRC) CDS380:** A detail report with at least three lines of data for each crash, including a row for every vehicle and participant in the crash. Summary includes location, date, collision type, injury severity, contributing factors, and more.

Data extracts are also available, providing unformatted full access to records for every crash, vehicle, and participant individually. Data extracts are available as a comma-delineated text document or as an Access database. The Access database includes code definitions and pre-defined report queries, making it a valuable resource for analysts familiar with database software.

The Comprehensive (PRC) CDS380 report or data extracts are recommended for use with the analysis procedures listed here, as these are the only formats that include the full injury severity scale (KABCO) and identify crashes involving pedestrians or bicyclists. The summary reports identify crashes involving pedestrians, but not bicyclists, and only include severity as fatal, nonfatal injury, or property damage. Summary reports may not include sufficient information for all analysis methods described in this chapter.

The crash data reports are heavily code-based and use of the Statewide Crash Data System Motor Vehicle Traffic Crash Analysis and Code Manual is required for a full understanding of the crash data. This manual and the online help documents provide additional important information about the crash data and reports and are available through the ODOT CAR Publications page or from within the Crash Data System.

Crash data are geocoded with latitude and longitude coordinate values. This allows for easy display on a map using a Geographic Information System (GIS) such as ArcMap, QGIS, or other online tools. If crash data are mapped using coordinate values, care should be taken to identify any records with the "Unlocatable_Flag" indicating that the coordinates are nonspecific default values.

Caution should be exercised when identifying actual crash locations from reported data. Crashes may be reported at the nearest integer milepoint or intersection even if they occurred hundreds of feet away. Crash data should be checked for discrepancies, such as where a crash occurred on a curve but where the reported location is a straightaway section.

4.2.2 Crash Characteristics, Trends, and Patterns

Crash analysis involves identifying trends and patterns on facilities. Analyzed crash types or severities may be all crashes or a more specific subset of crashes, such as fatal and serious injury crashes. These trends can then be used to identify applicable countermeasures for future mitigation. For guidance on performing a detailed on-site investigation and diagnosis of a safety trend, refer to the ODOT Safety Investigations Manual.

ODOT crash data includes many characteristics that can be used to identify trends and patterns. The analysis methods in this chapter primarily use the following characteristics:

- **Crash Location:** Geographical crash location is described by milepoint, distance to nearest intersection, and latitude and longitude coordinates.
- **Intersection-Related:** Crashes are identified as located at an intersection or related to the functioning of an intersection.
- **Driveway-Related:** Crashes are identified that are related to the use of a driveway.
- **Severity**: Crash severity is equal to the most serious injury sustained by anyone involved in the crash, based on the on-scene assessment (but which may not align with final medical determination of injuries). Severity is ranked on the KABCO scale:
    - K – Fatal injury, an injury that results in death
    - A – Incapacitating injury, a nonfatal injury that prevents the person from walking, driving, or doing activities they were capable of before the injury
    - B – Non-incapacitating evident injury, an injury that is evident to observers at the scene of the crash
    - C – Possible Injury, an injury or claim of an injury that is not evident to observers at the scene of the crash
    - O – No injury, also described as Property Damage Only (PDO)
- **Collision Type:** This field describes the intended movements of the vehicle(s) at the time of collision. Crashes are coded as one of the following collision types:
    - Angle – Vehicles collided while traveling on crossing or perpendicular paths, such as would occur if a vehicle ran a red light and crashed into a vehicle traveling on the crossing roadway
    - Head-On – Vehicles collided while traveling in opposite directions, their forward movement impeded while attempting to occupy a location simultaneously
    - Rear-End –Vehicles collided while traveling in the same direction, with one vehicle hitting the rear end of the second vehicle
    - Sideswipe-Meeting –Vehicles collided while traveling in opposite directions, with the side of at least one vehicle involved
    - Sideswipe-Overtaking – Vehicles collided while traveling in the same direction, with the side of at least one vehicle involved
    - Turning Movement – Collision involved one or more vehicles turning, originally traveling on parallel paths

- Parking Maneuver – Collision involved one or more vehicles entering or leaving a parked position. Parking begins when a vehicle first exits the traffic lane and ends when a vehicle resumes travel in the traffic lane.
  - Backing – Collision involving one vehicle backing in a traffic lane that struck another vehicle also in a traffic lane, does not include parking maneuvers
  - Fixed-Object or Other-Object – Collision where one vehicle struck a fixed object or other object (identified in the event field) on or off the roadway
  - Pedestrian – A collision where the first harmful event was an impact between a vehicle in traffic and a pedestrian. Does not include crashes where pedestrians are injured subsequent to the first impact, in which case pedestrians are coded as supplemental events to the crash.
  - Miscellaneous Collision – Any crash that does not fall into the other collision type categories, including collisions with animals
  - Non-Collision – Crash involved only one vehicle and is not classifiable as any other collision, for example, a roll-over
- **Bicyclist Involvement:** This information is contained in "Crash Type" field, separate from the "Collision Type" field described above. Bicyclist involvement is designated with the "Pedalcyclist" description.

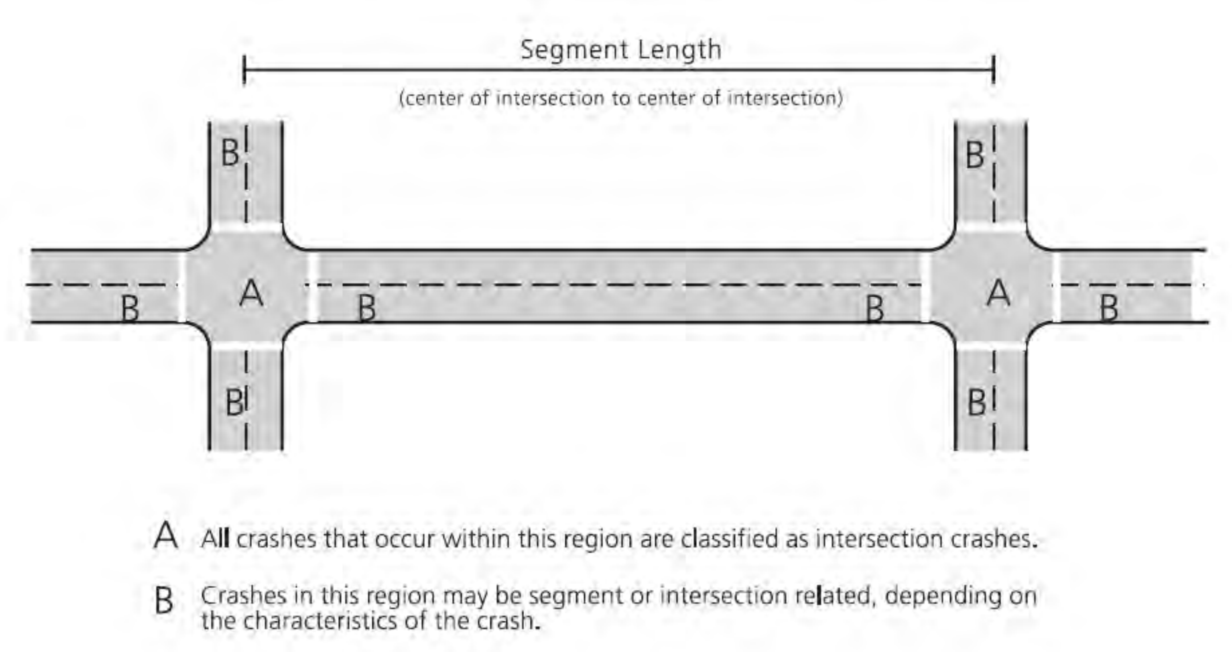
The crash data includes a wealth of other characteristics that may be of interest to the analyst, including direction of travel, lighting, weather and road conditions, work zones, vehicle and occupant details, use of alcohol and other drugs, use of safety restraints, cell phone use, speeding, and other contributing causes and events prior to the crash.

4.2.3 Assigning Crashes to Intersections and Segments

The safety analysis methods in this chapter require that each crash be uniquely assigned to an analysis site. Often this requires determining if the crash was intersection-related or roadway segment-related. The location and characteristics of the crash can be used to help determine the proper assignment for the crash.

Crashes that occur within an intersection are assigned to that intersection, as are crashes that occur on the intersection legs and are intersection-related in character. All crashes that are not assigned to an intersection are assigned to a segment. Exhibit 4-5 illustrates this concept. All crashes that occur in the "A" regions are intersection crashes. Crashes that occur in the "B" sections may be assigned to an intersection or the roadway segment on which they occur, depending on their characteristics.

Exhibit 4-5: HSM Definition of Roadway Segments and Intersections



Source: HSM Part C, Appendix A, Figure A-1

ODOT crash records include a field called "Intersection Related" that indicates a crash was related to an intersection, even though the crash was not coded as occurring at the intersection. (Note this field is not used for crashes coded as occurring at the intersection.) However, it may be that not all intersection-related crashes are identified with this field. The analyst should examine crashes near an intersection to determine if they have characteristics consistent with an intersection-related crash. For instance, rear-end collisions on an intersection approach are likely intersection-related. Conversely, run-off-the-road crashes or turning crashes near a driveway are likely segment-related. An understanding of the intersection's functional area (see APM Section 4.8.1) is also useful for assigning crashes to intersections and segments. The analyst also needs to observe operations in the field as part of the assessment, for example extent of queuing in determining intersection related crashes (see Chapter 3 for data collection procedures).

When performing both segment-based and intersection-based analyses, care should be taken to not double-count crashes. It is good practice to add a field identifying the analysis site to which each crash was assigned when including crash records in a project report.

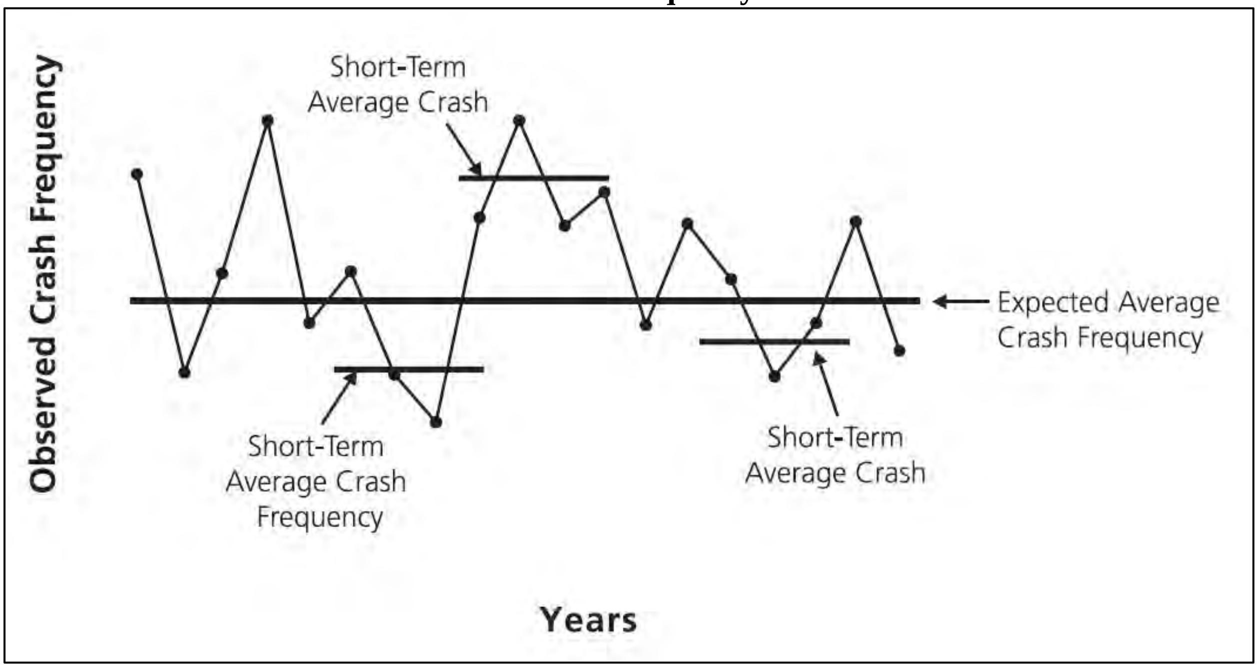
It should be noted that there may be more than one roadway segment analysis site between adjacent intersections, depending on roadway conditions. Segmenting, the process of dividing the study area into homogeneous analysis sites, is discussed in APM Sections 4.3.3 and 0.

Further information on assigning crashes to intersections and segments can be found in the HSM Part C, Appendix A, Section A.2.3.

4.2.4 Regression-to-the-Mean

Crashes are rare and random events, in that they represent a very small proportion of all events occurring on the transportation system and are partially influenced by factors that are unpredictable. As such, there is a natural variability in crash frequency at any specific location that should be considered when doing a crash analysis. This natural variation means that short-term crash frequencies using samples of one to five years of data may vary significantly from the long-term average crash frequency. Exhibit 4-6 illustrates this phenomenon. It is difficult to tell if the short-term sample represents a high, average, or low point in the natural variation of the crash frequency.

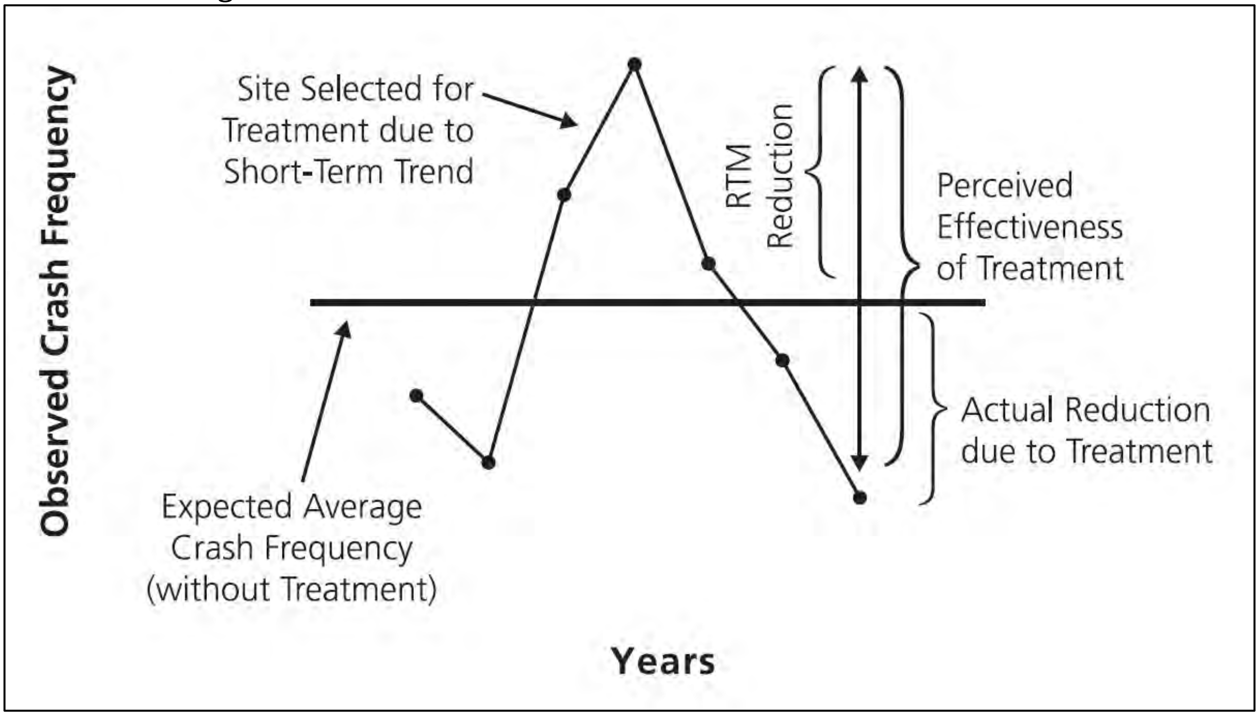
Exhibit 4-6: Variation of Short-Term Crash Frequency



Source: HSM Part A, Chapter 3, Figure 3-4

When a relatively high crash frequency is observed using a short-term sample, it is statistically probable that the next observation of that location will have a lower crash frequency. Similarly, when a low crash frequency is observed, it is likely that subsequent observations will be higher. This phenomenon is known as regression-to-the-mean (RTM), and is illustrated in Exhibit 4-7.

Exhibit 4-7: Regression-to-the-Mean Bias



Source: HSM Part A, Chapter 3, Figure 3-5

In a safety analysis, the effects of RTM can lead to a selection bias that prioritizes and evaluates countermeasures based on short-term trends. This may obscure locations with a higher expected average crash frequency or make it difficult to determine the actual reduction in expected average crash frequency due to a countermeasure.

Using a long-term sample is a way to account for RTM, and it is recommended that five years of crash data be used whenever possible. The predictive method described in this chapter uses statistical models to estimate the expected average crash frequency for a location with specific characteristics. Another way to account for RTM is to combine the predictive method with crash data [known as the Empirical-Bayes (EB) Method].

The analyst should be mindful of variations in conditions that may need to be accounted for when using multiple years of crash data—such as traffic volume changes, geometric and control changes, or disruptions due to construction. The predictive method is performed on a year-by-year basis, reflecting variations in conditions. When using screening methods, care should be taken to not include data from times with fundamentally different conditions.

4.2.5 Tools for Summarizing Crash Data

Crash Decoder Tool

The Crash Decoder Tool is an Excel-based spreadsheet with macros that uses the Comprehensive (PRC) CDS380 crash report (in Excel format) and the Excel look-up tables to translate the information. Once the sheet has decoded the information, filters can be applied to the dataset to investigate specific locations or issues. This tool also allows the analyst to create crash graphs that are helpful in both analysis and reporting. This tool can be used on all roadways. It is available on the ODOT Highway Safety Webpage. For more information refer to Appendix 4A.

Crash Graphing Tool (ODOT Employees Only)

The Crash Graphing Tool summarizes crash information of the “Direction” listing (in an Excel format) report from the State Highway Crash Reports and presents the information in standard graphs and charts. This report only analyzes state highways. Only ODOT employees can access this internal tool by contacting Information Services. For more information refer to Appendix 4A.

Crash Summary Database (ODOT Employees Only)

The Crash Summary Database, produced annually since 1990, is useful to generate quick summary reports that are often sufficient to answer questions when there is not time to do a detailed analysis. This software must be installed by an Information Services field technician. The crash summary database is a product of the most current Safety Priority Index System (SPIS) run so it uses the same three years of data. The crash summary gives an estimated crash rate based on the number of crashes, the average of the AADTs at the beginning and ending mile points of the segment and the numeric difference in the same mile points. This summary does not account for interruptions in the mile point distance (equations) or variation in the volumes when crossing multiple segments. The output reports only an estimated value along with the highest and number of SPIS sites within the section. It should not be used to report a formal crash rate unless all of the above items have been accounted for. Details on pulling crashes from the crash summary database, including use of ArcGIS, are found in Appendix 4A.

4.2.6 What Data to Report

When reporting the results of a crash analysis the narrative should describe the data and assumptions used, trends observed in the data, and the results of any crash analysis performed. Special consideration should be given to characterizing fatal and injury-A crashes and crashes involving transit, pedestrians, and bicyclists. The appendix should include data sufficient to reproduce the analysis, including the complete crash records used.

Try to use words like “more crashes than expected,” “over-representation of crashes,” “requires more investigation,” or “will likely reduce crashes” when speaking about safety review locations.

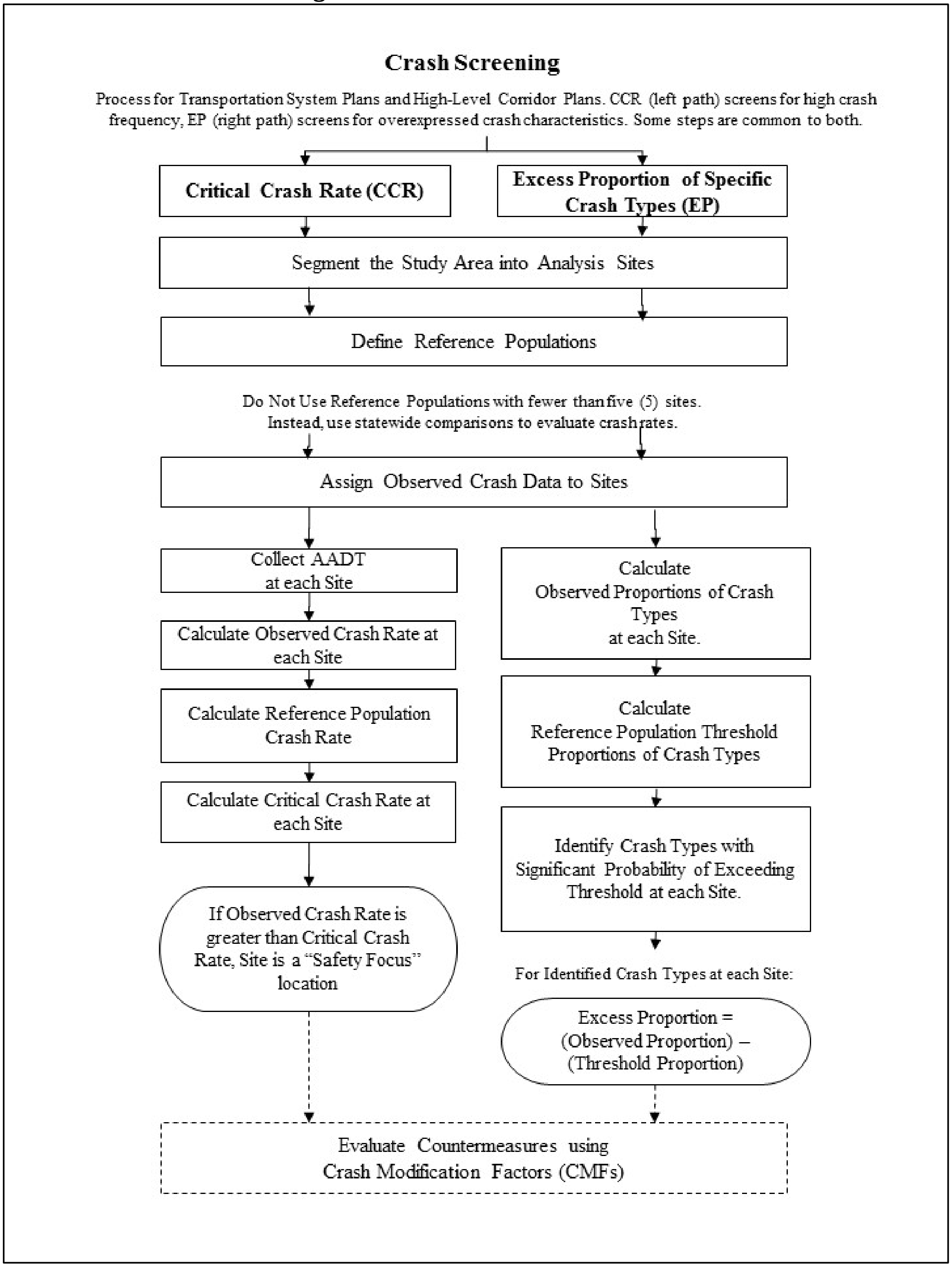
These words are more defensible because they are based on quantitative data and analysis. Avoid using subjective or qualitative words like “problem,” “hazard,” “unsafe,” etc.

- Crash Analysis Narrative
    - Data sources
    - Years of analysis
    - Assumptions used, including reference population descriptions
    - General crash trends and traffic conditions
    - Description of fatal, injury-A, transit, pedestrian, and bicycle crashes
- Crash Analysis Results Summary (as applicable)
    - Crash frequency per year at each study location by severity and by collision type
    - Crash rates and critical crash rates, including comparisons to statewide averages
    - Locations exceeding critical crash rates
    - Locations with an excess proportion of specific crash types
    - Excess expected average crash frequency
    - Predicted average crash frequency
    - PLANSAFE results
- Appendix (as applicable)
    - Crash records, including the corresponding study location assigned to each
    - Reference population statistics and descriptions
    - AADT values used for each study location
    - Unabridged critical crash rate results
    - Unabridged excess proportion of specific crash type results
    - Predictive method characteristics and results

4.3 Screening Methods

Screening methods are used to quickly characterize observed crash data from a large study area using a minimum of extra data. The results are used to identify a smaller set of locations that can then be analyzed in more detail. The HSM discusses network screening in more detail in Part B, Chapter 4. Screening methods are recommended for use with large-scale planning efforts such as Transportation System Plans (TSPs) and refinement plans. The flow chart in Exhibit 4-8 gives an overview of the process for crash data screening.

Exhibit 4-8: Crash Screening



4.3.1 Safety Priority Index System (SPIS)

Recommended Uses	All project types
Data Required	None
More Information	Safety Priority Index System website

The Safety Priority Index System (SPIS) is a statewide network screen for crash hotspots, using a methodology developed by ODOT in 1986 to flag potential safety issues on state highways. Major revisions have occurred from the original process. SPIS All-Roads covers all local roadways that have traffic volume data available in addition to state highways (typically these include all functionally classed public roads in Oregon).

The SPIS score is based on three years of crash data, and has three components: crash frequency, crash rate, and crash severity. ODOT bases SPIS analysis on 0.10-mile segments to account for variances in how crash locations are reported. To get a SPIS score, a segment must meet the segment qualifier of one of the following criteria:

- Three or more crashes have occurred at the same location over the previous three years.
- One or more fatal crashes have occurred at the same location over the previous three years.

Starting with the SPIS for crash years 2015-2017, it is planned to not include PDO (property damage only) crashes in the SPIS calculation, and for the segment qualifier to include one or more serious injury (Injury A) crashes.

Locations with the top 5% and 10% are determined for each year of data and reported as locations for further investigation. Detailed documentation on SPIS can be found on the Safety Priority Index System website.

As part of a complete safety analysis, the analyst needs to identify and report any top 5% sites in the study area along with a summary of crash trend information that may contribute to the SPIS score. Overlapping segments should be reported as one group using only the highest SPIS score. The analyst should identify relevant improvements and/or countermeasures and whether any interim roadway or operational changes have taken place since the data were collected. Identified SPIS sites may differ from those highlighted using the previously identified in HSM screening methods due to differences in methodology.

The top 5% SPIS ranking requires the Region Traffic offices to conduct a safety investigation each year to determine if there is an appropriate safety improvement fix to the problem. Contact the Region Traffic office to obtain any applicable safety investigations performed in the study area. The SPIS ranking can be determined by contacting the appropriate Region Traffic office for assistance or on the SPIS webpage.

4.3.2 Oregon Adjustable Safety Index System (OASIS) (ODOT Employees Only)

OASIS was developed as an online safety analysis tool that does the same calculations as SPIS, but provides the opportunity for the user to change some parameters and filters that can be modified in OASIS include:

- Crash Years (whether to use three of five years of crash data)
- Segment Length
- Segment Qualifier (minimum crash history required for inclusion in analysis)
- Road Jurisdiction
- Collision Type
- Weather condition
- Light Condition
- Road Surface Condition
- Special Conditions:
    - Work Zone Involved
    - Speed Involved
    - Alcohol or Drugs Involved
    - Pedestrian or Bike Involved
    - Truck Involved
    - Deer or Elk Involved
    - Cell Phone Involved
    - Intersection Involved
    - Roadway Departure Involved
    - Curved Involved
    - Signalized Involved
- Scoring Formula and Weighting

The OASIS tool allows the user to quickly create a statewide custom safety screening analysis, for example, examining only crashes involving pedestrians or bicyclists. This program is available only to Traffic Roadway staff and ODOT Region Staff.

Since parameters can be changed, OASIS data should be carefully reported to indicate parameters used, and only compared to results of the same input parameter set. Users should also be cautioned that OASIS top 5%, top 10% are only for the particular OASIS run. If data from more than one run are combined, the percentile cutoffs need to be checked and reset appropriately, such as if doing OASIS for the entire state, both local roads and state highways.

4.3.3 Reference Populations

Recommended Uses	Critical Crash Rate and Excess Proportions of Specific Crash Types
Data Required	Crash frequency for target crash types of interest Site characteristics such as: Land Use (Rural/Urban) Geometry (Number of through lanes, intersection legs, etc.) Traffic Control (Signalized, two-way stop control, etc.) AADT Traffic Volumes
More Information	HSM Part B, Chapter 4, Section 4.2.2

Reference populations are central to the HSM screening methods presented in this chapter. Reference populations are groups of study sites (such as intersections or roadway segments) that have similar characteristics and serve as a comparison for evaluating safety performance.

Reference populations represent the typical safety performance for a specific type of study site. The safety performance of an individual study site is compared to the average safety performance of a reference population, using either the Critical Crash Rate method or the Excess Proportion of Specific Crash Types method to establish a performance standard specific to the study site and reference population.

Potential characteristics that can be used to define reference populations include:

- Traffic control (e.g., signalized, two-way or four-way stop control, yield control, roundabout)
- Number of approaches (e.g., three-leg or four-leg intersections)
- Cross-section (e.g., number of through lanes and/or turning lanes)
- Functional classification (e.g., arterial, collector, local)
- Adjacent land use (e.g., urban, suburban, rural)
- Traffic volume ranges [e.g., total entering volume (TEV), peak hour volume, average annual daily traffic (AADT)]
- Terrain (e.g., flat, rolling, mountainous)
- Access density (e.g., driveway and intersection spacing)
- Median type and/or width
- Operating or posted speed

Defining characteristics for a reference population will vary depending on the amount of detail known about each study site, the size of the network to be screened, and the focus of the safety analysis. Each reference population must contain at least five sites to be statistically valid.

Reference populations may be composed of "internal" or "external" sites, or a combination of both. Internal sites are locations being studied for the safety analysis; external sites are locations not being studied for the safety analysis.

For example, a city TSP may be screening the safety performance of a variety of intersections within the city. An internal reference population might be composed of four-way stop-controlled intersections being screened in the city. An external reference population might be composed of four-way stop-controlled intersections from similar adjacent cities.

Internal reference populations are the minimum requirement for network screening. Since all data are being collected for the project, there is no need to gather extra data. Internal reference populations are best for prioritizing locations within a study area or for identifying outliers within a study area.

External reference populations may be used when it is desired to assess statewide safety performance conditions to assess statewide safety performance, use the mean crash rate from Exhibit 4-1.

When reporting screening results, clearly identify the reference populations used.

4.3.4 Critical Crash Rate

Recommended Uses	Transportation System Plans and Corridor Plans May be used for existing conditions assessment in development review or project development when predictive methods are unavailable
Data Required	Crash frequency by severity AADT traffic volumes Reference populations
More Information	HSM Part B, Chapter 4, Section 4.4.2.5

Crash rates describe crash frequency in relation to traffic volume. Crash rates at intersections are typically given in units of crashes per million entering vehicles (crashes/MEV). Crash rates for segments are typically given in units of crashes per million vehicle miles traveled [crashes/million vehicle miles traveled (MVMT)]. It is recommended that Annual Average Daily Traffic (AADT) values be used in calculating crash rates. APM Section 5.7 contains procedures for calculating AADT. If AADT data are not available, Average Daily Traffic (ADT) can be substituted.

The Critical Crash Rate analysis method evaluates each study site's crash rate compared to the average crash rate of that site's reference population. Study sites with significantly higher crash rates (exceeding the Critical Crash Rate) are identified for further analysis.

The Critical Crash Rate method evaluates the overall magnitude of the observed crash rate for one target crash type at a time. Two variations on Critical Crash Rate are commonly used, one evaluating total crash rate and one evaluating the crash rate considering only fatal and injury-A severity crashes.

The Critical Crash Rate method does not allow for estimates of future safety performance and cannot be used for evaluating alternatives. This method does not address RTM, so short time periods (less than three years) should not be used. Crash rates also do not account for the non-linearity of crash frequency with respect to traffic volumes. Analysts should be aware that crash rates may be reduced simply by an increase in traffic volumes alone. A reduction in crash rate at higher traffic volumes is often the expected roadway behavior and does not indicate a fundamental change in underlying safety performance. The predictive methods discussed in APM Section 4.4 can address all three of these limitations.

A spreadsheet is available from the safety analysis tools section of the ODOT Transportation Development Planning Technical Tools website that automates much of the Critical Crash Rate calculations. A planning level method for calculating critical crash rates using the ODOT Visum Safety Add-In tool is found in Appendix 4A.

The general procedure for network screening using the Critical Crash Rate method is as follows:

1. Identify analysis sites and assign observed crashes (see APM Section 4.2.3)
2. For each analysis site, using the target crash frequency and AADT traffic volume calculate the crash rate (or observed crash rate) on a MEV basis
3. Establish reference populations and calculate the average crash rate for each reference population
4. Choose the desired statistical significance level (95% is recommended)
5. For each analysis site, calculate the reference population critical crash rate. This value is unique for each analysis site and is a function of the reference population average crash rate, traffic volume at the site, and the desired statistical significance.
6. For each analysis site, calculate the statewide comparison critical crash rate. This value is unique for each analysis site and is a function of the mean crash rate from Exhibit 4-1, traffic volume at the site, and the desired statistical significance.
7. Identify any sites where the observed crash rate is greater than the calculated reference population critical crash rates
8. Identify any sites where the observed crash rate is greater than the calculated statewide comparison critical crash rates
9. Identify any sites where the observed crash rate is greater than the published statewide comparison rates. Intersections should also be compared with the 90th percentile rates in Exhibit 4-1. Segments should also be compared with the average rates in Crash Rate Table II.

This process is repeated for each area of interest. The analysis should be done using a critical crash rate based on total crashes. An additional suggested analysis is to use a critical crash rate based on only fatal and injury-A severity crashes. If multiple sets of reference populations are being used (e.g., internal and external), the process is repeated for each set of reference populations.

The process is generally the same for intersections and for segments. Intersection critical crash rate analysis should use only crashes associated with intersections and segment critical crash rate analysis should use only crashes associated with segments (see APM Section 4.2.3).

For intersections and segments, the crash rate is derived using different measures of traffic exposure. For intersections, crash rates are derived using Million Entering Vehicles (MEV), while segments are derived using Million Vehicle Miles Traveled (MVMT).

$$MEV = \frac{AADT \times 365 \times n}{1{,}000{,}000}$$

$$MEV = Million\ \ Entering\ \ Vehicles$$

$$n = Number\ of\ Years$$

$$MVMT = \frac{AADT \times L \times 365 \times n}{1{,}000{,}000}$$

$$MVMT = Million\ Vehicle - Miles\ of\ Travel$$

$$L = Segment\ Length$$

$$n = Number\ of\ Years$$

Segments also must be divided into sites that are similar in character, as described in APM Section 4.3.3 on reference populations. Segment boundaries should be placed where the reference population changes, at intersections that have observed crashes, and at other logical breakpoints in order for segments not to exceed two or three miles in length. For some very long corridors such as in rural eastern Oregon, segments could be up to five miles in length. Segments should ideally be close to one mile in length. For urban areas, obtaining one mile segments is difficult, however the majority of urban crashes are intersection related. Short sections less than a half mile in length typically skew the crash rates and should be avoided.

However, it may not be possible to avoid short segments in which case the length should be normalized to a mile and the rate recalculated. This recalculated rate would be compared to Table II to see if it still exceeds. For example, if a 0.25 mi segment had a crash rate of 8.0 crashes per MVMT, then the normalized rate would be 8.0 crashes per MVMT x 0.25 mile = 2.0 crashes per MVMT.

For segments on milepointed highways, length may be calculated based on Begin and End Milepoint. All crashes (both intersection and non-intersection) are included in a Table II-based segment crash analysis. Segment lengths need to be adjusted if they contain a milepoint equation. Milepoint equations can be found from the Equations and Milepoint Range Report.

When reporting the results of this method, include the following for each target crash type:

- Crash frequency for each analysis site
- AADT (or ADT) traffic volume for each analysis site
- Reference population characteristics and summary statistics
- Observed crash rate for each analysis site
- Critical crash rate for each analysis site
- Sites identified as exceeding their critical crash rate

For this method to be statistically valid, there needs to be at least five to ten sites in each reference population. If there are fewer than five sites available to create a reference population, these methods do not apply (since there are not enough sites to screen). This method may also produce small crash rate variances that may overlook sites with a significant crash problem. Also, the sites within the study area reference populations may have different variances when compared to similar sites statewide. To minimize these issues, crash rates should also be compared to published statewide crash rates as described below. The Critical Crash Rate method can also be used for a statewide comparison by replacing the reference population average crash rate with the appropriate mean crash rate from Exhibit 4-1.

Segment crash rates should be compared to the appropriate average crash rate from the Oregon State Highway Crash Rate Tables, annually published by the ODOT CAR Unit. In this document, crash rates for given segments of all state highways are calculated and listed for each of the last five years.

Crash Rate Table II is the primary table used in segment crash rate analysis. It shows statewide average crash rates for each of the last five years, by urban and rural area, and by roadway classification. Federal functional classifications can be found on the ODOT Federal Functional Classification (FC) webpage.

The following examples illustrate the calculation of MEV, segment crash rates, and the calculation of Critical Crash Rates for intersections and segments.

Example 4-1: Segment Crash Rate Calculation and Comparison

A 1.6-mile principal highway segment in a rural area has experienced 22 reported crashes over the last three years. The segment AADT from the State Highway Vehicle Classification Data Report is 23,000.

$$\text{Rate} = \frac{\text{Number of Crashes X 1,000,000}}{\text{Length (in miles) X AADT X (Yrs X 365)}}$$

$$= \frac{22 \text{ X } 1{,}000{,}000}{1.6 \text{ X } 23{,}000 \text{ X } 3\text{Yrs X } 365}$$

$$= 0.55 \text{ Crashes per Million Vehicle Miles (MVM)}$$

As shown in the table below, the statewide average crash rates are:

2005	0.67
2006	0.69
2007	0.68
Average	0.68

The segment crash rate of 0.55 is less than the average statewide rate of 0.68.

Statewide Crash Rate Table

TABLE II: FIVE-YEAR COMPARISON OF STATE HIGHWAY CRASH RATES

Table II presents a comparison of state highway crash rates for the past five years, for urban and rural areas, by functional classification. Mileage is shown for the current data year only.

See Table IV for information on official highway mileage and VMT data.

JURISDICTION AND FUNCTIONAL CLASSIFICATION	MILES*	2007 Rate	2006 Rate	2005 Rate	2004 Rate	2003 Rate
TOTAL STATE HWY SYSTEM	**7,455.23**	**0.85**	**0.85**	**0.87**	**0.79**	**0.98**
Interstate Freeways	729.57	0.38	0.39	0.41	0.38	0.42
Other Fwys/Expressways	52.26	0.73	0.78	0.80	0.78	0.87
Non-Freeways (combined)	6,673.40	1.27	1.26	1.25	1.13	1.45
Other Principal Arterials	3,281.04	1.28	1.29	1.28	1.16	1.52
Minor Arterials	1,964.61	1.20	1.14	1.14	1.02	1.20
Urban Collectors	8.69	1.10	0.68	1.19	1.23	2.08
Rural Major Collectors	1,382.20	1.25	1.11	1.14	0.93	1.25
Rural Minor Collectors	36.86	0.64	0.66	1.30	0.32	1.30
Rural Local	0.00	0.00	16.52	4.23	2.68	8.06
Rural Areas	**6,414.01**	**0.58**	**0.58**	**0.59**	**0.51**	**0.59**
Interstate Freeways	539.37	0.28	0.29	0.31	0.25	0.26
Non-Freeways (combined)	5,874.64	0.79	0.77	0.77	0.68	0.80
Other Principal Arterials	2,658.39	0.68	0.69	0.67	0.61	0.71
Minor Arterials	1,839.04	0.99	0.93	0.98	0.83	0.97
Rural Major Collectors	1,340.60	1.24	1.08	1.10	0.92	1.20
Rural Minor Collectors	36.61	0.69	0.36	1.40	0.35	1.40
Rural Local	0.00	0.00	16.52	4.23	2.68	8.06

* Couplet and Roadway 3 data are included. Frontage road and connection data are excluded.

A segment crash rate that exceeds the statewide average crash rate may be an indication that further investigation is necessary, although it is possible that upon further investigation it may be determined that no improvements are necessary. Likewise, cost-effective improvements to reduce crashes could still be identified even with a segment crash rate lower than the statewide average.

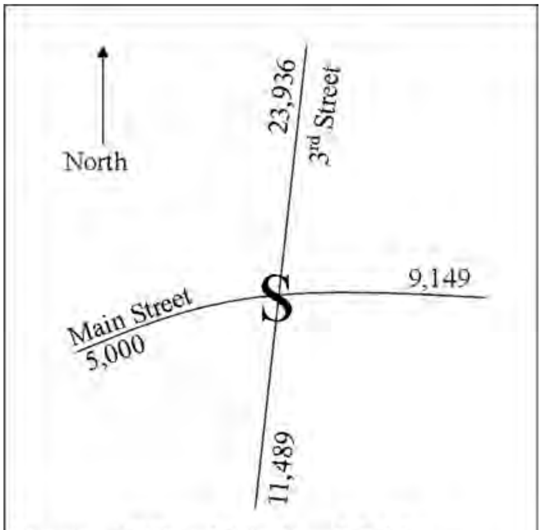
Example 4-2: Million Entering Vehicles (MEV)

As a part of an urban street modernization project, a safety analysis needs to be done for Main St. This street is a congested urban corridor with a mixture of unsignalized and signalized intersections with varying numbers of lanes.

Traffic counts were taken at each intersection and crashes for the previous five years have been compiled. In order to calculate the HSM Critical Crash Rate the total MEV is needed for each intersection.

Data Needs

Directional count data are needed for each leg of each intersection. If ADT-capable counts are not available and the best data available are the Traffic Volume Tables (TVT) or a non-directional tubular count, a 50/50 directional split can be assumed. For the intersection of Main St. and 3rd St., ADTs entering the intersection from each leg were developed and are shown in the diagram below. See Chapter 5 for the process to develop directional ADTs.



Entering ADTs at Main St and 3rd St

Step 1: Adjustment of ADTs to AADTs

The ADTs entering the intersection need to be converted to AADTs. See Chapter 5 for the process to develop the appropriate adjustment factors. Note: If using the TVT for volumes, this step does not apply. The volumes in the TVT have already been adjusted to AADT. In this case move directly to Step 2.

$$AADT = ADT \times Adjustment\ Factor^{*}$$

$$Entering\ AADTs\ at\ Main\ Street\ and\ 3rd\ Street$$

$$AADT_{North} = 23936 \times 0.94 = 22500$$

$$AADT_{South} = 11489 \times 0.94 = 10800$$

$$AADT_{East} = 9149 \times 0.94 = 8600$$

$$AADT_{West} = 5000 \times 0.94 = 4700$$

* See Chapter 5 for steps to calculate the adjustment factor.

Step 2: Million Entering Vehicles (MEV)

$$Total\ Entering\ AADT = \sum Entering\ AADTs$$

$$Total\ Entering\ AADT_{Main\ Street\ at\ 3rd\ Street} = 22500 + 10800 + 8600 + 4700 = 46{,}600$$

$$MEV = \frac{Total\ Entering\ AADT \times 365 \times n}{1{,}000{,}000}$$

$$AADT = Annual\ Average\ Daily\ Traffic$$

$$n = Number\ of\ Years$$

$$MEV_{Main\ Street\ at\ 3rd\ Street} = \frac{46{,}600 \times 365 \times 5}{1{,}000{,}000} = 85.0 MEV$$

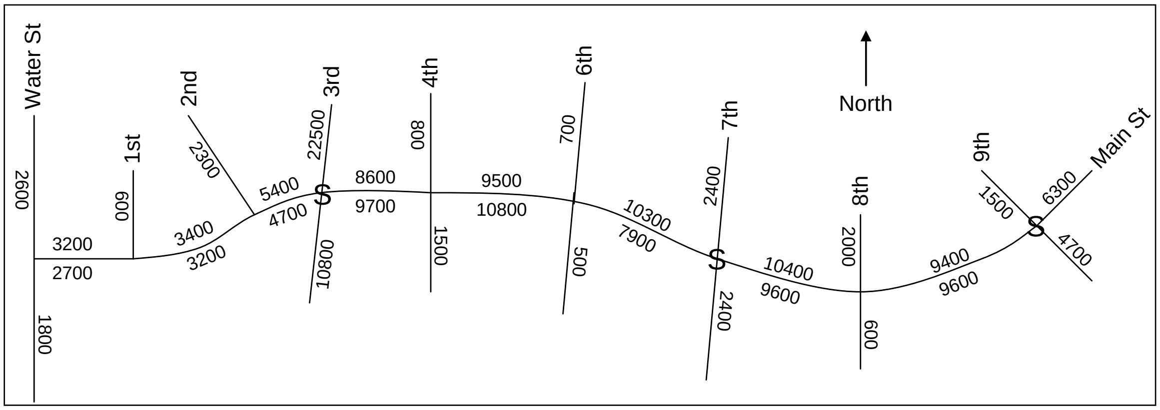
Example 4-3: HSM Critical Rate for Intersections with Internal Reference Population

As part of an urban street modernization project corridor plan, a safety analysis needs to be done for Main St. This street is a congested urban corridor with a mixture of unsignalized and signalized intersections with varying numbers of lanes.

The project engineer has created existing year average daily traffic (ADT) volumes from available intersection counts. The ADT counts were converted into AADT (an average value for 2005-2009 using appropriate seasonal factors and annual growth factors), which are shown as daily total entering volumes in the figure below. In addition, intersection crash data for the past five years are shown in the table below. Crash data are summarized by year and by severity.
A critical crash rate analysis will be performed considering all crashes and considering Fatal plus Injury-A crashes as the target crash type.

Data Needs

Existing Year Annual Average Daily Entering Traffic Volumes



Intersection Crashes per Year

Intersection	Type	Year					Total
		2005	2006	2007	2008	2009	
Water St.	Unsignalized	2	1	0	1	2	6
1st St.	Unsignalized	0	0	0	0	0	0
2nd St.	Unsignalized	0	0	0	0	1	1
3rd St.	Signalized	6	8	5	6	4	29
4th St.	Unsignalized	0	0	0	1	1	2
6th St.	Unsignalized	6	2	3	2	1	14
7th St.	Signalized	3	1	2	6	3	15
8th St.	Unsignalized	0	0	2	3	0	5
9th St.	Signalized	3	6	2	0	1	12
Total		**20**	**18**	**14**	**19**	**13**	**84**

Intersection Crashes by Severity

Intersection	Type	Severity					Total
		Fatal	Inj. A	Inj. B	Inj. C	PDO	
Water St.	Unsignalized	0	2	0	1	3	6
1st St.	Unsignalized	0	0	0	0	0	0
2nd St.	Unsignalized	0	0	0	0	1	1
3rd St.	Signalized	0	3	5	9	12	29
4th St.	Unsignalized	0	0	0	0	2	2
6th St.	Unsignalized	0	5	1	3	5	14
7th St.	Signalized	0	1	1	4	9	15
8th St.	Unsignalized	0	0	1	1	3	5
9th St.	Signalized	0	1	3	3	5	12
Total		**0**	**12**	**11**	**21**	**40**	**84**

The HSM Critical Rate screening method will be used to determine the intersections with the greatest need.

Note: All sample calculations given at the intersection of Water St. and Main St.

Step 1: At each intersection, calculate the volume on a Million Entering Vehicle (MEV) basis

(1) $$MEV = \frac{AADT \times 365 \times n}{1{,}000{,}000}$$

$$MEV = Million\ \ Entering\ \ Vehicles$$

$$n = Number\ of\ Years$$

$$MEV = \frac{7{,}600 \times 365 \times 5}{1{,}000{,}000} = 13.9\ MEV$$

Step 2: Calculate the crash rate at each intersection

(2) $$R = \frac{Crash\ Total}{MEV_n}$$

$$R = Observed\ Crash\ Rate$$

Crash rate for all crashes

$$R = \frac{6}{13.9} = 0.43\,\frac{Crashes}{MEV}$$

Crash rate for Fatal and Injury-A target crash type

$$R_{FA}\,\frac{2}{13.9} = 0.14\,\frac{Fatal + A\ Crashes}{MEV}$$

Intersection	Daily Volume	MEV (1)	Crash Total	F+A Crash Total	Crash Rate (2)	F+A Crash Rate (2)
Water St.	7,600	13.9	6	2	0.43	0.14
1st St.	6,700	12.2	0	0	0.00	0.00
2nd St.	10,900	19.9	1	0	0.05	0.00
3rd St.	46,600	85.0	29	3	0.34	0.04
4th St.	21,500	39.2	2	0	0.05	0.00
6th St.	22,300	40.7	14	5	0.34	0.12
7th St.	23,100	42.2	15	1	0.36	0.02
8th St.	19,800	36.1	5	0	0.14	0.00
9th St.	22,100	40.3	12	1	0.30	0.02

Step 3: Calculate the average crash per population

(3a) Divide the intersections into varying populations (groups) based on operational (i.e., unsignalized/signalized/roundabouts) or geometric (i.e., three-leg/four-leg) differences. Only one reference population of sufficient size exists for this example. Six intersections fall under the unsignalized type (Type 1) reference population. There are not enough three-leg or four-leg unsignalized intersections to further subdivide the reference populations. The critical rate method cannot be used for the signalized intersections (Type 2) since there are only three in the reference population. TPAU will be contacted to propose adding external reference sites to the signalized reference population (not included in this example).

$$(3b)\ R_a = \frac{\sum_{i=1}(N_{obs,i})}{\sum_{i=1}(MEV_i)}$$

$R_a = Average\ crash\ rate\ for\ reference\ population\ a$

$R_{FA(a)} = Average\ Fatal + A\ crash\ rate\ for\ reference\ population\ a$

Average Crash Rate for Unsignalized Intersections reference population:

$$R_1 = \frac{6+0+1+2+14+5}{13.9+12.2+19.9+39.2+40.7+36.1}$$

$$= 0.17\ \frac{Crashes}{MEV}$$

Average Fatal + Injury-a Crash rate for Unsignalized Intersections reference population:

$$R_{FA(1)} = \frac{2+0+0+0+5+0}{13.9+12.2+19.9+39.2+40.7+36.1}$$

$$= 0.04\ \frac{Crashes}{MEV}$$

Equation (3b) is a reduced version of Equation 4-10 found in the 1st edition Highway Safety Manual, Volume 1, Chapter 4.

Step 4: Calculate a critical crash rate for each intersection

$$(4) \quad R_c = R_a + Confidence\ Level \times \sqrt{\frac{R_a}{MEV_n} + \frac{1}{2 \times MEV_n}}$$

$$R_c = Critical\ crash\ rate$$

$$R_a = Weighted\ average\ crash\ rate\ for\ reference\ population$$

Critical crash rate for all crashes

$$R_C = 0.17 + 1.645 \times \sqrt{\frac{0.17}{13.9} + \frac{1}{2 \times 13.9}} = 0.39\ \frac{Crashes}{MEV}$$

Critical crash rate for Fatal + Injury-A crashes

$$R_{FA(C)} = 0.04 + 1.645 \times \sqrt{\frac{0.04}{13.9} + \frac{1}{2 \times 13.9}} = 0.17\ \frac{F + A\ \ Crashes}{MEV}$$

Table 4-9 in the HSM (page 4-36) gives P-levels to correspond to differing confidence levels. Typical use would be 95% confidence (P=1.645).

Intersection	Intersection Population Type (3a)	Crash Rate (2)	Critical Crash Rate (4)	Over Critical (5)
Water St.	1	0.43	0.39	Over
1st St.	1	0.00	0.41	Under
2nd St.	1	0.05	0.35	Under
4th St.	1	0.05	0.29	Under
6th St.	1	0.34	0.29	Over
8th St.	1	0.14	0.30	Under

Intersection	Intersection Population Type (3a)	F+A Crash Rate (2)	F+A Critical Crash Rate (4)	Over F+A Critical (5)
Water St.	1	0.14	0.17	Under
1st St.	1	0.00	0.18	Under
2nd St.	1	0.00	0.14	Under
4th St.	1	0.00	0.11	Under
6th St.	1	0.12	0.11	Over
8th St.	1	0.00	0.11	Under

Step 5: Compare observed crash rate with critical crash rate

Compare the critical crash rate with the crash rate for each intersection. Any intersection with a crash rate that exceeds its critical rate should be flagged for further review.
In the above example, the following intersections were flagged for further analysis:

- Water St. and Main St.
- 6^{th} St. and Main St.

Step 6: Compare observed crash rate with statewide 90^{th} percentile rates (Exhibit 4-1)

The crash rates at all study intersections (including those without a reference population) are compared to the statewide 90^{th} percentile crash rates for urban three-leg minor stop-controlled (U3ST), urban four-leg minor stop-controlled (U4ST), and urban four-leg signalized (U4SG) intersections.

Intersection	Intersection Population Type	Observed Crash Rate	Statewide 90^{th} Percentile Crash Rate
Water St.	U3ST	0.43	0.293
1^{st} St.	U3ST	0.00	0.293
2^{nd} St.	U3ST	0.05	0.293
3^{rd} St.	U4SG	0.34	0.860
4^{th} St.	U4ST	0.05	0.408
6^{th} St.	U4ST	0.34	0.408
7^{th} St.	U4SG	0.36	0.860
8^{th} St.	U4ST	0.14	0.408
9^{th} St.	U4SG	0.30	0.860

In the above example, the following intersection was flagged for further analysis:

- Water St. and Main St.

Step 7: Conclusions

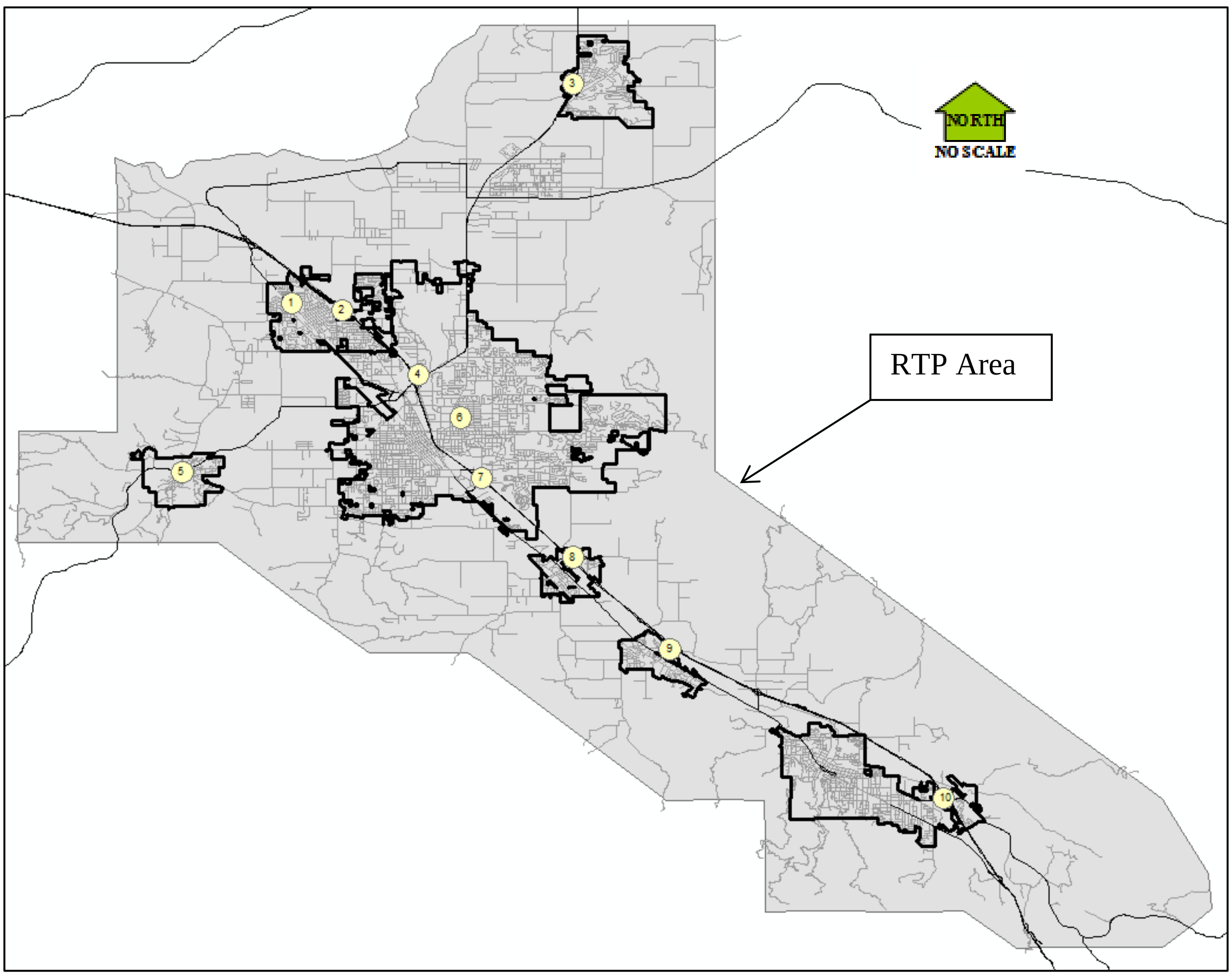
From the analysis, the intersections of Main St. and Water St. and Main St. and 6^{th} St. exceed the critical rate. Main St. and 6^{th} St. also exceeds the Fatal plus Injury-A critical rate, and the statewide 90^{th} percentile crash rate. These intersections are “safety focus” locations that need to be reviewed in more depth. At a minimum, crashes and patterns need to be identified and potential countermeasures indicated. More in-depth HSM predictive analysis could be done for the existing conditions and any later future conditions, which could also include finding the most cost-effective countermeasures.

Example 4-4: HSM Critical Rate for Intersections with Statewide Reference Population

As part of an RTP update, screening level statewide comparative safety analysis is to be performed on ten urban four-leg signalized intersections within the RTP area. The goal of this analysis is to identify intersections that have crash rates above other similar intersection types statewide. This analysis will consider all crash types and severities.
The project engineer has created existing year average daily traffic (ADT) volumes from available intersection counts and converted into AADT (total entering volumes). In addition, intersection crash data for the past five years are shown in the table below. Crash data are summarized by year and by severity.

Data Needs

Existing Year Annual Average Daily Entering Traffic Volumes



Intersection Crashes per Year

Intersection	Type	Year					Total
		2010	2011	2012	2013	2014	
1	Signalized	5	5	3	4	2	19
2	Signalized	4	3	5	4	2	18
3	Signalized	1	1	2	1	1	6
4	Signalized	4	1	1	5	5	16
5	Signalized	4	1	1	4	2	12
6	Signalized	3	3	4	3	3	16
7	Signalized	1	1	1	1	5	9
8	Signalized	4	1	4	5	4	18
9	Signalized	3	1	2	5	1	12
10	Signalized	1	4	1	2	1	9
Total		30	21	24	34	26	135

The HSM Critical Rate screening method will be used to determine the intersections with the greatest need.

Note: All sample calculations given for Intersection #1

Step 1: At each intersection, calculate the volume on a Million Entering Vehicle (MEV) basis

(1) $$MEV = \frac{AADT \times 365 \times n}{1{,}000{,}000}$$

$$MEV = Million \ \ Entering \ \ Vehicles$$

$$n = Number \ of \ Years$$

$$MEV = \frac{9{,}700 \times 365 \times 5}{1{,}000{,}000} = 17.7 \ MEV$$

Step 2: Calculate the crash rate at each intersection

(2) $$R = \frac{Crash\ Total}{MEV_n}$$

$$R = Observed\ Crash\ Rate$$

Crash rate for all crashes

$$R = \frac{19}{17.7} = 1.07 \ \frac{Crashes}{MEV}$$

Intersection	Daily Volume	MEV (1)	Crash Total	Crash Rate (2)
1	9,700	17.7	19	1.07
2	19,700	36.0	18	0.50
3	12,400	22.6	6	0.27
4	12,300	22.4	16	0.71
5	11,000	20.1	12	0.60
6	16,600	30.3	16	0.53
7	11,500	21.0	9	0.43
8	8,000	14.6	18	1.23
9	8,600	15.7	12	0.76
10	11,500	21.0	9	0.43

Step 3: Calculate the average crash per population

(3a) Because this is a statewide comparison, use the mean crash rate for Urban 4SG intersections from Exhibit 4-1.

Exhibit 4-1: Intersection Crash Rates per MEV by Land Type and Traffic Control

	Rural				Urban			
	3SG	**3ST**	**4SG**	**4ST**	**3SG**	**3ST**	**4SG**	**4ST**
No. of Intersections	7	115	20	60	55	77	106	60
Mean Crash Rate	**0.226**	**0.196**	**0.324**	**0.434**	**0.275**	**0.131**	**0.477**	**0.198**
Median Crash Rate	0.163	0.092	0.320	0.267	0.252	0.105	0.420	0.145
Standard Deviation	0.185	0.314	0.223	0.534	0.155	0.121	0.273	0.176
Coefficient of Variation	0.819	1.602	0.688	1.230	0.564	0.924	0.572	0.889
90th Percentile Rate	**0.464**	**0.475**	**0.579**	**1.080**	**0.509**	**0.293**	**0.860**	**0.408**

Source: Assessment Of Statewide Intersection Safety Performance, FHWA-OR-RD-18, Portland State University and Oregon State University, June 2011, Table 4.1, p. 47.

Note: Traffic control types include 3SG (three-leg signalized), 3ST (three-leg minor stop-control), 4SG (four-leg signalized), 4ST (four-leg minor stop-control).

Step 4: Calculate a critical crash rate for each intersection

$$(4) \quad R_c = R_a + Confidence\ Level \times \sqrt{\frac{R_a}{MEV_n}} + \frac{1}{2 \times MEV_n}$$

$R_c = Critical\ crash\ rate$

$R_a = Weighted\ average\ crash\ rate\ for\ reference\ population$

Critical crash rate for all crashes

$$R_C = 0.477 + 1.645 \times \sqrt{\frac{0.477}{17.7}} + \frac{1}{2 \times 17.7} = 0.78 \, \frac{Crashes}{MEV}$$

Table 4-9 in the HSM (page 4-36) gives P-levels to correspond to differing confidence levels. Typical use would be 95% confidence (P=1.645).

Intersection	Intersection Population Type (3a)	Crash Rate (2)	Critical Crash Rate (4)	Over Critical (5)
1	Signalized	**1.07**	**0.78**	**Over**
2	Signalized	0.50	0.68	Under
3	Signalized	0.27	0.74	Under
4	Signalized	0.71	0.74	Under
5	Signalized	0.60	0.76	Under
6	Signalized	0.53	0.70	Under
7	Signalized	0.43	0.75	Under
8	Signalized	**1.23**	**0.81**	**Over**
9	Signalized	0.76	0.80	Under
10	Signalized	0.43	0.75	Under

Step 5: Compare observed crash rate with critical crash rate

Compare the critical crash rate with the crash rate for each intersection. Any intersection with a crash rate that exceeds its critical rate should be flagged for further review. In the above example, intersections #1 and #8 were flagged for further analysis.

Step 6: Compare observed crash rate with statewide 90th percentile rates (Exhibit 4-1)

The crash rates at all study intersections (including those without a reference population) are compared to the statewide 90th percentile crash rates for urban three-leg minor stop-controlled (U3ST), urban four-leg minor stop-controlled (U4ST), and urban four-leg signalized (U4SG) intersections.

Intersection	Intersection Population Type	Observed Crash Rate	Statewide 90th Percentile Crash Rate
1	U4SG	**1.07**	**0.860**
2	U4SG	0.50	0.860
3	U4SG	0.27	0.860
4	U4SG	0.71	0.860
5	U4SG	0.60	0.860
6	U4SG	0.53	0.860
7	U4SG	0.43	0.860
8	U4SG	**1.23**	**0.860**
9	U4SG	0.76	0.860
10	U4SG	0.43	0.860

In the above example, intersections #1 and #8 are flagged for further analysis.

Step 7: Conclusions

From the analysis, intersections #1 and #8 exceed the critical rate based on a statewide comparison as well as the statewide 90th percentile crash rate. These intersections are "safety focus" locations that need to be reviewed in more depth. At a minimum, crashes and patterns need to be identified and potential countermeasures indicated. More in-depth HSM predictive analysis could be done for the existing conditions and any later future conditions, which could also include finding the most cost-effective countermeasures.

Example 4-5: Critical Crash Rate for Segments

A screening level safety analysis is to be performed on North Santiam Highway, a rural state highway corridor. The route is approximately 80 miles in length and has both level and rolling terrain . It contains multilane, two-lane, passing and climbing lane sections. It is desired to identify segments for further safety analysis. The Critical Crash Rate method will be applied to segments within the corridor. The study area is shown below.

Study Area

> *All sample calculations given are for the reference population 4-lane highway, and for Segment 5.*

Step 1: Obtain comprehensive (PRC) crash report in Excel format for corridor

Five years of crash data should be obtained using the PRC crash report, available for both state highways and local roads on the ODOT Internal Crash Reports website or the External Crash Reports website. The PRC report contains multiple records per crash. Only the crash level record (first record) is needed. The vehicle and participant level records should be filtered out so that only the crash level record remains for each crash.

Step 2: Filter out urban areas and rural intersections

The PRC report identifies intersection crashes using the RD_CHAR_SHORT_DESC field. Filter out these crashes. Intersection crashes can generally be assumed to be any that are at an intersection with at least one crash, plus 0.01 miles on either side of the intersection milepoint, as well as any coded as intersection-related. Segments are subdivided at intersections with at least one crash. Intersections with no crashes can be included within segments. The intersections with crashes can be analyzed separately using the Intersection Critical Rate method described previously.

Step 3: Obtain AADTs from the State Highway Vehicle Classification Report

For state highways, AADTs are obtained from the State Highway Vehicle Classification Report. For non-state highways, AADTs will need to be calculated following procedures in Chapter 5.

Step 4: Segmenting

Segments are developed based on reference populations. Segment boundaries should be placed where the reference population changes, at intersections that have crashes, and at other logical breakpoints in order for segments not to exceed two or three miles in length. For some very long corridors such as in rural eastern Oregon, segments could be up to five miles in length.

A variety of potential reference populations could be considered, including:

- Urban/rural
- Freeways/arterials
- Number of travel lanes (2, 3, 4, 5, etc.)
- Divided/undivided
- Presence of auxiliary lanes (passing lanes, climbing lanes)
- Terrain (level, rolling and mountainous)
- Geographic area or elevation
- AADT level

Data sources for determining reference population boundaries include TransGIS, Transviewer, the digital video log, and the State Highway Vehicle Classification Report.
At least five segments are needed for each reference population. If there are enough segments, it may be possible to have subgroupings, such as initially by number of lanes and then by terrain. It may also be desirable to separately analyze more than one reference population. This may help to further identify crash trends. Specific types of crashes could be examined as well, such as those involving snow and ice or fatal and injury crashes.

Freeway reference populations could include basic freeway lane sections, weave sections, and ramp merge or diverge sections.

Each segment is assigned a Begin and End Milepoint. Segment lengths need to be adjusted if they contain a milepoint equation. Milepoint equations can be found from the Equations and Milepoint Range Report.

Step 5: Identify the number of crashes in each segment

Each segment is assigned the total number of crashes within that segment. This process can be automated using the PRC crash data from Step 1 as a lookup table to sum the number of crashes in each segment between begin and end milepoints.

In this example, five segments were identified within the reference population of four-lane divided highway segments, as show in the following table.

Segment	Reference Population Type	Begin Milepoint	End Milepoint	5-Year Crash Total	AADT	Crash Rate
1	4 Lane Divided	4.10	6.84	21	24400	0.17
2	4 Lane Divided	6.88	8.89	7	19100	0.10
3	4 Lane Divided	8.93	11.53	8	19100	0.09
4	4 Lane Divided	11.54	13.23	8	13100	0.20
5	4 Lane Divided	13.24	13.80	5	8900	0.55

Step 6: Calculate the crash rate for each segment

The remaining steps can be automated using the Critical Rate Calculator. It is convenient if the data previously developed is formatted to paste directly into the calculator input cells.

$$(1) \quad MVMT = \frac{AADT \times L \times 365 \times n}{1{,}000{,}000}$$

$$MVMT = Million\ Vehicle - Miles\ of\ Travel$$

$$L = Segment\ Length$$

$$n = Number\ of\ Years$$

$$R = Crash\ Rate = \frac{Number\ of\ Crashes}{MVMT}$$

The MVMT for Segment 5 is calculated as follows:

$$MVMT = \frac{8900 \times (13.80 - 13.24) \times 365 \times 5}{1{,}000{,}000} = 9.10$$

The crash rate for Segment 5 is calculated as follows:

$$R = \frac{5}{9.10} = 0.55$$

Step 7: Calculate the average crash rate for each reference population

$$(2)\ R_a = \frac{\sum_{i=1}(N_{obs,i})}{\sum_{i=1}(MVMT_i)}$$

$$R_a = Average\ segment\ crash\ rate\ for\ reference\ population\ a$$

For the reference population of four-lane divided highway segments, the average crash rate is calculated as follows:

$$R_1 = \frac{21+7+8+8+5}{122.01+70.06+90.63+40.40+9.10}$$

$$= 0.15\ Crashes / MVMT$$

Step 8: Calculate a critical crash rate for each segment

$$(3)\quad R_c = R_a + Confidence\ Level \times \sqrt{\frac{R_a}{MVMT_n}} + \frac{1}{2 \times MVMT_n}$$

$$R_c = Critical\ crash\ rate$$

$$R_a = Weighted\ average\ crash\ rate\ for\ reference\ population$$

The segment 5 critical crash rate is calculated as follows:

$$R_C = 0.15 + 1.645 \times \sqrt{\frac{0.15}{9.10}} + \frac{1}{2 \times 9.10} = 0.41\ Crashes / MVMT$$

Step 9: Compare observed crash rate with critical crash rate

Segments can be ranked and prioritized by the amount the segment crash rate exceeds its critical rate. Mapping the safety priority locations may be desirable for visualization.

Segments identified for further analysis can then be analyzed in more detail by identifying specific crash types, causes, and locations within the segment. For example, for a climbing lane segment, each portion of the climbing lane can be examined—the uphill, crest, and downhill sides.

For Segment 5, the observed crash rate is 0.55, which exceeds Segment 5’s critical crash rate of 0.41.

Step 10: Conclusions

From the analysis, Segment 5 exceeds the critical rate and is a “safety focus” location that needs to be reviewed in more detail. At a minimum, crashes and patterns need to be identified and potential countermeasures indicated. More in-depth HSM predictive analysis could be done for the existing conditions and any later future conditions, which could also include finding the most cost-effective countermeasures. A variation of this method could be to calculate fatal and injury-only crash rates. This could help to identify segments with potential crash severity issues.

4.3.5 Excess Proportion of Specific Crash Types

Recommended Uses	Transportation System Plans and Corridor Plans May be used for existing conditions assessment in development review or project development when predictive methods are unavailable
Data Required	Crash frequency by collision type, and pedestrian and bicyclist involvement Reference populations
More Information	HSM Part B, Chapter 4, Section 4.4.2.10.

The example in HSM Section 4.4.2.9 has a number of errors and the APM spreadsheet should be followed instead.

The Excess Proportion of Specific Crash Types method quantifies the extent to which a specific crash type (the target crash type) is overrepresented at an analysis site, compared to the average representation within a reference population. Sites with significant overrepresentation are "safety focus" locations identified for further analysis. Since it does not require traffic volumes, it is well suited to large-scale regional analysis.

This analysis method can be used to assess any number of target crash types simultaneously. Analysis of collision type, pedestrian involvement, and bicyclist involvement are required. The analyst is encouraged to additionally include other crash characteristics that may be relevant to the project.

The Excess Proportion of Specific Crash Types analysis does not consider the overall frequency or rate of crashes, instead it considers only the types of crashes observed. It is useful for identifying locations that may benefit from targeted countermeasures. This method is best used in conjunction with Critical Crash Rate, as the two methods have complementary strengths and weaknesses.

This method does not allow for estimates of future safety performance and cannot be used for evaluating alternatives.

This method is generally unaffected by RTM. However, the analysis may be of limited usefulness for small study areas having low crash frequencies of the target crash type. For this method to be statistically valid, there needs to be at least five sites in each reference population. In addition, a minimum of two of those sites must have two or more observed crashes of the target crash type. It is preferred to have more than the minimum number of sites.

While the methodology will work with only two sites, caution should be exercised where there are fewer than 5 sites meeting the criteria. If this is an issue and the study area is a portion of an urban area, expanding the study area for this methodology to include the entire urban area should be considered. Also, expanding the number of years of crash records should be considered as long as no significant changes occurred within that timeframe that may have affected crash history.

A spreadsheet is available under safety analysis tools on the ODOT Transportation Development Planning Technical Tools webpage that automates much of the Excess Proportion of Specific Crash Types analysis. The analyst can provide input data in a summary table manually or can use automated extraction macros to analyze a PRC comprehensive crash summary report directly from ODOT. If the automated method is used, the results should be reviewed to ensure that all intersection-related crashes have been included, even those not identified as such in the records. The analyst must clean and format the results as described in the instructions tab before use. A planning level method for calculating critical crash rates using the ODOT Visum Safety Add-In tool is found in Appendix 4A.

Using the automated process still requires cleaning of the data as described in the instructions for the spreadsheet. The cleaning becomes more time consuming when using local roadways.

The general procedure for network screening using the Excess Proportion of Specific Crash Types method is as follows:

1. Identify analysis sites and assign observed crashes (see APM Section 4.2.3)
2. For each analysis site, determine the total crash frequency and the crash frequency for the target crash type
3. For each analysis site, calculate the proportion of total crashes for the target crash type. This is the number of target crashes divided by the number of total crashes (the observed proportion)
4. Establish reference populations and calculate the proportion of total crashes for the target crash type for each reference population (the threshold proportion)
5. Calculate the sample variance and “alpha” and “beta” parameters for the target crash type for each reference population
6. Using these results, at each site calculate the probability of the observed proportion exceeding the threshold proportion
7. Select the limiting probability for the analysis. Only sites with a probability over this limiting probability will be further evaluated. The recommended minimum value is 60%, but higher values can be used to limit the number of sites to a reasonable study size. The limiting probability can be interpreted as the likelihood that the expected long-term observed proportion actually exceeds the expected long-term threshold proportion.

8. For all sites that exceed the limiting probability, calculate the excess proportion of the target crash type by subtracting the threshold proportion from the observed proportion.

This process is repeated for each target crash type. If multiple sets of reference populations are being used (e.g., internal and external), the process is repeated for each set of reference populations.

Results can be used to help diagnose crash trends at each analysis site. For each target crash type, the greater the excess proportion, the greater the likelihood that the site will benefit from a countermeasure that addresses that crash type.

When reporting the results of this method, the following should be included:

- Crash frequency of each target crash type at each analysis site
- Reference population characteristics and summary statistics
- Limiting probability used
- Excess proportion of each target crash type at each analysis site

Example 4-6: HSM Excess Proportions at Intersections

As a part of a corridor safety project, a safety analysis needs to be completed for 25 intersections along Salem Highway (#072). The focus of this study is to increase safety by reducing angle crashes. The project is to reduce crashes at five intersections along the corridor. There are no ADT-capable counts along the corridor and the budget does not allow for taking any new counts. Crashes occurring along the corridor are examined, and those that are determined to be intersection crashes (see APM Section 4.2.3) are summarized by intersection and crash characteristics. The analyst also has the option of importing the length of the highway in question using the spreadsheet to summarize the data. Using the PRC data requires a cleaning process to take place after the data are pulled.

General & Site Information		
Analyst:		LJP
Agency/Company:		ODOT
Date:		1/7/16
Project Name:		APM Example 4-6
Highway Number and Name:		072 Salem Hwy
Mile Points:		All
Crash Years Pulled:		2008-2011

Calculate Probability

Intersection Crash Data Type of Crash						
MP	Reference Pop	Street 1	Street 2	Angle	Rear	Total
1.54	4SG	Salem Parkway	Verda Ln NE	5	14	22
3.16	4SG	Broadway St NE	Salem Parkway	13	17	36
3.55	3ST	Hickory St NE	Liberty St NE	1	0	2
3.56	3ST	Commercial St NE	Hickory St NE	4	0	5
3.57	3SG	Commercial St NE	Pine St NE	3	3	8
3.62	3SG	Liberty St NE	Pine St NE	18	2	20
3.73	3ST	Grove St NE	Liberty St NE	2	1	5
4.24	3ST	Commercial St NE	Hood St NE	7	3	13
4.30	3ST	Commercial St NE	Gaines St NE	2	8	10
4.36	3SG	Commercial St NE	Market St NE	5	13	24
4.79	3SG	Commercial St NE	Union St NE	14	3	22
4.84	3ST	Front St Parkway NE	Union St NE	5	3	13
4.85	3SG	Commercial St NE	Marion St NE	3	8	42
5.39	3SG	Commercial St NE	Ferry St SE	20	7	33
5.43	3SG	Commercial St NE	Trade St SE	8	6	31
5.47	3SG	Ferry St SE	Liberty St SE	8	3	19
5.52	3SG	Liberty St SE	Trade St SE	10	3	18
5.65	3SG	Church St SE	Ferry St SE	5	8	18
5.69	3SG	Church St SE	Trade St SE	3	1	7
5.93	4SG	Pringle Creek Parkwa	Winter St NE	3	6	13
6.20	4SG	Pringle Creek Parkwa	12th St SE	4	10	18
6.77	3SG	Mission St SE	17th St SE	1	23	33
7.52	4SG	Mission St SE	25th St SE	2	50	60
7.92	4SG	Mission St SE	Turner Rd SE	3	36	51
8.26	3SG	Hawthorne Ave SE	Mission St SE	3	68	84

Step 1: Organize Sites into Reference Populations

Intersections are divided into reference populations by traffic control and number of intersection legs (see APM Section 4.3.3). Reference populations are identified by the analyst during the initial steps of the spreadsheet. Each intersection is then placed into a reference population. Use of the digital video log, aerial photos and other electronic sources is suggested for placing intersections into reference population.

The next 6 steps are all done within the spreadsheet. Once the sites have been organized, the spreadsheet can calculate the following steps.

Step 2: Calculate Observed Proportions

$$p_i = \frac{N_{observed,i}}{N_{observed,i(total)}}$$

Where: P_i= Observed proportion at site i

$N_{observed,i}$= number of observed target crashes at i

$N_{observed,i(total)}$= Total number of crashes at i

Step 3: Calculate a Threshold Proportion

$$p^*_i = \frac{\sum N_{observed,i}}{\sum N_{observed,i(total)}}$$

Where: P_i= Threshold proportion

$\Sigma N_{observed,i}$= Sum of observed target crash frequency within the population

$\Sigma N_{observed,i(total)}$= Sum of total observed crash frequency within population

Step 4: Calculate Sample Variance[1]

$$Var(X/Y) = \frac{E^2(x)}{E^2(y)}\left[\frac{Var(X)}{E^2\,(X)} + \frac{Var(Y)}{E^2(Y)} - 2\frac{Cov(X,Y)}{E(X)E(Y)}\right]$$

Where:

$E(X) =$ Mean number of target crashes.

$E(Y) =$ Mean number of total crashes

$Cov(X,Y) =$ using the Excel Covariance function (COVARIANCE.S)

$Var(x) =$ using the Excel Variance function (VAR.S)

$Var(Y) =$ use the Excel Variance function (VAR.S)

If the number of sites with more than two type 'i' crashes is not greater than one, the variance is zero. There are also rare instances where the variance is calculated to be zero. If the variance is zero, the probability cannot be calculated because there is nothing to compare.

Step 5: Calculate the Alpha and Beta Parameters

$$\alpha = \frac{\overline{p^*_i}^2 - \overline{p^*_i}^3 - s^2\left(\overline{p^*_i}^2\right)}{Var(N)}$$

$$\beta = \frac{\alpha}{\overline{p_i^*}} - \alpha$$

Where:

$n_{sites} =$ Total number of sites being analyzed

$p^*_i =$ Threshold proportion

$p_i =$ Observed proportion

$Var(N) = s^2$

[1] This replaces Equation 4-20 in HSM 1st Edition 2010, which was found to have a number of errors.

Step 6: Use the Excel Function to Calculate the Beta Distribution

$$p\left(\frac{p_i > p_i^*}{N_{observed,i}, N_{observed,i(total)}}\right) = 1 - betadist(p_i^*, \alpha + N_{observed,i}, \beta + N_{observed,i\ (total)-N_{observed,i}}$$

$Where:$

$p*_i =$ Threshold proportion

$p_i =$ Observed proportion

$N_{observed(total)} =$ Total number of crashes for site i

$N_{observed,i} =$ Observed target crashes for site i

The resulting number is the probability of a specific crash type being greater than the long-term expected proportion of that crash type at the specified intersection type.

Steps 7 and 8: Choose Limiting Probability and Calculate the Excess Proportion

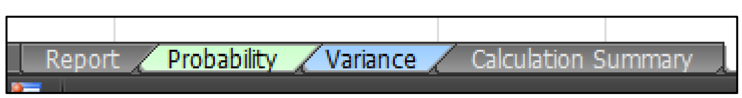
$p_{diff} = p_i - p*_i$

$Where:$

$p*_i = Threshold \quad proportion$

$p_i = Observed \quad proportion$

The spreadsheet first calculates the probability of each crash type exceeding a threshold proportion. . The calculated probability can be interpreted as the likelihood that the long-term expected proportion of a crash type at the intersection is greater than the threshold proportion. In order to calculate the excess proportion the analyst must use engineering judgement to set the limiting probability. According to the HSM "the selection of a limiting probability can vary depending on the probabilities of each specific crash type exceeding a threshold proportion. If many sites have a high probability the limiting probability can be set higher." The default limiting probability of the spreadsheet is 90% but is adjustable. The excess proportion that is calculated is simply the difference in the observed crash proportion and the threshold proportion for the reference population. The data output and a short explanation of the output for this example is shown here. The spreadsheet places the output onto the Probability tab. If the analyst chooses, they can use the "Create Report" button to create a printer friendly summary of the intersections with an excess proportion.



Step 9: Interpreting Results

The Report Tab summarizes those intersections with a probability greater than the set limiting probability. It separates out each crash type and sorts each crash type by descending Excess Proportion. According to the HSM, the greater the excess proportion, "the greater the likelihood that the site will benefit from a countermeasure targeted at the collision type under consideration."

General & Site Informaiton			
Analyst:	LJP	Highway Number and Name:	072, Old Salem Hwy
Agency/Company:	ODOT	Mile Points:	All
Date:	1/26/2016	Crash Years Pulled:	2008-2011
Project Name:	APM Example 4-6	Limiting Probability:	0.9

Notes

Angle Crashes					
					Excess
MP	RefPop	Street 1	Street 2	Probability	Proportion
3.62	3SG	Liberty St NE	Pine St NE	1.00	0.62
4.79	3SG	Commercial St NE	Union St NE	1.00	0.36
5.39	3SG	Commercial St NE	Ferry St SE	1.00	0.32
5.52	3SG	Liberty St SE	Trade St SE	0.99	0.27
3.16	4SG	Broadway St NE	Salem Parkway	1.00	0.21
5.47	3SG	Ferry St SE	Liberty St SE	0.90	0.14

Rear Crashes					
					Excess
MP	RefPop	Street 1	Street 2	Probability	Proportion
4.3	3ST	Commercial St NE	Gaines St NE	1.00	0.49
8.26	3SG	Hawthorne Ave SE	Mission St SE	1.00	0.40
6.77	3SG	Mission St SE	17th St SE	1.00	0.28
7.52	4SG	Mission St SE	25th St SE	1.00	0.17

					Excess
MP	RefPop	Street 1	Street 2	Probabilit	Proportion

Fix Crashes					
					Excess
MP	RefPop	Street1	Street2	Probability	Proportion
6.2	4SG	Pringle Creek Park	12th St SE	0.97	0.08
5.43	3SG	Commercial St NE	Trade St SE	0.95	0.06
4.85	3SG	Commercial St NE	Marion St NE	0.92	0.04
3.16	4SG	Broadway St NE	Salem Parkway	0.93	0.02

Page 1

Head Crashes					
					Excess
MP	RefPop	Street1	Street2	Probability	Proportion
5.43	3SG	Commercial St NE	Trade St SE	1.00	0.04
4.85	3SG	Commercial St NE	Marion St NE	1.00	0.02

NonCol Crashes					
					Excess
MP	RefPop	Street1	Street2	Probability	Proportion
5.43	3SG	Commercial St NE	Trade St SE	0.93	0.06
4.85	3SG	Commercial St NE	Marion St NE	0.95	0.06

SS-M Crashes					
					Excess
MP	RefPop	Street1	Street2	Probability	Proportion
4.85	3SG	Commercial St NE	Marion St NE	1.00	0.07
5.47	3SG	Ferry St SE	Liberty St SE	1.00	0.06
5.43	3SG	Commercial St NE	Trade St SE	1.00	0.05
4.79	3SG	Commercial St NE	Union St NE	1.00	0.04
6.77	3SG	Mission St SE	17th St SE	1.00	0.04
4.36	3SG	Commercial St NE	Market St NE	1.00	0.03

Turn Crashes					
					Excess
MP	RefPop	Street1	Street2	Probability	Proportion
3.57	3SG	Commercial St NE	Pine St NE	0.92	0.17
5.93	4SG	Pringle Creek Park	Winter St NE	0.94	0.11
4.85	3SG	Commercial St NE	Marion St NE	0.91	0.04

Using the Liberty Street and Trade Street intersection, the 0.274 means that there are 27% more observed angle crashes than the calculated threshold for three-leg stop-controlled intersections in this population.

Angle Crashes					
					Excess
MP	RefPop	Street 1	Street 2	Probability	Proportion
3.62	3SG	Liberty St NE	Pine St NE	1.000	0.619
4.79	3SG	Commercial St NE	Union St NE	1.000	0.355
5.39	3SG	Commercial St NE	Ferry St SE	1.000	0.325
5.52	3SG	Liberty St SE	Trade St SE	0.992	0.274
3.16	4SG	Broadway St NE	Salem Parkway	0.998	0.211
5.47	3SG	Ferry St SE	Liberty St SE	0.901	0.140

For this population of intersections, a total of 6 intersections have a greater than 90% chance of having a greater proportion of angle crashes than expected. Intersections with a probability greater than the limiting probability (default of 90%) and an excess proportion of at least 0.10 or a probability greater than the limiting probability and flagged by either the Critical Rate or the SPIS top 10% need to be further investigated.

In looking at the probability of 0.992, this means that there's a 99.2% chance that the long term expected proportion of angle crashes at Liberty St SE and Trade St SE will be greater than the long term expected proportion of angle crashes at 3-legged Signal-Controlled intersections when compared to the rest of this population of intersections. Then the 0.274 in the excess proportion column implies the "the likelihood that the site will benefit from a countermeasure targeted at the collision type under consideration." (HSM 4-58) The greater the excess proportion the greater the likelihood.

Turn Crashes					
MP	RefPop	Street1	Street2	Probability	Excess Proportion
3.57	3SG	Commercial St NE	Pine St NE	0.92	0.17
5.93	4SG	Pringle Creek Park	Winter St NE	0.94	0.11
4.85	3SG	Commercial St NE	Marion St NE	0.91	0.04

The spreadsheet will automatically grey out intersections with an excess proportion of less than 0.10. Unless these intersections are flagged by either the Critical Rate calculation or the SPIS 10% they may dropped from further investigation. Use engineering judgement to determine if there are other factors at the greyed out intersections to determine if they need to be further investigated.

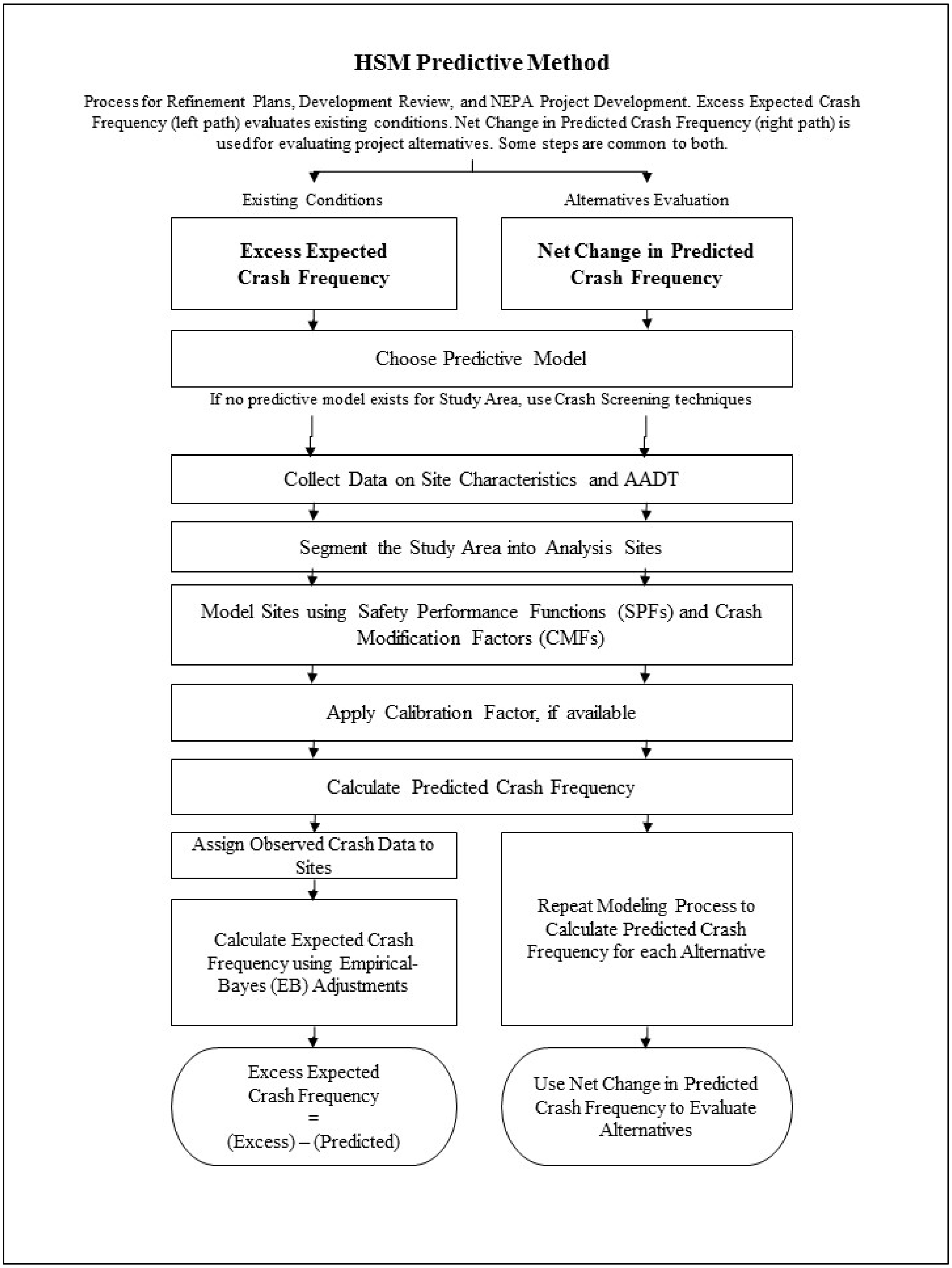
4.4 Predictive Methods

Predictive methods are used for detailed assessment of safety performance at the segment or intersection level. These methods are based on models developed for predicting crash frequency based on geometric and operational characteristics. The results are used to evaluate existing, future and alternative conditions. The HSM discusses predictive methods in more detail in Part C. Predictive methods are recommended for use with more detailed efforts such as Facility Plans, Development Review and Project Development.

Recommended Uses	Facility Plans, Development Review, and Project Development
Data Required	Crash frequency by severity, collision type, and pedestrian and bicyclist involvement [if using the Empirical Bayes (EB) Method] AADT traffic volumes for major and minor roads Pedestrian and bicyclist volumes or estimates Traffic control information Geometric design and roadway details Data requirements vary by predictive model and are discussed in APM Section 4.4.6. Complete HSM Part C data requirements can be found in HSM Part C, Sections 10.4, 11.4, and 12.4
More Information	HSM Part C, Chapters 10-12 ISATe User Manual PLANSAFE User Manual

The flow chart in Exhibit 4-9 gives an overview of the process for HSM Predictive Method.

Exhibit 4-9: HSM Predictive Method



The HSM has introduced a new way of evaluating safety performance of a roadway segment and/or intersections using substantive safety. This new predictive method, detailed in Part C of the HSM, was created from extensive research and analysis rather than relying on design standards (also known as nominal safety). The overall method estimates crashes per year and severity, depending on specific local conditions.

Predictive methods do not require observed crash data to derive quantitative safety evaluations, and therefore can be used with future scenarios or design alternatives that do not yet exist. This allows for a rigorous quantitative safety analysis in circumstances where it was previously impossible, such as development review or National Environmental Policy Act (NEPA) project development. Predictive methods also allow for a fine-grained safety analysis of locations without need of a reference population, which is beneficial for detailed corridor studies where an established safety concern exists.

The HSM predictive equation is located in HSM Section C.4 but is generalized to:
Predicted Crashes =

Safety Performance Function x Crash Modification Factor(s) x Calibration Factor

The HSM makes a distinction between the crashes based only on characteristics (Predicted Crashes) versus characteristics plus local crash history (Expected Crashes). Predicted Crash Frequency is based on the geometric design, traffic control and traffic volumes of the local conditions. The Expected Crash Frequency is the combination of the predicted crash frequency with the historical crash frequency (using the Empirical-Bayes methods).

Expected Crashes =
(EB Weighting Factor x Predicted Crashes) + [(1- EB Weighting Factor) x Observed Crashes]

The generalized process for the predictive method is as follows:

1. Divide the study area into homogenous analysis sites, called "segmentation," for intersections and roadway segments
2. Choose an appropriate predictive model for each site
3. Gather AADT or ADT traffic volumes, roadway characteristics, and traffic control features for each site
4. Characterize each site using SPFs and CMFs from Part C of the HSM
5. Apply calibration coefficients, if available
6. Calculate predicted crashes
7. If using Empirical Bayes (EB) method, assign crashes to sites and calculate expected crashes

This process is then repeated for each alternative to be evaluated. APM Section 0 describes the HSM Predictive Method process in more detail.
Because predictive methods have very specific methodology requirements and are under frequent revision, it is recommended that the analyst consult the appropriate source documentation (including published updates and errata) before beginning a predictive analysis.

The SPFs, CMFs, and calibration coefficients used for a predictive analysis are collectively referred to as the predictive model. Predictive models are specific to a particular type of roadway and require substantial research effort to produce. The HSM provides a methodology for jurisdictions interested in developing local predictive models.

4.4.1 Available Predictive Models and Current Limitations

Predictive models are available to analyze the following types of roadways:

- Rural two-way, two-lane roads (HSM Part C Chapter 10)
- Rural multilane highways (HSM Part C Chapter 11)
- Urban and suburban arterials (HSM Part C Chapter 12)
- Freeways, interchanges, and ramp terminals (ISATe / HSM Supplemental Chapters 18 and 19)

However, the predictive models mentioned above cannot be used in the following situations:

- Highways or arterials with six or more through lanes
- Rural freeways with eight or more through lanes
- Urban freeways with ten or more through lanes
- Interchange designs other than diagonal or partial clover (Parclo)
- Single-point urban, crossing/diverging diamond, or continuous flow interchanges
- Freeway ramp terminals on a one-way street, metered entrance, or roundabout
- All-way stop intersections
- Yield-control intersections
- Rural three-leg signalized intersections

ODOT is in the process of developing additional predictive models for:

- Access management (SPR 720)
- Roundabouts (SPR 733)
- Improved SPFs for signalized intersections (SPR 756)

When predictive models are not available or cannot be used for a location, the analyst should use the screening methods described in APM Section 4.3 and other techniques described in APM Section 4.8 to describe the existing conditions. The effect of potential countermeasures can be estimated using CMFs as described in APM Section 4.6.

If the predictive method can be applied to all but a small portion of the study area, it is acceptable to implement the predictive method where available. In this case, the narrative should explain the reasoning and expected impact of the omission and omitted locations should be clearly identified.

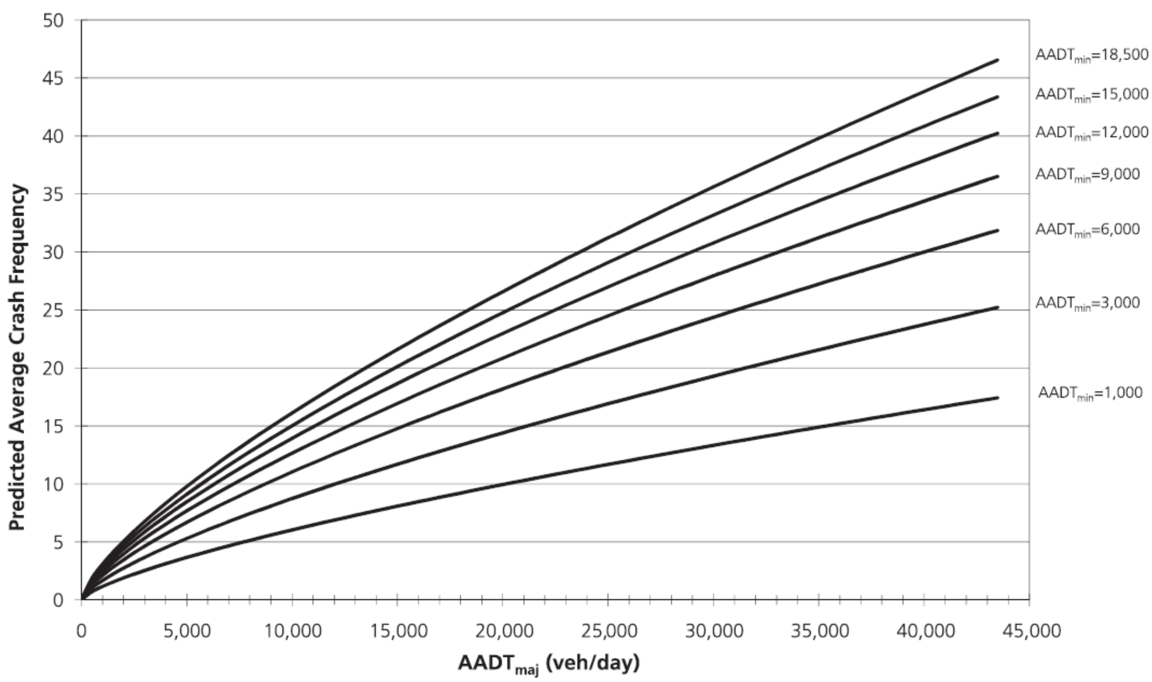
4.4.2 Safety Performance Functions (SPFs)

SPFs are developed using empirical research and regression models. SPFs predict crash frequency for a "base condition" roadway as a function of traffic volume. A unique SPF is developed for significant base condition variations, such as rural multilane highway segments or urban four-leg signalized intersections. CMFs adjust these predictions to account for differences from the SPF base conditions.

SPFs account for the fundamental non-linearity of crash frequency with respect to traffic volumes. An example of a non-linear SPF plot is shown in Exhibit 4-10. Crash rates alone do not capture the non-linearity of crash frequency with respect to traffic volumes. Analysts should be aware that crash rates may be reduced simply by an increase in traffic volumes alone. A reduction in crash rate at higher traffic volumes is often the expected roadway behavior, and does not indicate a fundamental change in underlying safety performance.

For example, consider the function in Exhibit 4-10 for a minor AADT of 1,000 vehicles/day (the bottom curve). At a major AADT of 8,000 vehicles/day, the predicted average crash frequency is approximately five crashes per year. This is a crash rate of 1.52 crashes per MEV. At a major AADT of 35,000 vehicles/day, the predicted average crash frequency is approximately 15 crashes per year. This is a crash rate of 1.14 crashes per MEV. Although the crash rate is lower in the second case, it is due to expected roadway behavior and not a change in underlying safety performance.

Exhibit 4-10: Rural Multilane SPF Four-leg Signalized Intersections (4SG)*



**Source: Fig 11-7 on p 11-23 in Ch. 11 of Volume 2, Part C, 1st Edition HSM*

4.4.3 Crash Modification Factors (CMFs)

CMFs describe the effect of specific treatments on the estimated crash frequency. In the HSM predictive method, CMFs adjust the "base condition" prediction of an SPF to account for additional characteristics of the local site. HSM predictive models require specific CMFs (from HSM Part C) that must be included in the model and developed in conjunction with the SPF. The required CMFs are further described in the HSM chapters for each predictive model and are included in the model spreadsheets or other computational tools.

In general, a CMF is multiplied with the crash frequency predicted by the SPF to account for any change in predicted crashes from the base condition. A CMF greater than 1.0 implies an increase in crashes, while a CMF less than 1.0 would have a decrease in crashes. A CMF equal to 1.0 means no change is expected.

For example a base model SPF may have four-foot shoulders but the actual site has eight-foot shoulders. In this example, one would expect a crash reduction because the roadway has a wider shoulder, so the CMF should be less than one. The actual value of the CMF will vary depending on facility type.

CMFs can also be used to estimate the effects of countermeasures or changes in conditions on observed crash frequency, independent from predictive models. CMFs for general use are included in the HSM Part D and CMF Clearinghouse, among other sources. These CMFs are based on independent research and are reviewed and approved by the HSM Task Force. There is a considerably wider selection of CMFs that can be used this way.

CMFs are an active area of research, and the best available CMF for a situation may change frequently. See APM Section 4.6 Countermeasure Selection and Evaluation for further information on selecting and applying CMFs to evaluate countermeasure effectiveness.

> *Part C CMFs were developed in conjunction with the model development and are unique to the HSM Predictive Method.*
>
> *CMFs in Part C should only be used with Part C SPFs and the HSM Predictive Method.*

The term crash reduction factor (CRF) was part of the early discussions of predictive crash methods (and is still frequently used in other literature) but was dropped in preference to the modification factors. CRFs relate to CMFs in the following way (see Section 4.6 for more information on CRFs):

CMF = 1- (CRF/100), so a CMF of 0.2 is a CRF of 80%.

4.4.4 Local Calibration Coefficients

Calibration coefficients are developed locally by comparing predictive results to locally observed results. These adjust the prediction to account for differences in geography, crash reporting, enforcement policy, and driver behavior between the general models provided in the HSM and the location of application. Oregon calibration coefficients are not yet available for all predictive models. In cases where a state- or region-specific SPF has been produced, calibration coefficients are not applied.

In Oregon, crash reporting is a driver responsibility with some enhancement for enforcement. This means that it is more likely to have a police report if there was a serious injury than for a crash involving property damage only. Oregon's required reporting threshold is typically higher than many other states (at $1,500) so many minor crashes are not reported and do not show up in the crash data. Because of these reporting requirements, Oregon conditions data should not be directly related to national averages or adjacent states (i.e., Washington), which have different reporting thresholds.

ODOT has created Oregon calibration factors for some of the HSM Part C models in the report, *Calibrating The Future Highway Safety Manual Predictive Methods For Oregon State Highways*. This report covers the Rural Two-Lane models, Rural Multilane models, and Urban/Suburban Arterial models for both segments and intersections. The ODOT Driveway Safety models do not require calibration, because they were developed using local Oregon data. PLANSAFE does not require calibration by the analyst, because the program self-calibrates using provided data. An attempt was made to calibrate the freeway and interchange Part C models but was unsuccessful. Results from the Enhanced Interchange Safety Analysis Tool (ISATe) must be reported uncalibrated, as described below.

The locally-derived calibration factors listed in Exhibit 4-11 adjust total predicted crash frequencies to a value that is representative of Oregon conditions. Predicted crashes are multiplied by the calibration factor to determine the calibrated predicted crashes. For example, if an uncalibrated model estimated 10 predicted crashes per year and had a local calibration factor of 0.50, the locally-calibrated result would be five predicted crashes per year.

Exhibit 4-11: Locally-Derived Oregon HSM Calibration Factors

Facility Type		Calibration Factor
Segments		
Rural Two-Lane		
R2	2-lane undivided	0.74
Rural Multilane		
MRU	Undivided	0.37
MRD	Divided	0.77
Urban/Suburban Arterials		
U2U	2-lane undivided	0.62
U3T	3-lane with TWLTL	0.81
U4D	4-lane divided	1.41 / 0.64 *
U4U	4-lane undivided	0.63
U5T	5-lane with TWLTL	0.64
Intersections		
Rural Two-Lane		
R3ST	3-leg, minor stop	0.31
R4ST	4-leg, minor stop	0.31
R4SG	4-leg, signalized	0.45
Rural Multilane		
MR3ST	3-leg, minor stop	0.15
MR4ST	4-leg, minor stop	0.39
MR4SG	4-leg, signalized	0.15
Urban and Suburban Arterials		
U3ST	3-leg, minor stop	0.35
U4ST	4-leg, signalized	0.45
U3SG	3-leg, signalized	0.73
U4SG	4-leg, signalized	1.05

* Value of 1.41 based on small sample size and geometric designs no longer used. Value of 0.64 should be used for all future new designs.

Source: Calibrating The Future Highway Safety Manual Predictive Methods For Oregon State Highways

Additional locally derived severity and crash type distributions are included in the calibration report linked to the above table. The best way to ensure that all appropriate Oregon calibration factors are accounted for is to use or refer to the HSM Spreadsheets that are pre-filled with Oregon calibration factors. These spreadsheets include tables with all recommended locally derived calibration factors and distributions, and are described in APM Section 4.4.13 below.

Where an Oregon calibration factor does not exist, the results of the predictive analysis should only be used for relative comparisons such as net difference in predicted crashes or percent change in predicted crashes. Uncalibrated predicted crashes should be identified as such and reported separately from calibrated predicted crashes or expected crashes.

4.4.5 Excess Expected Crash Frequency using Empirical Bayes (EB) Adjustments to Include Historical Crash Data

The HSM makes a distinction between the crashes based only on characteristics (predicted crashes) versus characteristics plus local crash history (expected crashes). Predicted crash frequency is based on the geometric design, traffic control and traffic volumes of the local conditions. The expected crash frequency is the combination of the predicted crash frequency with the historical crash frequency (using the EB Method). The benefit of the EB method is that it accounts for the RTM error, which is the natural fluctuation of crashes that occurs over the years independent of the contributing factors the analysis is trying to review. The EB method results in a more reliable estimate of crash frequency, while still accounting for unique site characteristics that influence safety but are not included in the predictive models.

The EB method requires a calibrated prediction model (with overdispersion factor) and substantial similarity between the analysis period for which crash data exist and the analysis period being used for the predictive method.

EB Method Can Be Used	EB Method Cannot Be Used
• Existing Conditions • Future conditions with traffic volume changes only • Future conditions with minor geometric changes (i.e., wider shoulders) or addition of turn or passing lanes.	• Uncalibrated predictive models, such as ISATe • Where no crash data from any time period are available • Future conditions with entirely new roadways where none existed before • Future conditions modifying existing roadways that include major alignment changes, the addition of through lanes, or a change in traffic control devices

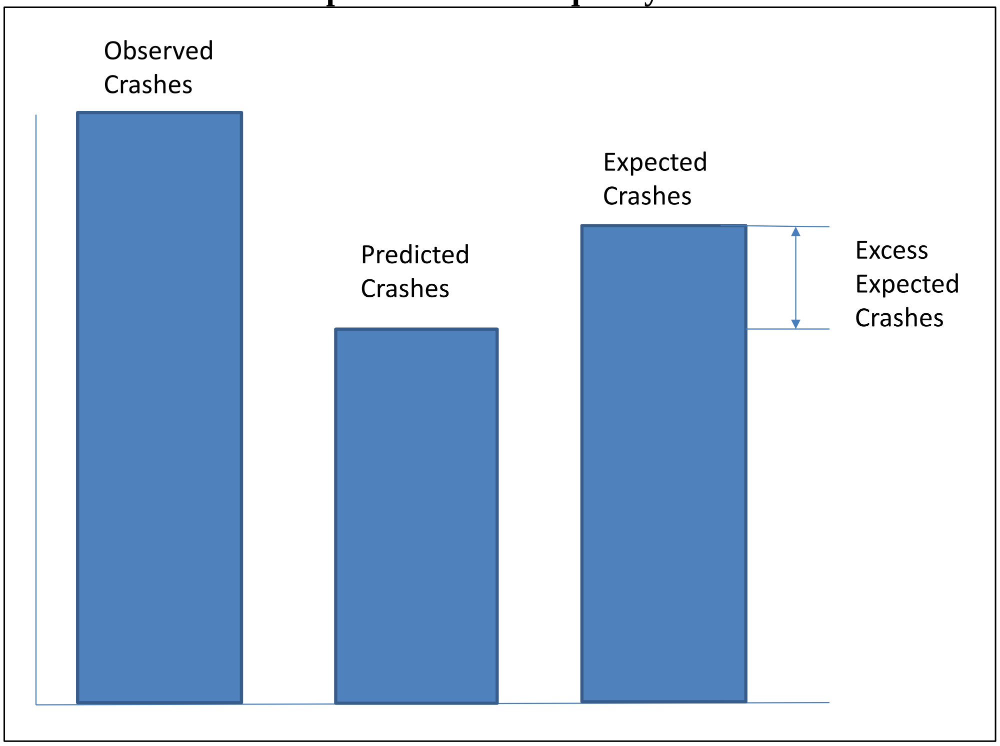
The EB method can be performed at a site-specific level or a project level. Site-specific EB adjustments are done for each analysis site and require the analyst to associate each crash with a specific site. Project EB corrections are done using crash data aggregated over all analysis sites and do not require crashes to be associated with a specific site. The site-specific EB method is preferred and should be used in most situations, since crash data available through ODOT are geocoded and can be associated with specific analysis sites.

When the EB method is performed, the result is an expected crashes value. This value is reported using the “Excess Expected Average Crash Frequency,” defined as:

Excess Expected Average Crash Frequency = Expected Crashes – Predicted Crashes

An example of Excess Expected Average Crash Frequency is illustrated in Exhibit 4-12. In this example, Excess Expected Crash Frequency is a positive number, meaning the long term average crash frequency at this site is greater than for comparable sites. The site should be investigated further for potential safety countermeasures. In other cases, Excess Expected Crash Frequency may be a negative value, meaning the long term average crash frequency at the site is less than for comparable sites.

Exhibit 4-12: Excess Expected Crash Frequency



APM Section 4.2.3 contains guidance for assigning observed crashes to an analysis site and determining if they are intersection-related or roadway segment-related.

Instructions and examples for using the EB method are provided in the HSM Part C, Appendix A, Section A.2.

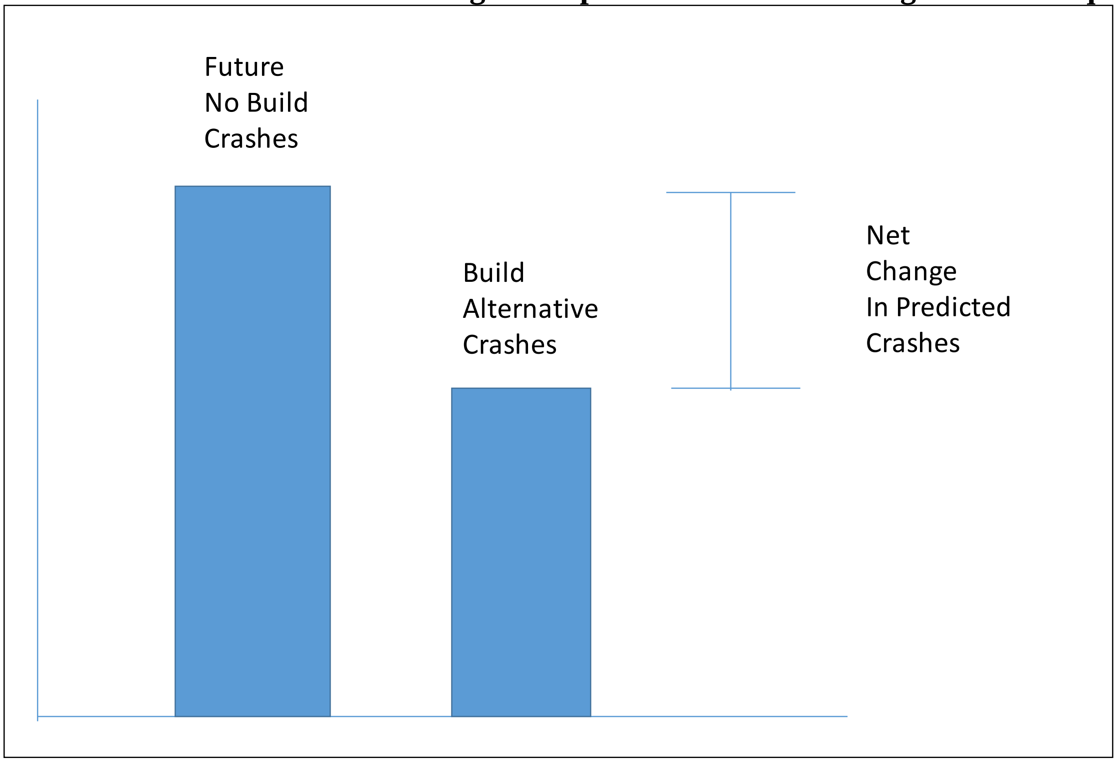
4.4.6 Net Change in Expected or Predicted Crashes

Alternatives should be compared using the net change in expected or predicted crashes, by severity, if possible. Expected crashes can be determined for alternatives if the only changes are in AADT (see APM Section 4.4.5 for more information). If expected crashes can be calculated for all alternatives under consideration, use net change in expected crashes. Otherwise, use net change in predicted crashes. Calculate expected or predicted crashes for each alternative, including the no-build alternative. Subtract the expected or predicted crash frequency of the no-build alternative from each build alternative to determine the net change in expected or predicted crashes for the build alternative. Any alternative with a negative net change in expected or predicted crashes indicates a modeled decrease in crashes. Alternatives with positive net change in expected or predicted crashes indicate a modeled increase in crashes.

Net Change In Expected/Predicted Average Crash Frequency = Expected/Predicted No Build Crash Frequency - Expected/Predicted No Build Crash Frequency

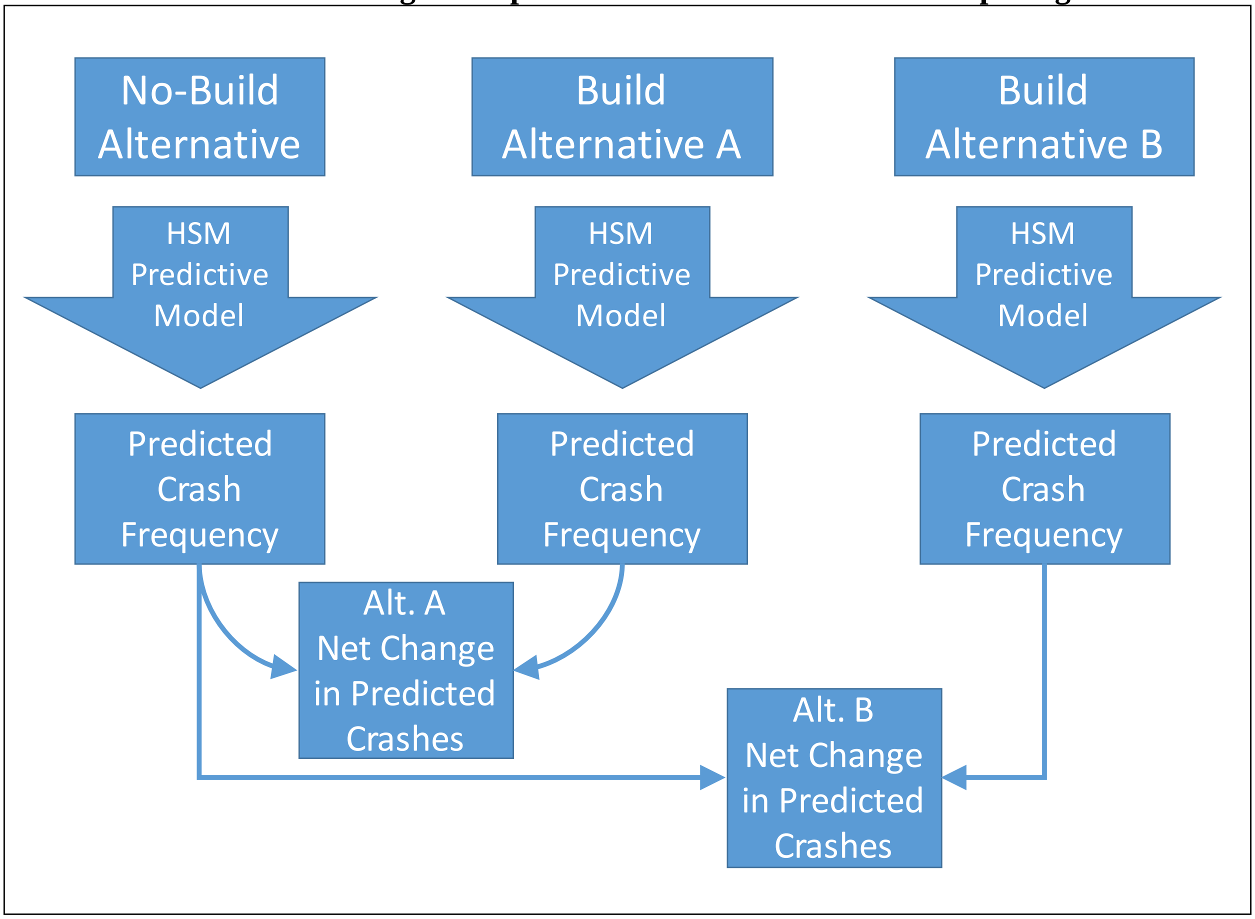
An example of Net Change in Expected/Predicted Average Crash Frequency is illustrated in Exhibit 4-13. In this example, the Net Change in Predicted Crashes represents a reduction in crash frequency, meaning that in comparison to the No Build, the Build Alternative will reduce the long term average crash frequency at this site by the amount of the Net Change.

Exhibit 4-13: Net Change in Expected/Predicted Average Crash Frequency



The Net Change in Crash Frequency can also be used to compare alternatives with each other, as shown in Exhibit 4-14.

Exhibit 4-14: Net Change in Expected/Predicted Crashes for Comparing Alternatives



4.4.7 Data Needs and Sources

HSM predictive methods require a substantial amount of roadway, geometric design, and traffic control data. Primary data are routinely collected during a project or are available in the ODOT TransGIS Database or the ODOT State Highway Inventory Reports. The ODOT Transportation Development Trans Data Portal provides a quick guide to many ODOT data sources. Other data will need to be collected through a desk survey using satellite imagery and street-level imagery (such as the ODOT Digital Video Log or Google StreetView) or through field visits. Some information may be infeasible to collect and can be estimated using the resources in this section.

Each HSM predictive model requires different data to estimate crashes, depending on the SPF base condition and available model CMFs. The analyst should consult the appropriate model methodology reference text before beginning an analysis to determine the data needed for each model and for the required data collection procedures.

- Rural two-lane roads (HSM Part C Chapter 10, Section 10.4)
- Rural multilane roads (HSM Part C Chapter 11, Section 11.4)
- Urban and suburban arterials (HSM Part C Chapter 12, Section 12.4)
- Freeways and interchanges (ISATe User Manual, Chapter 2, Page 19)

Based on sensitivity tests performed with the HSM models, accurate information on AADT volume data are the most important input. This includes minor street AADT for all models and pedestrian volumes for urban intersection models.

If minor street AADT is not available from the local jurisdiction, the analyst should provide a best estimate. If AADTs will be estimated, consult with TPAU to determine the appropriate methodology before beginning an analysis. In some situations, minor AADT may be inferred from nearby volumes or travel demand models may help provide an estimate. The major road AADT to minor road AADT ratio of similar intersections is another possible starting point for estimation. ITE trip generation rates may be appropriate for isolated minor roads.

If daily pedestrian volumes are not available, HSM Table 12-15 provides guidance on estimating daily pedestrian volumes based on general level of pedestrian activity.

Exhibit 4-15 is meant to give a general sense of the site characteristic data required for the HSM predictive method, but it is not intended to be exhaustive.

Exhibit 4-15: Data Needed for HSM Predictive Methods

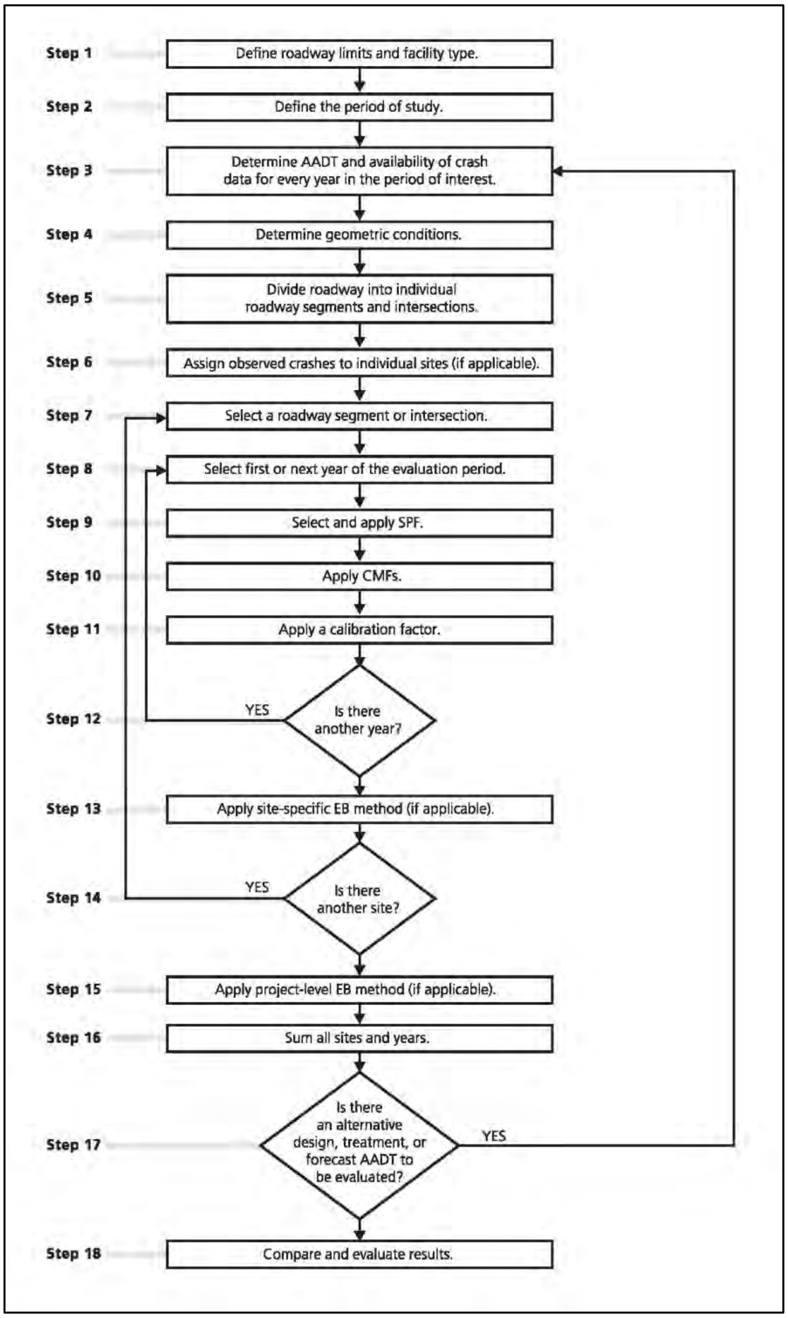
Information Needed	Data Source
Roadway Segments	
Length of Segment	TransGIS or Imagery
AADT	TransGIS or Inventory Report or Project
Lane Width	TransGIS or Inventory Report or Imagery
Shoulder Width and Type	TransGIS or Inventory Report or Imagery
Horizontal Curve Dimensions	TransGIS or Inventory Report or Imagery
Grade	Imagery or Inventory Report or Elevation Maps
Driveway Density and Type	TransGIS Imagery
Centerline and Edgeline Rumble Strips	Imagery
Passing Lanes	TransGIS or Inventory Report or Imagery
Two-Way Left-Turn Lanes	TransGIS or Inventory Report or Imagery
Roadside Hazard Rating	Imagery
Segment Lighting	Imagery
Automated Speed Enforcement	Imagery
Median Type and Median Width	Imagery or Inventory Report
Sideslope	Imagery
Type of On-street Parking	Imagery
Roadside Fixed Object Density and Offset	Imagery
Actual or Posted Speed	TransGIS or Imagery
Intersections	
Number of Intersection Legs	TransGIS or Imagery
Type of Traffic Control	Imagery
Intersection Skew Angle	TransGIS or Imagery
Approaches with Right-Turn Lanes	Imagery
Approaches with No Right-Turn on Red	Imagery
Approaches with Left-Turn Lanes	Imagery
Approaches with Left-Turn Signal Phasing	Imagery or Signal Timings
Intersection Lighting	Imagery
Most Traffic Lanes Crossed by Pedestrians	Imagery
Number of Bus Stops within 1,000 feet	TransGIS or Imagery
Presence of Schools within 1,000 feet	Imagery or Other Point-of-Interest Reference
Number of Alcohol Sales Locations within 1,000 feet	Imagery or Other Point-of-Interest Reference
Presence of Red Light Camera	Site Visit

4.4.8 Predictive Method Process

The HSM Predictive Method process consists of 18 steps that result in an estimation of crash frequency for a project area. Each analysis site is considered individually for each year of analysis. These individual results are summed to determine the study area total. This process is repeated for each alternative to be evaluated. Exhibit 4-16 illustrates this process graphically. HSM Section C.5 describes these steps:

- **Step 1:** Define the limits of the roadway and facility types in the study network, facility, or site for which the expected average crash frequency, severity, and collision types are to be estimated
- **Step 2:** Define the time period of interest
- **Step 3:** For the study period, determine availability of AADT and (if using the EB Method) observed crash data
- **Step 4:** Determine geometric design features, traffic control features, and site characteristics for all sites in the study network
- **Step 5:** Divide the roadway network or facility under consideration into individual roadway segments and intersections, which are referred to as sites
- **Step 6:** Assign observed crashes to the individual sites (if using the EB Method)
- **Step 7:** Select the first or next individual site in the study network. If there are no more sites to be evaluated, go to Step 15.
- **Step 8:** For the selected site, select the first or next year in the period of interest. If there are no more years to be evaluated for that site, proceed to Step 15.
- **Step 9:** For the selected site, determine and apply the appropriate SPFs for the site's facility type and traffic control features
- **Step 10:** Multiply the result obtained in Step 9 by the appropriate CMFs to adjust the predicted average crash frequency to site-specific geometric design and traffic control features
- **Step 11:** Multiply the result obtained in Step 10 by the appropriate calibration factor
- **Step 12:** If there is another year to be evaluated in the study period for the selected site, return to Step 8. Otherwise, proceed to Step 13.
- **Step 13:** Apply site-specific EB Method (if using the EB Method)
- **Step 14:** If there is another site to be evaluated, return to Step 7, otherwise, proceed to Step 15
- **Step 15:** Apply the project level EB Method (if the site-specific EB Method is not applicable)
- **Step 16:** Sum all sites and years in the study to estimate total crashes or average crash frequency for the network
- **Step 17:** Determine if there is an alternative design, treatment, or forecast AADT to be evaluated. Steps 3 through 16 of the predictive method are repeated for each alternative.
- **Step 18:** Evaluate and compare results

Exhibit 4-16: HSM Predictive Method Process



Source: HSM Section C.5, Figure C-2

4.4.9 Study Area Segmentation

Segmentation, step 5 in the predictive method, is the process of dividing the study area into smaller analysis sites that are each relatively uniform in character. Sites should be comprehensive, collectively covering all of the study area. When a segment ends or begins at an intersection, the segment lengths are measured from the center of the intersection.

Each intersection with a public road should be an individual site, even if it is a minor local road. Roadway segments, such as highways, streets, freeways, and ramps, should be divided into individual sites at a minimum at every intersection. Segments between intersections may need to be split into multiple sites, so that each site is uniform in character for the predictive model. Segments do not need to be split at driveways. Limiting the minimum segment length to 0.10 miles is appropriate to keep the analysis manageable.

Segments should always be split into distinct sites where any of the following change:

- Number of through lanes
- AADT
- Land use (rural/urban)
- Presence or type of median
- Posted speed limit

Continuous values such as lane width, shoulder width, or grade are generally rounded to some extent for the purposes of segmenting. The unrounded values are then averaged (weighting by length) for the predictive analysis. Some characteristics, such as the presence of curves and rumble strips, may not require the creation of a new site in some models and can instead be expressed as a percent of the site with that characteristic.

Segmenting requirements vary by predictive model, and the analyst is referred to the appropriate predictive model methodology reference text for further details before starting an analysis.

- Rural two-lane roads (HSM Part C Chapter 10, Section 10.5)
- Rural multilane roads (HSM Part C Chapter 11, Section 11.5)
- Urban and suburban arterials (HSM Part C Chapter 12, Section 12.5)
- Freeways and interchanges (ISATe User Manual, Chapter 2, Page 34)

4.4.10 Multiple-Year Analysis

The predictive method requires each year of the analysis period to be modeled individually. For future alternatives, a single year of analysis is adequate. For historical analysis, three to five years of crash data and matching site characteristics should be used. Often, most site characteristics do not change over the analysis period aside from the AADT.
When AADT is not available for every year in the analysis period, the following rules can be used for estimating AADT:

- For years between two known AADT values, use a linear interpolation
- For years after the latest known AADT value, use the latest known AADT value
- For years before the first known AADT value, use the first known AADT value

---

Example 4-7: HSM Predictive Analysis

<u>Predictive Method for Urban and Suburban Arterial Intersections</u>

The predictive method for urban and suburban arterials is described in the 1st edition HSM, Volume 2, Chapter 12, Section 12.4. The HSM lists an 18-step process for the urban/suburban arterial predictive method.

Note: The predictive method example is based on Adams St. and 128th St., which is a four-leg signalized intersection. The example is done only for the year 2010. A full analysis would analyze each year in the study period, from 2006-2010.

Step 1: Determine the limits of the roadway and facility types that will be included in the study network, facility, or site
Step 2: Define the time period of interest
Step 3: Determine AADT and availability of crash data for each year in the period of interest
Step 4: Determine geometric design features, traffic control features, and site characteristics for all sites in the study network

Data for the case study of the Adams St. and 128th St. intersection are shown in the following table for use in this example problem. The figure below shows these data in the HSM spreadsheet tool.

Data Requirement	Intersection of Adams St. and 128th St.
AADT (vehicles/day)	AADTs were found through the local agency's traffic counts information page. Major street AADT is 23,150 vpd, and minor street AADT is 12,300 vpd.
Pedestrian volumes	The pedestrian volume was estimated as 50 pedestrians per day. Table 12-15 of the 1st edition HSM, Volume 2, Chapter 12 gives estimates of pedestrian crossing volumes based on the general level of pedestrian activity.
Number of intersection legs	The intersection has four legs.
Type of traffic control	The intersection is signalized.
Number of approaches with a left-turn lane	All four of the approaches to the intersection have left-turn lanes.
Number of approaches with left-turn signal phasing and type of left-turn signal phasing for each movement	All four of the left-turn lanes have protected/permissive phasing.
Number of approaches with a right-turn lane	None of the approaches to the intersection have right-turn lanes.
Number of approaches with right-turn-on-red operation prohibited	None of the approaches have prohibited right-turn-on-red movements.
Presence/absence of intersection lighting	There is intersection lighting.
Presence/absence of Red Light Cameras	No red light cameras.
Maximum number of traffic lanes to be crossed by a pedestrian in any crossing maneuver at the intersection considering the presence of refuge islands	The maximum number of lanes to be crossed by a pedestrian is five lanes.
Number of bus stops within 1,000 feet of the intersection	There are four bus stops located within 1,000 feet of the intersection.
Presence of schools within 1,000 feet of the intersection	There are no schools located within 1,000 feet of the intersection.
Number of alcohol sales establishments that sell alcohol within 1,000 feet if the intersection	There are four different establishments that sell alcohol within 1,000 feet of the intersection.

HSM Spreadsheet Data Input

Worksheet 2A -- General Information and Input Data for Urban and Suburban Arterial Intersections			
General Information		**Location Information**	
Analyst	TPAU	Roadway	Adams Street
Agency or Company	ODOT	Intersection	128th Street
Date Performed	01/01/11	Jurisdiction	
		Analysis Year	2010
Input Data		**Base Conditions**	**Site Conditions**
Intersection type (3ST, 3SG, 4ST, 4SG)		--	4SG
AADT $_{major}$ (veh/day)	$AADT_{MAX}$ = 67,700 (veh/day)	--	23,150
AADT $_{minor}$ (veh/day)	$AADT_{MAX}$ = 33,400 (veh/day)	--	12,300
Intersection lighting (present/not present)		Not Present	Present
Calibration factor, C_i		1.00	1.05
Data for unsignalized intersections only:		--	
Number of major-road approaches with left-turn lanes (0,1,2)		0	
Number of major-road approaches with right-turn lanes (0,1,2)		0	
Data for signalized intersections only:		--	
Number of approaches with left-turn lanes (0,1,2,3,4) [for 3SG, use maximum value of 3]		0	4
Number of approaches with right-turn lanes (0,1,2,3,4) [for 3SG, use maximum value of 3]		0	0
Number of approaches with left-turn signal phasing [for 3SG, use maximum value of 3]		--	4
Type of left-turn signal phasing for Leg #1		Permissive	Protected / Permissive
Type of left-turn signal phasing for Leg #2		--	Protected / Permissive
Type of left-turn signal phasing for Leg #3		--	Protected / Permissive
Type of left-turn signal phasing for Leg #4 (if applicable)		--	Protected / Permissive
Number of approaches with right-turn-on-red prohibited [for 3SG, use maximum value of 3]		0	0
Intersection red light cameras (present/not present)		Not Present	Not Present
Sum of all pedestrian crossing volumes (PedVol) -- Signalized intersections only			50
Maximum number of lanes crossed by a pedestrian (n_{lanesx})		--	5
Number of bus stops within 300 m (1,000 ft) of the intersection		0	4
Schools within 300 m (1,000 ft) of the intersection (present/not present)		Not Present	Not Present
Number of alcohol sales establishments within 300 m (1,000 ft) of the intersection		0	4

Step 5: Divide roadway into individual roadway segments and intersections

- Since there is only one site to be analyzed, this step is unnecessary.

Step 6: Assign observed crashes to individual sites

- It is useful to summarize the observed crashes to match with the crash prediction categories used by the predictive model. The Urban/Suburban Intersection model predicts multiple vehicle crashes and single vehicle crashes separately using different SPFs. It also provides predicted crash breakdowns by severity, fatal and injury (FI) or property damage only (PDO). Vehicle-bicycle crashes and vehicle-pedestrian crashes are independently identified.
- A total of nine observed crashes in 2010 were assigned to this site:
  - 4 FI crashes
  - 5 PDO crashes
  - 8 multiple-vehicle crashes
  - 1 single-vehicle crash
  - 1 vehicle-bicycle crash (also considered a single-vehicle crash)

Step 7: Select a roadway segment or intersection

- The intersection of Adams St. and 128th St. has been selected for further study.

Step 8: Select first or next year of the evaluation period

- Year 2010 is being considered in this example.

Step 9: Select and apply SPF

- SPFs relevant to Urban and Suburban Arterial Intersections are found in Section 12.6.2 of the 1st edition, HSM, Volume 2.
- Multiple-vehicle collisions: 6.884 total, 2.284 FI, 4.601 PDO
- Single-vehicle collisions: 0.435 total, 0.113 FI, 0.322 PDO
- Vehicle-bicycle collisions (predicted as a fixed portion of all vehicle collisions): 0.070 total, 0.070 FI, 0 PDO
- Vehicle-pedestrian collisions: 0.029 total, 0.029 FI, 0 PDO

Step 10: Apply CMFs

- Intersection left-turn lanes: CMF_{1i} (Table 12-24 in the 1st edition, HSM, Volume 2, Chapter 12)
  - For the intersection of Adams St. and 128th St., CMF_{1i} = 0.66
- Intersection left-turn signal phasing: CMF_{2i} (Table 12-25 in the 1st edition, HSM, Volume 2, Chapter 12)
  - For the intersection of Adams St. and 128th St., CMF_{2i} = 0.96
- Intersection right-turn lanes: CMF_{3i} (Table 12-26 in the 1st edition, HSM, Volume 2, Chapter 12)
  - For the intersection of Adams St. and 128th St., CMF_{3i} = 1.00
- Right-turn-on-red: CMF_{4i}
  - $CMF_{4i} = 0.98^{n_{prohib}}$

  Where:

  CMF_{4i} = crash modification factor for the effect of prohibiting right turns on red on total crashes

  n_{prohib} = number of signalized intersection approaches for which right-turn-on-red is prohibited

  - For the intersection of Adams St. and 128th St., CMF_{4i} =1.00
- Lighting: CMF_{5i}
  - $CMF_{5i} = 1 - 0.38 \times p_{ni}$

  Where:

  CMF_{5i} = crash modification factor for the effect of intersection lighting on total crashes

p_{ni} = proportion of total crashes for unlighted intersections that occur at night Default values for p_{ni} can be found in Table 12-27 in the 1st edition HSM, Volume 2, Chapter 12. It is recommended to replace the default values with locally derived values.

- For the intersection of Adams St. and 128th St., CMF_{5i} =0.95 using a locally-derived value (p_{ni} =0.154)

- Red-light cameras: CMF_{6i}
  - CMF_{6i} is based on the presence of red-light cameras. The base condition is their absence. There are no red-light cameras at the intersection of Adams St. and 128th St., so for the example CMF_{6i} =1.00. The formula for CMF_{6i} is (12-37) found in the 1st edition HSM, Volume 2, Chapter 12.

CMF_{1i}	CMF_{2i}	CMF_{3i}	CMF_{4i}	CMF_{5i}	CMF_{6i}	CMF_{i}
0.66	0.96	1.00	1.00	0.95	1.00	0.60

The CMFs above apply to vehicle crashes and have a combined value of 0.60.
The CMFs below apply only to vehicle-pedestrian crashes and have a combined value of 4.65.

- Bus stops: CMF_{1p} (Table 12-28 in the 1st edition, HSM, Volume 2, Chapter 12)
  - There are more than three (two bus stops at 128th and Adams St., one a block north on 128th and one half a block west on Adams St.) bus stops within 1,000 feet of Adams St. and 128th St., so CMF_{1p} =4.15.

- Schools: CMF_{2p} (Table 12-29 in the 1st edition, HSM, Volume 2, Chapter 12)
  - There are no schools within 1,000 feet of Adams St. and 128th St., so CMF_{2p} =1.00.

- Alcohol sales establishments: CMF_{3p} (Table 12-30 in the 1st edition, HSM, Volume 2, Chapter 12)
  - There are four (two stores, one restaurant, and one gas station) alcohol sales establishments within 1,000 feet of Adams St. and 128th St., so CMF_{3p} =1.12.

CMF_{1p}	CMF_{2p}	CMF_{3p}	CMF_{p}
4.15	1.00	1.12	4.65

Step 11: Apply a calibration factor

- The recommended Oregon Highway Safety Manual Calibration factor for urban and suburban four-way signalized intersections is 1.05.
- The resulting predicted crash values are:
    - 4.4 predicted multiple-vehicle crashes
    - 0.3 predicted single-vehicle crashes
    - 0.14 predicted pedestrian crashes
    - 0.07 predicted bicycle crashes

Step 12: Is there another year? If yes, return to step 8. (Only one year is included in this example.)

Step 13: Apply site-specific EB Method (This step is not included in this example.)

- Observed crashes are matched with the output of each SPF, and the EB Method is performed using the overdispersion parameter of the SPF.
- The following graphic shows the results of the site-specific EB Method calculation for vehicle crashes using the HSM Spreadsheets.

HSM Spreadsheet Results with Site-Specific EB Method

Worksheet 3A -- Predicted Crashes by Severity and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Collision type / Site type	Predicted average crash frequency (crashes/year)			Observed crashes, $N_{observed}$ (crashes/year)	Overdispersion Parameter, k	Weighted adjustment, w	Expected average crash frequency,
	$N_{predicted}$ (TOTAL)	$N_{predicted}$ (FI)	$N_{predicted}$ (PDO)			Equation A-5 from Part C Appendix	Equation A-4 from Part C Appendix
INTERSECTIONS							
Multiple-vehicle							
Intersection 1	4.367	1.450	2.918	8	0.390	0.370	6.656
Single-vehicle							
Intersection 1	0.276	0.071	0.204	1	0.360	0.910	0.341
COMBINED (sum of column)	4.643	1.521	3.122	9	--	--	6.997

Worksheet 3B -- Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

(1)	(2)	(3)
Site Type	N_{ped}	N_{bike}
INTERSECTIONS		
Intersection 1	0.141	0.073
COMBINED (sum of column)	0.141	0.073

Worksheet 3C -- Site-Specific EB Method Summary Results for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)
Crash severity level	$N_{predicted}$	N_{ped}	N_{bike}	$N_{expected (VEHICLE)}$	$N_{expected}$
Total	$(2)_{COMB}$ from Worksheet 3A	$(2)_{COMB}$ from Worksheet 3B	$(3)_{COMB}$ from Worksheet 3B	$(8)_{COMB}$ Worksheet 3A	(3)+(4)+(5)
	4.6	0.1	0.1	7.0	7.2
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 3A	$(2)_{COMB}$ from Worksheet 3B	$(3)_{COMB}$ from Worksheet 3B	$(5)_{TOTAL} * (2)_{FI} / (2)_{TOTAL}$	(3)+(4)+(5)
	1.5	0.1	0.1	2.3	2.5
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 3A	--	---	$(5)_{TOTAL} * (2)_{PDO} / (2)_{TOTAL}$	(3)+(4)+(5)
	3.1	0.0	0.0	4.7	4.7

Step 14: Is there another site? If yes, return to step 7. (This step is not included in this example.)

Step 15: Apply project-level EB Method (This step is not included in this example.)

Step 16: Sum all sites and years

- Results from all sites and all years would be summed together for the study area total.
- For this intersection, the calibrated predictive method results for 2010 are:
    - 7.2 expected crashes
    - 2.5 expected fatal and injury crashes
    - 4.7 expected property damage only crashes
    - 0.14 predicted pedestrian crashes (assumed fatal or injury)
    - 0.07 predicted bicycle crashes (assumed fatal or injury)
- Results should ultimately be reported as excess expected, which is the expected crash value minus the predicted crash value.
    - 2.6 excess expected crashes
    - 1.0 excess expected fatal and injury crash
    - 1.6 excess expected property damage only crashes

Step 17: Repeat for alternative designs, treatments, or forecast AADT to be evaluated

- The predictive method is repeated for each alternative to be evaluated; however, no alternatives are included in this example.
- Countermeasures that are included in the predictive model can be evaluated by changing the CMFs used in the predictive method. This includes the addition of dedicated turn lanes, left-turn signal phasing, lighting, etc. This can be quickly achieved by duplicating and modifying the HSM spreadsheet used for the original analysis.
- Countermeasures that are not included in the predictive model can be evaluated by using CMFs from other sources. No more than three CMFs should be evaluated simultaneously in this fashion.
- See APM Section 4.6 for further information on countermeasures.
- See APM Section 4.4.5 to determine if the EB Method can be applied to alternative designs, treatments, or forecast AADT.

Step 18: Evaluate and compare results

- The evaluation and reporting of predictive method results is discussed below, in APM Section 0.

Note: Although the steps seem quite lengthy, the calculations can be expedited through the use of spreadsheets or other tools, discussed below in APM Section 4.4.13.

4.4.11 Reporting Predictive Method Results

When reporting predictive results, include the predictive models used along with a table of sites and characteristics. Depending on the availability of calibration factors and the use of the EB Method, the predictive method will result in one of these types of estimates for each site:

- Uncalibrated predicted crash frequency
- Calibrated predicted crash frequency
- Expected crash frequency
- Excess expected crash frequency

Depending on the predictive model used, these results may be reported by crash severity and/or collision type. Within a study area, this may result in a mix of these estimate types. When summing the predictive results for a project area total, each estimate type (uncalibrated predicted, calibrated predicted, expected, and excess expected) should be summed individually by severity and identified in the reporting. Any sites that are analyzed uncalibrated should be denoted.

Existing conditions evaluations should be reported using excess expected crash frequency, by severity, if possible. A positive excess expected crash frequency indicates that the site is performing more poorly than the HSM models suggest is normal for that site. Higher values of excess expected crash frequency indicate more potential for improvement.

Alternatives should be compared using the net change in expected or predicted crashes, by severity, if possible. Expected crashes can be determined for alternatives in limited situations (see APM Section 4.4.5 for more information).

Predicted or expected crashes should be reported over the study period as the total number of crashes in the study period, rounded to the nearest integer (for example, 15 crashes over five years). Alternatively a long-term yearly average frequency could be reported, for example 3.4 crashes per year.

4.4.12 Project-Induced Volume Changes

A project build alternative will often create new capacity and relieve congestion in the study area. As a result, the new roadway can attract previously latent demand. The project build alternative will thus have higher forecast traffic volumes than the no-build alternative. For large projects with regional impact, the difference in forecast volumes can be substantial.

It may be valuable to report and discuss the safety impacts of a project's design, independent of the effects of increased vehicle exposure. If there are significant (more than 10%) volume changes due to project build alternatives, the analyst has the option of reporting predictive results using both the actual build volumes and the future no-build volumes on the build alternative. The use of future no-build volumes must be clearly identified as such, and the report should acknowledge that the results are not the actual forecast estimates and are for discussion purposes only.

4.4.13 Tools for Implementing Predictive Methods

Computational tools are available to help in implementing the HSM Predictive Method. These tools are recommended to ensure uniform and transparent application of analysis techniques.

HSM Spreadsheets

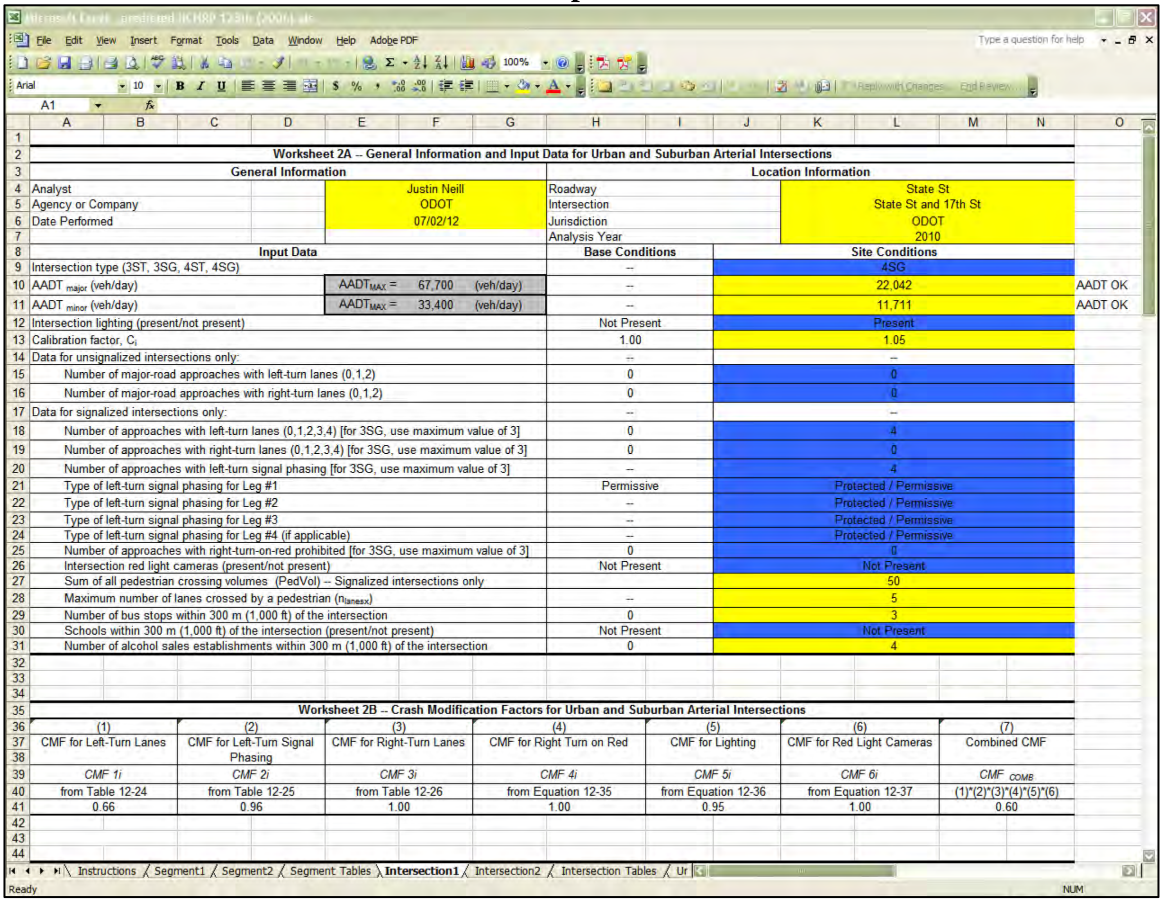
The HSM spreadsheets were developed as part of National Cooperative Highway Research Program (NCHRP) Project 17-38, Highway Safety Manual Implementation and Training Materials, to aid in training the HSM Part C predictive method (see Exhibit 4-17). These spreadsheets are designed for only two segments and two intersections for one year, but can be modified to include additional segment and intersection capacity. However, adding worksheets to increase the number of segments and intersections in an analysis can be moderately time consuming.

The spreadsheet displays results by segments and intersections and also separates the results into crashes by single-vehicle, multiple-vehicle non-driveway, and multiple-vehicle driveway-related. The worksheets are free to download, but do not have any technical support, at this time, for any changes or updates to the HSM. Data needs are consistent with the normal HSM Part C requirements.

The basic NCHRP spreadsheets allow the user to apply the EB Method for the current analysis period, but post-processing is required to apply the EB Method to alternatives where no observed crash data are available. See the discussion in APM Section 4.4.5 for more information on applying the EB Method.

The spreadsheets containing Oregon-specific calibration factors and crash proportions can be downloaded from the ODOT HSM Webpage. The general, non-calibrated, spreadsheets can be downloaded from the HSM Website.

Exhibit 4-17: HSM Spreadsheet Screenshot



Worksheet 2A -- General Information and Input Data for Urban and Suburban Arterial Intersections				
General Information		**Location Information**		
Analyst	Justin Neill	Roadway	State St	
Agency or Company	ODOT	Intersection	State St and 17th St	
Date Performed	07/02/12	Jurisdiction	ODOT	
		Analysis Year	2010	
Input Data		**Base Conditions**	**Site Conditions**	
Intersection type (3ST, 3SG, 4ST, 4SG)		--	4SG	
AADT major (veh/day)	$AADT_{MAX}$ = 67,700 (veh/day)	--	22,042	AADT OK
AADT minor (veh/day)	$AADT_{MAX}$ = 33,400 (veh/day)	--	11,711	AADT OK
Intersection lighting (present/not present)		Not Present	Present	
Calibration factor, C_i		1.00	1.05	
Data for unsignalized intersections only:		--	--	
Number of major-road approaches with left-turn lanes (0,1,2)		0	0	
Number of major-road approaches with right-turn lanes (0,1,2)		0	0	
Data for signalized intersections only:		--	--	
Number of approaches with left-turn lanes (0,1,2,3,4) [for 3SG, use maximum value of 3]		0	4	
Number of approaches with right-turn lanes (0,1,2,3,4) [for 3SG, use maximum value of 3]		0	0	
Number of approaches with left-turn signal phasing [for 3SG, use maximum value of 3]		--	4	
Type of left-turn signal phasing for Leg #1		Permissive	Protected / Permissive	
Type of left-turn signal phasing for Leg #2		--	Protected / Permissive	
Type of left-turn signal phasing for Leg #3		--	Protected / Permissive	
Type of left-turn signal phasing for Leg #4 (if applicable)		--	Protected / Permissive	
Number of approaches with right-turn-on-red prohibited [for 3SG, use maximum value of 3]		0	0	
Intersection red light cameras (present/not present)		Not Present	Not Present	
Sum of all pedestrian crossing volumes (PedVol) -- Signalized intersections only			50	
Maximum number of lanes crossed by a pedestrian (n_{lanesx})		--	5	
Number of bus stops within 300 m (1,000 ft) of the intersection		0	3	
Schools within 300 m (1,000 ft) of the intersection (present/not present)		Not Present	Not Present	
Number of alcohol sales establishments within 300 m (1,000 ft) of the intersection		0	4	

Worksheet 2B -- Crash Modification Factors for Urban and Suburban Arterial Intersections						
(1)	(2)	(3)	(4)	(5)	(6)	(7)
CMF for Left-Turn Lanes	CMF for Left-Turn Signal Phasing	CMF for Right-Turn Lanes	CMF for Right Turn on Red	CMF for Lighting	CMF for Red Light Cameras	Combined CMF
CMF 1i	*CMF 2i*	*CMF 3i*	*CMF 4i*	*CMF 5i*	*CMF 6i*	*CMF COMB*
from Table 12-24	from Table 12-25	from Table 12-26	from Equation 12-35	from Equation 12-36	from Equation 12-37	(1)*(2)*(3)*(4)*(5)*(6)
0.66	0.96	1.00	1.00	0.95	1.00	0.60

Cost-Effectiveness Index Analysis spreadsheet

The Cost-Effectiveness Index Analysis spreadsheet is an adaption of the HSM spreadsheets to analyze countermeasures for bicycle and pedestrian crashes on urban and suburban arterials. This spreadsheet allows up to 9 segments and 8 intersections to be analyzed. If countermeasure costs are known a cost effectiveness index can be calculated.

ISATe

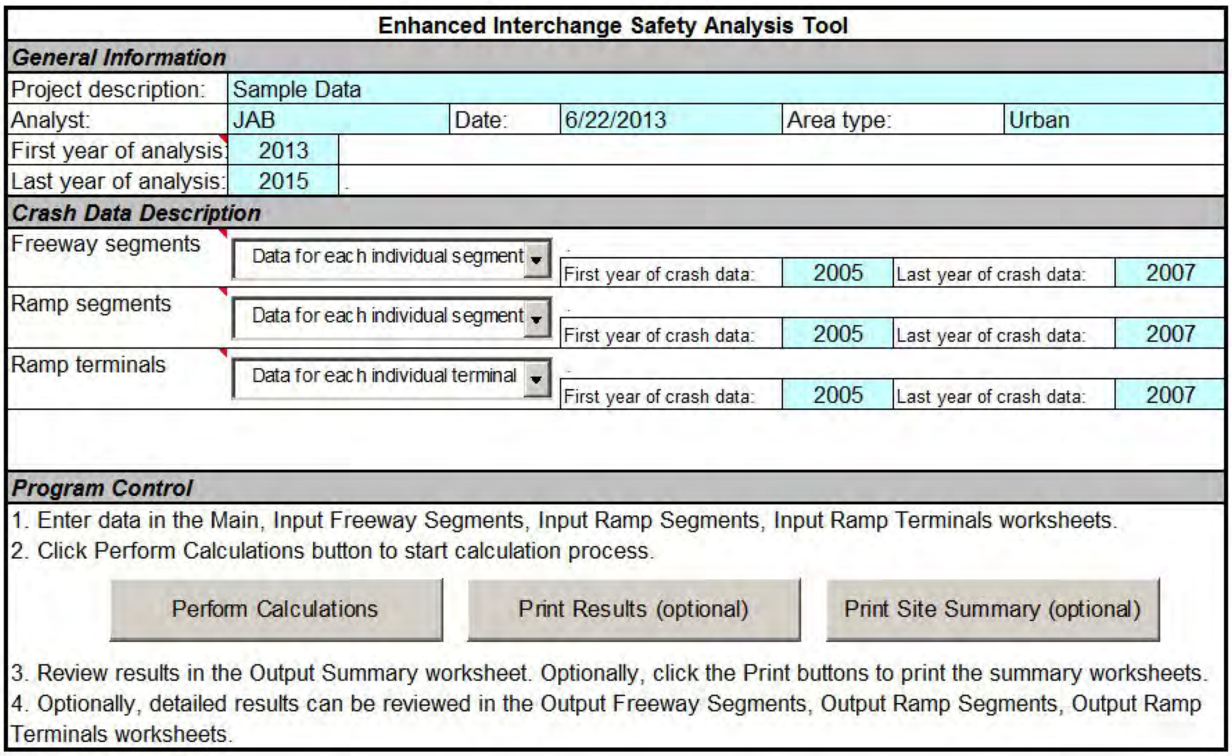
The Enhanced Interchange Safety Analysis Tool (ISATe) is a predictive model for freeway mainline segments and common interchange elements in both urban/suburban and rural contexts, implemented as a macro-enabled spreadsheet. The methodology used is consistent with the HSM Part C Predictive Method. ISATe has been incorporated in the HSM 2014 Supplement as Chapters 18 and 19. The ISATe spreadsheets and users guide are available for download from the HSM website as one of the spreadsheet tools.

ISATe can be used to analyze a single type of site, such as an extended length of freeway mainline. It can also be used to analyze a collection of interconnected sites of different types, such as a complex interchange. If the study area includes sites that are not covered by ISATe, other HSM Part C predictive methods can be used to complete the study area analysis. For example, the urban arterial predictive model may be used to evaluate the freeway overpass or neighboring intersections as applicable.

Predicted crashes are reported by each of the KABCO severity levels and by crash type. Expected crashes can be reported using the site-specific or project-level EB Method if crash data are available. The spreadsheet automates applying the EB Method with crash data from any time period, not only the analysis time period. The EB Method should be used only when study conditions are similar to those during the crash period, as described in APM Section 4.4.5.

As with other HSM predictive methods, ISATe requires the analyst to segment the facility into homogenous sites. Segmentation requirements for ISATe are somewhat more specific than other HSM models. Additional details are available in the "Segmentation Criteria" section of the ISATe User Manual on page 34. ISATe screen captures are shown Exhibit 4-18 through Exhibit 4-20.

Exhibit 4-18: ISATe Main Screen



Enhanced Interchange Safety Analysis Tool

General Information

Project description:	Sample Data				
Analyst:	JAB	Date:	6/22/2013	Area type:	Urban
First year of analysis:	2013				
Last year of analysis:	2015				

Crash Data Description

Freeway segments	Data for each individual segment	First year of crash data:	2005	Last year of crash data:	2007
Ramp segments	Data for each individual segment	First year of crash data:	2005	Last year of crash data:	2007
Ramp terminals	Data for each individual terminal	First year of crash data:	2005	Last year of crash data:	2007

Program Control

1. Enter data in the Main, Input Freeway Segments, Input Ramp Segments, Input Ramp Terminals worksheets.
2. Click Perform Calculations button to start calculation process.

Perform Calculations | Print Results (optional) | Print Site Summary (optional)

3. Review results in the Output Summary worksheet. Optionally, click the Print buttons to print the summary worksheets.
4. Optionally, detailed results can be reviewed in the Output Freeway Segments, Output Ramp Segments, Output Ramp Terminals worksheets.

Exhibit 4-19: ISATe Input Worksheet (Partial)

Input Worksheet for Freeway Segments		Segment 1		Segment 2		Segment 3		Segment 4	
Clear / Echo Input Values / Check Input Values	(View results in Column AV) (View results in Advisory Messages)	Crash Period	Study Period	Crash Period	Study Period	Crash Period	Study Period	Crash Period	Study Period
Basic Roadway Data									
Number of through lanes (n):		5	5	5	5	4	4	4	4
Freeway segment description:		Station 0+00.00		Station 4+75.20			Alt seg		
Segment length (L), mi:		0.09	0.09	0.05	0.05	0.08	0.08	0.25	0.25
Alignment Data									
Horizontal Curve Data	See note								
1	Horizontal curve in segment?:	No	No	No	No	No	No	Both Dir.	Both Dir.
	Curve radius (R_1), ft:							2800	2800
	Length of curve (L_{c1}), mi:							0.23	0.23
	Length of curve in segment ($L_{c1,seg}$), mi:							0.15	0.15
2	Horizontal curve in segment?:							No	No
	Curve radius (R_2), ft:								
	Length of curve (L_{c2}), mi:								
	Length of curve in segment ($L_{c2,seg}$), mi:								
3	Horizontal curve in segment?:								
	Curve radius (R_3), ft:								
	Length of curve (L_{c3}), mi:								
	Length of curve in segment ($L_{c3,seg}$), mi:								
Cross Section Data									
Lane width (W_l), ft:		10.8	12	10.8	10.8	12	12	12	12
Outside shoulder width (W_s), ft:		4	4	5	5	9	9	10	10
Inside shoulder width (W_{is}), ft:		2.5	2.5	2.5	2.5	9	9	9	9
Median width (W_m), ft:		7	7	7	7	21	21	22	22
Rumble strips on outside shoulders?:		No	No	No	No	Yes	Yes	Yes	Yes
	Length of rumble strips for travel in increasing milepost direction, mi:					0.08	0.08	0.25	0.25
	Length of rumble strips for travel in decreasing milepost direction, mi:					0.08	0.08	0.25	0.25
Rumble strips on inside shoulders?:		No	No	No	No	Yes	Yes	Yes	Yes
	Length of rumble strips for travel in increasing milepost direction, mi:					0.08	0.08	0.175	0.175
	Length of rumble strips for travel in decreasing milepost direction, mi:					0.08	0.08	0.175	0.175
Presence of barrier in median:		Center	Center	Center	Center	Center	Center	Center	Center

Exhibit 4-20: ISATe Output Summary (Partial)

Output Summary								
General Information								
Project description:	Sample Data							
Analyst:	JAB	Date:	12/16/2014		Area type:		Urban	
First year of analysis:	2013							
Last year of analysis:	2015							
Crash Data Description								
Freeway segments	Segment crash data available?			Yes	First year of crash data:			2005
	Project-level crash data available?			No	Last year of crash data:			2007
Ramp segments	Segment crash data available?			Yes	First year of crash data:			2005
	Project-level crash data available?			No	Last year of crash data:			2007
Ramp terminals	Segment crash data available?			Yes	First year of crash data:			2005
	Project-level crash data available?			No	Last year of crash data:			2007
Estimated Crash Statistics								
Crashes for Entire Facility		**Total**	**K**	**A**	**B**	**C**	**PDO**	
Estimated number of crashes during Study Period, crashes:		120.5	0.3	1.7	10.9	40.3	67.4	
Estimated average crash freq. during Study Period, crashes/yr:		40.2	0.1	0.6	3.6	13.4	22.5	
Crashes by Facility Component	**Nbr. Sites**	**Total**	**K**	**A**	**B**	**C**	**PDO**	
Freeway segments, crashes:	4	24.5	0.1	0.4	2.2	4.3	17.5	
Ramp segments, crashes:	6	4.9	0.0	0.1	0.7	1.1	3.0	
Crossroad ramp terminals, crashes:	6	91.1	0.1	1.2	8.0	34.9	46.9	
Crashes for Entire Facility by Year	**Year**	**Total**	**K**	**A**	**B**	**C**	**PDO**	
Estimated number of crashes during the Study Period, crashes:	2013	40.1	0.1	0.6	3.6	13.4	22.4	
	2014	40.2	0.1	0.6	3.6	13.4	22.5	
	2015	40.2	0.1	0.6	3.6	13.4	22.5	

PLANSAFE

PLANSAFE is a free GIS-based tool that estimates the anticipated safety impact of changes in traffic flow, demographics, and safety policy at a regional scale. PLANSAFE is not an HSM-based tool, although it is based on a similar methodology that has been modified for macroscopic implementation. The PLANSAFE SPFs were designed for use with the limited data typically available for a long-range regional analysis.

The tool uses outputs from a travel demand model and historic crash data with macro-level predictive analysis to determine the predicted future safety performance at a census block group or Transportation Analysis Zone (TAZ) unit of resolution. The results can be used to evaluate relative safety across an entire study area, neighborhoods/districts, a city, or a region.

PLANSAFE is available for download on the PLANSAFE TRB webpage. An updated version is in development but is not yet publically available.

PLANSAFE is an option for use on TSPs that involve major transportation network modifications or significant demographic changes. It provides a system-wide safety assessment for future scenario analysis that is unavailable with any other tool.

Minimum data requirements for PLANSAFE are the following GIS layers (geocoded) for baseline and alternative conditions:

- Road network (arterials and higher) with traffic volumes
- Intersection locations
- Baseline safety performance measure, such as total observed crash frequency
- At least one measure of demographic information, such as number of housing units

Safety performance predictions can be reported using various target crash types including all crashes, fatal and serious injury crashes, bicyclist crashes, pedestrian crashes, and others. Results are reported for each census block group or TAZ and can be exported in tabular or map form. PLANSAFE includes a variety of SPFs for each target crash type. These SPFs are dynamically evaluated and self-calibrated using observed crash data to provide the best performance with the data available. The analyst should always choose to use the SPF with the highest “Goodness of Fit” value. Additional data can be included to improve the accuracy of predictions.

What data are recommended varies based on the target crash type being predicted. Recommended datasets for each target crash type are shown in Exhibit 4-21.

Exhibit 4-21: PLANSAFE Recommended Data

Variable	TC	Int	NInt	KA	KAB	Ped	Bike	Deer
Target Crashes/Polygon	X	X	X	X	X	X	X	X
Total Number of Intersections/Polygon		X		X	X	X		
Total Roadway Length/Polygon (mile)		X						
VMT/Polygon	X	X	X	X	X	X	X	
Number of Intersections/Mile	X					X		X
Population between 16 and 64/Polygon	X	X	X					
Proportion Urban Population/Polygon	X			X	X	X	X	
Proportion Minority Population/Polygon	X			X	X	X		
Housing Units/Polygon				X	X	X	X	X
Density of Children in K12/Polygon				X	X	X		
Number of Schools/Polygon							X	
Average Household Income/Polygon				X				
Proportion Population in Urban Areas/Polygon								X
Rural Minor Arterial/Polygon (mile)								X
Rural Major Collector/Polygon (mile)								X
Sum of Combined Freeways, Principal Arterial, Rural Minor Arterial/Polygon (mile)	X							

TC = Total Crashes, Int = Intersection Crashes, NInt = Non-Intersection Crashes, KA = Injury Level K and A, KAB = Injury Levels K, A, and B. Ped = Pedestrian Crashes, Bike = Bike Crashes, Deer = Large Animal Crashes

What data are recommended varies based on the target crash type being predicted.

Using all recommended data, PLANSAFE SPF predictions are of a similar accuracy to HSM models, though the results are not as geographically specific as the HSM Predictive Method. Baseline and future roadway and traffic data are typically derived from a travel demand model. Baseline demographic data are available from the U.S. Census or American Community Survey. The analyst will need to provide future demographic conditions forecast through a travel demand model or by other methods. Data GIS layers are summarized by TAZ or census block group polygons using the PLANSAFE GIS tools or manually following the methodology in the PLANSAFE documentation.

PLANSAFE prediction sensitivity to demographic conditions vary based on the underlying SPF. Variables in the SPFs used in PLANSAFE generally have elasticities between 0 and 2, which describe how responsive the SPF prediction is to change in that variable. At an elasticity of 1, an X% increase in the variable will result in an X% increase in the predicted crashes. For example, consider an SPF predicting fatal and serious injury crashes that includes a variable for the percentage of population in urban areas at an elasticity of 1. An increase in urban population from 25% to 35% would increase the prediction from 100 to 110 fatal and serious injury crashes per year.

CMFs for countermeasures can be manually added to the future predictions to specific geographic regions or as systemic actions to the study area as a whole. Although a table of CMFs is provided in the PLANSAFE software, it is the responsibility of the analyst to ensure that any CMF is used appropriately and of sufficient quality (see APM Section 4.6). For example, a CMF that is intended for use with intersection crashes should not be applied to all crashes throughout the region.

At the time of this writing, some of the GIS toolbox components for PLANSAFE do not function with recent versions of ArcGIS. Without these tools, PLANSAFE can be very time-intensive and is best performed by analysts familiar with the software model. Consult the PLANSAFE manual for required GIS methodology.

Exhibit 4-22 gives an overview of the data flow within PLANSAFE and Exhibit 4-23 summarizes the analysis steps.

Exhibit 4-22: PLANSAFE Data Flow

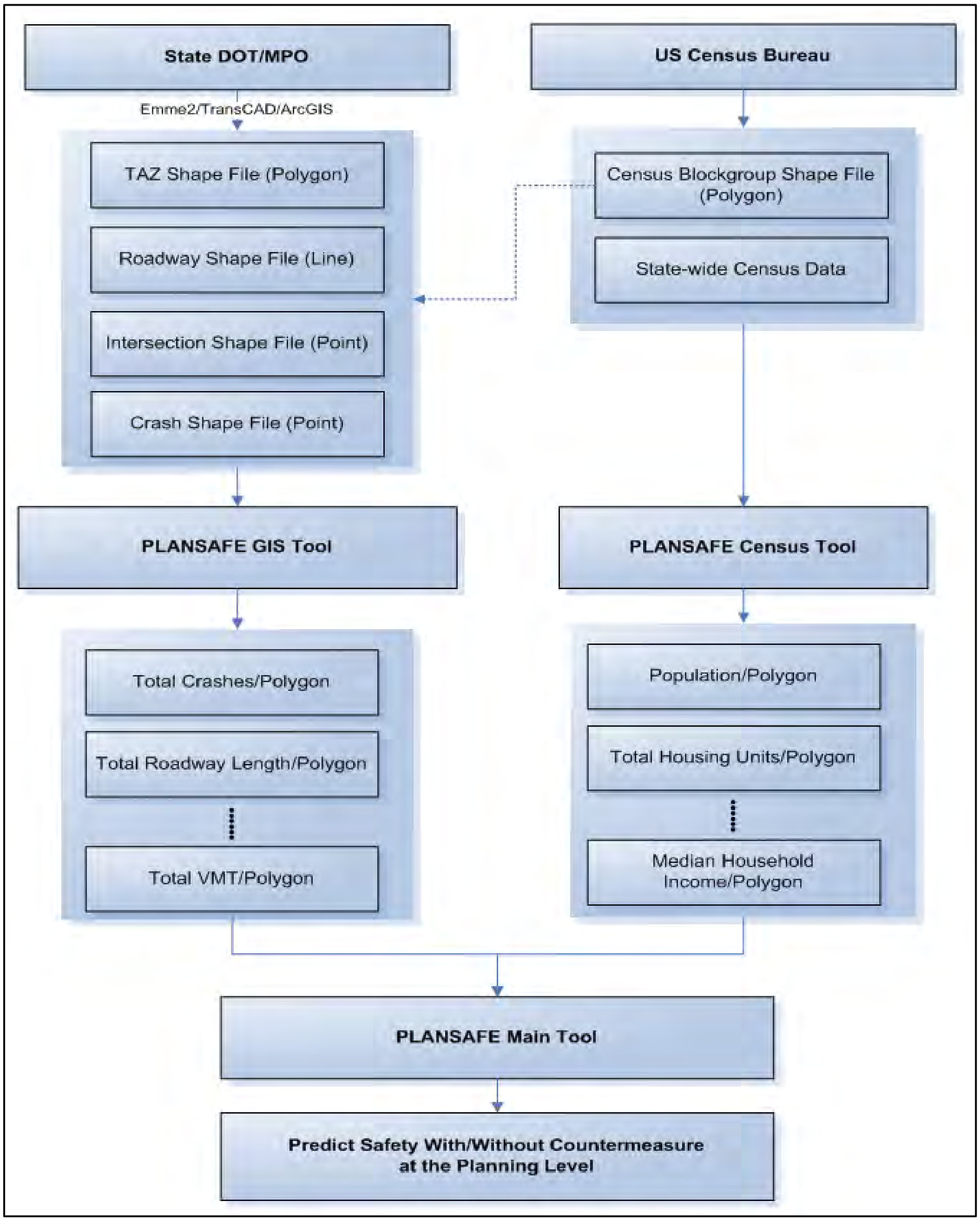
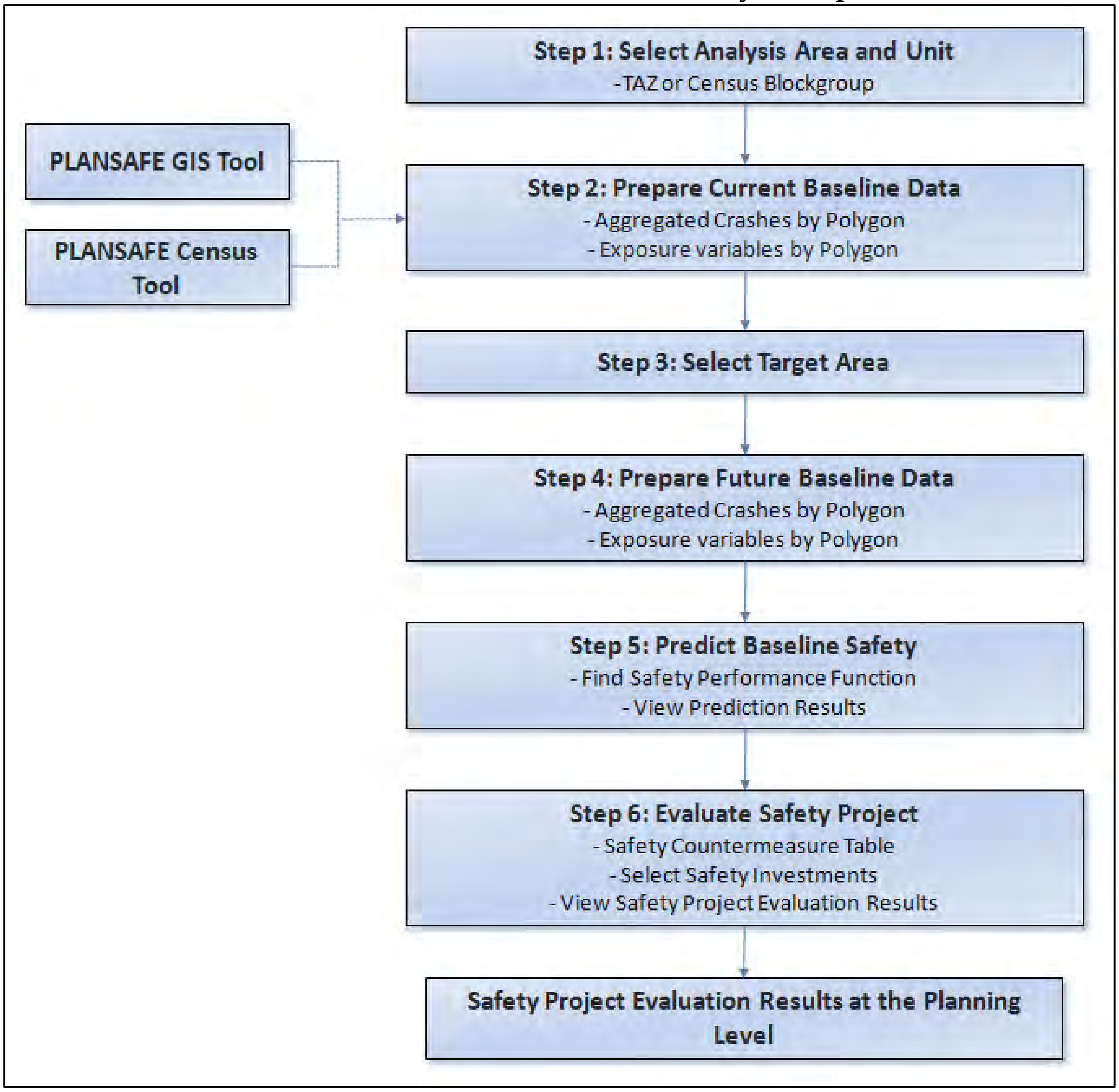


Exhibit 4-23: PLANSAFE Analysis Steps



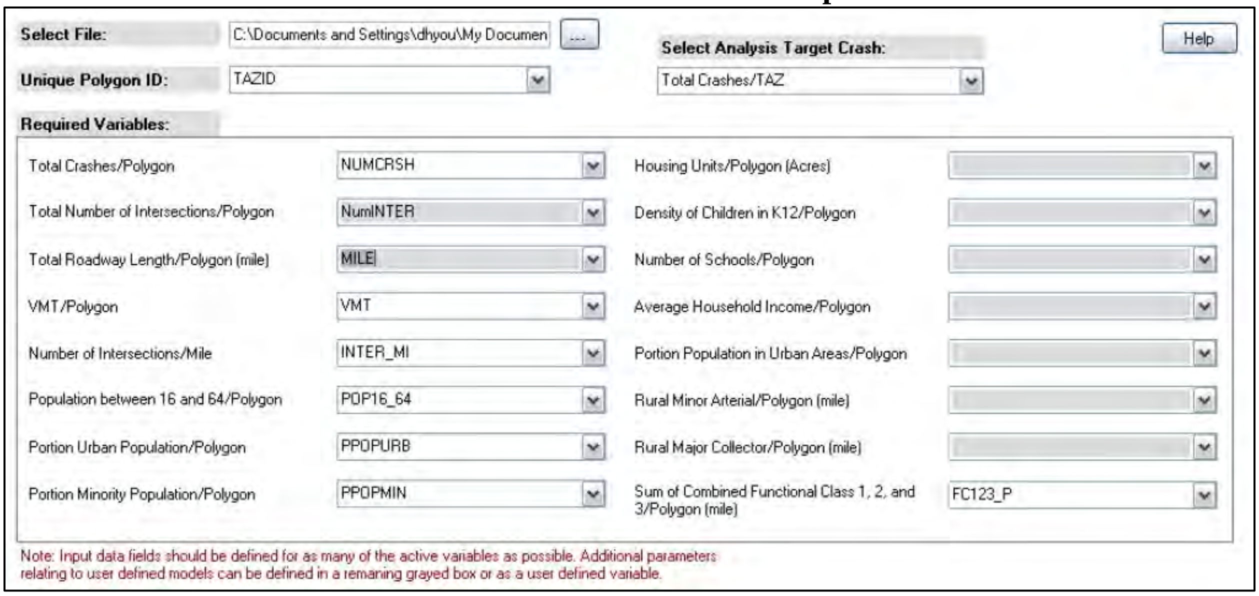
Example 4-8: PLANSAFE Regional Safety Analysis

The City of Klamath Falls is preparing an update to its TSP that includes plans for considerable new residential development. In addition to an intersection-based safety analysis, it would like to perform a PLANSAFE predictive analysis to assess the overall impact of residential growth, set citywide safety performance goals, and help direct its systematic safety countermeasures.

The results of a travel demand model were exported by TAZ for the current year and for the horizon analysis year. A desktop GIS client was used with the PLANSAFE GIS tool and the PLANSAFE Census tool to post-process crash data, census data, and roadway data into the format needed for analysis in PLANSAFE.

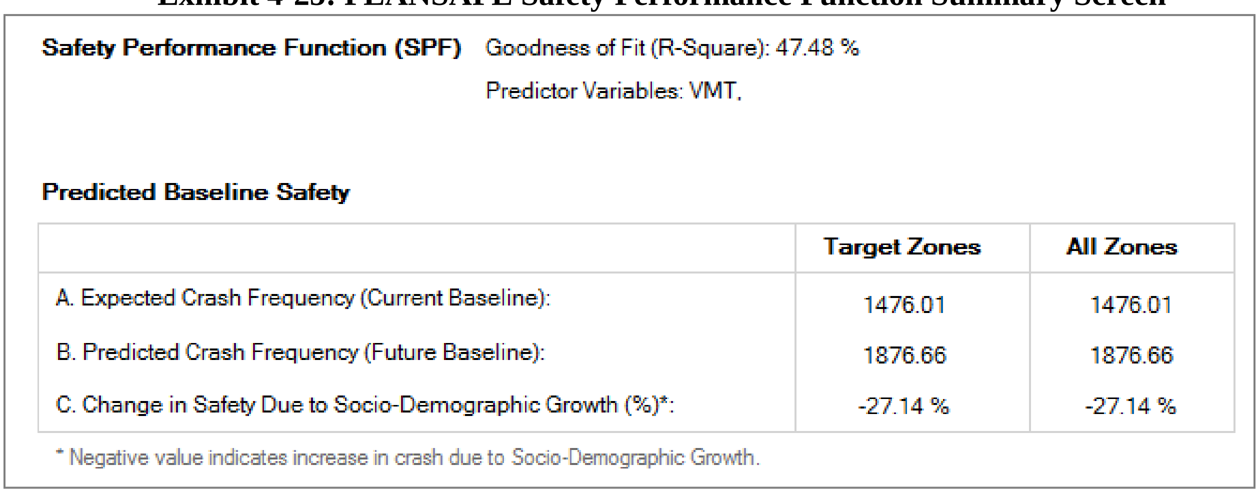
The City's TSP safety policy is focused on reducing the total number of crashes, so this was chosen as the target crash type for the PLANSAFE analysis. Exhibit 4-24 shows the data input screen where field names from the GIS shapefile are matched with the required variables, shown with white input boxes. Grey boxes are optional, but including them increases model performance.

Exhibit 4-24: PLANSAFE Data Input Screen



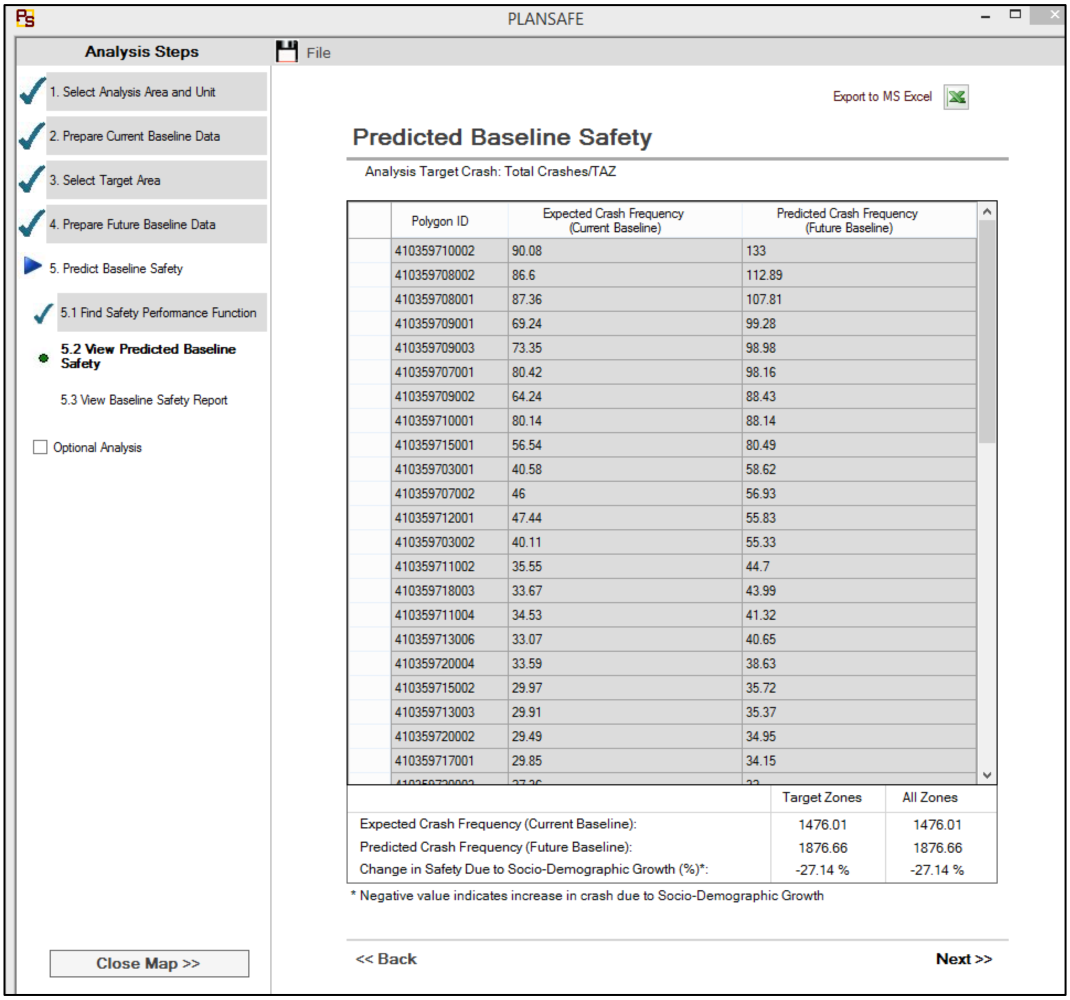
After the variables in the GIS file have been identified for the current baseline data, the analysis zone is selected and the process is repeated for the future baseline data. PLANSAFE recommends the best available SPF based on the data available, which is chosen by the City analysts. Exhibit 4-25 shows the SPF performance summary screen.

Exhibit 4-25: PLANSAFE Safety Performance Function Summary Screen



Using the selected SPF, PLANSAFE predicts the current expected crash frequency (current baseline) and future predicted crash frequency (future baseline). The results can be viewed in tabular form and with a map. Exhibit 4-26 displays the results by TAZ in table form, while Exhibit 4-27 shows the results on a map, with red indicating high predicted crash growth.

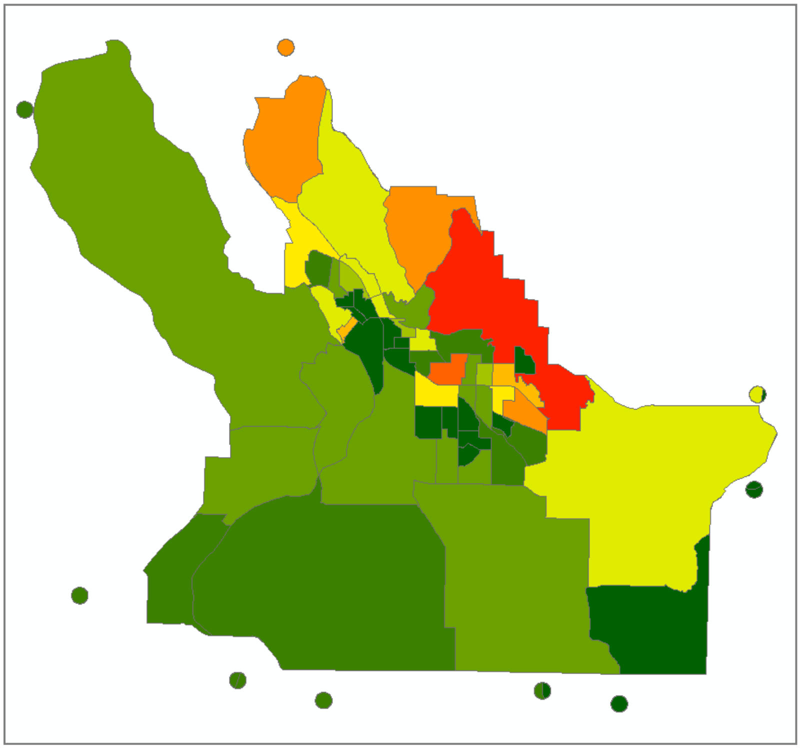
Exhibit 4-26: PLANSAFE Tabular Results



Polygon ID	Expected Crash Frequency (Current Baseline)	Predicted Crash Frequency (Future Baseline)
410359710002	90.08	133
410359708002	86.6	112.89
410359708001	87.36	107.81
410359709001	69.24	99.28
410359709003	73.35	98.98
410359707001	80.42	98.16
410359709002	64.24	88.43
410359710001	80.14	88.14
410359715001	56.54	80.49
410359703001	40.58	58.62
410359707002	46	56.93
410359712001	47.44	55.83
410359703002	40.11	55.33
410359711002	35.55	44.7
410359718003	33.67	43.99
410359711004	34.53	41.32
410359713006	33.07	40.65
410359720004	33.59	38.63
410359715002	29.97	35.72
410359713003	29.91	35.37
410359720002	29.49	34.95
410359717001	29.85	34.15

	Target Zones	All Zones
Expected Crash Frequency (Current Baseline):	1476.01	1476.01
Predicted Crash Frequency (Future Baseline):	1876.66	1876.66
Change in Safety Due to Socio-Demographic Growth (%)*:	-27.14 %	-27.14 %

Exhibit 4-27: PLANSAFE Results Map



Based on the analysis, it is anticipated that the 24% growth in vehicle miles traveled (VMT), 15% growth in population and 18% growth in housing units in the region will, on average, result in a predicted increase in crashes by 27%, to 626 per year by the year 2025.

The analysis results estimate that without additional systemic countermeasures, there will be a 27% growth in the total number of crashes in the city due to demographic growth. Based on this, the City implements a safety program in its TSP with the goal of keeping the city's total crash growth below 15% through the forecast horizion.

Later in the TSP process, the City will evaluate systemic countermeasure options using PLANSAFE to quantify the expected results.

4.5 Multimodal Safety Analysis

Pedestrians and bicyclists are considered to be vulnerable road users, and identifying and improving conditions for these road users is an important part of many transportation plans and projects. Safety analysis for these modes is more difficult than for typical motor vehicle crashes because of practical limitations such as:

- Compared to motor vehicle crashes, fewer pedestrian and bicycle crashes occur and only a portion are reported. With fewer reported crashes to study, identifying statistically significant crash patterns is more difficult than for other types of vehicular crashes.

- Exposure data, such as pedestrian and bicycle volumes, are not widely available and travel patterns vary by user.
- Few systematic countermeasures for reducing pedestrian and bicycle crashes have been studied.

Although most frequently discussed in the context of pedestrian and bicycle safety, these limitations apply to other modes with limited data—such as motorcycles, freight, and transit—that may also be of interest in a safety analysis.

These practical limitations may limit the multimodal utility of the screening and predictive analysis methods included in this chapter. Additionally, subjective safety (the perceived safety by the traveling public) is an important component of multimodal planning and design. A person's choice of mode and route is strongly affected by how safe and comfortable the person feels about them.

4.5.1 Using Screening and Predictive Methods with Multimodal Crashes

When an analysis has few records of crashes involving pedestrians and bicyclists, reporting the details of those crashes with a narrative may be the only option available. In instances where sufficient crash data and/or exposure data exist, the screening and predictive analysis methods described in this chapter may be used.

4.5.2 Screening

SPIS does not include explicit consideration of multimodal crashes, but OASIS can be used to quickly identify hot-spots of crashes involving pedestrians or bicyclists across a large geographic region. This is done by adjusting the OASIS crash conditions criteria to include only pedestrian- or bike-involved crashes. The same can also be done for truck-involved crashes.

The critical crash rate method can be applied with a multimodal focus in two ways. One way is to use the traditional critical crash rate methodology with multimodal crashes as the target crash type. This would evaluate the pedestrian and bicycle crash frequency while controlling for vehicular exposure. Since vehicles are involved in all recorded multimodal crashes and vehicular exposure is usually at least an order of magnitude higher than multimodal exposure, this is generally sufficient.

Another potential way is to use pedestrian or bicycle volumes in the analysis. This should be limited to situations where multimodal traffic is generally high and varies in the project area, with available good historic data. An example is a popular mixed-use path with many street crossings. This analysis uses the same critical crash rate methodology described in APM Section 4.3.4, but with a crash rate calculated for a specific target mode, such as vehicle-bicycle crashes per million entering bicycles. Considerations would need to be given for historically and seasonally adjusting the pedestrian and bicycle volumes. This alternative analysis should be in addition to a traditional critical crash rate.

The excess proportions of specific crash types method is well suited for multimodal crash analysis, as this method does not require exposure data. As long as the target mode is included as a crash type to be considered, the analysis will flag locations where crashes of that mode are overrepresented. However, the analyst should be aware that this method may overlook crash types that are not well represented in the overall sample, which may hinder multimodal analysis particularly for small study areas.

4.5.3 Predictive

Most HSM Part C predictive analysis methods report pedestrian and bicycle crashes separately from other crash types and thus can be used for multimodal analysis. In general, the pedestrian and bicycle HSM predictive methods are less well developed than the motor vehicle predictive methods. Pedestrian crashes at urban and suburban signalized intersections are characterized by a unique SPF, considering vehicle and pedestrian volumes and pedestrian crossing distance. However, all other pedestrian and bicycle crashes are predicted using a locally derived crash adjustment factor. This factor is developed for each basic road configuration, considering vehicle speed, and predicts pedestrian or bicycle crashes as a simple fixed percentage of motor vehicle crashes.

ISATe does not predict pedestrian or bicycle crashes. Although freeways are generally not designed for pedestrians or bicyclists, interchange terminals may have high amounts of multimodal activity. Safety for these users is not evaluated through ISATe.

PLANSAFE does allow for pedestrian or bicycle crash prediction, though results are reported aggregated by census block group or TAZ.

4.5.4 Risk-based Multimodal Safety Analysis

The recent ODOT Pedestrian and Bicycle Safety Implementation Plan includes an effort to address the limitations inherent in multimodal safety analysis through a risk-based screening methodology. The risk-based process identifies roadway characteristics that have contributed to pedestrian and bicycle crashes in the historical crash data then evaluates the road network based on the presence of high-risk characteristics. This allows for a prioritization of safety projects that is proactive, addresses perceived safety by users, and is applicable where no or few pedestrian or bicycle crashes have been recorded.

Identified high-risk screening characteristics include:

- Posted speed
- Number of lanes
- Presence of bicycle facilities
- Number of driveways
- Presence of transit stops
- Occurrence of pedestrian or bicycle crashes
- Annual average daily traffic
- The presence of signalized intersections or pedestrian activated systems

The risk-based method of the plan is limited by the availability of data for the roadway network. Although the risk of serious pedestrian crashes is probably related to factors such as pedestrian volume, pedestrian age, and volume of turning vehicles, these factors were not included in the method because the data were not available across the roadway network.

This risk-based screening method is distinct from the HSM screening and predictive methods. Unlike the HSM screening methods, it is not based on crashes and does not require any crashes to have been recorded in the study area. Unlike the HSM predictive methods, the aim is not to quantify the effects of any particular treatment or roadway design. Risk-based screening is a complement to HSM analysis methods.

Although risk-based safety analysis methods can be useful in many planning contexts, the results cannot be used to apply for funding from the FHWA's Highway Safety Improvement Program (HSIP). Risk-based analysis results can be used to support an application to the "Enhance" portion of ODOT's State Transportation Improvement Program (STIP) but not to the "Fix-It" portion of the STIP.

The analyst is encouraged to contact TPAU if a risk-based multimodal safety analysis is being considered for a project or system plan.

4.6 Countermeasure Selection and Evaluation

Most crash analysis projects will specify general or potential ranges of countermeasures. The results of the analysis methods in this chapter can be used as a starting point for identifying countermeasures. Sites that demonstrate an excess of a specific crash type are more likely to benefit from countermeasures targeted at that crash type.

There are many resources that provide potential countermeasures for a given crash pattern.

- The initial source for countermeasures should be the ODOT approved set of proven countermeasures and associated CRFs that are used for the All Roads Transportation Safety (ARTS) Program. Use of these CRFs allow for all countermeasures to be evaluated consistently and fairly. If the desired countermeasure is not in the ARTS CRF list, a CMF from Part D or from the CMF Clearinghouse (studies with 3 star rating or better) may be used.
- Information on countermeasure selection is included in the ODOT Safety Investigations Manual and Chapter 6 of the HSM.
- Part D of the HSM is also an extensive resource of countermeasure treatments and includes quantitative CMFs that estimate the expected change in crash frequency resulting from implementation.
- The CMF Clearinghouse (http://www.cmfclearinghouse.org) is an active online database maintained by the FHWA providing CMFs and supporting research. The FHWA has developed a list of proven systemic countermeasures that are widely applicable to common situations.
- PedBikeSafe.org is an interactive FHWA guide to pedestrian and bicycle safety countermeasures. The website guides users to targeted countermeasures based on crash trends, patterns, and the road context.

Some CMFs are applicable only for specific facility types, AADT ranges, or base conditions. Engineering judgment is critical in selecting the appropriate CMF for a project. CMFs are an active area of research, and the best available CMF for a situation may change frequently.

> *If CMFs are being used that are not derived from the ARTS CRF list, the ODOT CMF standard is to use CMFs with quality ratings of three stars or better (star rating is a rating of the type of research and more stars indicate better, more reliable results). For many countermeasures, there may be multiple CMFs available with different levels of crash reduction.*

Care should be taken not to just pick the CMF with the highest reduction because a particular CMF may apply to all crashes, or just severe crashes, or for a particular crash type. The CMF study parameters may limit applicability to a particular roadway configuration or to a specific volume range. The CMF AADT range should be no more than +/- 10% away from the countermeasure location AADT. Values greater than this indicate that the subject roadway likely does not have the same characteristics as the one in the CMF study.

Certain countermeasures may require coordination, review, or approval by the Region or State Traffic Engineer (see list of ODOT Traffic Engineering Authorities). Region Traffic will likely perform a detailed safety investigation of the crash patterns later during project delivery/design phase. When selecting countermeasures, the analyst should coordinate with Region Traffic and/or ODOT TRS.

Countermeasures can generally be grouped into four categories: education, enforcement, emergency medical services and engineering.

- **Education** is a variety of public information campaigns using a broad range of media to reach a target audience. These campaigns can be effective in reducing driver error or problematic behaviors by making motorists aware of the risks and consequences of certain driving behaviors and environments encountered. These can be handled by the Region Public Information office (for a specific area) or by the Traffic Safety Division (for programmatic issues).
- **Enforcement** involves increased policing activity to encourage compliance with existing traffic controls and regulations. Increased enforcement is often implemented due to frequent driver violations. The application of enforcement countermeasures is typically coordinated by Region Traffic Sections.
- **Emergency Medical Services (EMS)** involves working with EMS providers to improve response time to incidents, which plays a role in the severity of the crash.
- **Engineering** is a broad range of improvements to the transportation system to improve roadway safety. These may include geometric improvements, ITS applications, changes to traffic controls (signing, striping, signals, etc.), changes to roadway surfacing, or operational enhancements.

Specific investigation into countermeasures should be at the appropriate level for the analysis being conducted.

Countermeasure options in a system plan or alternative design concepts should be screened based on the relative safety improvement expected, which may be reported as a range of feasible values. This reduction can be determined using CMFs applied to historic crash values. Reporting should indicate the source and quality of CMFs used. Reporting should also identify what types of crashes the CMFs are expected to reduce. When possible, report crash reductions by severity level with an emphasis on fatal and serious injury crashes.

4.7 Multimodal Mixed-Use Areas (MMAs)

A recent amendment to Oregon Administrative Rule (OAR) 660-012-0060 (Plan and Land Use Regulation Amendments) introduced the Multimodal Mixed-Use Area (MMA) designation that local governments can use to gain new flexibility in applying transportation performance standards in specific locations. An MMA allows a local government to amend a functional plan, comprehensive plan, or land use regulation without consideration of motor vehicle congestion, delay, or travel time.

Amendments within an MMA must still comply with transportation standards and policies promoting safety for all modes.

In evaluating a local request for ODOT concurrence with a proposed MMA designation near an interchange, ODOT must consider the following safety factors per OAR 660-012-0060(10)(a):

- Whether the interchange area has a crash rate that is higher than the statewide crash rate for similar facilities. The statewide crash rate tables are available on the ODOT Crash Analysis and Reporting Unit Publications webpage.
- Whether the interchange area is in the top 10% of locations identified by the SPIS. SPIS locations can be found on the SPIS webpage or through TransGIS.
- Whether existing or potential future traffic queues on the interchange exit ramps extend onto the mainline highway or the portion of the ramp needed to safely accommodate deceleration. Procedures for estimating queue lengths are found in Section 7.5 of the APM Version 1. Intersection functional areas are discussed in APM Section 4.8.1.

It is strongly encouraged that these considerations be taken into account during ODOT review of all proposed MMA designations, including those that are not near an interchange. In addition, a predictive analysis is encouraged for roadways within the proposed MMA to determine the excess expected average crash frequency. This crash frequency can be used in addition to crash rates to characterize the prevailing safety conditions of the MMA area.

Additionally, for local governments, it is suggested that safety performance standards using one of the predictive methods described in this chapter be established for considering land use amendments within designated MMAs. Standards could be based on reducing or maintaining within a predetermined range the predicted or expected crash frequency. Increases in estimated crashes caused by exposure would be offset by proposed safety countermeasures or payment into a safety fund.

4.8 Other Safety-related Techniques

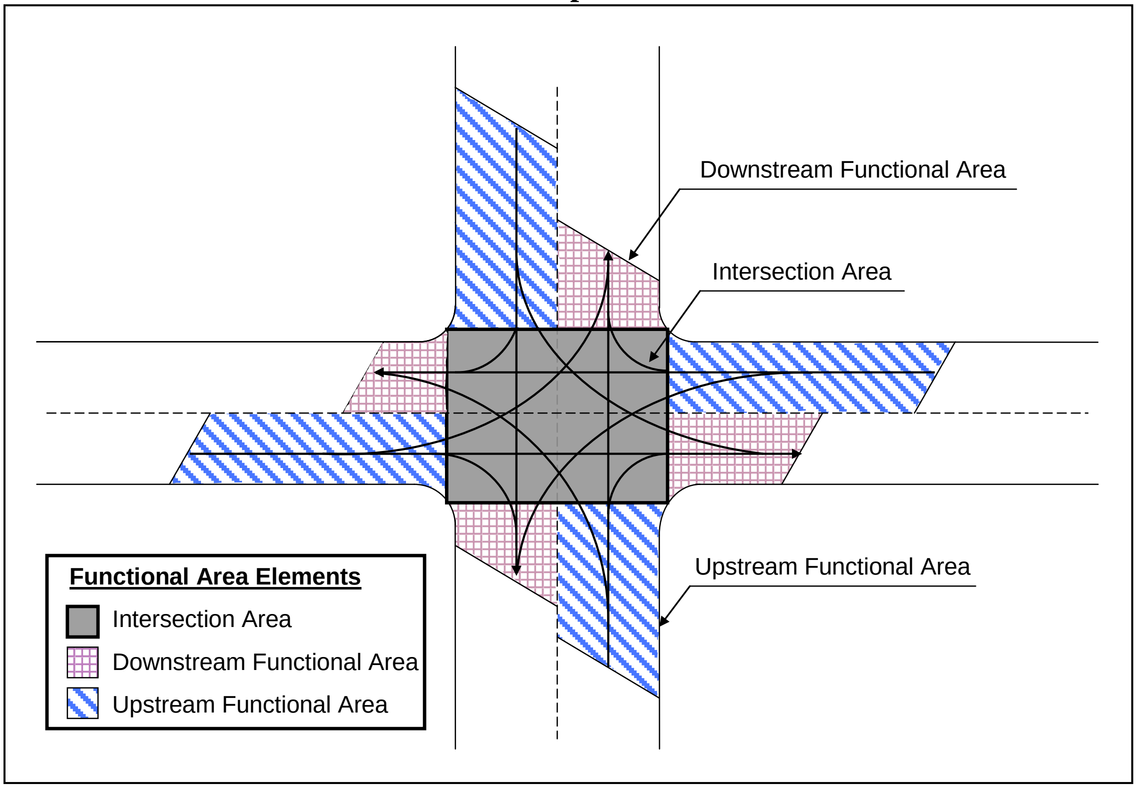
The techniques listed in this section provide a detailed review of the safety impact associated with intersection functional areas, sight distance, intersection conflict points, and segment access management. These techniques can be used in addition to the screening and predictive tools identified in this APM chapter to assist in evaluating a proposed build alternative or safety mitigation. The application of these techniques incorporates necessary human factors into the analysis. Functional area, sight distance, conflict points and other techniques focus on the need to spread apart the necessary driver information processing points. Drivers can be confused or miss obstacles if information about driveways, intersections, and other elements are too closely spaced. In addition, as the driving population ages, the importance of analyzing the human factor cannot be overstated. The analyst is strongly encouraged to be familiar with Chapter 2 of the HSM.

4.8.1 Functional Area of an Intersection

A functional area analysis should be done to evaluate the impact of closely-spaced intersections, access points, or any combination of both. This can be for either existing or proposed (alternative) conditions. Areas with long queues should also be reviewed for functional area impacts. The analysis should also be done when adding new connections to a roadway to verify that functional area overlap does not occur and vehicle maneuvers can be performed lawfully. The functional area of an intersection is the area in which an intersection affects vehicle paths. The intersection functional area—defined as the physical area where two roads overlap—influences driver decisions, vehicle movements, and vehicle queues. The sections beyond the intersection area are composed of upstream and downstream functional areas. The upstream functional area for vehicles moving toward the intersection has four maneuvering elements. The downstream functional area for vehicles traveling away from the intersection has one. These maneuvering elements are listed in the next section. Each element is unique in its contribution to the functional area. Exhibit 4-28 shows the functional area of an intersection.

The upstream functional area described in this section is defined similarly to the "influence area" used in the Highway Safety Manual (HSM). The HSM does not use the downstream functional area definition. In addition, the functional area in this section is not the same as the weaving distance calculation used in the approach permitting (development review) process

Exhibit 4-28: Components of the Functional Area[2]



Both upstream and downstream functional areas may need to be studied for an intersection improvement or any project with access in the immediate intersection area. However, only the upstream functional area needs to be studied if an access is opened upstream of an intersection and only the downstream functional area needs to be studied if an access is opened downstream of an intersection. Functional area analysis may determine the placement of an access, the provision of turn movements, or the number of travel lanes.

Upstream Functional Area

Four elements make up the distance a vehicle travels as it approaches an intersection:

- **Distance traveled during the perception-reaction time (d_1):** The perception-reaction time has four phases—perception, intellection, emotion, and volition (PIEV). This distance involves the driver seeing the intersection, thinking about their options, making a decision, and initiating their response. The perception-reaction time is 2.0 seconds for desirable conditions and 1.0 seconds for limiting conditions as set by the Transportation Research Institute (TRI) of Oregon State University in Discussion Paper No. 7, Functional Intersection Area (January 1996) (1), which was prepared for ODOT to

[2] Data referenced through exhibits in this section were obtained from the Discussion Papers presented to the Oregon Department of Transportation by the Transportation Research Institute (TRI) of Oregon State University.

support its policies, practices, and procedures. A table of perception-reaction distances for varying time intervals is shown in Exhibit 4-30.

- **Distance traveled while the driver decelerates or brakes and moves laterally into a turn bay (d_2):** The limiting condition for a vehicle traveling laterally over a 12-foot lane is three seconds in urban areas with an assumed lateral movement at four feet per second (fps). For each 12-foot lane, three seconds of travel time should be added. Four seconds of travel time per 12-foot lane should be assumed for rural conditions, with an assumed lateral movement at three fps.
- **Distance traveled during full deceleration (d_3):** These maneuver distances are based on a 6.7 fps^2 deceleration rate accommodating 85% of drivers. The limiting condition accommodates 50% of drivers with a deceleration rate of 9.2 fps^2 or higher. The distances of d_1, d_2, and d_3 are dependent on vehicle speed. Maneuver distances ($d_2 + d_3$) and PIEV plus maneuver distance ($d_1 + d_2 + d_3$) are based on the intersection functional area approaches from the ODOT's Access Management Manual. Values for just the maneuver distance and PIEV plus maneuver distance are developed from the uniform acceleration formulas and are listed in the table in Exhibit 4-31. Note that storage length, d_4, is not included in the values of Exhibit 4-31. Perception-reaction time may not always be included in an upstream functional area analysis if decisions are made prior to the driver's approach to the intersection.
- **Storage length (d_4):** Calculated by the 95th percentile queue for turning or through traffic, whichever is greater.

Exhibit 4-29 depicts the succession of these movements.

Exhibit 4-29: Upstream Functional Area; d_1, d_2, d_3, and d_4

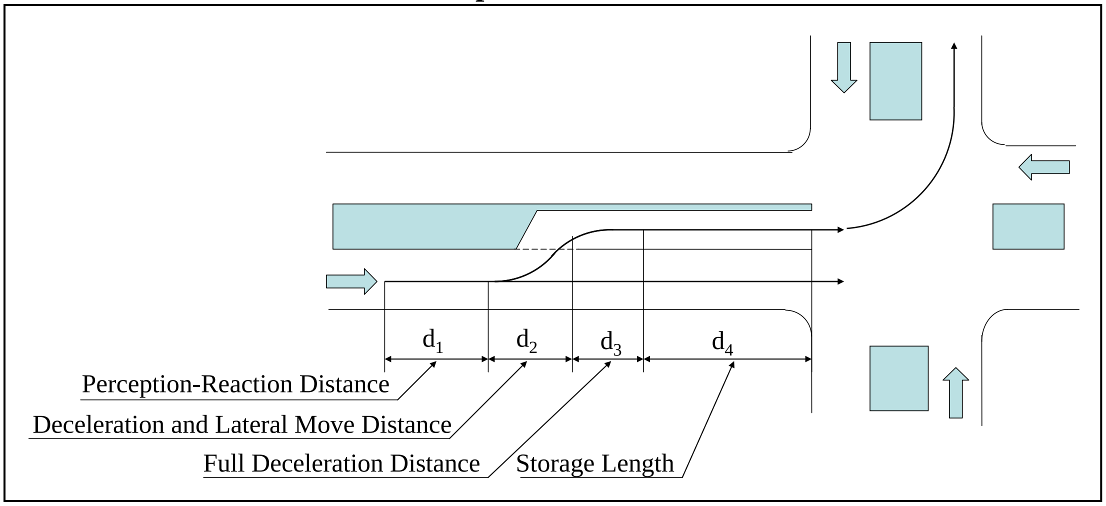


Exhibit 4-30: Perception-Reaction Time, d_1

Distance Traveled During Perception-Reaction					
US Customary Units (Feet)(1)(2)					
Speed (mph)	Perception-Reaction Time (Seconds)(3)				
	1.0	2.0	3.0	4.0	5.0
30	45	90	130	175	220
40	60	115	175	235	295
45	65	130	200	265	330
50	75	145	220	295	370
60	90	175	265	355	440
70	105	205	310	410	515

(1) Rounded to 5 feet

(2) US Customary: distance (feet) = 1.47*(speed in mph)*t

(3) Distance traveled in t-seconds

A functional area analysis has four possible values:

- The unfamiliar path under desirable conditions
- The unfamiliar path under limiting conditions
- The familiar path under desirable conditions
- The familiar path under limiting conditions

Limiting conditions are used for projects that have design constraints. A project using limiting conditions must justify this reasoning and provide appropriate documentation. From one to all of these scenarios may need to be checked depending on driver types and roadway conditions.

The familiar vehicle path would be used by drivers who would anticipate the intersection and know the lane to be in to complete the turn. This is more likely near grocery stores or residential areas. The unfamiliar vehicle path would be used by drivers who may not know anything about the intersection before approaching. This is more likely near developments such as tourist areas and regional stores.

Exhibit 4-31 demonstrates the difference between desirable conditions and limiting conditions. Desirable conditions allow for a longer perception-reaction time and a lesser rate of deceleration. Area demographics may affect these values because older, younger, and unfamiliar drivers may have longer perception-reaction times.

Exhibit 4-31: Upstream Functional Intersection Area, $d_1 + d_2 + d_3$

Upstream Functional Intersection Area Excluding Storage, in Feet(1)				
Speed (mph)	Desirable Conditions		Limiting Conditions	
	Maneuver Distance(2) (6) (ft)	PIEV(3) Plus Maneuver Distance (ft)	Maneuver Distance(4) (6) (ft)	PIEV(5) Plus Maneuver Distance (ft)
	$d_2 + d_3$	$d_1 + d_2 + d_3$	$d_2 + d_3$	$d_1 + d_2 + d_3$
20	70	130	70	100
25	110	185	105	140
30	160	250	145	190
35	215	320	190	240
40	275	395	245	305
45	345	475	300	365
50	425	570	365	440
55	510	670	435	515
60	605	780	510	600
65	710	900	590	685

(1) Rounded to 5 feet

(2) 10 mph speed differentials, 5.8 fps^2 deceleration while moving from the through lane into the turn lane; 6.7 fps^2 average deceleration after completing lateral shift into the turn lane

(3) 2.0 second perception-reaction-time

(4) 10 mph speed differential; 5.8 fps^2 deceleration while moving from through lane into the turn lane; 9.2 fps^2 average deceleration after completing lateral shift into the turn lane

(5) 1.0 second perception-reaction time

(6) Assumes turning vehicle has “cleared the through lane” (a following through vehicle can pass without physically encroaching on the adjacent through lane when the turning vehicle has moved laterally 10 ft. Also assumes a 12 ft. lateral movement will be completed in 3.0 seconds

Turn lanes may be installed at unsignalized intersections to improve safety and at signalized intersections to expand the roadway capacity. The minimum length for a turn bay (including the taper) is the deceleration and lateral move distance and the full deceleration distance, plus the storage length ($d_2 + d_3 + d_4$). The upstream functional area increases/decreases with the number of lanes (related to d_2), the rate of deceleration (related to d_3), and the queue (d_4). Vehicles that change lanes at an intersection expand the influence area of the intersection and the intersection functional area.

Turn lanes remove turning vehicles from the general flow of traffic allowing through vehicles to proceed without significant slowing or stopping. Research indicates that the crash potential between turning vehicles and through traffic increases exponentially as the speed differential increases. It is desirable to have no more than a 10 mph speed differential between vehicles in the through lanes and vehicles entering turn bays. Providing a turn lane with adequate deceleration distance significantly lowers the speed differential between turning vehicles and through traffic.

Turn lanes at a signalized intersection serve as capacity expanders and are constructed where demand approaches or exceeds capacity. The typical urban turn bay is 100 feet unless capacity or speeds require it to be longer, while the typical rural turn bay is 150 feet. The 95th percentile queue is generally calculated by traffic analysis software for signalized intersections. Chapter 13 contains the procedures to consider turn bays for intersections and estimating the length for right-turn and left-turn vehicle queues.

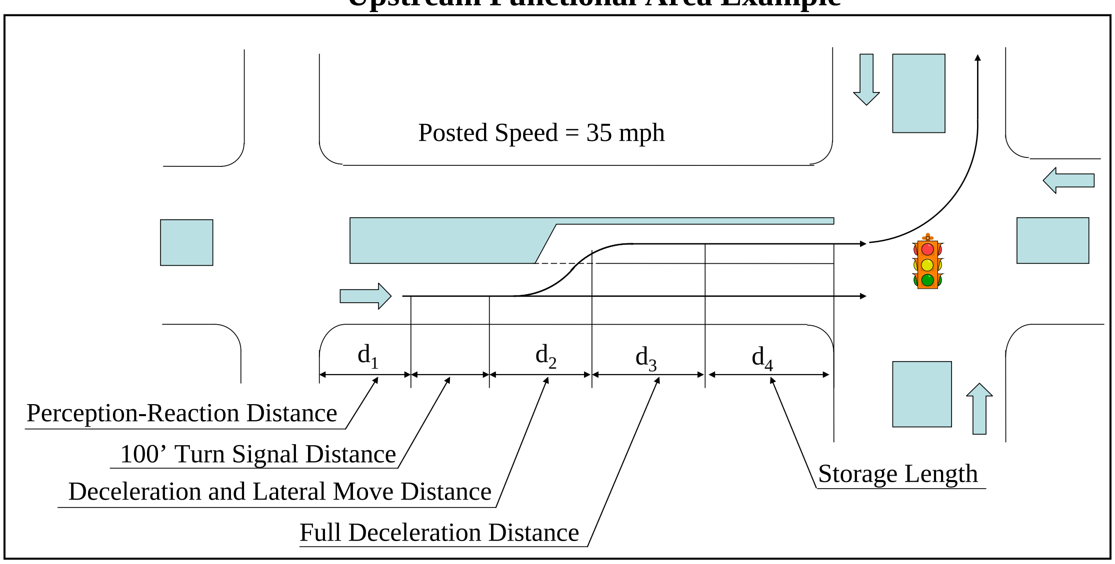
Example 4-9: Upstream Functional Area Distance Calculation

A development proposes to access the roadway upstream of a signalized intersection. The intersection shown has a volume of 100 vph using the left-turn lane.

The road has a posted speed of 35 mph. How close to the intersection can a proposed driveway be placed?

Assume the signal has a 120-second cycle length.

Upstream Functional Area Example



Solution

PIEV Plus Maneuver Distance ($d_1 + d_2 + d_3$)

Check values from Exhibit 4-31:

Limiting Condition

$d_{L1} = d_1 + d_2 + d_3 = 240$ feet

Desirable Condition

$d_{D1} = d_1 + d_2 + d_3 = 320$ feet

Storage Length (d_4)

For this signalized intersection, the Left-Turn Movement Queue Estimate Technique from Chapter 14 was used.

Assume each cycle is 120 seconds (30 per hour)

Assume the constant, t, is 1.85 to find the 95th percentile queue. (See Chapter 7 for background information)

$$\text{Length} = \frac{\text{volume}}{\#\text{ of cycles/hour}} * t * 25\text{feet}$$

$$\frac{100\text{ vph}}{30\text{ cycles/hr}} * 1.85 * 25 \text{ feet} = 154.17 \text{ feet} = 154 \text{ feet (rounded)}$$

Turn Signal Length

Another 100 feet must be added to provide distance for the turn signal to be used.

Total Functional Area Length

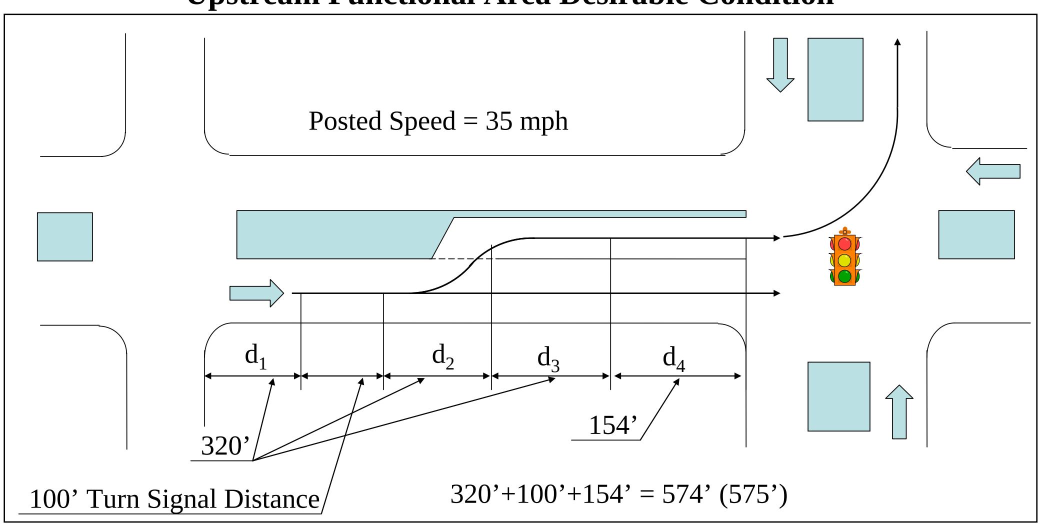
Limiting Condition

$d_{LTotal} = d_1 + d_2 + d_3 + d_4 +$ Signal Distance = 240’ + 154’ + 100’ = 494 feet

Desirable Condition

$d_{DTotal} = d_1 + d_2 + d_3 + d_4 +$ Signal Distance = 320’ + 154’ + 100’ = 574 feet

Upstream Functional Area Desirable Condition



Using the values for a 35-mph speed in Exhibit 4-31, the desirable conditions path is 575 feet long and the limiting conditions path is 495 feet long. The desirable condition calculated using the value from Exhibit 4-31 is the greatest distance and is the closest location to access the highway with respect to the intersection. The driveway should be no less than 575 feet from the intersection.

Downstream Functional Area

As a vehicle travels away from an intersection the driver needs a minimum stopping sight distance (d_5) before approaching another intersection or driveway. The stopping sight distance is the distance traveled while braking to avoid an unexpected obstacle. Stopping sight distance is determined by AASHTO by the speed, brake reaction time, and the deceleration rate. A table developed from the following AASHTO equation is shown in Exhibit 4-32:

$$d = 1.47 * V * t + 1.075 * \left(\frac{V^2}{a}\right)$$

Where: V – speed, mph
t – brake reaction time, 2.5s
a – deceleration, ft/s^2

If an acceleration lane is present, the stopping sight distance is measured from the end of the taper. The downstream intersection functional area includes the distance traveled during acceleration before merging into the general traffic flow. Acceleration lanes are rarely provided for at-grade arterials. Lane drops that have an auxiliary lane longer than the distance traveled during acceleration before merging will not be included in the functional area analysis.

Exhibit 4-32: Downstream Functional Area

Downstream Intersection Functional Area	
Speed (mph)	**AASHTO Stopping Sight Distance (Feet)**
20	115
25	155
30	200
35	250
40	305
45	360
50	425
55	495
60	570

Note that the downstream functional area analysis is a check for adequacy in safety and legality. Further analysis will be necessary to ensure the adequacy of the design.

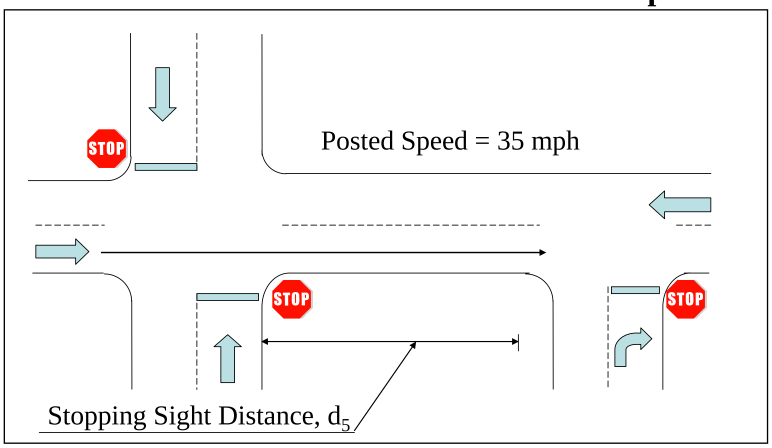
Functional Intersection area is detailed in the ODOT Access Management Manual and further information is contained in Discussion Paper No. 7, Functional Intersection Area (1), Transportation Research Institute (TRI) of Oregon State University (January 1996).

Example 4-10: Downstream Functional Area Distance Calculation

A driveway is located 350 feet downstream of the intersection shown. The main street has no traffic control and a speed of 35 mph.

Is there adequate spacing between the intersection and the driveway? What is the stopping sight distance (d_5) for this intersection? The following figure shows a general diagram of the intersection area.

Downstream Functional Area Example



Solution

Stopping Sight Distance, d_5

Check values from Exhibit 4-32: AASHTO Stopping Sight Distance at 35 mph

d_5 = 250 feet

The driveway must be at least 250 feet from the next downstream intersection to avoid a stopping sight distance conflict. Keep in mind that the downstream functional area analysis is a check for adequacy in safety and legality. Further analysis will be necessary to ensure the adequacy of the design.

Functional Area Application

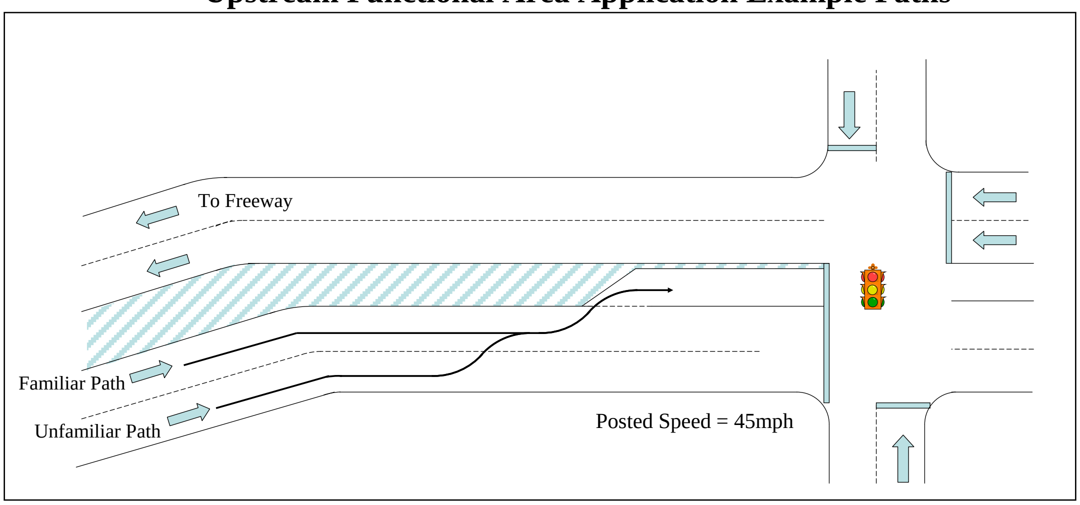
The principles of functional area can be used to test geometric and operational adequacy before detailed analysis starts. The primary objective is to check vehicle paths for adequate length to perform safe and legal maneuvers. For example, a path that connects a right turn onto a roadway to a turn left at the next intersection, or two paths come from two roadways merging together and terminating at a signal, may require lane changes that have safety and legal constraints. Although a functional area analysis may reveal potential conflicts, simulation is used to ensure the adequacy of design in the detailed analysis. Generally, functional area overlaps will appear in simulation results as slowdowns or bottlenecks.

Example 4-4: Functional Area Application – Geometric Adequacy

There is a two-lane ramp transitioning from a freeway to an arterial and has geometry similar to an interchange. An intersection is proposed on the arterial near this ramp as shown in the following figure. The queue at a speed limit of 45 mph is estimated at 400 feet.

Test the adequacy of the design for a driver in either lane of the exit ramp to turn left into the driveway. Can movements from the off-ramp to northbound intersection leg occur safely in this design? Check both the familiar path and the unfamiliar path.

Upstream Functional Area Application Example Paths



Solution

Familiar drivers will generally use the familiar path, the lane closest to the turn bay, in anticipation of the left turn. Unfamiliar drivers may take the unfamiliar path, which starts from the furthest lane or the "wrong lane" and must change lanes into the turn bay. The maneuver distance over one lane is 198 feet when a three-second maneuver time is assumed.

<u>At Speed Maneuver Distance</u>

$$\left(\frac{45\text{miles}}{\text{hr}}\right)*\left(\frac{1\,\text{hr}}{3600\,\text{s}}\right)*\left(\frac{5280\text{ft}}{1\,\text{mile}}\right)*3\text{s} = 198\text{ft} \;\; \text{(200 feet)}$$

Perception-reaction (PIEV) distances are found in Exhibit 4-30. Assume a two-second PIEV (130 feet) for desirable conditions and a one-second PIEV (65 feet) for limiting conditions as set by the TRI.

The maneuver distances for the turn bay include the deceleration and lateral move distance along with the full deceleration distance. The maneuver distances are found in Exhibit 4-31. For a speed of 45 mph, there should be 300 feet of distance to meet the limiting conditions and 345 feet is desirable.

Lane changes and turn movements should be signaled for 100 feet prior to the action. If an unfamiliar driver follows the unfamiliar path, a lane change must be signaled to move laterally into the near lane and the turn bay separately. The following figures show the components of the unfamiliar path and familiar path for desirable and limiting conditions.

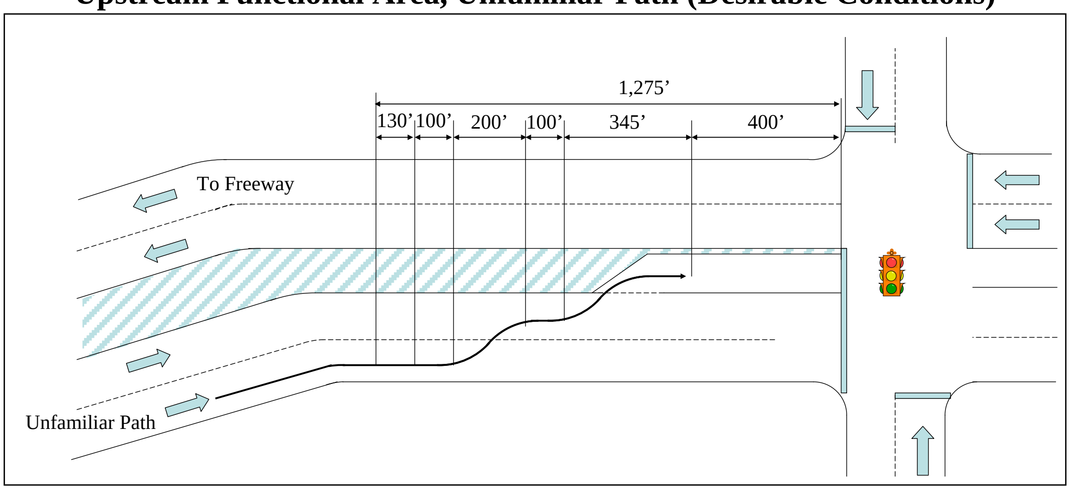
<u>Unfamiliar path (desirable conditions)</u>

d_{UD} = 130’ (PIEV) + 100’ (turn signal) + 200’ (at speed maneuver) + 100’ (turn signal) + 345’ (desirable distance into the turn bay) + 400’ (queue)
d_{UD} = 1,275’

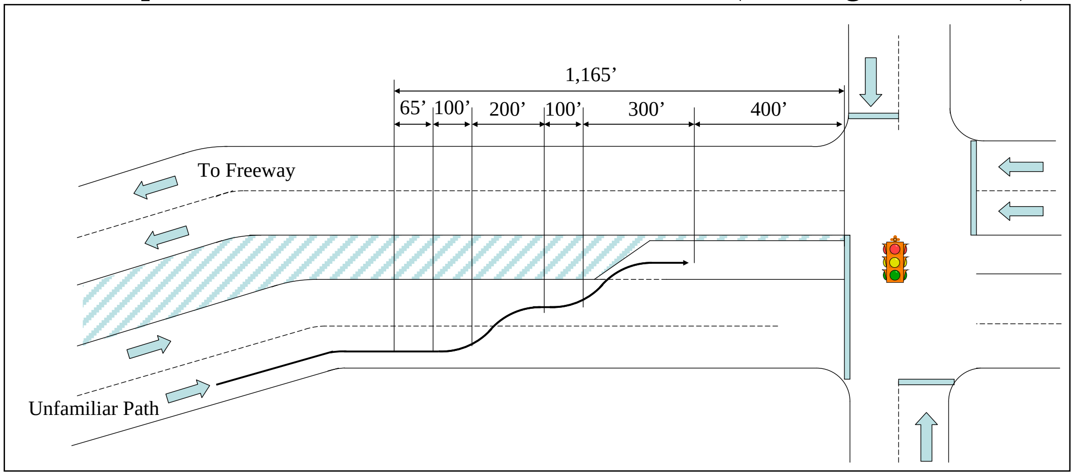
<u>Unfamiliar path (limiting conditions)</u>

d_{UL} = 65’ (PIEV) + 100’ (turn signal) + 200’ (at speed maneuver) + 100’ (turn signal) + 300’ (limiting distance into the turn bay) + 400’ (queue)
d_{UL} = 1,165’

Upstream Functional Area, Unfamiliar Path (Desirable Conditions)



Upstream Functional Area, Unfamiliar Path (Limiting Conditions)



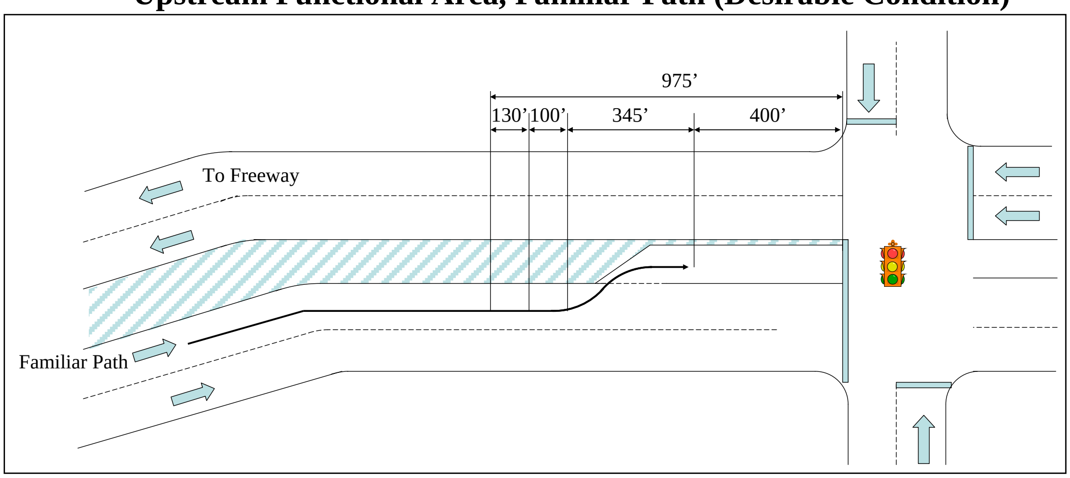
Familiar path (desirable conditions)

d_{FD} = 130’ (PIEV) + 100’ (turn signal) + 345’ (desirable distance into the turn bay) + 400’ (queue)
d_{FD} = 975’

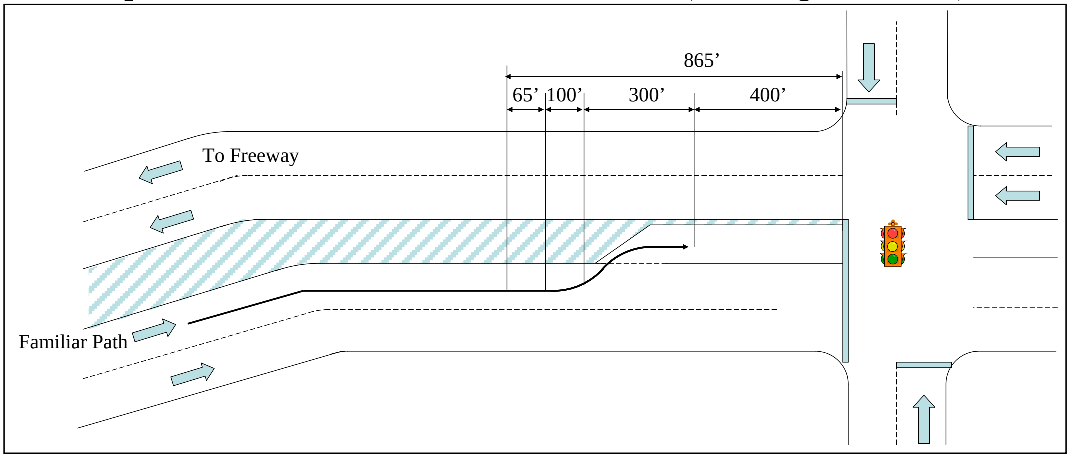
Familiar path (limiting conditions)

d_{FL} = 65’ (PIEV) + 100’ (turn signal) + 300’ (limiting distance into the turn bay) + 400’ (queue)
d_{FL} = 865’

Upstream Functional Area, Familiar Path (Desirable Condition)



Upstream Functional Area, Familiar Path (Limiting Condition)



Unfamiliar path (desirable conditions): d_{UD} = 1,275’
Unfamiliar path (limiting conditions): d_{UL} = 1,165’
Familiar path (desirable condition): d_{FD} = 975’
Familiar path (limiting condition): d_{FL} = 865’

Ideally, the design would need to allow 1,275 feet between the stop bar at the intersection back to the gore point. If the distance available was between 865 feet and 1,165 feet, then drivers using the unfamiliar path would be subject to high speed differentials.

4.8.2 Sight Distance

The length of roadway visible to a driver is referred to as "sight distance." The amount of visible roadway needed by a driver at any given time depends on the maneuvers or decisions that must be made at that moment. The four basic categories of sight distance are:

- Intersection sight distance
- Stopping sight distance
- Decision sight distance
- Passing sight distance

Note: Some portions of the access management process use different sight distance methods than what would be normally used for operations and project development. See OAR 734-051 for more information.

Although each of these is briefly described below, intersection and stopping sight distance are most frequently examined in traffic analysis. For additional information on sight distance refer to ODOT's Highway Design Manual or Section 3.2 of the AASHTO Green Book.[3]

- **Intersection sight distance** is considered adequate when drivers at or approaching an intersection have an unobstructed view of the entire intersection and of sufficient approach lengths of the intersecting roadways to see oncoming vehicles and select appropriate turning gaps. Sight distance must be unobstructed along both approaches at an intersection and across the corners to allow the vehicles simultaneously approaching to see each other and react in time to prevent a collision. Intersection sight distance should be obtained at every road approach, whether it is a signalized intersection or private driveway. In no case should the sight distance be less than safe stopping sight distance (minimum).
- **Stopping sight distance** is the minimum distance required for a vehicle traveling at a particular design speed to come to a complete stop after an obstacle on the road becomes visible. This distance is used frequently for fatal-flaw screening in project analysis.
- **Decision sight distance** should be provided at locations where multiple information processing, decision making, and corrective actions are needed. Sample locations where decision sight distance is needed include unusual intersection or interchange configuration and lane drops.
- **Passing sight distance** is the minimum distance required for a vehicle to safely and comfortably pass another vehicle. If adequate passing sight distance opportunities cannot be accommodated in the project design, passing lanes or climbing lanes should be considered.

[3] AASHTO A Policy on Geometric Design of Highways and Streets, 6th Edition, 2011.

4.8.3 Conflict Points

Introduction

It is good practice to determine the conflict points at intersections and major accesses in the study area for most analysis work other than TSPs. Bicycle and pedestrian conflicts especially should be determined in areas of high multimodal demand such as transit-oriented developments (TODs) and MMAs. Areas with functional area overlaps or overrepresentation of crashes should have the conflict points quantified. The number of conflict points can also be a good alternative screening evaluation criteria.

Every roadway access creates conflict points for drivers, pedestrians, and bicyclists. Conflict points are locations where one vehicle path impacts another. Each conflict point is a possible crash location. Crashes occur at conflict points when one roadway user fails to yield to another. The crash potential associated with each conflict point varies depending on the complexity, volume of the movements, and speed. Multilane highways have more conflict points and a higher crash potential because of the increased exposure area, exposure time, and potential for obstructed sight distance by vehicles in adjacent lanes. Reducing the number of conflict points decreases crash potential.

Conflict points are classified as diverging, merging, weaving, turning, and crossing. Crossing paths are major conflicts. Diverging, merging, and weaving paths are minor conflict points. Diverging conflicts occur where one path separates into two. Merging conflicts occur where two paths come together. Weaving conflicts involve vehicles changing paths. Both major and minor conflict points may occur at high speeds, but minor conflicts typically involve vehicles traveling in the same direction. Vehicles crossing paths at high speeds may not have the sight distance or ability to minimize the severity of the crash.

Turning and crossing conflicts can also involve pedestrians and bicyclists. A crossing conflict point, which involves pedestrians and bicyclists, occurs where a vehicle path passes through a crosswalk or bike path; a turning conflict point occurs where the turning vehicle path passes through a crosswalk or bike path. Pedestrian crashes are not limited by locations. The pedestrian-vehicle conflicts are counted separately from vehicle-vehicle conflict points.

This section is a reference only. Every intersection is unique and must be analyzed appropriately. The analysis should consider geometry, permitted turn movements, the level of control of non-permitted movements, and the number of lanes for each movement. Reducing conflict points allows drivers to move through an area with less distraction so that traffic flows smoothly at constant speeds. With fewer conflict points, drivers can better maintain their attention on roadway conditions.

Conflict points can be reduced through three measures:

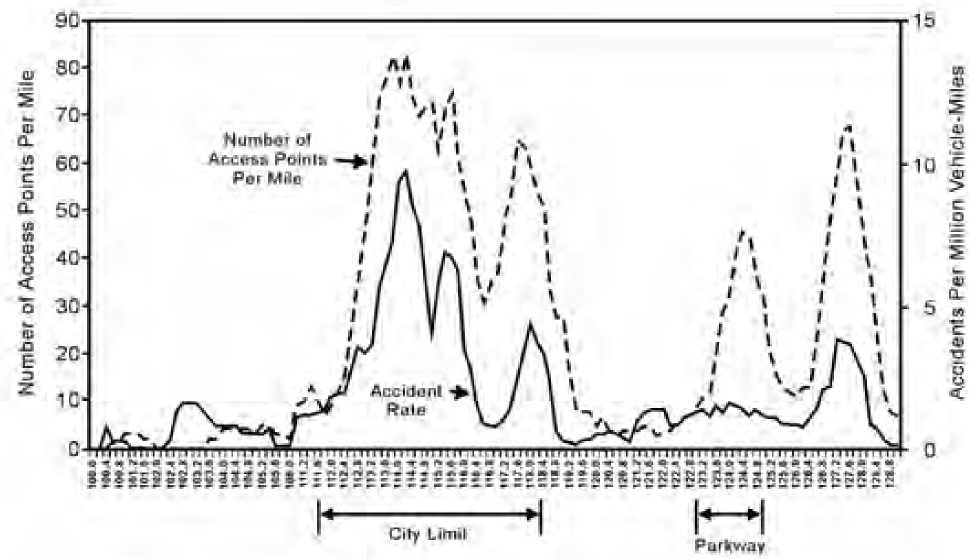
- Limit the number and/or type of access points
- Install medians, channelization, and other control devices (e.g., roundabouts) to restrict or control turning movements
- Grade separate traffic flows

Limit the Number and/or Type of Access Points

Limiting driveways or access points should be considered at locations with limited sight distance, high crash frequency, high volume-to-capacity ratios, or poor access to facilities. Combining driveways reduces access and conflict points. Accommodating entering traffic at one location simplifies driver tasks. Combining multiple driveways into one joint-use driveway directs traffic more safely, clearly, and efficiently.

Exhibit 4-33 shows the relationship between access density and crash rates in Lincoln City and Lincoln Beach. The crash rates increase as the density of access points increases. The area labeled City Limit in Exhibit 4-33 is Lincoln City on US101, which has a high density of access points. The area labeled Parkway is in Lincoln Beach where a non-traversable landscaped median limits access to driveways and side streets. Crash rates in the Parkway section are greatly reduced.

Exhibit 4-33: Access Points Per Mile vs. Crashes Per Mile



Comparison of Access Connections Per Mile with
Crashes Per Mile on a Segment of US 101 in Oregon

From Analysis of Traffic Accidents Within the Functional Area of Intersections and Driveways
Technical Report Trans 1-95 1995

Locating driveways on lower classification roadways or backage/frontage roads also reduces the number of conflict points along the main roadway. Removing driveways from an arterial or a collector decreases delay caused by turning vehicles. Diverting traffic to local roads directs traffic to one access point and simplifies conditions.

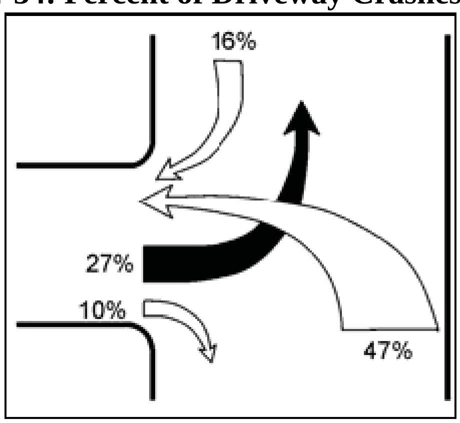
Refer to OAR 734, Division 51 for information concerning signal spacing, backage/frontage roads, or access rights. Sometimes access rights to individual parcels are obtained. Rules regarding obtaining property access rights to state highways, signal spacing, and backage/frontage roads are found in OAR 734, Division 51 and on the ODOT Access Management website.

Non-Traversable Median or Traffic Control Devices to Restrict Turning Movements

Medians are a roadway element intended to separate traffic traveling in opposite directions. A median can be traversable or non-traversable. A traversable median may be a painted or concrete mountable median that allows emergency traffic to cross over it. A non-traversable median is a physical barrier (examples include the Jersey barrier, landscaped, or grassy median) that separates opposing traffic and prohibits movement across the median. Installing a non-traversable median restricts turning and crossing movements at roadway accesses. Non-traversable medians reduce conflict points by eliminating turn movements that impact the general traffic flow. More information about median types can be found in Chapter 5.5 of the Oregon Highway Design Manual (HDM).

A median that impacts the State Highway Freight System must comply with the ORS 366.215 which states that the Oregon Transportation Commission (OTC) may not permanently reduce the vehicle-carrying capacity of an identified freight route when altering, relocating, changing, or realigning a state highway unless safety or access considerations require the reduction. According to the Transportation Research Board, Access Management Manual 2003, 47% of crashes involve left-turn ingress movements and 27% of crashes involve left-turn egress movements, shown in Exhibit 4-34. Controlling turn movements can allow one turn direction and divert others elsewhere.

Exhibit 4-34: Percent of Driveway Crashes by Movement



Non-traversable medians reduce overall crash frequency, improve pedestrian safety, and enhance visuals. They restrict traffic from making complex left turns and provide median openings at designated locations. They may also provide a pedestrian refuge between directions of traffic and decrease pedestrian clearance intervals. Including a raised median can significantly reduce pedestrian collisions at uncontrolled locations, with CMFs of 0.61 when used with an unmarked crosswalk and 0.54 when used with marked crosswalks.[4] Replacing a two-way left-turn lane with

[4] Zegeer, C. V., Stewart, R., Huang, H., and Lagerwey, P., "Safety Effects of Marked Versus Unmarked Crosswalks at Uncontrolled Locations: Executive Summary and Recommended Guidelines." FHWA-RD-01-075, McLean, Va.,

a raised median in urban environments has demonstrated a CMF of 0.77 for all crashes.[5] Landscaping large medians improves the aesthetics of the roadway. Although medians improve the roadway, they are costly and may require the acquisition of right-of-way.

Grade-Separated Roadways

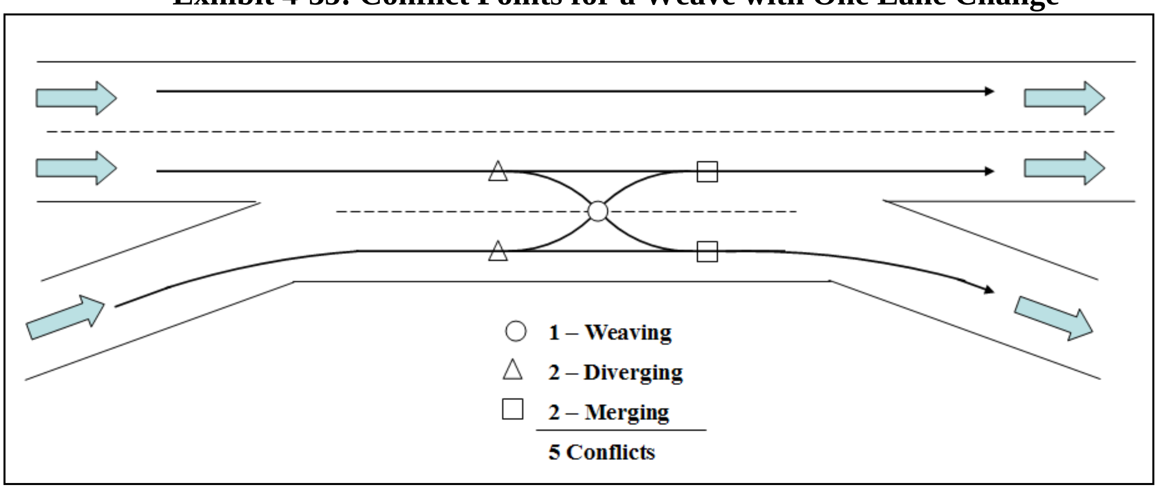
Grade-separated roadways are locations where one roadway crosses over the other. Grade separation should be considered where an at-grade signal cannot accommodate traffic capacity or where crash history indicates a need for grade separation. Conflict points are decreased by removing major flow grade crossings and by rerouting turning traffic. Interchanges are grade-separated connections of two or more roads. Interchanges reduce conflict points and the severity of crashes. Although interchanges occupy a large amount of space and require costly structural work, they reduce delay, reduce crashes, and improve efficiency of a corridor.

It can be difficult to accommodate pedestrians at interchanges because of the separation of paths. For more information about accommodating pedestrians or bicyclists, refer to the Oregon Bicycle and Pedestrian Plan and the related Bicycle & Pedestrian Design Guide.

Weaving Segments

Weaving segments, where vehicles traveling in the same direction cross paths, create several conflict points. Exhibit 4-35 and Exhibit 4-36 show two examples of weaving vehicle paths entering and exiting the roadway. A single-lane change weaving segment has five minor conflict points. A lane-balanced two-lane change weaving segment has seven minor conflict points. More information about weaving segments can be found in Chapter 12.

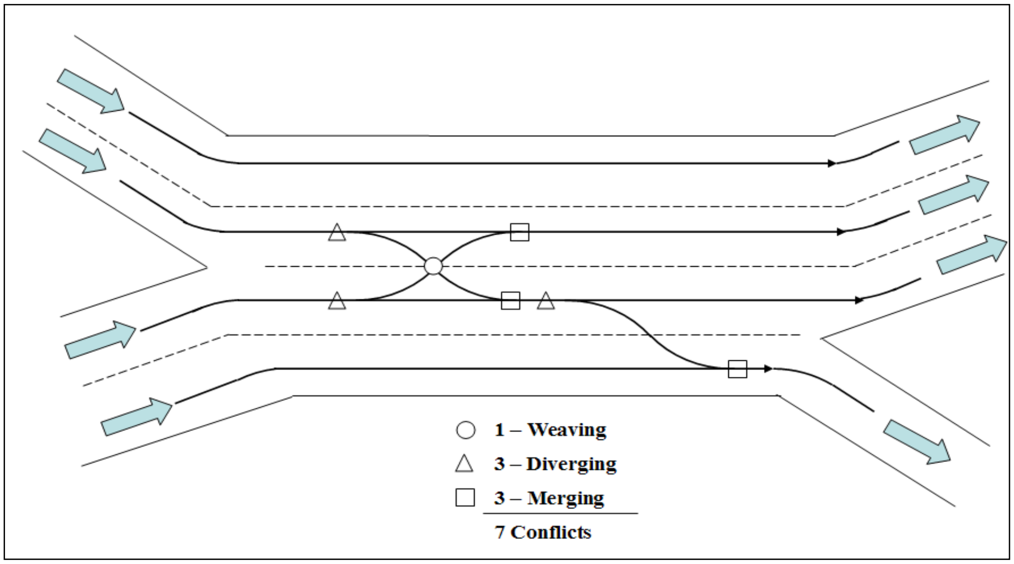
Exhibit 4-35: Conflict Points for a Weave with One Lane Change



Federal Highway Administration, (2002)

[5] Mauga, T. and Kaseko, M., "Modeling and Evaluating the Safety Impacts of Access Management (AM) Features in the Las Vegas Valley." Transportation Research Record: Journal of the Transportation Research Board 2171, pp. 57-65, 2010

Exhibit 4-36: Conflict Points for a Weave with Two Lane Changes



Unchannelized Intersections

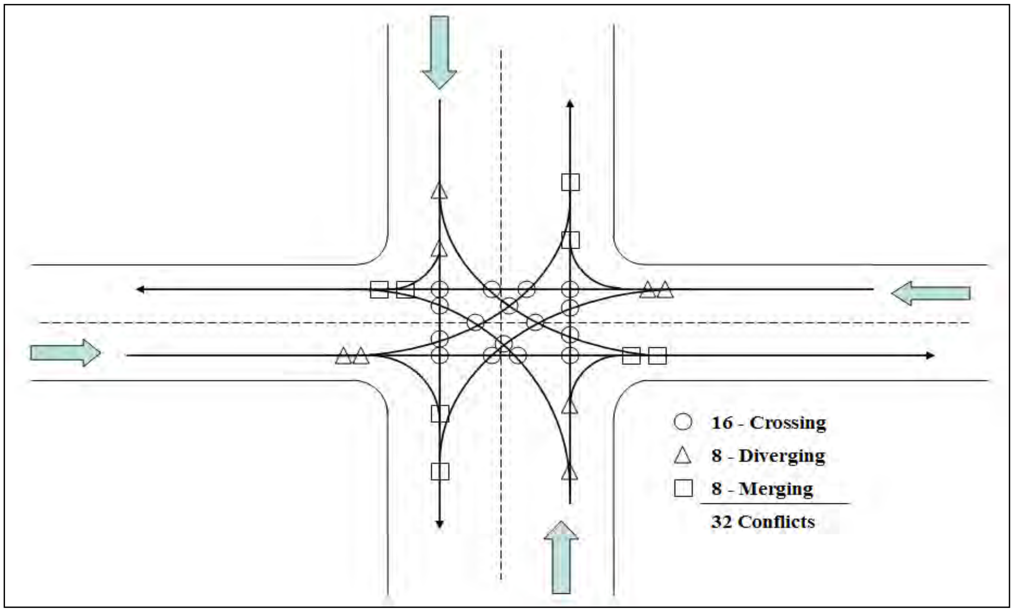
Unchannelized intersections are where two roadways without medians and/or turn restrictions intersect and that allow full access to traffic. Every such intersection should be analyzed as a unique situation. Unchannelized intersections may have other forms of traffic control such as a traffic signal. Although a traffic signal does not reduce the number of conflict points, it increases driver communication and awareness. For movements that operate with separate signal phases, the exposure to conflicts is significantly reduced compared to movements that operate with unsignalized control or permissive signal phasing.

Note: The intersections analyzed and illustrated in the figures are typical configurations; however, for clarity, turn lanes are not included. The addition of single turn lanes should not increase the number of conflict points. Any additional through or dual turn lanes must be included in the analysis.

Four-Leg Intersection

The four-leg intersection with no medians, shown in Exhibit 4-37, has the most conflict points of all intersections. The four-leg intersection allows movement in all directions and is the most familiar to drivers.

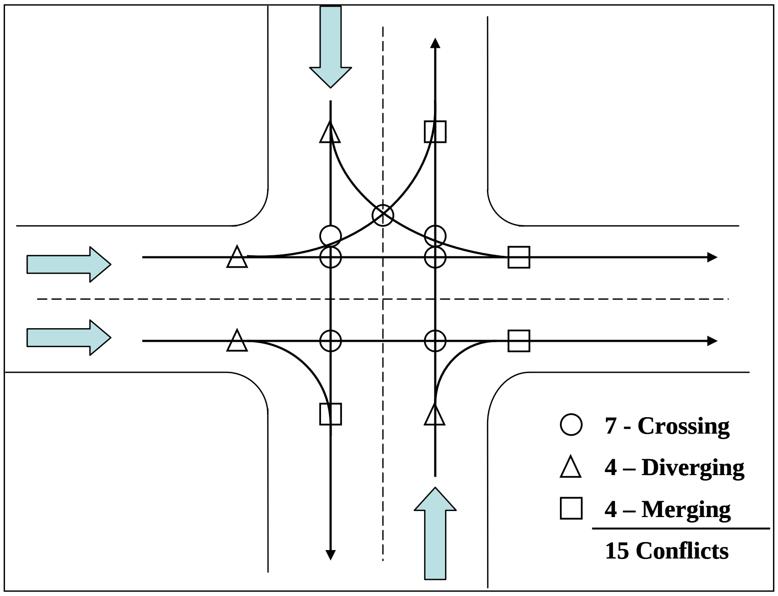
Exhibit 4-37: Conflict Points for a Four-Leg (Both Two-Way Roads) Intersection



Four-Leg Intersection of a Two-Way Road and a One-Way Road

A four-leg intersection of a two-way road and a one-way road is shown in Exhibit 4-38. The one-way road may be part of a couplet. The one-way road limits turns and reduces conflict points.

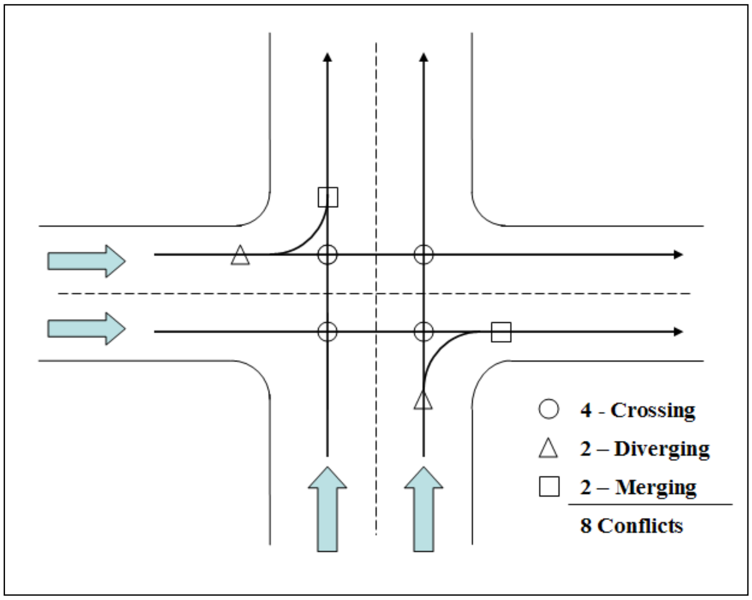
Exhibit 4-38: Conflict Points for a Four-Leg Intersection of a Two-Way Road and a One-Way Road



Four-Leg Intersection of Two One-Way Roads

A four-leg intersection of two one-way roads is shown in Exhibit 4-39. The one-way roads may be part of a couplet. The one-way roads limits turns and reduce conflict points.

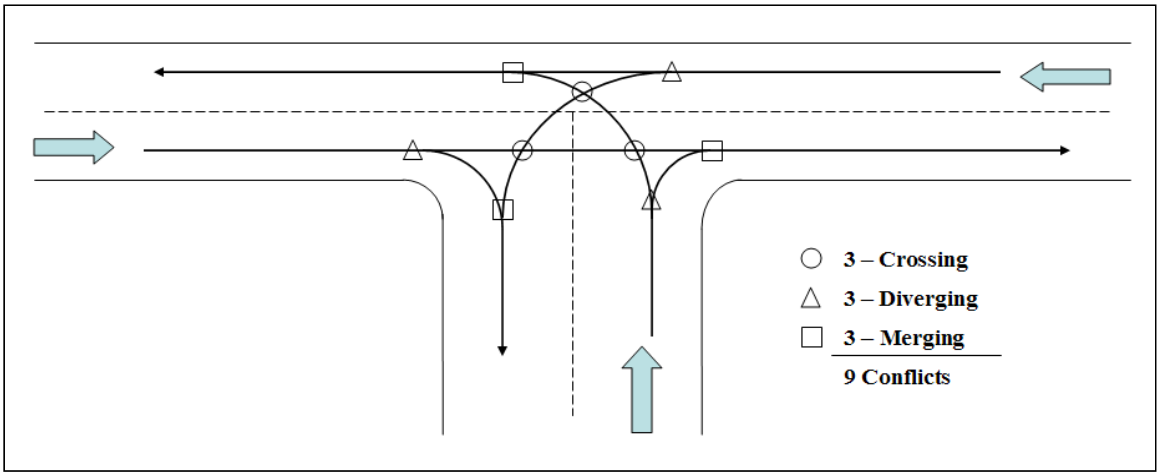
Exhibit 4-39: Conflict Points by Lane for a Four-Leg Intersection of Two One-Way Roads



T-Intersection

A T-intersection is a location where one roadway ends at its intersection with another roadway. The T-intersection shown in Exhibit 4-40 permits turns in all directions. The intersection in the figure has nine conflict points, three of which are major.

Exhibit 4-40: Conflict Points for the T-Intersection



Channelized Intersections

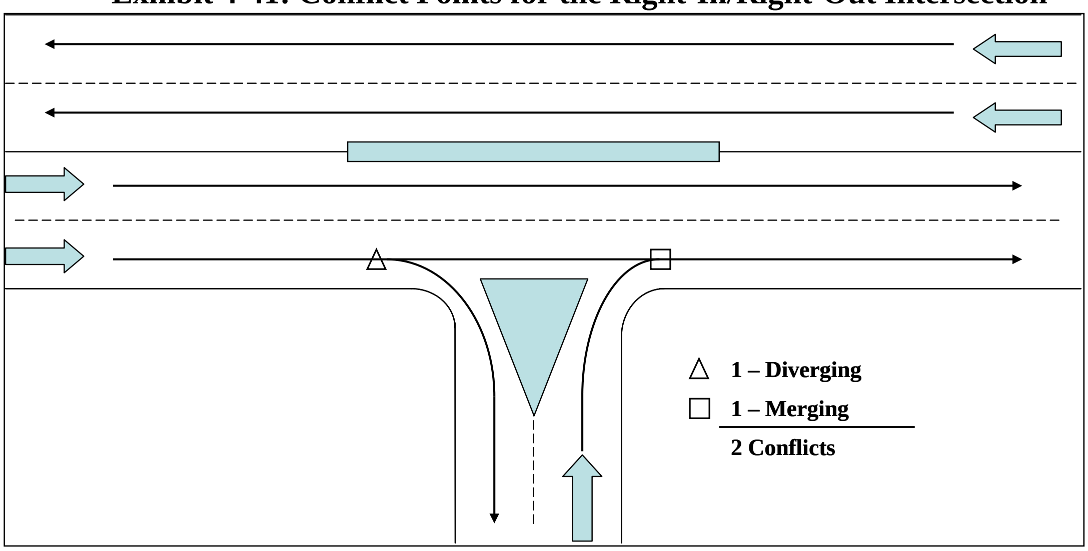
A channelized intersection restricts turn movements by signs, pavement markings, medians, or some other type of traffic control. Channelized intersections include, but are not exclusive to, right-in/right-out intersection, non-traversable median separated four leg intersection, left-turn ingress intersection, left-turn egress intersection, and roundabout. Both ends of the non-traversable median should be analyzed for the required traffic control in order to meet traffic needs safely.

Intersections and driveways that are restricted to right-in/right-out have two conflict points, but complex channelized intersections may have up to eleven conflict points.

Right-In/Right-Out (RIRO) Intersection

The right-in/right-out (RIRO) geometry shown in Exhibit 4-41 restricts traffic to right-turn movements only and forces roadway users to complete a left turn at another location either in a permitted U-turn or at a completely different intersection or roadway. Additional lane changing/weaving may also be necessary. This reduces crashes at the location of the right-in/right-out intersection, but requires left-turning vehicles to travel farther to get to their destination. The analyst needs to address where the left-turning vehicles will end up so potential safety issues are not created elsewhere.

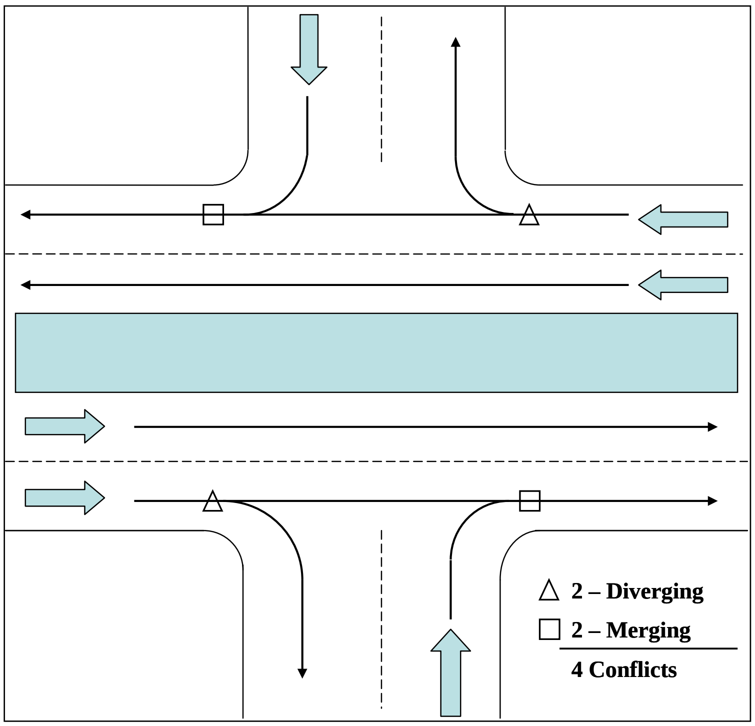
Exhibit 4-41: Conflict Points for the Right-In/Right-Out Intersection



Non-Traversable Median Separated Four-Leg Intersection

Installing a non-traversable median changes the traffic flows of the typical four-leg intersection by effectively creating two right-in/right-out intersections as shown in Exhibit 4-42. Left-turning or crossing vehicles must complete those maneuvers at another location, eliminating the major conflict points. This reduces the conflict points from the typical 32 to just four. The intersection is restricted to right turns, which improves safety and operations but also adds out-of-direction travel.

Exhibit 4-42: Conflict Points for a Median Separated Four-Leg Intersection



Roundabout

Roundabouts are considered at locations where speeds and volumes may not require a traffic signal for smooth operations. The roundabout, shown in Exhibit 4-43, directs all traffic to move in a counter-clockwise direction that allows movements in all directions. This limits conflict points to merging and diverging movements. It also reduces vehicular travel speed, which reduces the severity of any associated crashes. Bypass lanes would produce additional conflict points at their respective merge and diverge locations. A multilane roundabout, shown in Exhibit 4-44, has more conflict points and increased capacity. Chapter 13 contains capacity analysis procedures for roundabouts and bypass lanes.

Exhibit 4-43: Conflict Points for a Single Lane Roundabout

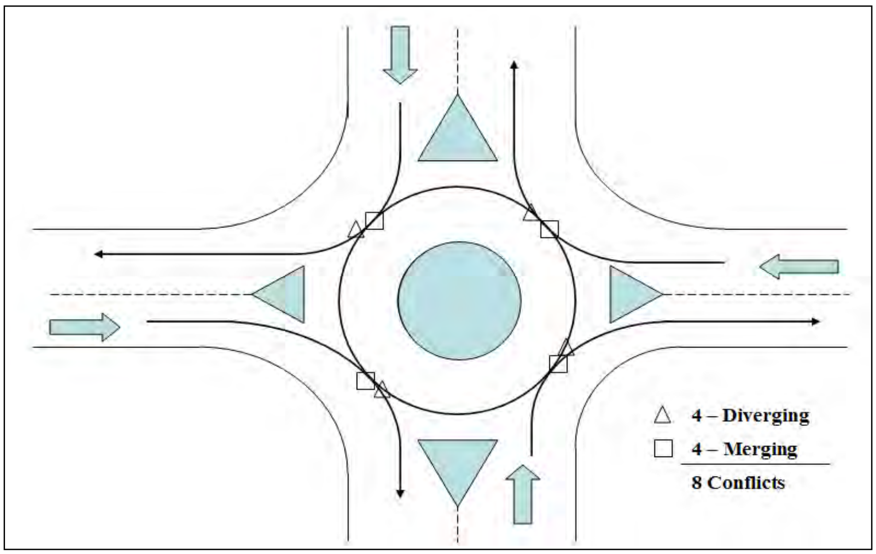
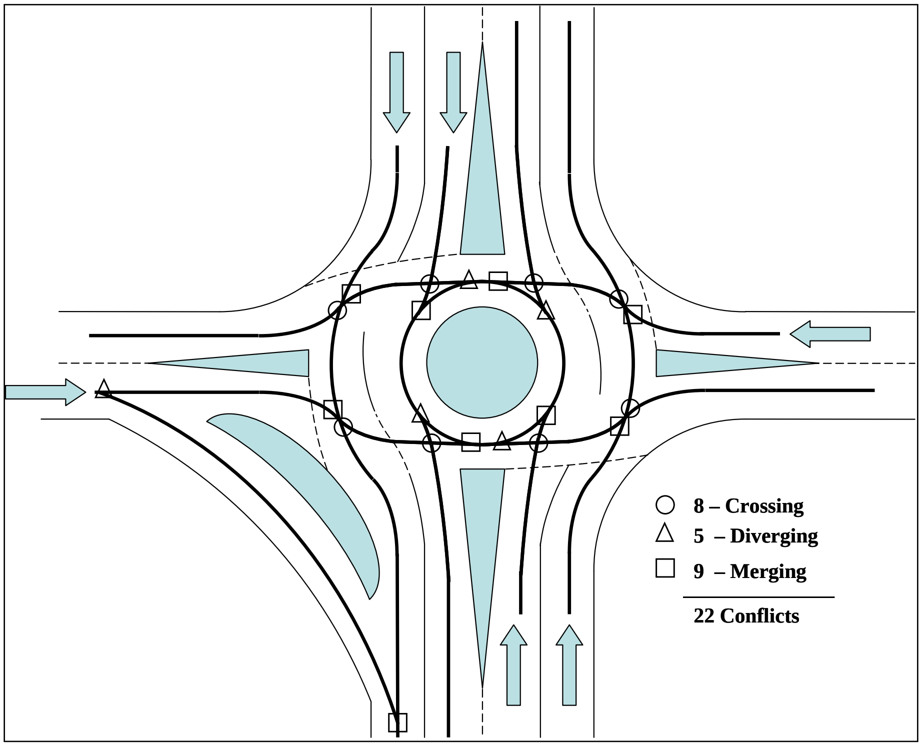


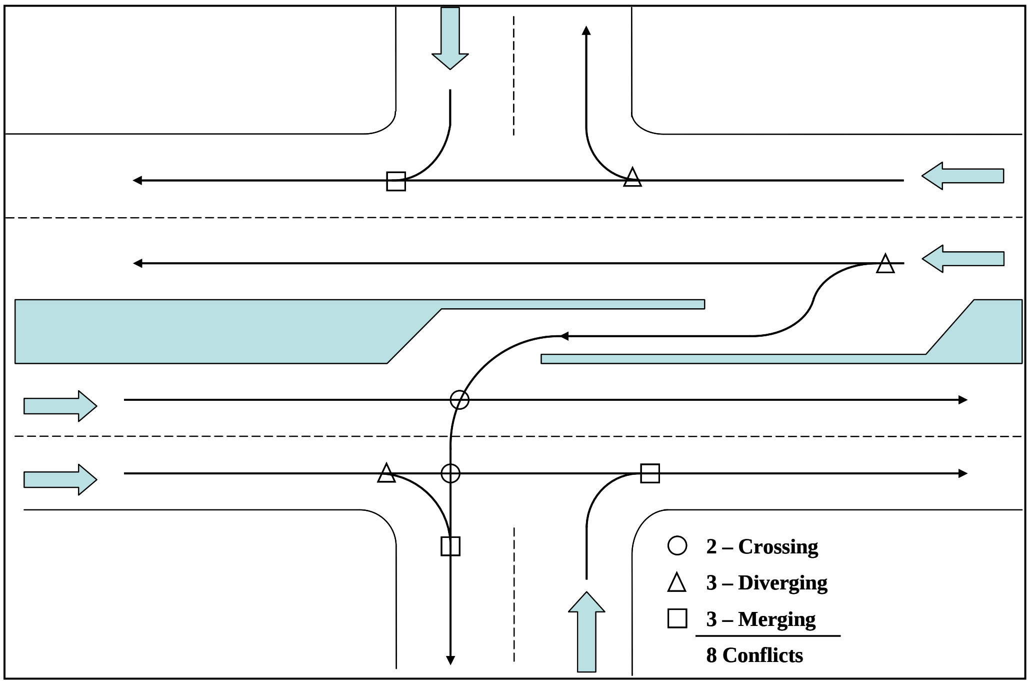
Exhibit 4-44: Conflict Points for a Multilane Roundabout



Left-Turn Ingress Intersection

The left-turn ingress intersection shown in Exhibit 4-45 permits one direction of traffic to turn left, from a turn bay, while the opposing left and crossing movements are prohibited. The permitted left turn may have significantly higher volumes or may service a critical access.

Exhibit 4-45: Conflict Points for a Median with One Left-Turn Ingress Intersection



Two Left-Turn Ingresses Intersection

Exhibit 4-46 shows the geometry for two left-turn ingresses. Only the traffic turning left into the access street is permitted through the median. Vehicles can remain in the median until there is a sufficient gap to complete the turn. More left-turn egress and left-turn ingress examples are shown in Exhibit 4-47 through Exhibit 4-49. Locations that provide a median restricting egress or ingress turns require median openings for vehicles to make U-turns. The median openings add a merge and a diverge point to the segment or intersection. Any intersection along a median that permits U-turns must be analyzed for conflict points, with the inclusion of the merge and diverge conflict points caused by the U-turn.

Exhibit 4-46: Conflict Points for a Median with Two Left Turn Ingresses Intersection

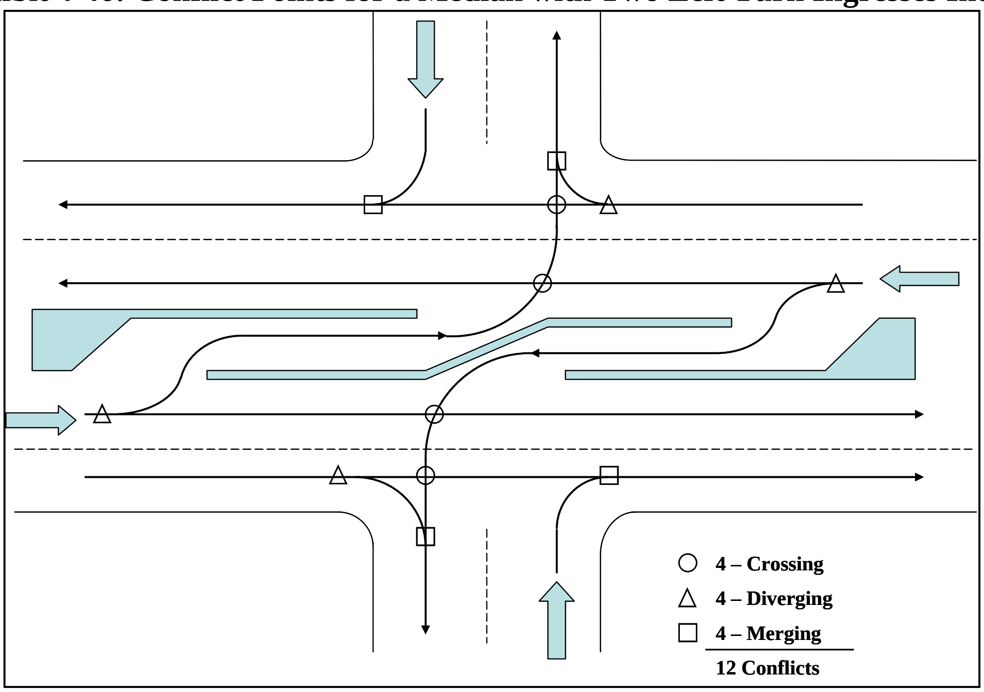


Exhibit 4-47: Conflict Points for a Median with a Left-Turn Ingress and Egress Intersection

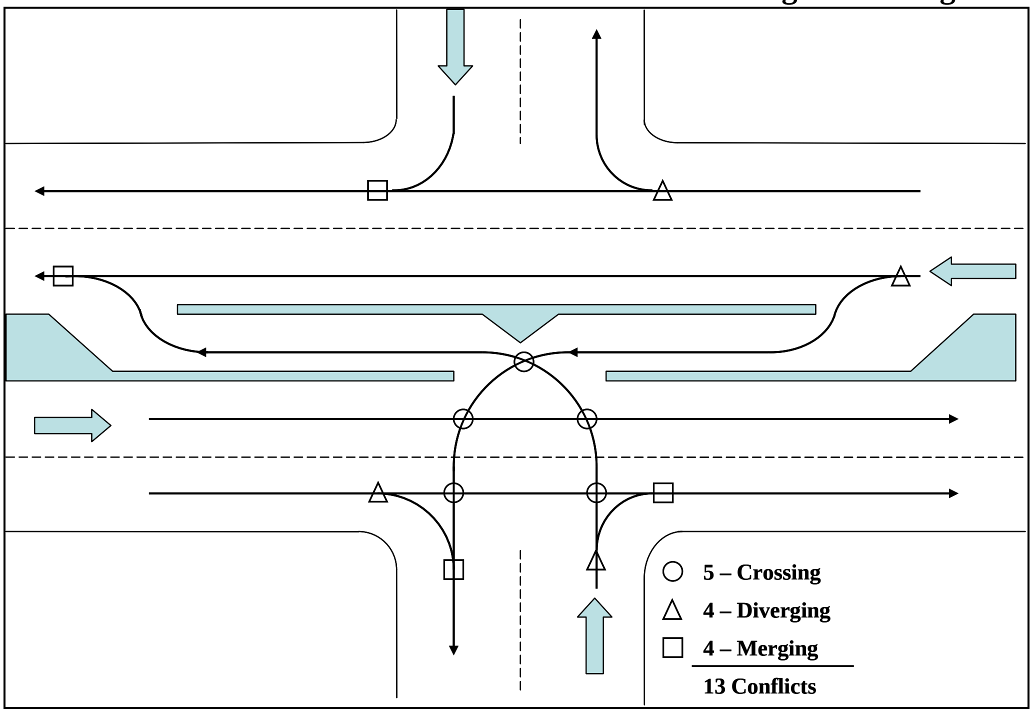


Exhibit 4-48: Conflict Points for a Median with One Left-Turn Egress Intersection

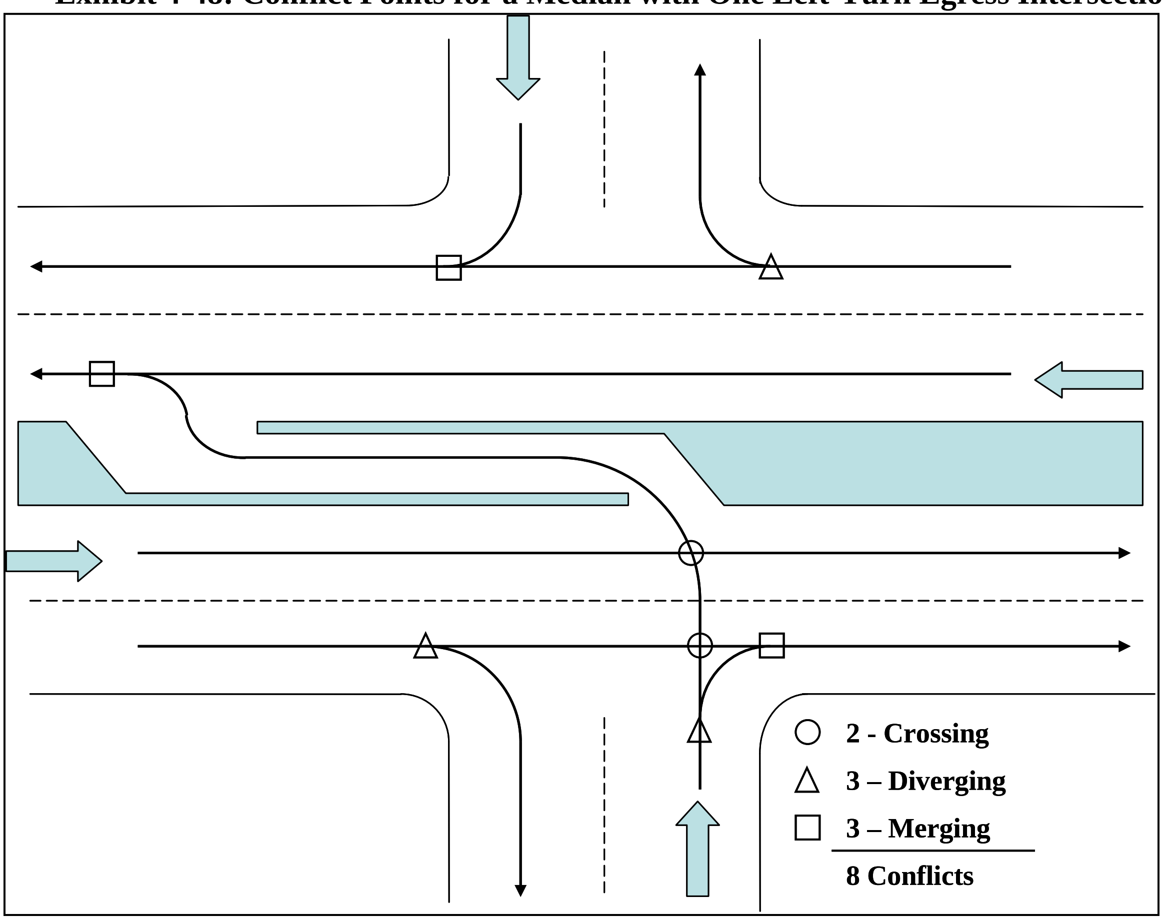
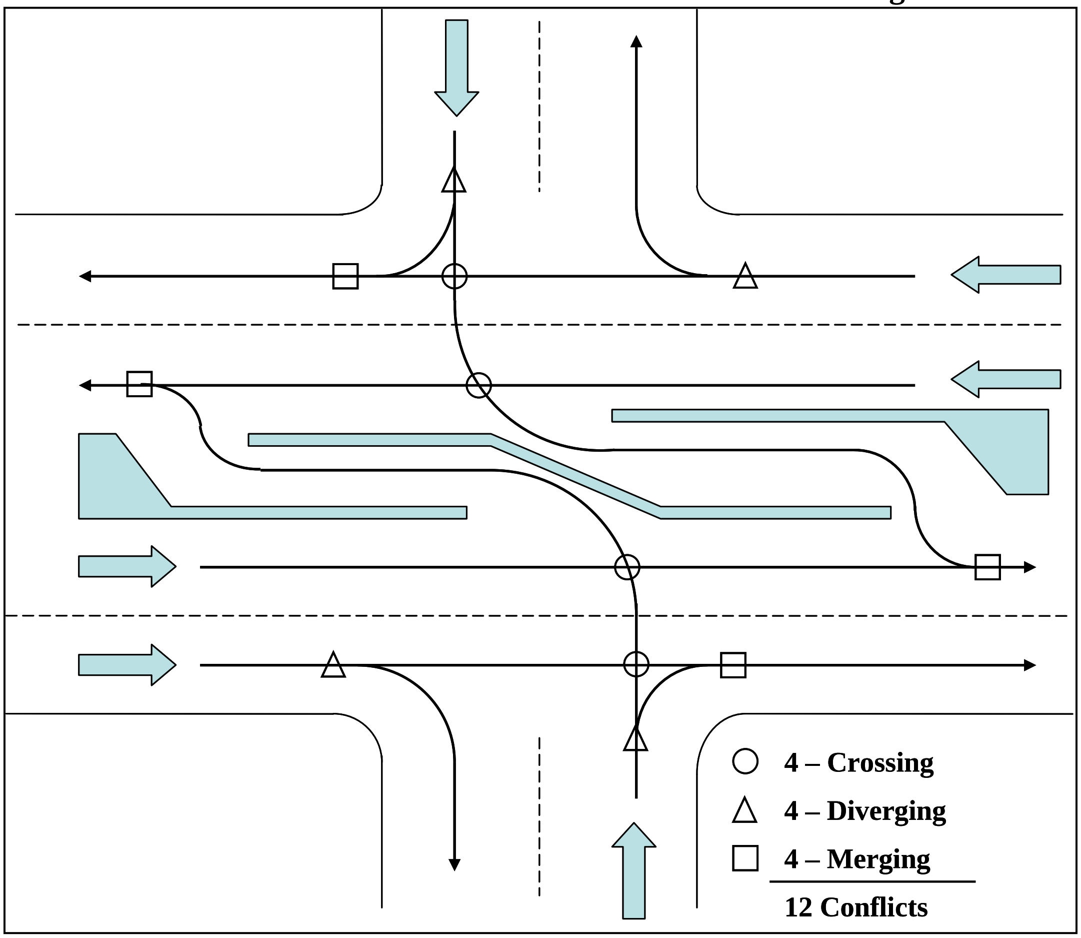


Exhibit 4-49: Conflict Points for a Median with Two Left-Turn Egresses Intersection



Indirect Left Turns

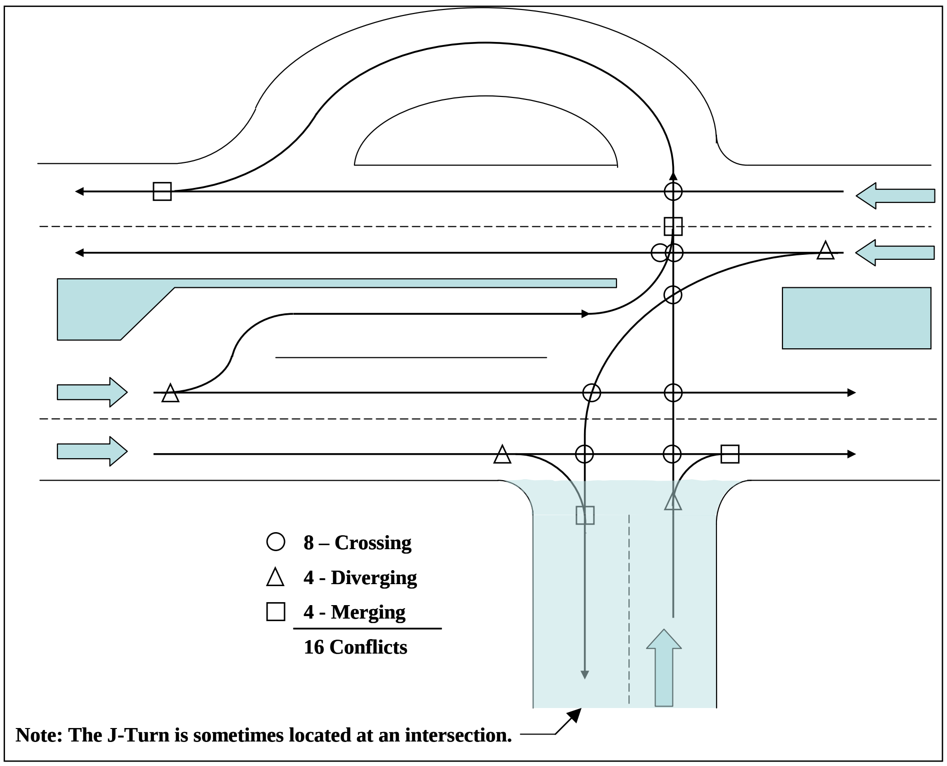
Some intersections require creative accommodations for left turning vehicles. The treatment of left turns must be considered at intersections with restricted turns, high volumes or high speeds to achieve the greatest capacity and safety for traffic.

J-Turn

The J-turn is an opportunity for larger vehicles to turn left on either side of a non-traversable median. The median restricts intersections and driveways to right-in/right-out, which reduces the conflict points. Vehicles can turn left at median openings which may be accompanied by a stand-alone J-turn, a J-turn intersection, or a signalized J-turn intersection. The J-turn intersection shown in Exhibit 4-50 has sixteen conflict points. Locations where the J-turn does not meet a crossing roadway or driveway have three conflict points: one diverge, one crossing, and one merge conflict point.

The J-turn intersection also allow turning vehicles to join traffic moving in its desired direction. An add-lane at the J-turn will eliminate the merge conflict point, shown in Exhibit 4-50, and increase the capacity of the segment. The J-turn reduces the congestion and improves the safety of median openings. A J-turn intersection may or may not have pedestrian crossings.

Exhibit 4-50: Conflict Points for a J-Turn Intersection

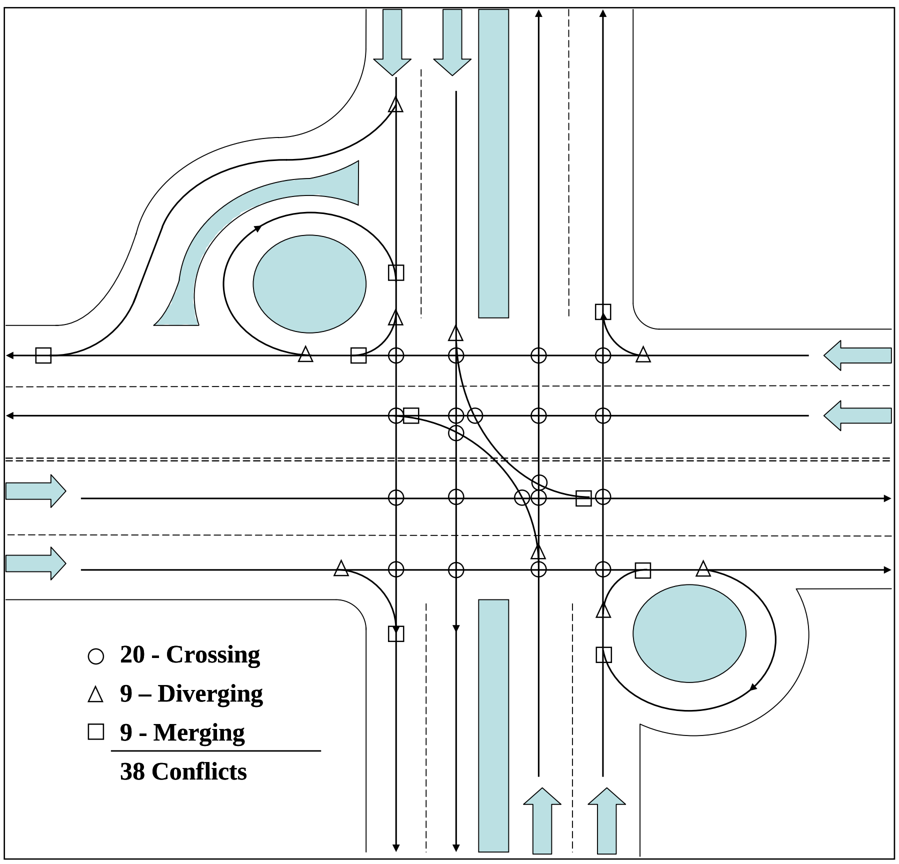


Jug Handle Intersection

The jug handle intersection, shown in Exhibit 4-51, is commonly used when there are high left-turn volumes which cannot be accommodated by a signalized intersection. The left-turning traffic passes through the intersection as a through movement both before and after diverging onto a loop ramp to complete the left turn. This reduces the congestion by removing phases from the traffic signal and improves the safety of the intersection by reducing the number of conflict points. The jug handle can be located in various quadrants of the intersection depending on the restricted turn movements.

Note: Jug handle configurations are not always installed as pairs and are not always accompanied with right turn bypass lanes.

Exhibit 4-51: Conflict Points for a Jug Handle Intersection



Pedestrian Conflict Points

Pedestrian conflict points are counted separately from vehicle-vehicle conflict points. Pedestrian conflict points are located at the intersection crosswalks or midblock crossings. Turning vehicles and crossing vehicles are counted separately. Intersections with wide cross-sections, such as a median separated four-way intersection, are more attractive to pedestrians when a curb-protected pedestrian refuge is provided between directions of travel. Exhibit 4-52 through Exhibit 4-54 show examples of pedestrian conflict points for intersections of two-way roads with and without a median and a four-way intersection with restricted left-turn ingresses.

Exhibit 4-52: Pedestrian Conflict Points for a Four-Leg Intersection

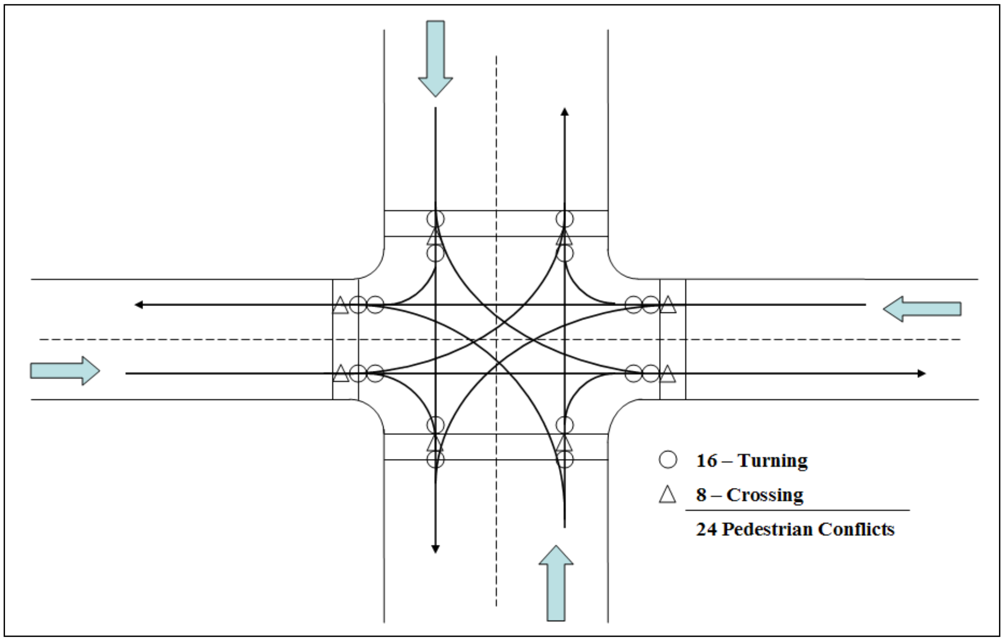


Exhibit 4-53: Pedestrian Conflict Points for a Median Separated Four-Leg Intersection

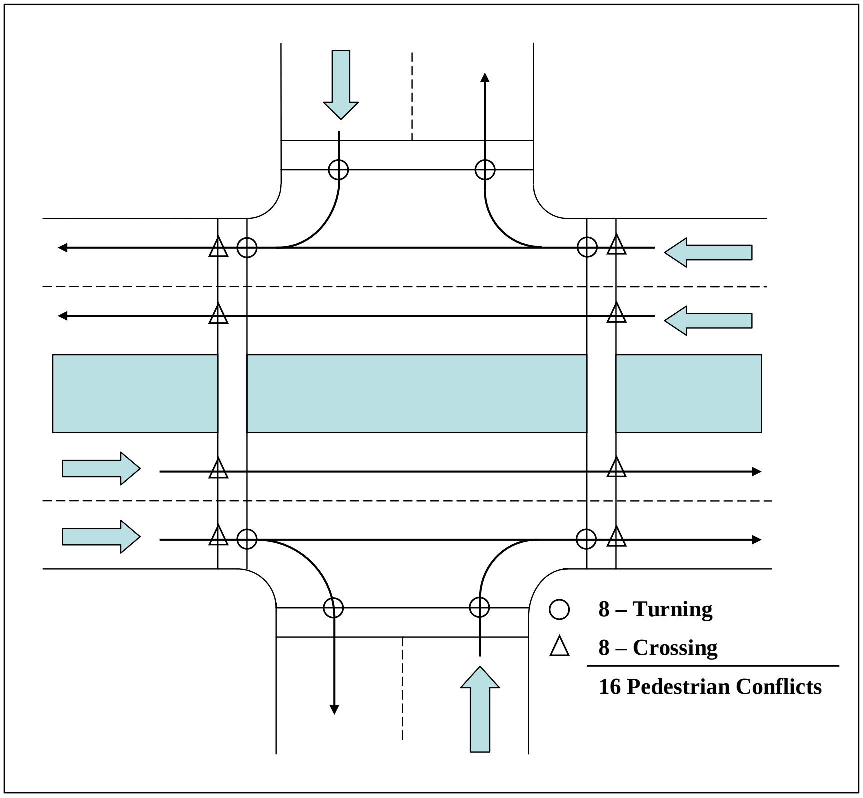
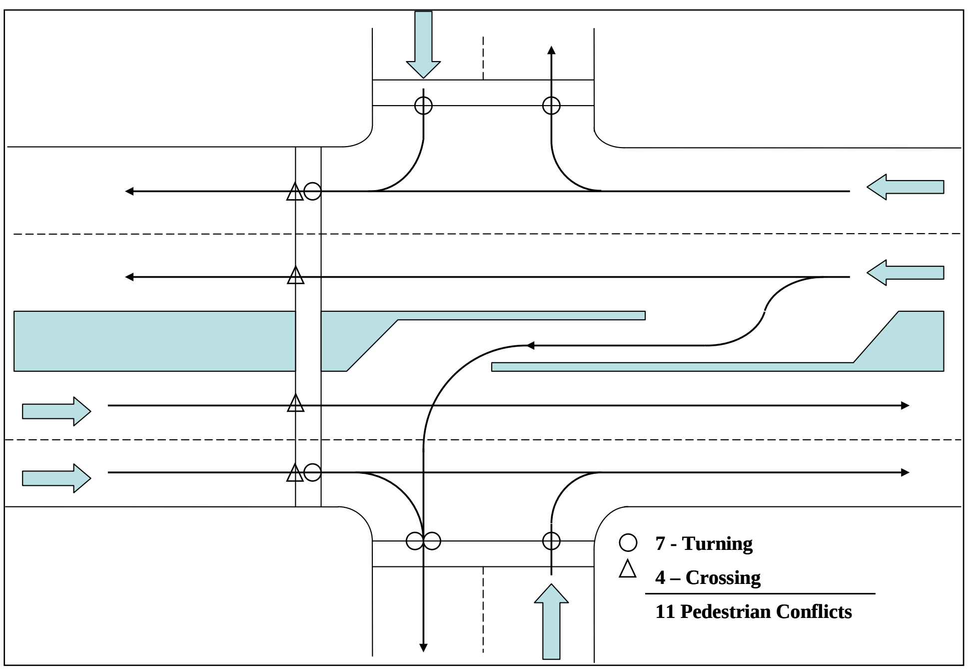


Exhibit 4-54: Pedestrian Conflict Points for a Median with One Left-Turn Ingress Intersection



Grade-Separated Access

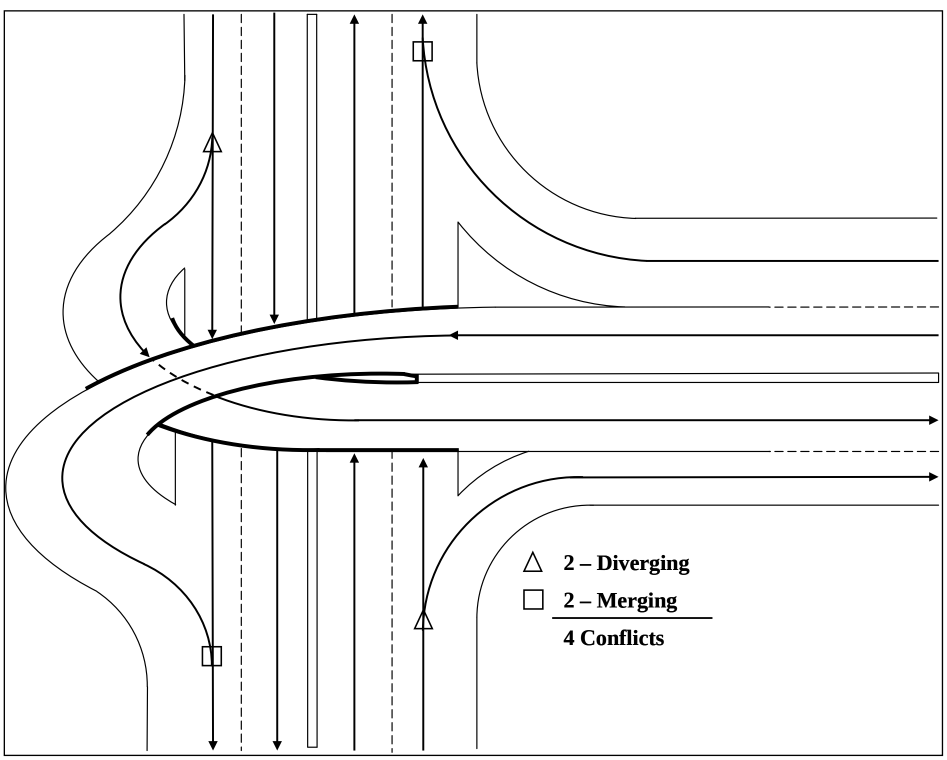
High-speed and high-volume junctions may need grade-separated access to meet traffic capacity while reducing crashes. Interchanges remove major flow grade crossings increasing capacity and reducing conflict points. Interchanges may have as few as six conflict points (directional interchange) or as many as 28 conflict points (left-turn flyover).

Directional Interchange

The directional interchange has all free flow ramps with only four (three diverging and three merging) minor conflict points in the system as seen in Exhibit 4-55. It is similar to a T-intersection with large volumes and high speeds. Traffic does not cross paths due to the grade separation. Each free-flow connection has a merge and diverge conflict point. Due to the high volumes and speed, pedestrian crossing does not occur at the street level.

The full-directional interchange, shown in Exhibit 4-55, has three levels of grade separation. A partial directional interchange is a junction where one leg has lower speeds and is accommodated by a loop ramp. A partial directional interchange has only two levels of grade separation. Since each free-flow ramp has one merge and one diverge conflict point, the partial directional interchange has the same conflict points as the full-directional interchange. Likewise, a four-way directional interchange would have eight conflict points, four merge and four diverge. Add itional conflict points could be introduced if ramps are closely spaced.

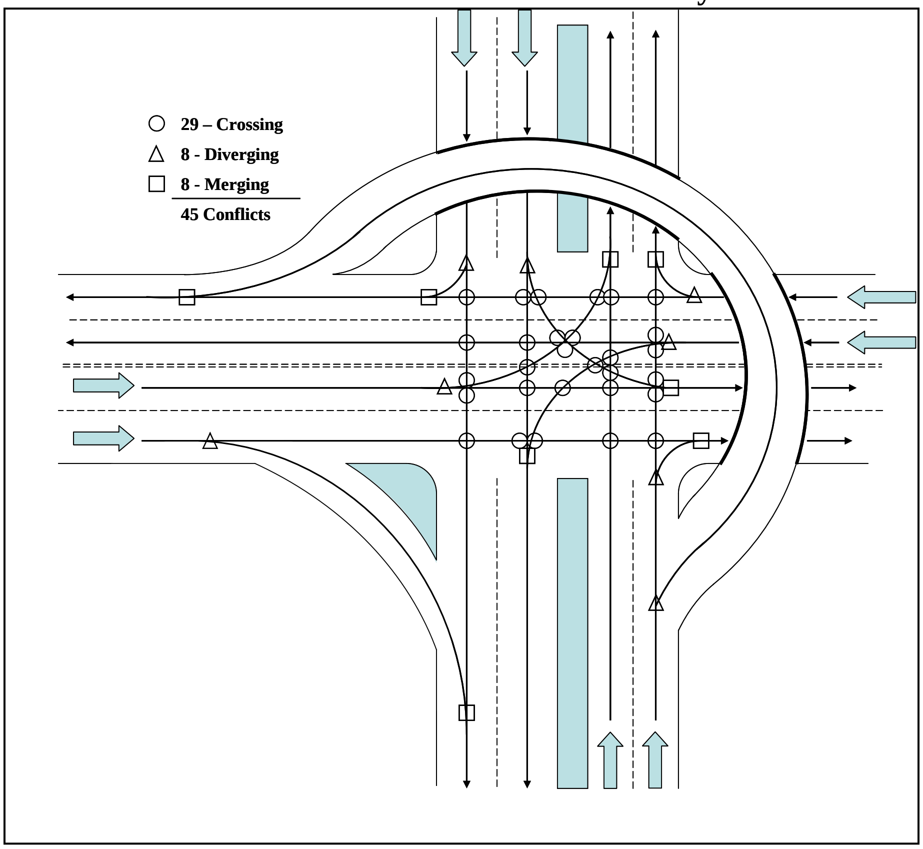
Exhibit 4-55: Conflict Points for a Directional Interchange



Left-Turn Flyover Intersection

The left-turn flyover intersection, shown in Exhibit 4-56, is commonly used when one direction has high left-turn volumes that cannot be accommodated by a signalized intersection. The left-turning traffic is grade-separated as it crosses over the opposing traffic, reducing conflict points and congestion. Pedestrian crossings may or may not be modified from the standard intersection.

Exhibit 4-56: Conflict Points for a Left-Turn Flyover Intersection

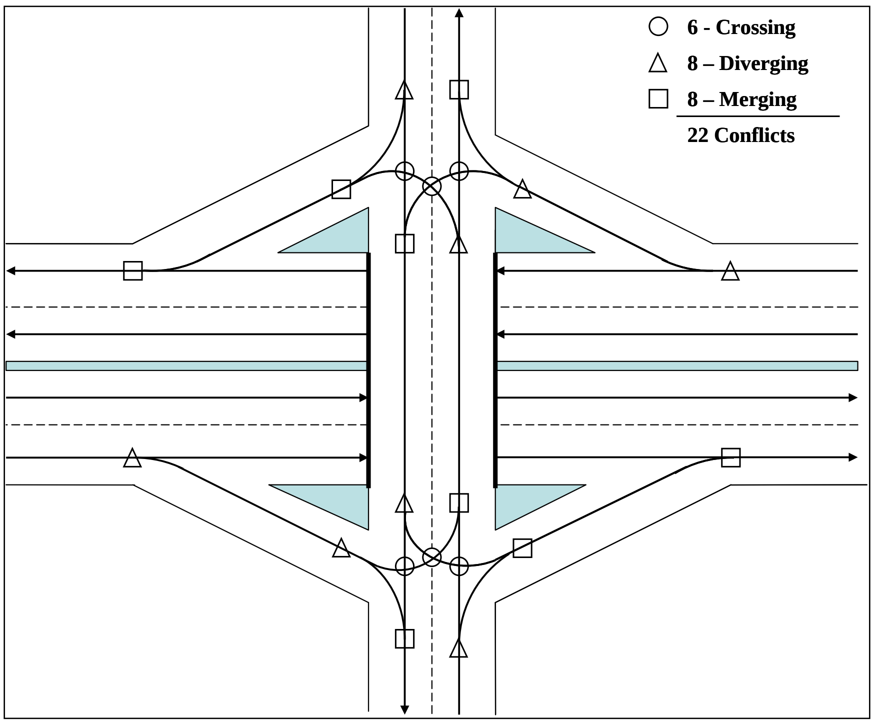


Diamond Interchange

The diamond interchange has four ramps and may have traffic control at the minor road. Traffic is directed to turn on or off each ramp, which creates conflict points. There is potential for weaving conflicts if this interchange has two or more lanes in each direction. Conventional, compressed, and tight diamond interchanges all travel the same paths and the conflict points are the same. Exhibit 4-57 shows the conflict point configuration for a conventional diamond interchange.

Pedestrian crossings are generally not provided at locations along the major, free-flowing movement.

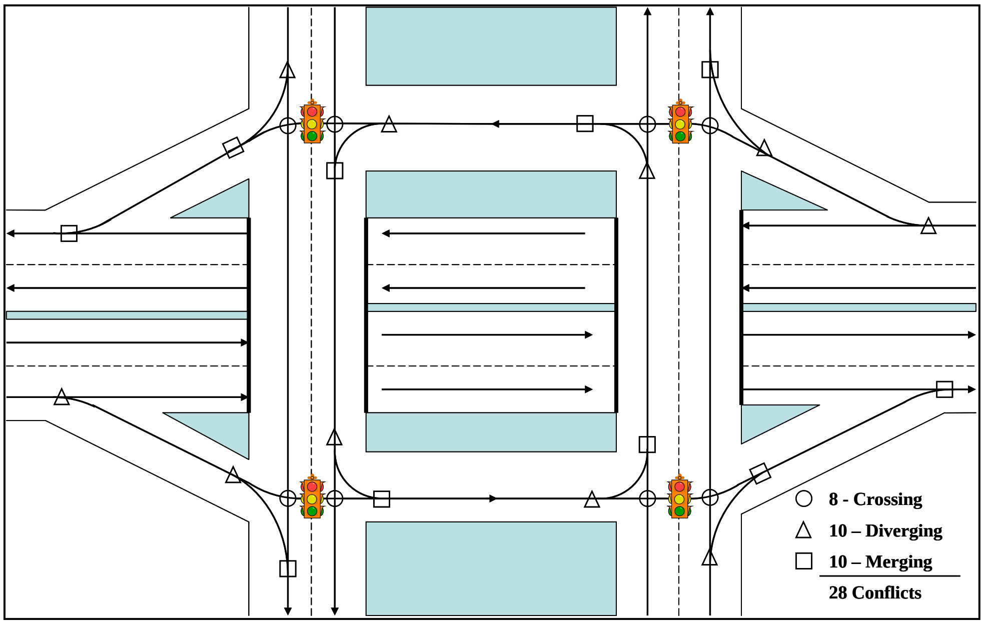
Exhibit 4-57: Conflict Points for a Diamond Interchange



Split Diamond Interchange

A split diamond interchange, shown in Exhibit 4-58, has only four ramps that connect to each other with segments that travel parallel to the major roadway. This type of interchange is appropriate where minor roads are one-way streets and will most likely be accompanied with traffic signals at the ramp terminals. It may be furnished on a regular grid system where the minor streets are two-way as well.

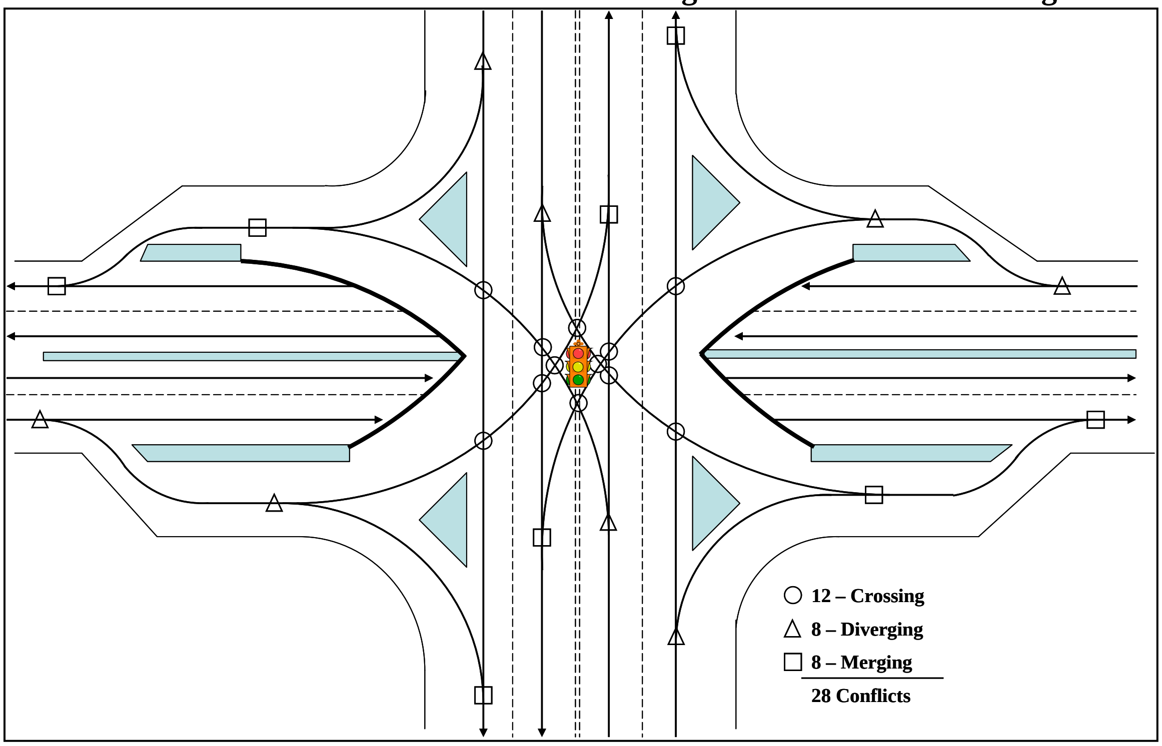
Exhibit 4-58: Conflict Points for a Split Diamond Interchange



Single Point Urban Interchange (SPUI)

The single point urban interchange has four ramps that converge to one point that is controlled by a traffic signal. All minor movement vehicles must travel through the same grade-separated intersection. This type of interchange conserves space and provides large capacity, since the signal operates with fewer phases. These conflict points are shown in Exhibit 4-59.

Exhibit 4-59: Conflict Points for a Single Point Urban Interchange

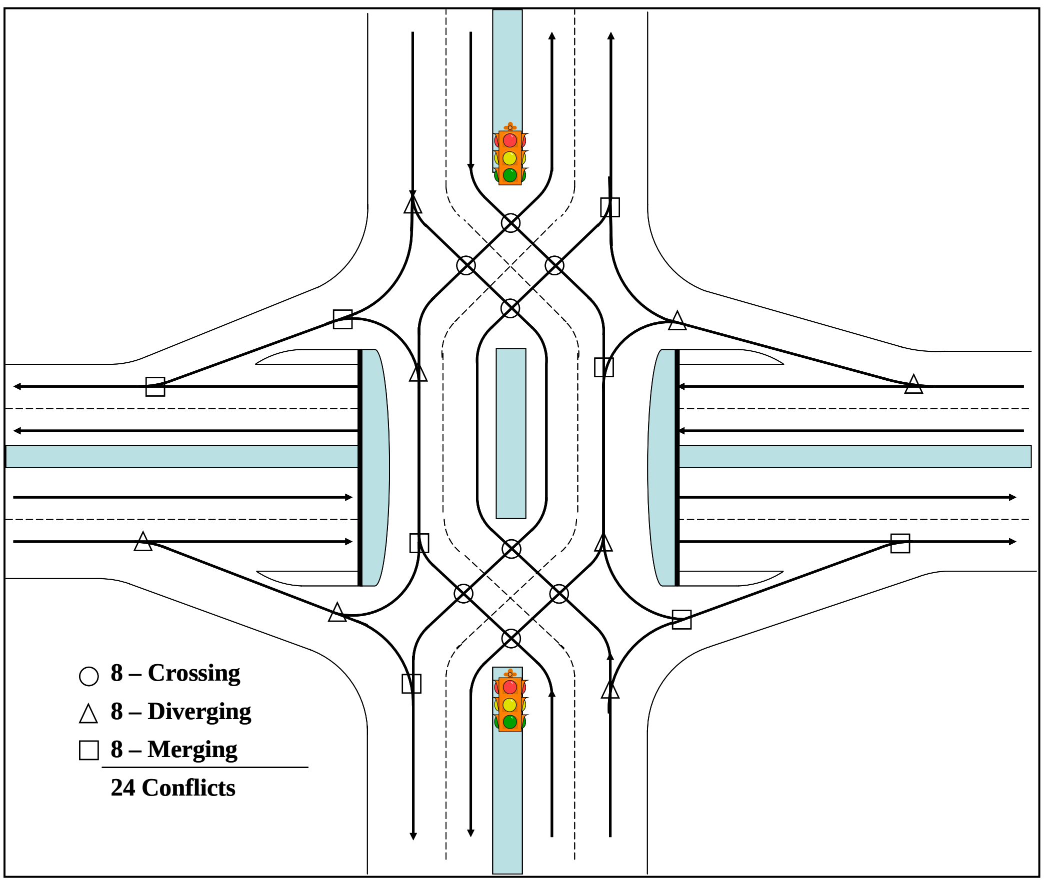


Divergent Diamond Interchange

The divergent diamond interchange has four ramps where vehicles that want to turn right may get on or off the roadway, as shown in Exhibit 4-60. The divergent diamond is designed so that as traffic on the minor roadway approaches the interchange intersections, the opposing lanes change the side of the road that is being used. This allows turn conflicts to be merge/diverge rather than crossing. Vehicles that want to turn left follow the appropriate traffic flow and merge into the receiving lane without interference of opposing traffic. The divergent diamond overlap allows vehicles to turn left at the designated signalized intersections reducing crossing paths and conflict points. This configuration also allows the use of fewer phases in the traffic signal operation.

Due to high speeds and high volumes, the divergent diamond must consider appropriate pedestrian crossings. There have been designs that show pedestrian crossings over the ramps and down the middle of the minor leg.

Exhibit 4-60: Conflict Points for a Divergent Diamond Interchange

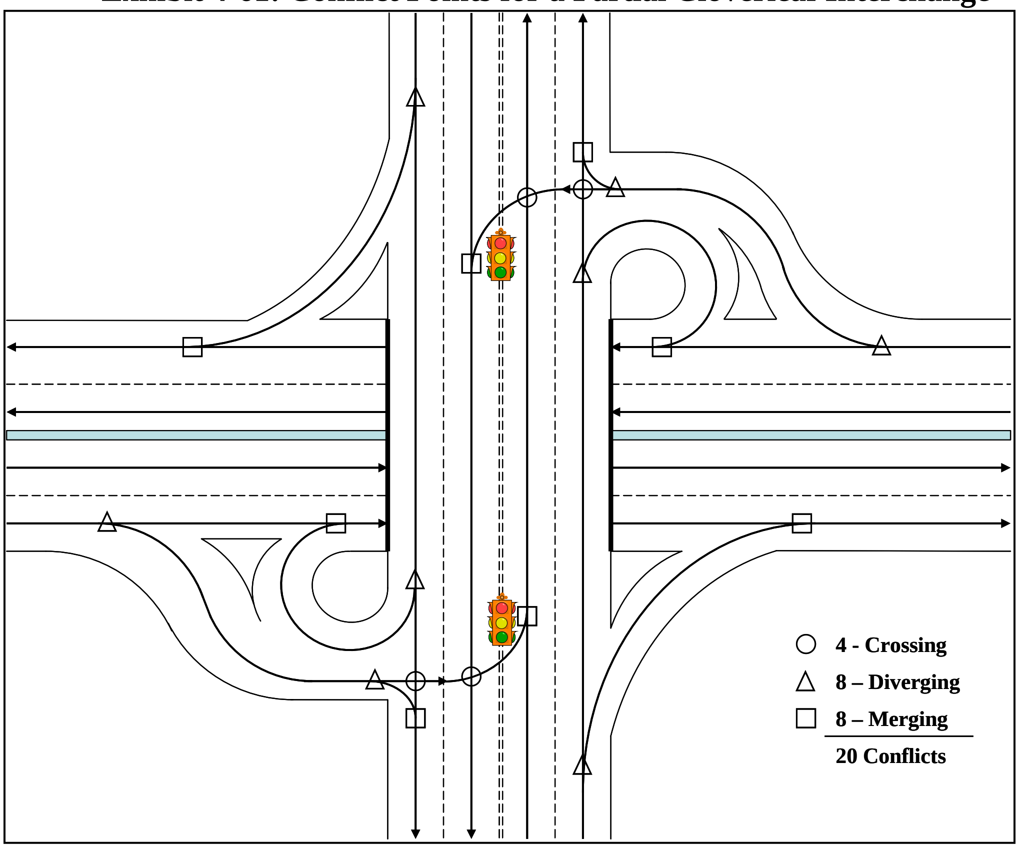


Partial Cloverleaf Interchange

The partial cloverleaf, shown in Exhibit 4-61, reduces conflict points by providing elevated ramps to maneuver on and off the roadway. The on ramps are free flow but the off-ramps are controlled by traffic signs or signals. Vehicle paths cross only for left-turning traffic from the ramps. There is potential for weaving conflicts if this interchange has two or more lanes in each direction.

Although a full cloverleaf configuration is a possible design, Oregon and many other states no longer use them because of the short distance between the merging/diverging paths of the ramps. If present, pedestrian crossings would direct people over the ramps where the speeds are lowest and drivers have adequate sight distance.

Exhibit 4-61: Conflict Points for a Partial Cloverleaf Interchange



4.8.4 Access Management

Access management is the location, spacing, design, and operation of driveways, median openings, interchanges, and street connections to a roadway. Appropriate access management promotes the safe and efficient use of the transportation network while providing access to adjacent land uses. Access management can often be a low-cost solution to congestion and reduce crashes.

In 1948, the Oregon State Highway Department was tasked with policing access to Oregon's state highways. The state highway approach permitting process was created to regulate the number of private driveways or public approaches onto a state highway. The Highway Division was also required to regulate the design of driveways, in order to ensure they complied with current design standards. When ODOT was established in 1973, access management remained a department duty with processes specified in the Oregon Administrative Rules (OARs). The goal of this policy was and is to protect viability of the state's highway infrastructure and to provide the motoring public with safe and reasonable access.

Note: Conflict points are discussed in detail in Section 4.8.3

4.8.5 Effects of Access Management Implementation

Good access management techniques improve both the roadway and adjacent land use. Details and research citations on the safety, operational, economic and other effects of access management as noted below are documented in the TRB's Access Management Manual[6].

Safety Effects

- Reduction of crashes involving vehicles as well as pedestrians and bicyclists[7]
- Limit number of traffic conflict points
- Separate conflict areas
- Preserve functional area of intersections
- Fewer conflicts for bicyclists
- Medians reduce crashes as well as provide pedestrian dwelling areas for two-stage crossings
- Reduced speed differentials resulting from through traffic meeting with turning traffic.
- Improved sight distance

Operational Effects

- Proper spacing of access points and intersections improves the flow of traffic. Poor spacing, design and location of driveways may reduce average travel speeds by up to 5 mph to 10 mph from desired speeds.[8]
- Reduced access density improves free-flow speed and reduces delay and congestion
- Up to 40% fewer vehicle hours of delay on access controlled roadways as compared to uncontrolled roadways
- Preserve integrity of the roadway system
- Extend functional life of the roadway

[6] Access Management Manual, Transportation Research Board, 2003.
[7] HSM Part D, Chapter 13, Section 13.14 includes CMFs for access management on rural two-lane roads and urban/suburban arterials. See also ODOT Driveway Safety Models.
[8] Access Management and the Relationship to Highway Capacity and Level of Service, Florida DOT, 1996

Economic Effects

- Access improvements on corridors have been shown to result in increased property values by decreasing travel time.
- Predictable travel times benefit service industries and manufacturing facilities operating under "just in time" delivery contracts
- Combining driveways creates more room for parking and landscaping and may result in lower maintenance. Providing cross-access between retail parking lots often encourages multistop business trips by customers who otherwise may not have stopped.
- Non traversable median projects generally have little or no overall adverse impact on business activity.

Other Effects

- Improved traffic flow resulting from access management reduces vehicle emissions and fuel consumption.

Driveway Safety Assessment Predictive Models

ODOT has produced a predictive model to assist with assessing driveway safety on rural and urban arterial roadways in Oregon. The Revised ODOT Driveway Safety Models are available as a spreadsheet tool with instructions, with background information available in the Validation Report and Original SPR 720 Report.

The Revised ODOT Driveway Safety Models spreadsheet can be used to quantify and predict the effect of access management changes on non-intersection crashes. It includes a model for urban arterials and for rural arterials. The model has the following data requirements:

- Segment length
- Annual average daily traffic
- Speed limit
- Total number of through travel lanes
- Presence of a two-way left-turn lane (urban only)
- Number of commercial plus industrial driveways (urban only)
- Total driveways in segment (rural only)
- Number of industrial driveways (rural only)
- Total number of driveway clusters (rural only, methodology included in spreadsheet)

The spreadsheet provide the number of predicted crashes in five years, split into a baseline exposure value and an additional roadside and driveway effect value.

Rules, Policies, and Guidance

Laws pertaining to the control of access to public highways in Oregon are found in Oregon Revised Statutes (ORS) Chapter 374.

Administrative rules for highway approaches, access control, spacing standards and medians are found in OAR Chapter 734, Division 51 (OAR 734-051). Division 51 establishes procedures, standards, and approval criteria used by ODOT to govern highway approaches, access control, spacing standards, medians and restriction of turning movements.

The Oregon Highway Plan (OHP) serves as the policy basis for implementing OAR 734-051 through three goals:

- **Goal 1: System Definition.** To maintain and improve the safe and efficient movement of people and goods and contribute to the health of Oregon's local, regional, and statewide economies and livability of its communities.
- **Goal 2: System Management.** To work with local jurisdictions and federal agencies to create an increasingly seamless transportation system with respect to the development, operation, and maintenance of the highway and road system that:
    - Safeguards the state highway system by maintaining functionality and integrity;
    - Ensures that local mobility and accessibility needs are met; and
    - Enhances system efficiency and safety.
- **Goal 3: Access Management.** To employ access management strategies to ensure safe and efficient highways consistent with their determined function, ensure the statewide movement of goods and services, enhance community livability and support planned development patterns, while recognizing the needs of motor vehicles, transit, pedestrians and bicyclists.

The ODOT Access Management program website provides guidance on access management.

Types of Access Management Efforts

Access management encompasses a variety of different activities. The following is a brief description of each area. Detailed guidance on these efforts can be found on the ODOT access management website. It is important to coordinate with the Region Access Management Engineer (RAME) and other access management staff early in planning and project development processes.

Standards

Section 4020 of Division 51 contains standards for approval of private approaches, including the access spacing tables for roadways and interchanges. The spacing standards are also contained in OHP Appendix C. In addition, Table 12 of OHP Appendix C contains standards for spacing between interchanges.

Planning

Access management is one of the principal goals of the OHP (Goals 1, 2 & 3). Plans involving public road connections to the highway should be coordinated with the RAME as they are not subject to the approach permitting process. A resource for more information on access management in planning is NCHRP Report 548, *A Guidebook for Including Access Management in Transportation Planning*.

ODOT encourages the development of access management plans and interchange area management plans to maintain and improve highway performance and safety by improving system efficiency and management before adding capacity.

An **Access Management Plan (AMP)** is a plan adopted by the OTC for managing access on a designated section of highway or the influence area of an interchange to maintain and improve highway performance and safety. Detailed guidance on AMPs can be found on the ODOT access management website.

An **Interchange Area Management Plan (IAMP)** is a plan to determine transportation solutions or land use/policy actions needed in an interchange area and how best to balance and manage transportation and land use issues over time. It is an important tool in protecting the function and operations of state highway interchanges and the supporting local street network. Assistance in the preparation of IAMPs is available in the ODOT Interchange Area Management (IAMP) Guidelines.

Project Development

Access management in project delivery in addressed in Division 51 Section 734-051-5120. ODOT encourages the development of access management strategies and access management plans during project delivery to maintain and improve highway performance and safety by improving system efficiency and management before adding capacity.

An **Access Management Strategy (AMStrat)** is a product developed by the project team that identifies the location and type of approaches and other necessary improvements that will occur as part of the project. The strategy may range from general statements on a preservation project to maps showing specific access closures in a modernization project. Division 51 requires access management strategies for modernization projects, projects within an influence area of an interchange where the project includes work along the crossroad, or projects on an expressway. Access management strategies may be developed for other highway projects.

The goal of an AMStrat is to improve safety and operations through measures taken during project delivery. These may include physical improvements to reduce conflicts such as medians and deceleration lanes, with treatments of existing approaches to mitigate, modify, relocate, consolidate, or close them as needed and as resources allow.

Operational notices provide requirements and guidance on access management in project development and delivery. PDLT Notice 03 addresses access management in project development. PDLT Notice 03 addresses access management specifically for pavement preservation projects.

A tabular evaluation of spacing standards should be conducted as part of existing conditions, future no-build, and alternative analyses and documented in the appropriate technical memorandums and narratives. This should cover all applicable roadway segments in the study area. Local jurisdiction's spacing standards shall be used for non-state roadways.

Example 4-5 Access Spacing Standards Reporting

This is an example of a typical access spacing standards reporting at the project development level included in a technical report.

The spacing and location of intersections and driveways affect traffic safety and operations. These access points introduce conflicts and are frequently the cause of slowing or stopping vehicles that can significantly degrade the flow of traffic and reduce the efficiency of the transportation system. Appendix C in the OHP has spacing standards for public road approaches and private access to be used in the planning process.

The following spacing standards apply to this project:[9]

- **Interchange to interchange:** Two miles for freeway interchanges with two-lane crossroads in a rural area, measured between interchange lane tapers.
- **Next intersection adjacent to ramp terminal:** 1,320 feet for a two-lane crossroad in a rural area next to a full or right-in/right-out intersection.
- **Street spacing:** 1,320 feet for a rural statewide highway at 55 mph and 500 feet for a rural district highway at 45 mph. There is no standard for private accesses as they are discouraged on state highways.

The table below shows the comparison between roadway segments and their appropriate spacing standard. Existing street and ramp spacing is well below standards for all segments in the project area. In the example, on I84 the distance between the Biggs-Junction and Rufus interchange is approximately five miles, which is compliant with standards. Bargeway Lane directly accesses the I84 westbound on-ramp acceleration lane. The access point is less than 300 feet downstream of the I84 westbound ramp terminal intersection and approximately 600 feet upstream of the freeway entrance. Extending the length of the on-ramp, so that the acceleration lane begins west of the Bargeway Lane, will reduce the potential for accelerating vehicles to crash with slow moving and/or turning vehicles entering/exiting Bargeway Lane.

On US97, the distance between the I84 EB ramp terminal and the Celilo-Wasco/Biggs Rufus Frontage Rd and US97 intersection is less than half the spacing standard. Queuing and blocking on this segment will become an issue as volumes grow; however, because of the topography and waterway constraints, meeting the spacing standards for this segment will not be possible even under the best conditions. On the segment south of the Celilo-Wasco/Biggs Rufus Frontage Rd. and US97 intersection, there are two driveways for the Shell Gas and Travel Stop; the first (for cars) is approximately 550 feet south of the intersection and the second (for trucks) is approximately 900 feet south of the intersection.

On the Celilo-Wasco Spur, there are multiple driveways on both sides of the highway. The first, for McDonalds and the Grand Central Travel stop on the opposite side of the road, is approximately 150 feet west of the Celilo-Wasco/Biggs Rufus Frontage Rd and US97 intersection. The second, for both locations, is approximately 300 feet west of the intersection.

[9] OAR 734-051, Highway Approach Permitting, Access Control, and Access Management Standards. Effective June 30, 2014.

Spacing Standard Summary

Access Management Classification	Roadway	Segment	Spacing Standard*	Existing Conditions**
Interchange to Interchange	I84	Between Biggs Junction and Rufus Interchange	2 miles	5 miles
Next intersection adjacent to ramp terminal	US97	Between I84 EB ramp terminal and Celilo-Wasco/Biggs Rufus Frontage Rd and US97 intersection	1320 feet	600 feet
Street Spacing: Statewide Highways	US97	Between Celilo-Wasco/Biggs Rufus Frontage Rd and US97 intersection and entrance to Shell Gas and Travel Stop	1320 feet	550 feet
Street Spacing: District Highways	Celilo-Wasco Spur	Between Celilo-Wasco/Biggs Rufus Frontage Rd and US97 intersection and the entrance to McDonalds / Travel Stop	500 feet	150 feet

** OHP Spacing Standards*
***Approximate approach spacing*
Black-shaded cells indicate that the spacing standard is not met.

Permits

An Application for State Highway Approach is required for new approaches and in other circumstances. The outcome of the permitting process is issuance of a permit to construct the approach. Once construction is approved, a final Permit to Operate, Maintain and Use a State Highway Approach is issued. Details on the permitting process are found in Division 51 as well as Chapter 4 of ODOT's Access Management Manual.

Access Management Techniques

A variety of administrative and design techniques can be applied to preserve and enhance the safety and operational character of a roadway segment and to mitigate the traffic problems at many types of locations. The appropriate technique depends on the context of the roadway, traffic, land use, and environmental characteristics. There is no one-size-fits-all solution. The following is not an exhaustive list. HDM Section 8.2 contains additional information on the design of road approaches.

- **Minimize:** Relocate driveways to farthest edge of property, consolidate driveways to increase separation, close, acquire access rights, joint and cross access, provide secondary access
- **Administrative:** Access policies and codes, subdivision and partition review, vehicle use limitations, purchase of access control, Division 51, spacing tables
- **Alternate Access:** Backage or frontage roads

- **Restrictions:** Painted/Non Traversable medians, porkchops to preclude direct left turns. Raised medians remove major conflict points from direct left turns. Studies indicate significant crash reductions from installation of raised medians.[10] Raised medians also significantly benefit pedestrian safety.[11]
- **Location:** Sight distance, spacing, corner clearance, signal spacing
- **Design:** Width reduction, definition, throats vs. aprons, curb radius, directional median openings, shoulder bypass, jug handles, frontage roads, service roads, illumination, visual cues at driveways. Indirect turns such as U-turns and J-turns or roundabouts are safer than direct left turns.[12] Frontage and backage roads facilitate traffic circulation by separating local from through traffic.
- **Channelization:** Left- and right-turn lanes on adjacent roadway and on access. Providing space for left turns away from through traffic can significantly reduce crashes.[13]

4.8.6 Turn Lane Criteria and Traffic Control

Certain crash types and the overall crash frequency at a location may be addressed by mainline turn lanes or improved traffic control. Analysis of the existing or no-build conditions should identify these safety deficiencies that are addressed in the build alternative analysis. Left- and right-turn lanes at unsignalized locations can be installed to reduce high-speed rear-end collisions by removing slower turning vehicles from through traffic. The criteria for turn lanes are outlined in detail in Chapter 7.

High frequency of turning or angle crashes may indicate a need to investigate traffic control improvements. Traffic control improvements may remove simultaneous conflict points such as an all-way stop, roundabout, or a traffic signal. Conflict points could also be removed or minimized by using medians, channelization, or grade separation. Traffic control may also include bike or pedestrian islands and devices. Traffic control is discussed in detail in subsequent chapters.

4.9 Online Safety Resources

4.9.1 ODOT Safety Resources

Oregon Highway Safety Website

Overview webpage describing ODOT safety programs and resources. Includes links to many analysis tools and data resources, as well as information on funding sources and ODOT research.

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[10] Mauga, T. and Kaseko, M., "Modeling and Evaluating the Safety Impacts of Access Management (AM) Features in the Las Vegas Valley." Transportation Research Record: Journal of the Transportation Research Board 2171, pp. 57-65, 2010

[11] Zegeer, C. V., Stewart, R., Huang, H., and Lagerwey, P., "Safety Effects of Marked Versus Unmarked Crosswalks at Uncontrolled Locations: Executive Summary and Recommended Guidelines." FHWA-RD-01-075, McLean, Va., Federal Highway Administration, (2002)

[12] Xu, L., "Right Turns Followed by U-Turns Versus Direct Left Turns: A Comparison of Safety Issues." ITE Journal, Vol. 71, No. 11, Washington, D.C., Institute of Transportation Engineers, (2001) pp. 36-43.

[13] Lyon, C., B. Persaud, N. Lefler, D. Carter, and K. Eccles. "Safety Evaluation of Installing Center Two-Way Left-Turn Lanes on Two-Lane Roads." TRB 87th Annual Meeting Compendium of Papers CD-ROM. Washington, D.C., 2008.

Oregon Highway Safety Manual Website
Oregon-specific HSM tools, research, and implementation information. Includes links to pre-calibrated HSM Predictive Method spreadsheets.

Highway Safety Investigations Manual
The Highway Safety Investigations Manual is a resource to assist ODOT traffic investigators and analysts with detailed highway safety project screening and evaluations. The manual includes checklists and analysis procedures suitable for a variety of field and office safety investigations and assessments. A set of worksheets is available containing tools and forms to facilitate the analysis.

Low-Cost Systemic Safety Countermeasures
Guidance and fact sheets on the use of research-proven low-cost safety countermeasures that can be deployed on a systematic basic. Through the collective efforts of ODOT's Traffic Operations Leadership Team (TOLT) and Highway Safety Engineering Committee (HSEC), many of these safety countermeasures are thoroughly integrated into the options the ODOT Regions must consider when addressing highway safety issues to reduce fatal and serious injury crashes throughout Oregon.

Transportation Development Trans Data Portal
Index page that lists many transportation data sources available through ODOT.

4.9.2 ODOT Safety Plans

Transportation Safety Action Plan (TSAP)
The TSAP is the guiding safety policy document for ODOT and includes the specific actions that are being taken to provide a safer travel environment. The document also serves as the federally mandated Strategic Highway Safety Plan (SHSP).

Roadway Departure Safety Implementation Plan
Roadway Departure crashes account for approximately 66% of all fatalities in Oregon. Data analysis of Oregon crashes was combined with cost-effective strategies to identify locations for the most effective use of funds to achieve an approximate 20% reduction in roadway departure fatalities. This systematic approach involves deploying large numbers of relatively low-cost, cost-effective countermeasures on targeted segments of road with a history of roadway departure crashes.

Intersection Safety Implementation Plan
This plan developed by Oregon and FHWA focuses on reducing fatal and major injury crashes at intersections. In Oregon, an average of 72 fatalities at intersections occurs each year. Although this number is declining, this plan is geared towards reducing this number even more. Using cost-effective strategies to apply both systemic improvements as well as hot spot improvements can be used to reach approximately a 13% reduction in intersection fatalities.

Bicycle and Pedestrian Safety Implementation Plan
Provides a systemic approach to reducing pedestrian and bicycle risks and crashes in Oregon. Includes two network screening methods, a traditional crash-based process and a risk-based systemic safety planning process using roadway characteristics that have contributed to pedestrian and bicycle crashes over the study period. The plan provides a list of candidate priority locations and a toolbox of relevant countermeasures.

4.9.3 National Safety Analysis Resources

Official AASHTO Highway Safety Manual Website
Includes purchasing information, case studies, implementation tools, and online training guides for the HSM. Also includes user forums for practitioners and published corrections to the HSM.

Crash Modification Factors Clearinghouse
Companion tool to HSM Part D Crash Modification Factors. Provides a searchable database of up-to-date CMFs with details on applicability and links to the original research. This is a free web site funded by FHWA and maintained by the University of North Carolina Highway Safety Research Center.

PEDBIKESAFE.org
FHWA website for the Pedestrian Safety Guide and Countermeasure Selection System (PEDSAFE) and Bicycle Safety Guide and Countermeasure Selection System (BIKESAFE). The site includes practitioner guidebooks, proven countermeasure information, and various case studies covering treatment implementation.

Appendix 4A – Crash Attribution and Automation

(1) Discussion Paper No. 7, Functional Intersection Area, Transportation Research Institute, Oregon State University, January, 1996

5 DEVELOPING EXISTING YEAR VOLUMES

> *Disruptive events such as the 2020 COVID-19 pandemic can cause major changes in traffic patterns for extended periods of time. Under these conditions taking new traffic counts for the project will often not be advised and state and local traffic count programs will likely have been suspended. Refer to Appendix 3E for guidance on volume development in planning and project development when disruptive conditions are present.*

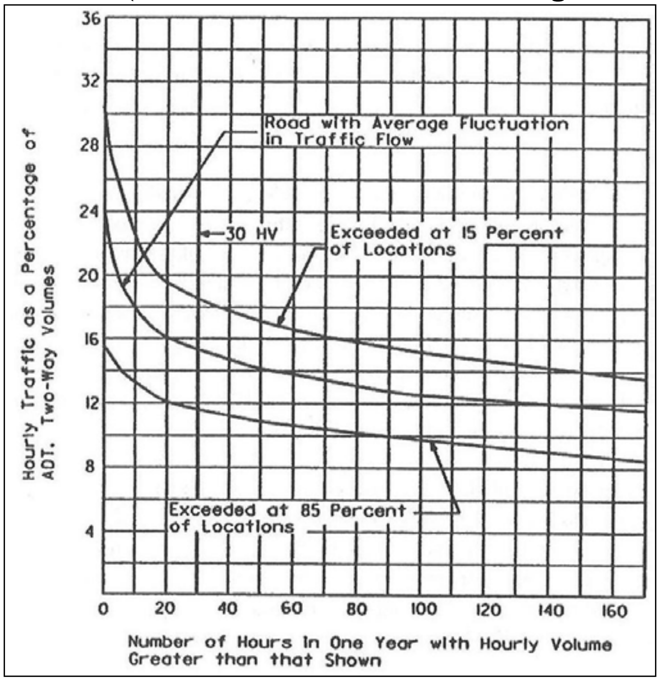
5.1 Purpose

Traffic counts alone should not be used for design or operational analysis of projects. This chapter will outline procedures for developing 30th highest hour volumes (30HV) and average daily traffic volumes (ADT) for planning and project level analysis. Existing and count-level data are also needed to support other volume –related analysis inputs such as peak hour factors and truck percentages. This chapter also provides the basis for developing the future volumes shown in Chapter 6.

5.2 30th Highest Hour Volumes

For most traffic studies, the 30th highest hour volumes (30HV) should be used to represent existing and future volumes. Future volumes are often referred to as Design Hour Volumes (DHV). The 30HV is the target hour based on the concept that designs are not done to the absolute highest hour of the entire year, but to design to meet most of the needs. Plotting an entire year's hourly volume data ranked from highest to lowest at a specific point will yield a flow curve. The break point between the steep and shallow regions of the curve is typically located at the 30th highest hourly volume as illustrated in the Exhibit 5-1 below. Alternatives are generally analyzed at 30HV to be consistent with accepted analysis methods and comparisons.

Exhibit 5-1 Relation between Peak-Hour and Average Daily Traffic Volumes on Rural Arterials (2011 AASHTO Green Book Figure 2-28)



The overall process for developing the 30HV is as follows:

1. Document raw count volumes, types and durations (see Section 5.6.3).
2. Identify a system peak hour (see Section 5.3).
3. Adjust for axle factors if necessary (see Section 5.4)
4. Calculate seasonal adjustments; bring counts to a common month (see Section 5.5).
5. Calculate historical adjustments; bring counts to a common year (see Section 5.6).
6. Balance the 30HV across the network (see Section 5.6.1)
7. Round the 30HV (see Section 5.6.2)
8. Document on figures for the 30HV process (see Section 5.6.3)

The peak hour from a manual count is converted to the 30HV by applying a seasonal factor. The 30HV is then used for design and analysis purposes. Experience has shown that the 30HV in large urban areas usually occurs on an afternoon on a weekday during the peak month of the year. The Metropolitan Planning Organization's (MPO) of Metro, Salem and Eugene are large enough that the average weekday peak hour approximates the 30HV. Smaller MPO's such as Corvallis and Medford do not have the steady flow throughout the year and the 30HV is concentrated into peak periods of the year. For the Bend MPO, the average weekday peak hour 30HV approximation is limited to commuter-flow dominated high volume arterials such as US97; other areas are subject to normal seasonal fluctuations. The 30HV for an urban area typically ranges from 9- to 12-percent of the Average Annual Daily Traffic (AADT). For a recreational route, the 30HV usually occurs on a summer weekend and ranges from 11- to 25-percent of the AADT.

It is recommended a top 200- to 500-hour count listing of the Automatic Traffic Recorders (ATR) is obtained from the Transportation Systems Monitoring Unit. The 30HV at the ATR(s) will be included in the list so that it will be possible to determine approximately when the 30HV occurs during the day and in the week. Since the actual 30HV determination is based off of at least the previous year's (ideally at least three years) data, the actual time and day when it occurs for the base year cannot be determined. Manual counts can then be timed for the period when the 30HV will likely occur, minimizing seasonal adjustments.

> *OTMS is unable to accept estimated values as previously created for Automatic Traffic Recorder (ATR). If MADT = MAWDT, then ATR month was estimated.*

5.3 System Peak Hour Selection

Daily traffic volumes, while useful for planning purposes, cannot alone be used for design or operational analysis purposes. Once all the traffic counts have been obtained, the intersection counts should be adjusted to a single system peak hour. The peak hour is the single hour of the day that has the highest hourly volume. Using the 15-minute breakdowns in the traffic counts is necessary in order to determine the true peak hour, resulting in a time period such as 4:00 PM to 5:00 PM or, just as easily, 4:45 PM to 5:45 PM. The final selection of a peak hour may be based on a simple majority of counts that have the same peak hour, using a controlling intersection, or the count(s) that the analyst believes are the most accurate. Counts that have longer durations or that are taken close to the 30HV are generally more accurate.

Generally, PM peak hour volumes are higher than AM peak hour volumes. In areas where there are large industries with shift changes, the hour during the shift change may be as high as or higher than the PM peak hour for the remainder of the transportation network. If this is true, another set of volumes should be developed. Volumes for the non-standard peak hour should be developed along with the PM peak hour volumes so that all the volumes may be analyzed at a later date. Multiple sets of volumes may be necessary in these circumstances, which may include areas of heavy industrial, retail or recreational uses; coastal routes; or on routes with highly directional commuter flows.

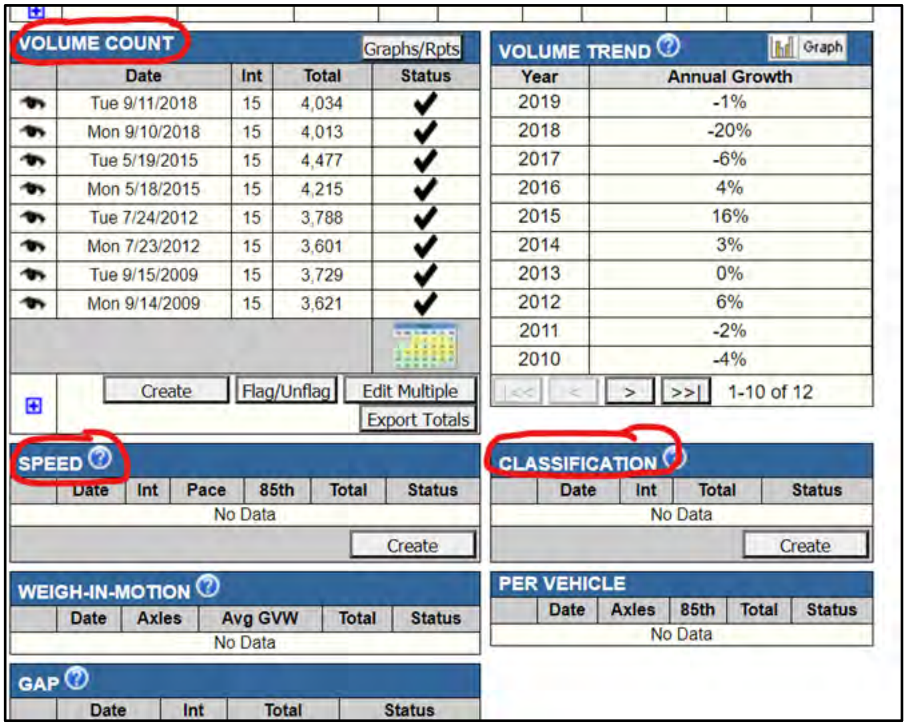
5.4 Axle Factors

> *Axle hit counts (volume only road tube counts) obtained from the Oregon Traffic Monitoring System (OTMS) program must be axle factored before use. For each volume only count, OTMS identifies a representative classification count to use for the axle factors. Refer to Appendix 17A for more information.*

OTMS contains Axle Hit Counts (sometimes referred to volume-only counts) which are counts of axle hits divided by two. These counts overestimate the number of vehicles since some vehicles have more than two axles, so the overestimation degree depends on the volume of heavy vehicles of three axles or more. To estimate the number of vehicles, an axle factor must be applied, outside of OTMS, before using these counts. For each of these counts, OTMS identifies a suggested representative road tube classification count to use for the axle factor.

In OTMS, an axle factor is needed when the “classification” and “speed” list are empty for the same count shown in the “volume count” list as shown in Exhibit 5-2. The axle factor that should be used is shown in Exhibit 5-3 under Axle Factor Group. If no axle factor group is listed then a classification count with a minimum of 16 hours on a similarly functionally classed road with approximately the same AADT should be used. If needed, please contact the TSM unit for clarification on an axle factor or group.

Exhibit 5-2 OTMS Count List



VOLUME COUNT (Graphs/Rpts)

Date	Int	Total	Status
Tue 9/11/2018	15	4,034	✔
Mon 9/10/2018	15	4,013	✔
Tue 5/19/2015	15	4,477	✔
Mon 5/18/2015	15	4,215	✔
Tue 7/24/2012	15	3,788	✔
Mon 7/23/2012	15	3,601	✔
Tue 9/15/2009	15	3,729	✔
Mon 9/14/2009	15	3,621	✔

Create | Flag/Unflag | Edit Multiple | Export Totals

VOLUME TREND (Graph)

Year	Annual Growth
2019	-1%
2018	-20%
2017	-6%
2016	4%
2015	16%
2014	3%
2013	0%
2012	6%
2011	-2%
2010	-4%

<< < > >>| 1-10 of 12

SPEED

Date	Int	Pace	85th	Total	Status
No Data					

Create

CLASSIFICATION

Date	Int	Total	Status
No Data			

Create

WEIGH-IN-MOTION

Date	Axles	Avg GVW	Total	Status
No Data				

PER VEHICLE

Date	Axles	85th	Total	Status
No Data				

GAP

Date	Int	Total	Status

Exhibit 5-3 Axle Factor Group

On NHS	No
LRS ID	9148
SF Group	27005
AF Group	26203
GF Group	27005
Class Dist Grp	26203
Seas Clss Grp	Statewide
WIM Group	

FHWA has developed axle factors for each vehicle type classification. These factors can be used with an OTMS road tube classification count to determine the site axle factor. Within each vehicle classification, the axle factor is applied to the count to estimate the vehicle overcount that would have occurred if an axle hit count was taken. The vehicle overcount estimate is then summed across all vehicle classifications. The site axle factor in both directions is calculated as the total volume of vehicles divided by the total estimated vehicle overcount. Exhibit 5-4 below is an example of an OTMS axle factor sheet for a Turning Movement Count (TMC) showing the axle factors for each vehicle classification and the calculation of site overcount. At this time, directional axle factors from straightaway class counts will need to be hand calculated using the "Reports → Class →Volume By Class – by Direction" report. The axle factors and vehicle overcount lines will need to be added to the report and calculated to produce the calculated axle factor as shown in Exhibit 5-5. If a combined direction axle factor is required, the "Reports → Class → Axle Factor by Month (Chart)" should be pulled as shown in Exhibit 5-6.

Exhibit 5-4 OTMS TMC Axle Factor Sheet

Axle Factor Report
10/13/2009 Through 10/13/2009

Intersection ID: 22032009
County: Linn
City: Albany
LRS ID: 05800I00
LRS Location Point: 1.24

Date: 10/13/2009
Hour: 6:00 AM - 10:00 PM
Legs: Burkhart St. (SB), OR99E(Paciifc Blvd.) (WB), Burkhart St. (NB), OR99E(Pacific Blvd.) (EB)
Location: Burkhart St. at OR99E(Paciifc Blvd.) and OR99E(Pacific Blvd.)
Notes: Weather: Clear

						Single Unit Truck			Single Trailer Truck			Multi Trailer Truck			
Leg	From To	Motorcycle	Car	Lt Truck	Bus	2 Axles	3 Axles	4+ Axles	4- Axles	5 Axles	6+ Axles	5- Axles	6 Axles	7+ Axles	Total All Vehicle
East	East-North	1	1254	291	7	8	1		6	3	1			3	1575
	East-South		341	160	1	10			1	1					514
	East-West	10	6727	1999	59	158	38	2	28	94	68	2		15	9200
	North-East	1	931	299	3	13	1		5	5					1258
	South-East		349	146		6				1					502
	West-East	9	6503	1941	57	145	38	2	38	111	74		1	10	8929
	Total Volume	21	16105	4836	127	340	78	4	78	215	143	2	1	28	21978
	Axle Factor	1	1	1	1	1	1.5	2	2	2.5	3	2.5	3	3.5	0.965
	Vehicle Over Count	21	16105	4836	127	340	117	8	156	538	429	5	3	98	22783

Axle Factor

Exhibit 5-5 Straightaway Axle Factor Calculation

Oregon Traffic Monitoring System

Volume by Class - by Direction for 1/1/2019 - 12/31/2019
Criteria: Type = I-SECTION, Is Perm Station = 0, From 1/1/2018 To 12/31/2020 Include Class Counts

District: Region 3 **Station:** 10011 **Route:** 5
County: Douglas **City:** - **ADT:** 20,949 **Located On:** PACIFIC HIGHWAY NO. 1
At Road: Glendale Interchange and Barton Road

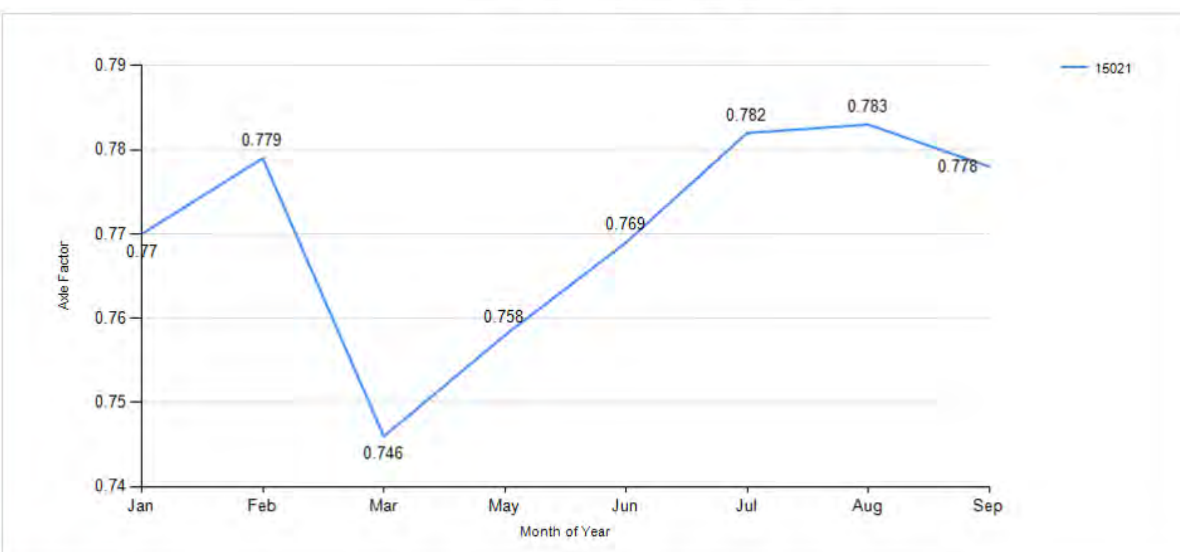
RB	Coll Type	Dir		MCycle	Car	Pickup	Bus	SU 2A-6T	SU 3 Axle	SU 4+ Axle	STT 3-4 Axle	STT 5 Axle	STT 6+ Axle	MTT 5 Axle	MTT 6 Axle	MTT 7+ Axles	UC	Total
ML		NB	#	30	4,058	2,101	7	688	107	0	21	2,897	158	72	75	198	0	10,412
			%	0.3	39.0	20.2	0.1	6.6	1.0	0.0	0.2	27.8	1.5	0.7	0.7	1.9	0.0	100.0
				1.0	1.0	1.0	1.0	1.0	1.5	2.0	2.0	2.5	3.0	2.5	3.0	3.5		0.655
				30	4058	2101	7	688	161	0	42	7243	474	180	225	693		15901
ML		SB	#	44	3,721	2,605	11	820	102	0	20	2,672	214	70	51	207	0	10,537
			%	0.4	35.3	24.7	0.1	7.8	1.0	0.0	0.2	25.4	2.0	0.7	0.5	2.0	0.0	100.0
				1.0	1.0	1.0	1.0	1.0	1.5	2.0	2.0	2.5	3.0	2.5	3.0	3.5		0.668
				44	3721	2605	11	820	153	0	40	6680	642	175	153	725		15769
	Total		#	**74**	**7,779**	**4,706**	**18**	**1,508**	**209**	**0**	**41**	**5,569**	**372**	**142**	**126**	**405**	**0**	**20,949**
	Percent		%	**0.4**	**37.1**	**22.5**	**0.1**	**7.2**	**1.0**	**0.0**	**0.2**	**26.6**	**1.8**	**0.7**	**0.6**	**1.9**	**0.0**	**100.0**
				1.0	1.0	1.0	1.0	1.0	1.5	2.0	2.0	2.5	3.0	2.5	3.0	3.5		0.661
				74	7779	4706	18	1508	314	0	82	13923	1116	355	378	1418		31670

Lines added to report to calculate axle factor

Exhibit 5-6 Straightaway Axle Factor Report[1]

Oregon Traffic Monitoring System

Axle Factor by Month for 1/1/2020 - 12/31/2020
Criteria: Type = I-SECTION, Is Wim Station = 1



Factor Group	Station	Weight	Jan	Feb	Mar	May	Jun	Jul	Aug	Sep
	15021		0.770	0.779	0.746	0.758	0.769	0.782	0.783	0.778
	Average of Weighted Factors		**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**

[1]For short-Term sites, there will only be one value.

Axle hit counts may be from a city or county and not found in OTMS. These counts also need to be axle factored before use. It is recommended to contact the source of the count to determine whether the count is axle factored or not and if the numbers represent axle hits divided by two or some other measure. Most privately collected counts are axle factored. The axle factor needs to be from a representative count site. For counts of axle hits divided by two, it is desirable to use an axle factor from a representative road tube classification count in OTMS if available. A representative count would ideally be located on the same roadway and with an AADT within 10 percent. If an OTMS axle factor is not available, one can be developed from a classification count on a facility with the same functional classification and volume within 10%.

Example 5-1 Axle Factors

For a study, a local jurisdiction volume-only count was obtained. The count showed a total volume of 12,384 vehicles per day was obtained.

From OTMS, a classification count was obtained on a nearby highway section, which was determined to be representative of the general area. From the OTMS TMC report, the axle factor was shown to be 0.862.

The adjusted ADT volume is calculated as 12,384 x 0.862 = 10,675 vpd.

5.5 Seasonal Factors

Since manual counts are taken throughout the year, data derived from a count taken in a particular month may need to be converted to the peak month by applying a seasonal factor. The Highway Capacity Manual 2010 Chapter 3 has background information on variations in traffic flow. Seasonal factors can be created using data collected from the ODOT ATR stations.

As of November 2022, there are 181 active ATR stations and 3 proposed ATR sites throughout the State Highway System. Most of these locations have loops in the roadway that count traffic flows for 24 hours a day, 365 days a year, and many have been in operation for many years. In addition, some ATR sites have been replaced with Automatic Vehicle Classifiers (AVC) which classifies vehicles continually in addition to counting. As of the date mentioned, there are 9 AVC sites throughout the system. In the future, it may be possible to create seasonal factors for freight movements using data from the AVCs. ATR information is available from the ODOT Transportation (Traffic) Volume Tables (TVT) located on the Traffic Counting web site, as well as the ATR Characteristic Table and the Seasonal Trend Table located on the Analysis Procedures Manual website.

Localized seasonal factors could be derived from other sources of data such as the PORTAL system on the Portland area-freeways and from the 170/2070 traffic signal controller downloads for local non-state streets.

ATRs provide the percentage of AADT that occurs in the count month and in the peak month. AADT is the annual average value for all days of the year including holidays. This information can then be used to develop a seasonal adjustment that may be applied to the manual count using one of the following three methods.

- On-Site ATR Method
- ATR Characteristic Table Method
- ATR Seasonal Trend Table Method

The On-Site ATR Method is the best and most accurate method to use, followed by the ATR Characteristic Table Method and then the ATR Trend Table Method. All of the seasonal adjustment tables and ATR information are updated annually.

Seasonal factors greater than 30% should be avoided. Factors such as these indicate that a count was NOT taken at or close to the time that the 30HV occurs. Using a winter count with a high seasonal factor to represent the peak summer period will likely not represent traffic turning movements accurately, as driving patterns change in the winter compared to the summer. If alternate periods are used other than the typical summer like what would be used for an alternate mobility standard, then counts need to be taken in those alternate periods so resulting seasonal factors do not exceed 30%. As an example, suppose a count was taken at a rural intersection in the winter months with one of the minor legs of the intersection serving a campground beyond the intersection. The turning volume in the direction of the campground may be small or non-existent; say 5 vph. Even with a seasonal factor of 50%, this would result in an adjusted volume of only 8 vph, compared to an actual summer 30HV that may be 20 vph. Simply factoring for the season would still leave the turning movements too low.

5.5.1 On-Site ATR Method

The On-Site ATR Method is used when an ATR is within or near the project area. If located outside of the project area, there should be no major intersections between the ATR and the project area, and it should be within a minimal distance so that the traffic characteristics such as road class, number of lanes, rural/urban area, etc., are comparable. It is also important to check that the project area's AADT in the Transportation Volume Table is within +/- 10% of the ATRs AADT.

When using the ATR Summaries from the TVTs, the analyst should note both the average weekday and average daily percentages. Average weekday traffic (AWD) percentages include values for Monday through Thursday while average daily traffic (ADT) includes all days of the week. When there is little variation between the AWD and ADT percentages, using AWD supports the notion that the peak is likely on an average weekday (see Exhibit 5-7). If the ADT is much larger than the AWD, then the peak is likely on a weekend day. Check the Weekly Traffic Trend column from the ATR Characteristic table to aid in this calculation.

Exhibit 5-7 Percent of AADT to use from ATR Trends

	Weekday Trend	Weekend Trend
Percent of AADT to use	Average Weekday Traffic (AWD)	Average Daily Traffic (ADT)

Example 5-2 Seasonal Factor – On-Site ATR

On-Site ATR in Project Area

A traffic count was taken on a June weekend along Kings Valley Highway No. 191 (OR 223) at MP 28.00.

- **Step 1: Transportation Volume Table** - ATR 02-005, located on Kings Valley Highway at MP 26.40, can be used.
- **Step 2: Check ATR Characteristic Table** weekly traffic trend column to identify whether to use the ADT or AWDT for seasonal factors. In this case, the Characteristic Table identifies this as a Weekend ATR; therefore the ADT column is used.
- **Step 3: ATR Trend Summary** - The ATR number corresponds to a table in the last half of the TVT that contains yearly summaries for each ATR. Since the Average Daily percentages are much higher than the average weekday, select a value from the column titled "Average Daily Traffic/Percent of ADT" (Exhibit 5-7). Both the count month and the peak month percentage of ADT should be recorded. This information should be obtained from the ATR Summaries in the TVT's for the past five years. The peak month is the month with the highest percentage. The highest and lowest percentages should be eliminated to account for construction activity that may have occurred in the vicinity of the ATRs during the five-year period. An average percentage of ADT is then calculated for the remaining three years. The percentages shown in the TVT represent the 15th day of the month, so interpolation is needed if the count was taken near the beginning or end of a month.

Seasonal Adjustment Using ATR #02-005

	2012	2011	2010†	2009	2008
Peak Month (September)	115%	119%	124%	121%	126%
Count Month (June)	104%	104%	103%	106%	106%

† – Sometimes the Peak Month may vary between months, depending on the year. As in this case, 4 of the 5 Peak Months were in September. Therefore, you assume September is the Peak Month for every year of ATR data used.

As shown above, the percentage of ADT values listed during June and September for the past five years are reviewed to calculate the average. The highest and lowest values, shown as shaded, are dropped from this calculation. The average monthly factors are as follows:

- The average peak month (September) is: (121%+124%+119%) / 3 = 121%.

- The average count month (June) is: (106%+104%+104%) / 3 = 105%.
- The seasonal adjustment is September/June = 121% / 105% = 1.15.

Therefore, traffic volumes in the month of September are 1.15 times greater than in June. To convert the June traffic data to the 30HV:

30HV = (June PHV) x (Peak Month Percent of ADT/Count Month Percent of ADT).

If one of the peak hour turning movement volumes was 75 vph in June, then the 30HV for September would be 1.15 x 75 vph = 91 vph.

When there are two or more ATRs within the project area use the following guidance to determine the appropriate seasonal adjustment:

Scenario # 1: The project area has two ATR's one at each end. The project area ADT, roadway characteristics and roadway functional class are the same as at both ATR's. To seasonally factor the peak hour volumes within the project area, an average of the two ATR seasonal factors recommended.

Scenario # 2: The project area has two ATRs on the same highway within the project area but has a major roadway between them. The roadway has different characteristics on each side of the major intersecting roadway. With this scenario, each side should be seasonally factored using the ATR on that side.

Scenario # 3: The project area has two (or more) ATRs located on different roadways. If the roadway has the same characteristics as the ATR, then use that one, if not average the ATR's. NOTE: If the ATR's report dramatically different roadways (i.e. freeways and local streets), then they should not be averaged together.

5.5.2 ATR Characteristic Table Method

The ATR Characteristic Table provides general characteristics for each ATR in Oregon and should be used when there is not an ATR on-site. The Characteristic Table is a filterable Excel table that will often provide more than one ATR with similar characteristics. To use the table, filter through the column characteristics from left to right to create a list of ATRs with similar characteristics. See example in Exhibit 5-8 which is an excerpt of the Summer<2500 trend.

Averaging multiple ATRs with similar characteristics will yield a more appropriate factor than if only one ATR is used. Follow the steps described in the on-site ATR Method for averaging count and peak months over 5 years for each ATR with similar characteristics. The factor used to convert the traffic data to 30HV will be an average of these similar characteristic ATR factors. Seasonal Traffic Trend groupings for the table were constructed by plotting the monthly percent of AADT for each ATR. The plots were then grouped into trends with the greatest influence in traffic patterns. Like with the On-Site method, review Exhibit 5-7 along with the weekend/weekday trend columns to determine whether AWD or ADT should be used.

Exhibit 5-8 ATR Characteristic Table Example

2013 ATR Characteristic Table										
Seasonal Traffic Trend	**Area Type**	**# of Lanes**	**Weekly Traffic Trend**	**2012AADT**	**OHP Classification**	**ATR**	**County**	**Highway Route, Name, Location**	**MP**	**State Highway Number**
Summer < 2500	Rural	2	Weekday	790	District Highway	01-001	Baker	US 30, La Grand	37.70	66
Summer < 2500	Rural	2	Weekday	210	District Highway	01-007	Baker	OR 203, Medical	36.86	340
Summer < 2500	Rural	2	Weekend	390	District Highway	13-005	Harney	OR 205, Frenchglen	0.01	440

It is important to note that the trends provided in the table are not the only trends attributed to each ATR but are the dominant trends. Over time, some ATRs may shift trend categories which are mainly the Summer/Summer<2500 and Recreational Summer/Summer<2500 which are typically small changes. Typically, ATR characteristics stay constant from year to year. After the seasonal traffic trend characteristic is selected, other trend groupings, including area type (e.g., urban, rural), the number of lanes and weekly traffic trends are broken down to provide more comparable sub-groupings.

ATRs are characterized by only one of eleven seasonal trends, described below and illustrated in Exhibit 5-9. Project areas should be characterized by these trends in the order listed below.

1. **Interstate Urbanized**: ATRs located on any section of urbanized (areas of population > 50,000) interstate. (Example: I-5, Iowa Street - ATR #26-016.)
2. **Interstate Non-Urbanized**: ATRs located on any non-urbanized interstate section. (Example: I-84, west of Troutdale - ATR #26-001.)
3. **Commuter**: ATRs characterized by small seasonal changes in traffic patterns and commuting between city pairs. (Example: OR 22, West Salem Bridges - ATR #24-014.)
4. **Coastal Destination**: ATRs characterized by summer peaks to/or within larger coastal city destinations as well as favorable routes from the valley. Favorable routes for Coastal Destinations include: Salmon River Highway (OR 18), Corvallis-Newport Highway (US 20/OR 34), Alsea Highway (OR 34), and Florence-Eugene Highway (OR 126). (Example: OR 18, east of Valley Junction - ATR #27-001.) Note: This grouping does not include the Sunset Highway.
5. **Coastal Destination Route**: ATRs characterized by high summer peaks on predominantly rural routes to/or between large coastal cities and coastal destinations. Rural routes include the Sunset Highway (US 26) from the Wilson River Hwy. junction, Umpqua Highway (OR 38), and Redwood Highway (OR 199). (Example: US 101, south of Rockaway - ATR #29-001.)
6. **Agriculture**: ATRs characterized by peaking in the late summer and fall harvest months. (Example: Kings Valley Highway - ATR #02-005.)
7. **Recreational Summer**: ATRs characterized by high summer peaks in recreational areas. (Example: Crater Lake Highway, south of Fort Klamath - ATR #18-021.)
8. **Recreational Summer/Winter**: ATRs characterized by both summer and winter peaks in recreational areas. (Example: Timberline Highway - ATR #03-008.)
9. **Recreational Winter**: ATRs characterized by high winter peaks in recreational areas. (Example: OR35, Mt Hood Highway – ATR #03-007)

If the project area trend does not fall into Trends 1 through 9, either Trend 10 or 11 should be used.

10. **Summer**: ATRs characterized by a smaller summer increase in traffic patterns when compared to Recreational Summer. (Example: US 26, south of Warm Springs - ATR #16-006.)
11. **Summer < 2,500 ADT**: ATRs with less than 2,500 ADT characterized by a smaller summer increase in traffic patterns when compared to Recreational Summer. (Example: OR36, Mapleton-Junction City Highway - ATR #20-004)

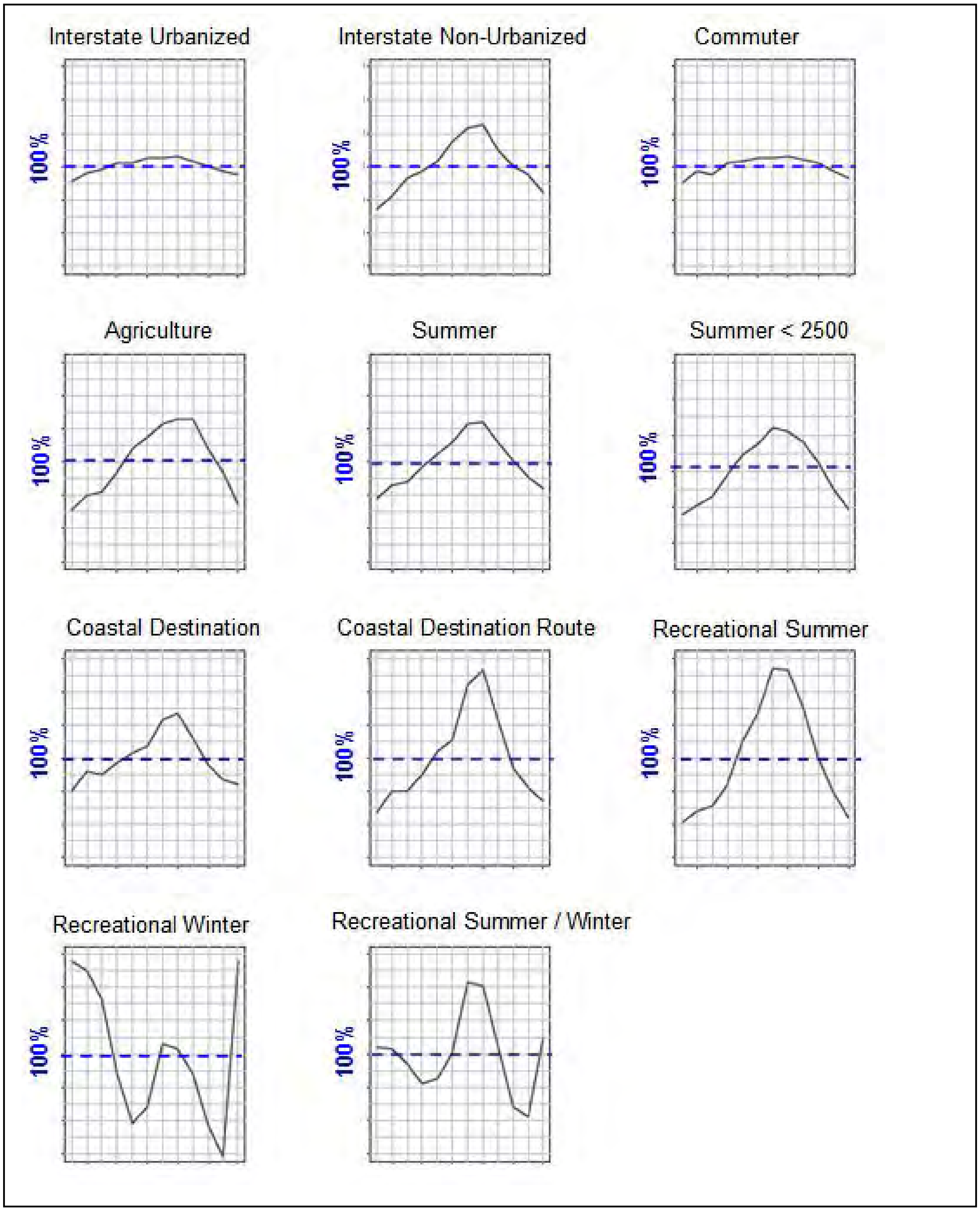
5.5.3 Non-State and other Roadway Trends

Many of the state highways mimic characteristics on rural county roads and urban streets. Some of the ATR Characteristic trends can be used to approximate seasonal factors on non-state highway approaches.

- Commuter: Non-state streets in urbanized cities (Example: US20/OR99E, Albany Junction City Highway - ATR # 22-022)
- Summer: Non-state streets in small cities (Example: OR42, Coos Bay Roseburg Highway - ATR #10-006)
- Summer<2,500 ADT: Rural off-system county roads. (Example: OR36, Mapleton-Junction City Highway - ATR #20-004)

Interchange ramps mix characteristics of the mainline freeway and the intersecting cross-roads, so seasonal factors for ramps should be created by averaging mainline and crossroad factors. State frontage roadways should use the applicable seasonal trend other than Interstate, or the bullet list above.

Exhibit 5-9 Seasonal Trends



ATRs are also characterized by weekly traffic trends and ADT/AWD.

- **Weekday:** Day of the week when the 30HV occurs; typical occurs on weekdays (Monday through Thursday) for commuter trends and urban areas.
- **Weekend:** Day of the week when the 30HV occurs; typical occurs on weekends (Friday through Sunday) for recreational trend and coastal destination trend.

ATRs are also characterized by area type and number of lanes.

- **Urbanized:** ATRs within areas of population > 50,000. (Examples: Portland and Salem) Urban Fringe: ATRs influenced by an urban area, such as an MPO area. (Example: Wilsonville)
- **Small Urban:** ATRs within areas of population between 5,000 and 49,999. (Examples: Albany and Pendleton)
- **Small Urban Fringe:** ATRs influenced by a small urban area. (Examples: US 101 south of Coos Bay and I-5 north of Albany)
- **Rural:** ATRs on routes outside of areas with population <5,000.
- **Rural Populated:** ATRs in cities with a population of less than 5,000. This also includes unincorporated communities. (Examples: Sisters and Tillamook)

To use the table, filter through the column characteristics from left to right to create a list of ATRs with similar characteristics. The table **must** be filtered starting on the far left, otherwise the groupings at the end will not be correct. Starting with the "Seasonal Traffic Trend" column, filter out the traffic trend that best describes the project area. Next, filter the area type, number of lanes, and weekly traffic trend. Make sure that the section of highway where the ATR(s) is located and the project area for which the seasonal adjustments are being made have similar traffic characteristics. To be considered comparable, the AADT of the characteristic ATR should be within +/- 10% of the Transportation Volume Table AADT for the project area.

Example 5-3 Seasonal Factor – ATR Characteristics Table

ATR Characteristic Table Method for a Project Area

A count was taken June 15th–18th along Corvallis-Lebanon Highway No. 210 (OR 34), west of I-5 at MP 9.75. The 2012 Transportation Volume Table AADT is 24,900.

- **Step 1: Transportation Volume Table**: There are no ATRs on this section of the highway.
- **Step 2: ATR Characteristic Table**: This section of highway can be categorized as Commuter/Small Urban Fringe/Five-Lanes/Weekday. Filtering through the ATR Characteristic Table from left to right, two ATRs have similar characteristics to the project area. However, ATR 31-003 has an AADT of 11,300; as previously noted, characteristic AADT counts should be within +/- 10% of the Transportation Volume Table AADT in order to be considered comparable to the project area. Alternatively, ATR 09-020 has identical field conditions to the project site and has an AADT of 26,300, which is within 10% of the TVT AADT. The characteristics of these two representative

locations are summarized below.

Example ATR Characteristic Table (Year 2013)

Characteristics	ATR Location 1	ATR Location 2
Seasonal Traffic Trend	Commuter	Commuter
Area Type	Small Urban Fringe	Small Urban Fringe
Number of Lanes	5	5
Weekly Traffic Trend	Weekday	Weekday
2010 ADT	11,300	26,300
OHP Classification	Statewide Hwy	Statewide Hwy
ATR	31-003	09-020
County	Union	Deschutes
Highway Route, Name and Location	OR82, Wallowa Lake Hwy	US97, The Dalles-CA Hwy
ATR Milepoint	1.74	124.40
State Hwy Number	10	4

Step 3: ATR Trend Summary: Data from ATR #09-020 are located in the ATR summary in the back of the TVT and under the "ATR Trend Summaries" on ODOT's Traffic Counter Program website TDD Transportation Data Traffic Counting Program. The count was taken on June 15th, which is in the middle of the month, so the ATR percentages from the TVT can be used directly without interpolation. The weekly traffic trend is "Weekday" so this means that AWD should be used and the ATR percentages should be pulled from the Average Weekly Traffic column. The peak month was found to be July for three of the three years.

Seasonal Adjustment Using ATR #09-020

	2012	2011	2010	2009	2008
Peak Month (July)	112%	115%	119%	119%	118%
Count Month (June)	116%	113%	115%	115%	115%

Removing the high and the low years for the peak and count months results in:

- The average peak month (July) is: (115%+119%+118%) / 3 = 117%.
- The average count month (June) is: (115%+115%+115%) /3 = 115%.
- The seasonal adjustment is July/June = 117% / 115% = 1.02.

Therefore, weekday traffic volumes in the month of July are 1.02 times greater than in June. To convert the June traffic data to the 30 HV: 30 HV = (June PHV) x (Peak Month Percent of AWD / Count Month Percent of AWD).

If one of the peak hour turning movement volumes were 100 vph in June, then the 30 HV for July would be 1.02 x 100 vph = 102 vph.

5.5.4 Seasonal Trend Method

The seasonal trend table is used when there is not an ATR nearby or in a representative area. The Seasonal Trend Table was constructed by averaging seasonal trend groupings from the ATR Characteristic Table and is based on ADT. Essentially, by using a factor from the table, the average for the entire trend grouping is applied to the project area as shown in Exhibit 5-10.

Exhibit 5-10 Example ATR Seasonal Trend Table (Year 2013)

	Jan 1	Jan 15	Feb 1	Feb 15	Dec 15	Peak Period Seasonal Factor
Recreation Summer/Winter	1.0783	0.9668	0.9719	0.9771	0.9091	0.7038
Recreation Winter	0.7454	0.6408	0.6536	0.6663	0.6398	0.6398

To determine the appropriate seasonal trend, select from the list the trend that best describes the project area. The seasonal trends have been placed graphically on an ATR Characteristic Trend Map on the APM website for ease of reference. Trends should be characterized in the same order as previously described in the ATR Characteristic Table Method. The Seasonal Factor Table is updated yearly. It is not necessary to average five years' worth of seasonal factors for this method, or compare AADTs because, as previously stated, this method uses an average of all ATRs in the characteristic trend. In certain areas, averaging seasonal trends may yield a more appropriate factor than just a single trend. These areas include:

- **Coastal Destination and Coastal Destination Route Trends:** It may be necessary to average trends in areas such as Warrenton, Depoe Bay and Yachats. While these cities are destinations along the Oregon Coast, they do not have the summer influx of traffic associated with larger coastal destinations such as Lincoln City and Seaside. A Coastal Destination Trend Factor for these areas may be too high, while a Coastal Destination Route Trend Factor may be too low. When analyzing coastal cities such as these, it is appropriate to average the trends to yield a more reasonable factor.
- **Summer and Commuter Trends**: It may be necessary to average trends when analyzing mid-sized cities such as Philomath, Dallas and Sutherlin. For urbanized areas the commuter trend is appropriate, while for smaller areas the summer trend is appropriate. However, for mid-sized areas such as these, the summer or commuter trends may alone be too high or too low. A more reasonable factor would be obtained by averaging the summer and commuter trends.
- **Interstate and Interstate Urbanized Trends**: It may be necessary to average trends

when analyzing interstates in small urban and fringe areas (urban and small urban) such as Albany, Wilsonville and north of Roseburg. For rural areas the interstate trend is appropriate, while for urbanized areas the interstate urbanized trend is appropriate. For small urban and fringe areas such as these, however, these trends may alone be too high or too low. A more reasonable factor would be obtained by averaging the interstate and interstate urbanized trends.

It is important to note that these are the only trend grouping pairs that would be appropriate to average, Interstate should only be averaged with interstate urbanized and should never be averaged with Coastal or Recreational. The same is true for the other trends not listed in the above examples. The Seasonal Trend Table is located on the Transportation Analysis webpage of the TDD Planning website.

Factoring count data to the peak month requires dividing the seasonal factor for the count period by the seasonal factor for the peak period. The peak period seasonal factor for a traffic trend is the lowest value in the row and is highlighted in the last column in the table.

Seasonal factors are given for the 1st and the 15th of each month so if the count date is not at the beginning/end or in the middle of a month interpolation is needed.

Example 5-4 Seasonal Factor – Seasonal Trend Table

This example demonstrates the Seasonal Trend Method for a Project Area.

A count of 11,000 was taken July 1st – 5th along Oregon Coast Highway No. 9 (US 101) at MP 63.19 which is just north of Tillamook city limits.

- **Step 1: Transportation Volume Table**: There are no ATRs on this section of the highway.
- **Step 2: ATR Characteristic Table**: This section of highway can be categorized as Coastal Destination/Rural/Two-Lanes. Filtering through the ATR Characteristic Table, from left to right, four ATRs have similar characteristics to the project area. However, none of the characteristic ADT values are within +/- 10% of the Transportation Volume Table ADT for the project area. Refer to the table below for details regarding these four candidate locations.

Example ATR Characteristic Table (Year 2013)

Characteristics	ATR Location 1	ATR Location 2	ATR Location 3	ATR Location 4
Seasonal Traffic Trend	Coastal Destination	Coastal Destination	Coastal Destination	Coastal Destination
Area Type	Rural	Rural	Rural	Rural
Number of Lanes	2	2	2	2
Weekly Traffic	Weekday	Weekend	Weekend	Weekend
2012 ADT	2000	5700	4700	17200
OHP Classification	District Hwy	Statewide Hwy	Statewide Hwy	Statewide Hwy
ATR	02-003	20-005	21-006	27-001
County	Benton	Lane	Lincoln	Polk
Highway Route, Name and Location	OR34, Alsea Hwy No. 27, southwest of Corvallis-Newport Hwy No. 33 (US20/OR34)	OR126, Florence-Eugene Hwy No. 62, west of Territorial Hwy No. 200 (OR200)	US20, Corvallis-Newport Hwy No. 33, west of Lincoln-Benton County Line	OR18, Salmon River Hwy No. 39, east of Three Rivers Hwy No. 32 (OR22)
ATR MP	53.89	43.86	34.24	23.23
State Hwy Number	27	62	33	39

- **Step 3: Seasonal Trend Table**: Since there are no ATRs with similar characteristics, the Seasonal Trend Table must be used. The correct values are obtained by following the "Coastal Destination" row to the "Jul_1" count month column, and to the "Peak Period Seasonal Factor" column at the end of the table, as summarized below.

Seasonal Trend Table (Year 2013)

	Jun 15	Jul 1	Jul 15	Aug 1	Peak Period Seasonal
Coastal Destination	0.9345	0.8749	0.8153	0.8005	0.7857

- The peak period seasonal factor is 0.7857.
- The count date seasonal factor (July 1st) is 0.8749.
- The seasonal adjustment is: Count Date Seasonal Factor/Peak Period Seasonal Factor = 0.8749/0.7857 = 1.11.

Therefore, the peak period volumes for a Coastal Destination are 1.11 times greater than volumes for the 1st – 5th of July.

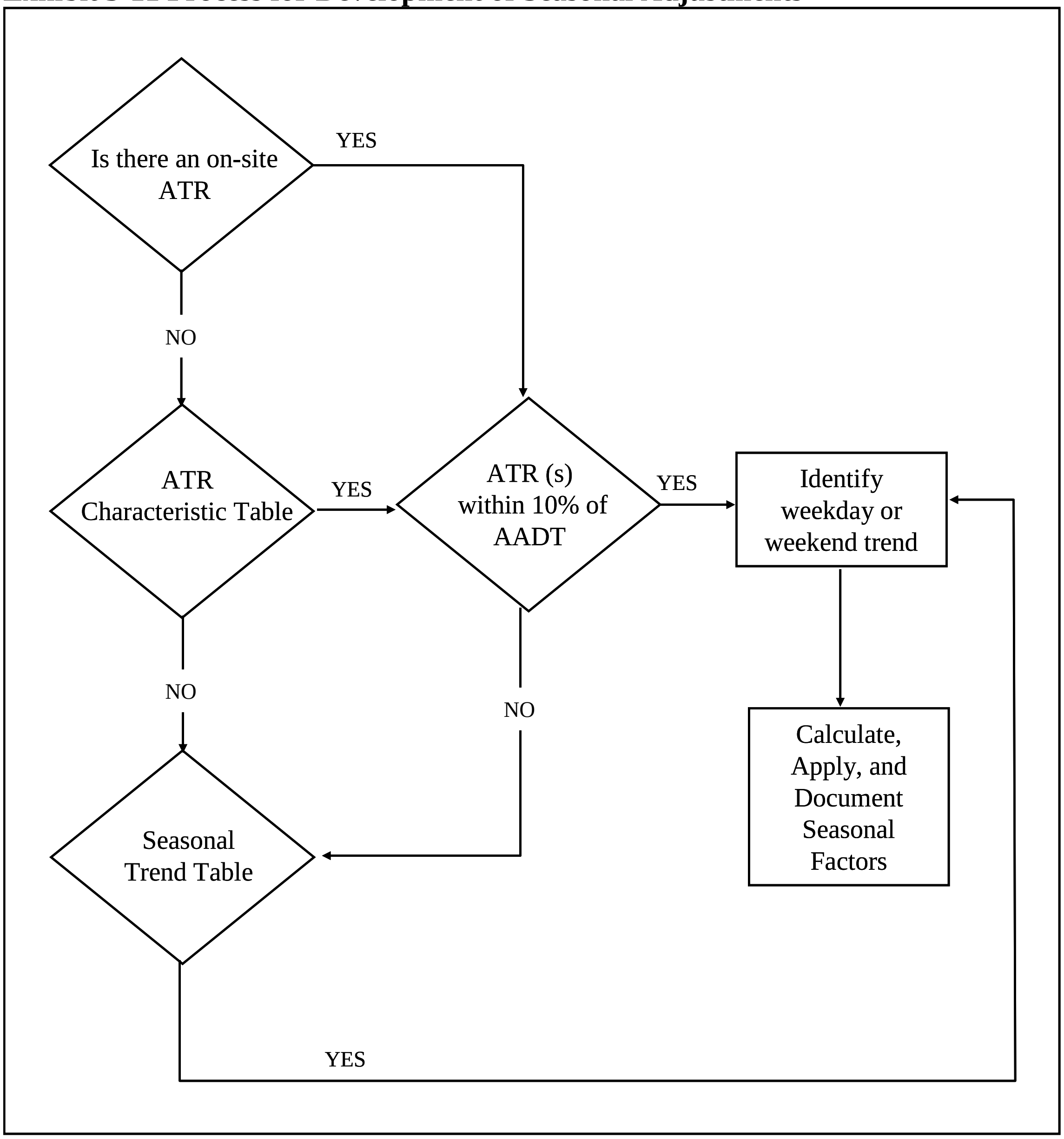
To convert the July traffic data to the 30HV:

30HV = (July PHV) x (Count Date Seasonal Factor / Peak Period Seasonal Factor).

If one of the peak hour turning movement volumes were 100 vph in July, then the 30HV would be 1.11 x 100 vph = 111 vph.

Exhibit 5-11 below is a simplified flow chart of the process for seasonal adjustments.

Exhibit 5-11 Process for Development of Seasonal Adjustments



Seasonal Adjustment for Alternate Periods

If it is desired to factor count data to an annual average time of year (Oct or May typically) for determination of an alternate mobility standard, the values in the Seasonal Trend table can still be used. This is the same process that the Transportation Data Section uses to create average annual daily traffic (AADT) to create the Transportation Volume Tables.

Example 5-5 Seasonal Characteristic Table Adjustment for Alternate Period

This is a continuation of Example 5-3 Seasonal Factor – ATR Characteristics Table (count taken June 15th–18th along OR 34 west of I-5). It is desired to obtain average annual peak hour volumes for this location. It was previously determined using the ATR Characteristic Table method that ATR #09-020 should be used for seasonal adjustments for this site.

Seasonal Adjustment Using ATR #09-020

	2012	**2011**	**2010**	**2009**	**2008**
Count Month (June)	109%	107%	108%	108%	107%

For this site location, the ratio of (Count Month ADT)/(AADT) from ATR #09-020 is used. For the five years of this example, dropping the high and low, the average count month % of AADT was found to be 109%.

$$\frac{\text{Count Month ADT}}{\text{AADT}} = 1.09$$

To convert to average annual volumes, count month volumes are divided by this ratio (1.09), i.e., the average annual peak hour volume = (Site June PHV) / (ATR June ADT/ATR AADT)

If one of the site peak hour turning movement volumes were 100 vph in June, then the site average annual peak hour turning movement volume would be 100 vph / 1.09 = 92 vph.

Example 5-6 Seasonal Trend Table Adjustment for Alternate Period

This is a continuation of Example 5-4 Seasonal Factor – Seasonal Trend Table (count of 11,000 taken July 1st – 5th along US 101 north of Tillamook). It is desired to obtain the average annual peak hour volumes for this location. The seasonal trend table was used for this location. The applicable seasonal trend was determined to be Coastal Destination.

The factors in the seasonal trend table are ratios of AADT to the month ADT (AADT/Month ADT). The ratio of the AADT/Month ADT is assumed to be the same for the site peak hour volumes. To convert to the average annual peak hour, the site month peak hour volume is multiplied by that ratio for that month.

Site Average Annual Peak Hour Volume = Site Month peak hour volume x Trend Table Factor for that month

In this example, the count was taken the week of July 1st. The seasonal factor for July 1st was found to be 0.8749. If one of the site peak hour turning movement volumes were 100 vph in July, then the site average annual peak hour turning movement volume would be 100 vph x 0.8749 = 87 vph.

5.6 Historical Factors (Factoring to Current Year)

In addition, to adjusting count volumes seasonally, they need to be adjusted to the current or base year of the project. This might mean increasing or decreasing the count peak hour volume. The historical factor process uses the Future Volume Table to adjust the volume regardless of what process is used to create the future volumes (see Chapter 6).

The Future Volumes Table is updated annually. The table is based on long-term 20-year trends of traffic counting sites (for the TVT) on Oregon highways. Each individual record trend has been analyzed to eliminate any unexplained spikes or dips in the data to improve its overall correlation with the 20-year regression curve. The trends are based on linear regression best-fit trends and are extrapolated out 20 years. Over the 20-year period, the linear trends show a steady growth and should not grossly under or overestimate the future 20-year value outside of travel demand model areas. Use of other curve extrapolation models (i.e. compound growth) shall not be used.

Within the table, the milepoint location for the project highway that most closely resembles the traffic flows for the section being analyzed is selected. Sometimes, another mile point may be closer to the section being analyzed, but there may be a cross street that affects traffic volumes on the highway so that the growth rate is different.

There are three columns in the table for the most recent three years of traffic count data. Only one of the columns is filled; corresponding to the last year that the location was counted. The far-right column contains the R-squared value for the regression equation that was used to estimate the historical trend. The R-squared value measures the degree of correlation between the dependent variable (historical traffic volumes) and the independent variable (time). A value of 1.0 indicates an exact linear relationship between the historical counts and time. Ideally, the R-squared value should exceed 0.75. However, values higher than 0.50 are still acceptable, if there is nothing else available. Conversely, a low R-squared value indicates a weak relationship. In this case, the table should be investigated for a nearby location having similar traffic characteristics and a more acceptable R-squared value.

In areas that have travel demand models, the R-squared value is replaced by the “MODEL” code. This code indicates that the historical trend has been replaced by the model growth rate (future year divided by the base year). In larger areas, use of the historical growth rate can overestimate volumes, so the use of the model growth rate allows for consideration of capacity constraints.

These model growth rates are updated as the models are updated (typically every five years or so).

For non-state roadways, or ramps/frontage roads, historical factors can be averaged from similar roadways with the same characteristics. The seasonal factor sections in this chapter can apply to help locate highway sections with similar characteristics. For example, an interchange ramp is affected by growth on the mainline freeway as well as the intersecting crossroad, so the ramp historical adjustment would be the average of the two.

Local land use changes should also be reviewed if they have occurred between the time that the count was taken and the desired base year of the project. In this case, the Future Volume Table should not be used because it is likely that double-counting would occur. The additional trips from the background and development sections in the traffic impact analysis document need to be added to the count volumes to approximate the existing traffic levels.

Example 5-7 Adjusting Counts to Current Year

The project base year is 2013 but the counts available were counted in 2010. The Future Volume Table is used to adjust the counts to the base year.

For the Madras ATR (16-002) located on US 97 at MP 96.92, The following table shows the 2011 traffic volumes, projected Year 2032 traffic volumes and the R-squared value.

Example Future Volumes Table

Hwy#	DIR	MP	Description	2011	2032	RSQ
4	1	96.46	0.02 mile N of Fairgrounds Road	17700	23200	0.9150
4	1	96.92	ATR 16-002 - Madras	12200	12500	0.7037
4	1	97.31	0.02 mile S of US 26	9300	13800	0.3994
4	1	103.61	0.02 mile N of SW Iris Lane	8200	10600	0.5093
4	1	105.63	0.10 mile N of Culver Hwy	8700	13000	0.7844

RSQ = Root Mean Square

Based on the data above, the 21-year growth factor would be 12500 / 12200 = 1.0246. (Note that numbers used in intermediate calculations are not rounded.) Assuming linear growth in the future, the annual growth factor would be (1.02459 –1.0) / 21 = 0.00117, or 0.117%. The R-squared value of 0.7037 is acceptable. To convert the 2010 30HV, for example a minor approach left volume of 115 vph, to a 2013 DHV, the 2010 30HV is multiplied by the 3-year growth factor.

2013 DHV = 2010 30HV x [(3 x Annual Growth Rate) + 1] = 115 vph x [(3 x 0.00117) + 1]
= 115 vph x 1.0035
= 115 vph

5.6.1 Network Balancing

The 30HV network needs to be balanced. Balancing is somewhat of an "art" that techniques vary from project to project and within areas on a project, so it is hard to list out a specific process. The exhibits and examples below illustrate some different balancing techniques. Balancing is simply, "what goes into an intersection or segment needs to come out." Without balancing, it is possible to have two intersections with nothing between them with the volume that leaves one intersection and enters the next one be 200 vph or more different.

Interstates and expressways with interchanges or roadway segments with no accesses need to balance perfectly from one intersection or interchange to another. Roadways with accesses probably will not balance perfectly, but should be consistent from intersection to intersection. Roadways with many non-counted accesses (especially major ones) should not be forced to balance as this will artificially increase or decrease the adjacent intersection volume beyond the realistic point. If a segment fails to balance because of accesses, it may be best to perform a peak hour count at the point or points to help reduce the balancing needed.

The timing of the traffic counts can help determine how easy a network is to balance. Counts that are spread throughout the allowable three-year span taken at different times of the year will be harder to balance than counts all taken on the same day or within a week of each other. Counts that have similar peak hours will be easier to balance than counts of varying peaks.

Network balancing should be done initially on paper as it is easier to track differences and proportional changes as the process proceeds. An intersection may have to be adjusted more than once in a single balancing process, so a record of the "chronological" adjustments/steps is needed for the analyst to retrace or a reviewer to follow. Trying to follow a spreadsheet-based balancing process is difficult and it becomes more of an effort to identify whether the right formula was used versus the right thought process was used. If a spreadsheet is used, then the thought process needs to be shown via comments/notes. Once the network is balanced, then the results should be rounded and placed into spreadsheets or figures. The important point is that the balancing thought process be reasonable, logical and well documented. The balanced network must be reviewed before the values are used in any further analysis or construction of future year volumes.

When balancing, it is important to keep the general proportions of turn splits and leg directional factors relatively constant unless the analyst knows that the counts did not capture the true conditions.

Identifying High/Low Imbalances

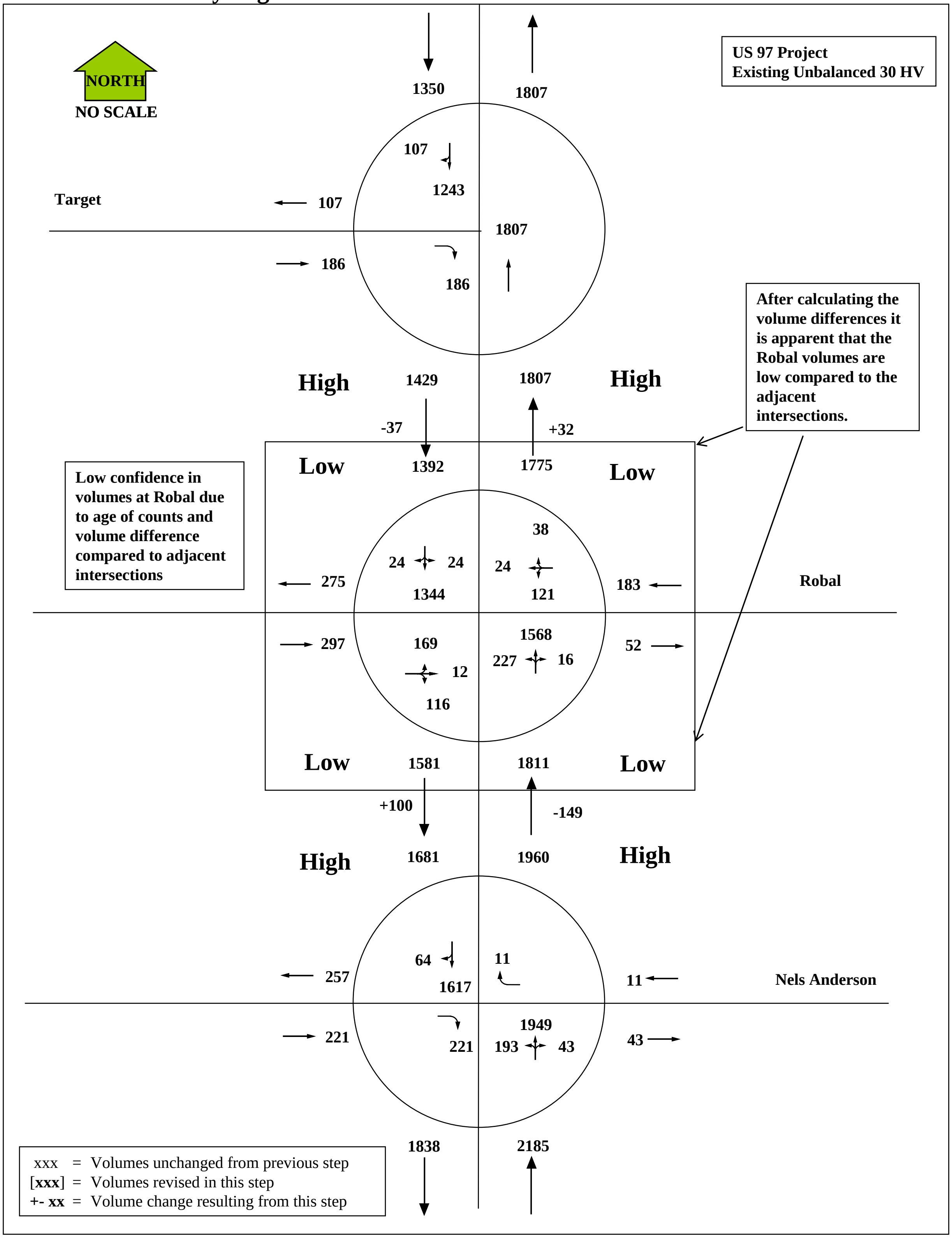
A good way to start the balancing process is to first identify which intersections/segments are high or low and likely techniques that could be used. Sum up the inflows and outflows on each approach and exit leg and note the differences between them across each intersection. Doing this graphically via sketches on paper with different colors for high and low segments can expedite the process. Note whether the differences seem to be on through movements or turn movements. The count dates, durations, and the analyst's overall confidence in the intersection counts should also be documented.

Small differences overall likely mean that a single point could be held, and the differences proportionally distributed. Large differences likely mean that differences will need to be split between intersections first. Intersections with whole approaches that are low or high, especially on minor approaches, may mean that the whole approach will need to be adjusted. The average project study area will have many of these conditions, so a combination of methods is necessary.

Intersections that are identified to have a high-low-high-low pattern generally have a better success rate in balancing cleanly versus a high-high-high or low-low-low pattern which might create too much of a difference at the network edge.

The following series of exhibits illustrates a small network consisting of three intersections on US 97 and will be used to demonstrate many of these network balancing techniques. Volumes shown in Exhibit 5-12 have been previously developed by factoring counts to a common year and applying seasonal adjustment factors. The exhibit identifies the high and low imbalances between intersections. There are no driveways between intersections so volumes should balance.

Exhibit 5-12 Identify High/Low Imbalances in Network



Holding an Intersection Constant

If differences are relatively small between intersections, holding an intersection constant and pushing the differences away proportionally may be used. Another use of this method is if the analyst has good confidence in a major intersection count based on count year or duration. For example, long duration counts generally are better to hold than short duration counts. Larger intersections (number of legs and/or volumes) are generally better to hold than smaller intersections. Depending on the situation, it may be best to start with the intersection with the greatest differences.

The selected intersection is held constant and the differences are pushed to the next intersection. The difference is resolved by identifying the splits between the approach turn movements from the next intersection's subject approach and applying those splits to the difference. The difference in the opposing direction is obtained from the proportional splits that make up the movements going onto the exit leg. This is repeated for each segment of the held intersection. Then, the difference is moved away from the held intersection and the difference is proportionally reduced the further away it gets as portions are removed by each turn movement.

It is possible that as the difference is worked downstream, the increases/decreases at an intersection become too great or seem to be going away from convergence (getting worse). In this case, either another intersection could be selected to be held, an entire approach may need to be adjusted, or better, the differences need to be split instead of held.

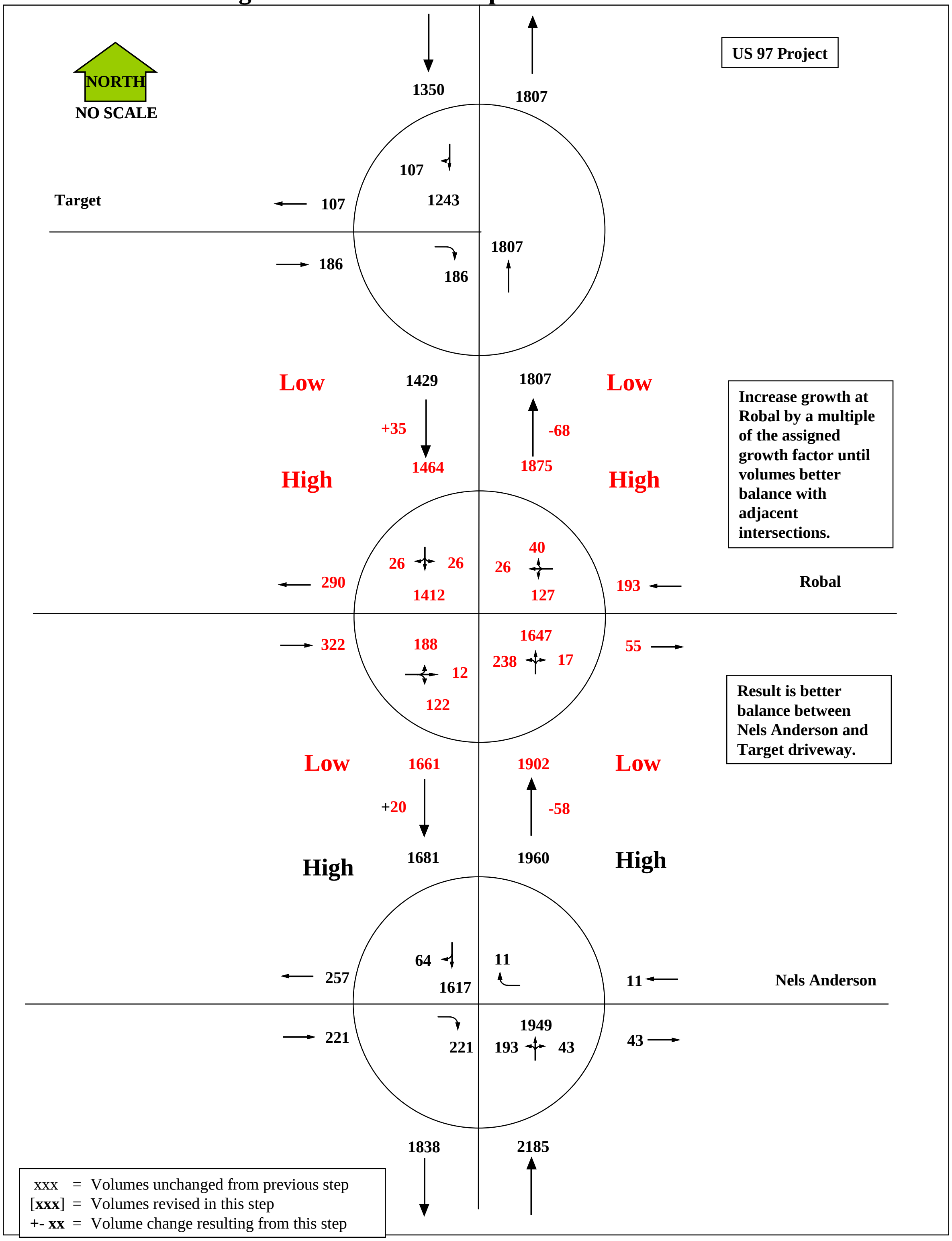
This method can also be used to just hold particular approaches constant if desired. This is useful if the analyst believes that the minor street leg growth factors seem to be off and have more confidence in the major approaches.

Factoring Approaches/Legs/Intersections

Sometimes, after identifying the high and low imbalances, an approach, or a leg, or an entire intersection appears to be significantly low or high for some reason. These areas may have been influenced by driveway closures, nearby road construction/diversion, incidents, or special events. Sometimes the count might have an error so this should be checked thoroughly even up to replacing the count with a new one if necessary. If no obvious errors /issues exist, then these can be factored up or down to more closely agree with adjacent intersections which the analyst has more confidence in. If these are on state highways, this is most easily done by using the Future Volume Tables to adjust the appropriate leg or intersection further with a growth factor as similarly done in Section 5.6. If a travel demand model exists, then this can be used to create annual growth factors for local roadways.

Exhibit 5-13 illustrates a situation where the volumes on US 97 at Robal are low in comparison to adjacent intersections. In this case there is less confidence in the volumes at Robal because they are based on an older count even though they have been adjusted to a common base year. There is greater confidence in the volumes at Nels Anderson and the Target driveway. In this case volumes at the entire intersection of Robal volumes are increased by a factor of 1.057 based on the Future Volume Table growth along this segment. This factor represents three multiples of the annual growth factor to bring the intersection to as close as possible without making it too high. The volume imbalance is then recalculated. After this adjustment, Robal is now high compared to the Target driveway to the north, and low compared to Nels Anderson intersection to the south. But the imbalance is now smaller and more realistic.

Exhibit 5-13 Factoring an Intersection to Improve Balance



Splitting the Difference

Large differences between intersection exit leg and the next intersection approach leg need to have the difference split. In this way the difference is moved proportionally in both directions rather than in just one. The proportional adjustments are the same as described above. This is repeated for every intersection until the edge of the network is reached. It may be necessary to re-adjust previously balanced sections of a particular leg does not balance cleanly (does not converge), or if the network circulates the difference back around to the starting spot. Some small differences may be possible to assign to minor streets or non-counted locations.

Exhibit 5-14 illustrates splitting the difference between Robal and Nels Anderson in both the northbound and southbound directions. To avoid increasing the imbalance to the north, only turn movement volumes at Robal to and from the south are adjusted. As balanced volumes are created, they are rounded to the nearest five vehicles.

Next, the difference between Robal and the Target driveway to the north is addressed by splitting the difference as shown in Exhibit 5-15. Again, the through movement volumes at Robal are held to avoid re-introducing an imbalance to the south. The difference is proportionately assigned at Robal to the turn movements to from the north only.

In the next step as shown in Exhibit 5-16, the volume adjustments previously made are pushed off the network.

In reviewing the resulting volumes, the drop in turning movement volumes at Robal is determined to be excessive. An iteration is performed by holding the southbound turning movement volumes at Robal and instead applying the difference to the southbound through movement as shown in Exhibit 5-17. That volume adjustment is then pushed off the network. This results in more realistic volumes and illustrates how the balancing process is often iterative, i.e., initial assumptions may need to be changed during the process to improve the balance.

Once satisfied that no further improvements in network balance can be made, final balanced volumes are shown as in Exhibit 5-18. Any remaining volumes that were not adjusted and rounded in previous steps were rounded.

Exhibit 5-14 Splitting the Difference

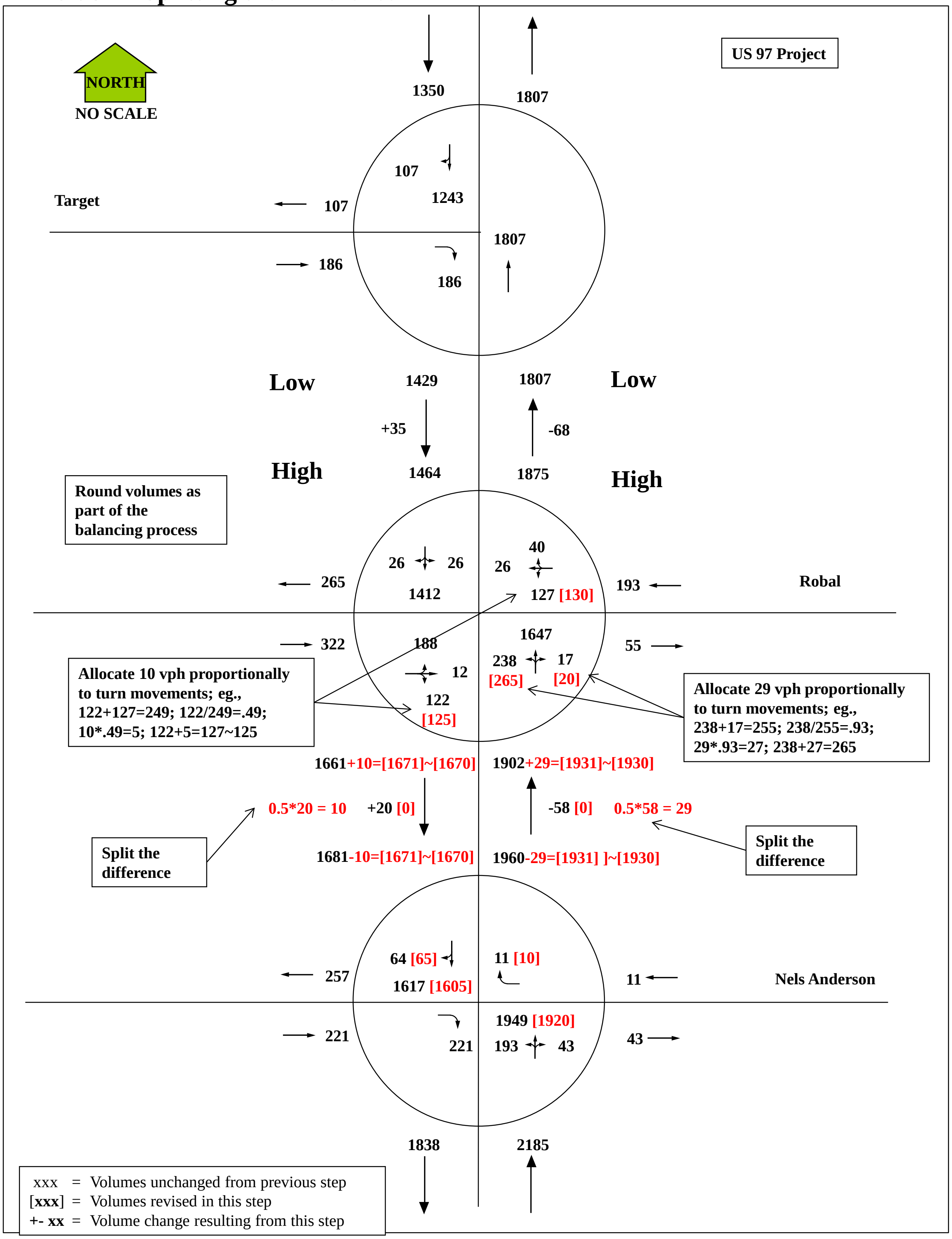


Exhibit 5-2 Adjustment Between Robal and Target Driveway

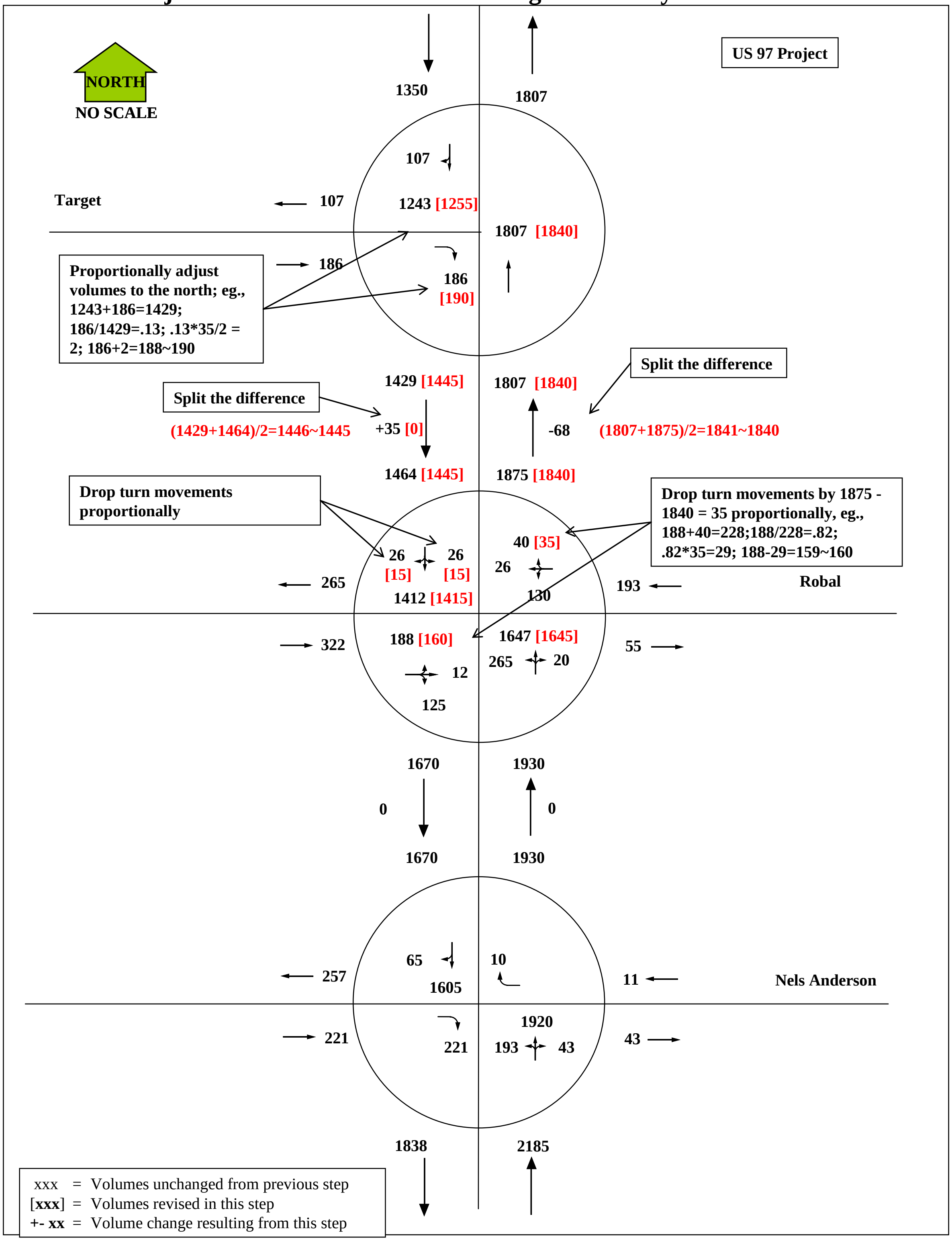


Exhibit 5-3 Adjusting volumes on Side Streets and Off the Network

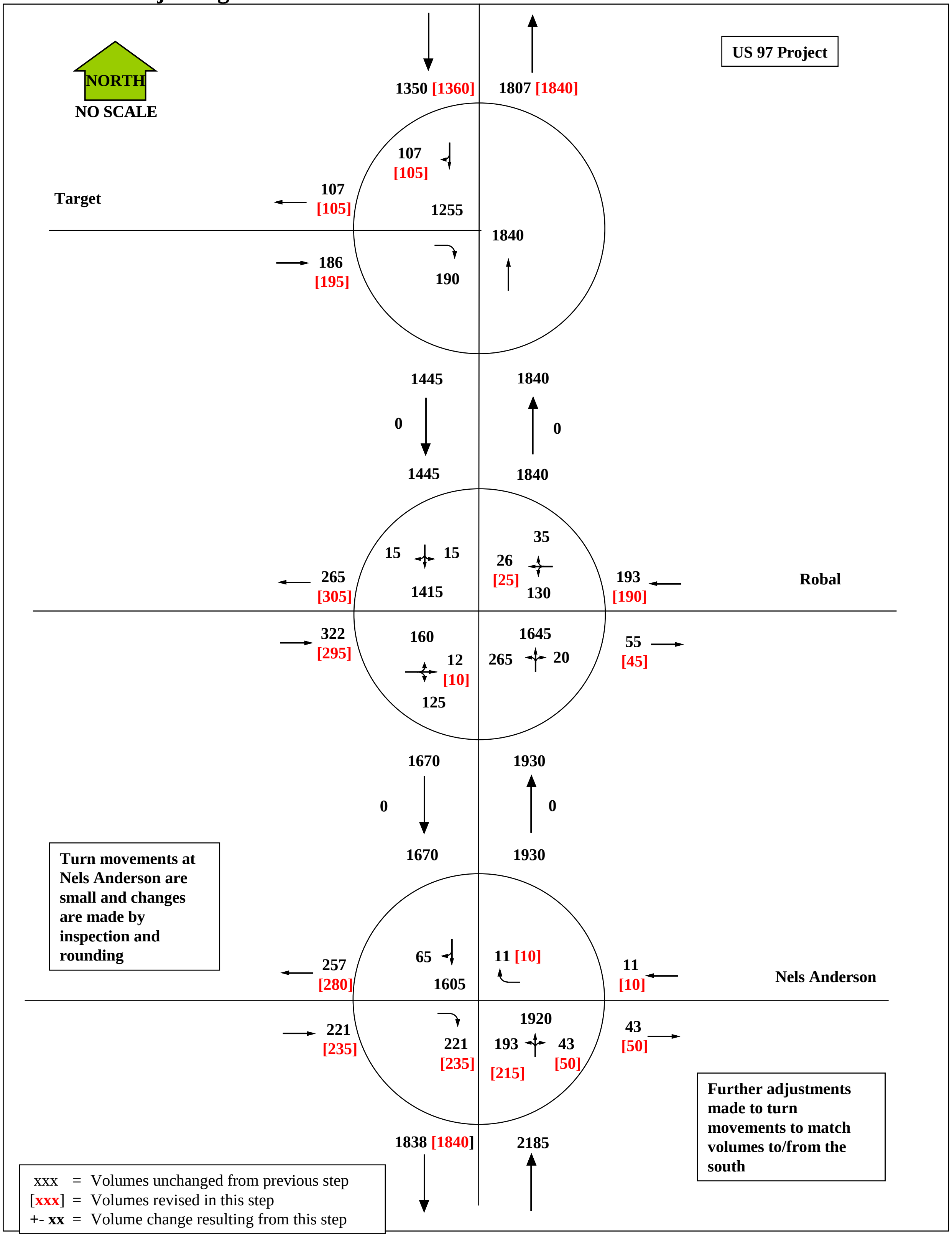


Exhibit 5-4 Revising Initial Assumption to Improve Balance

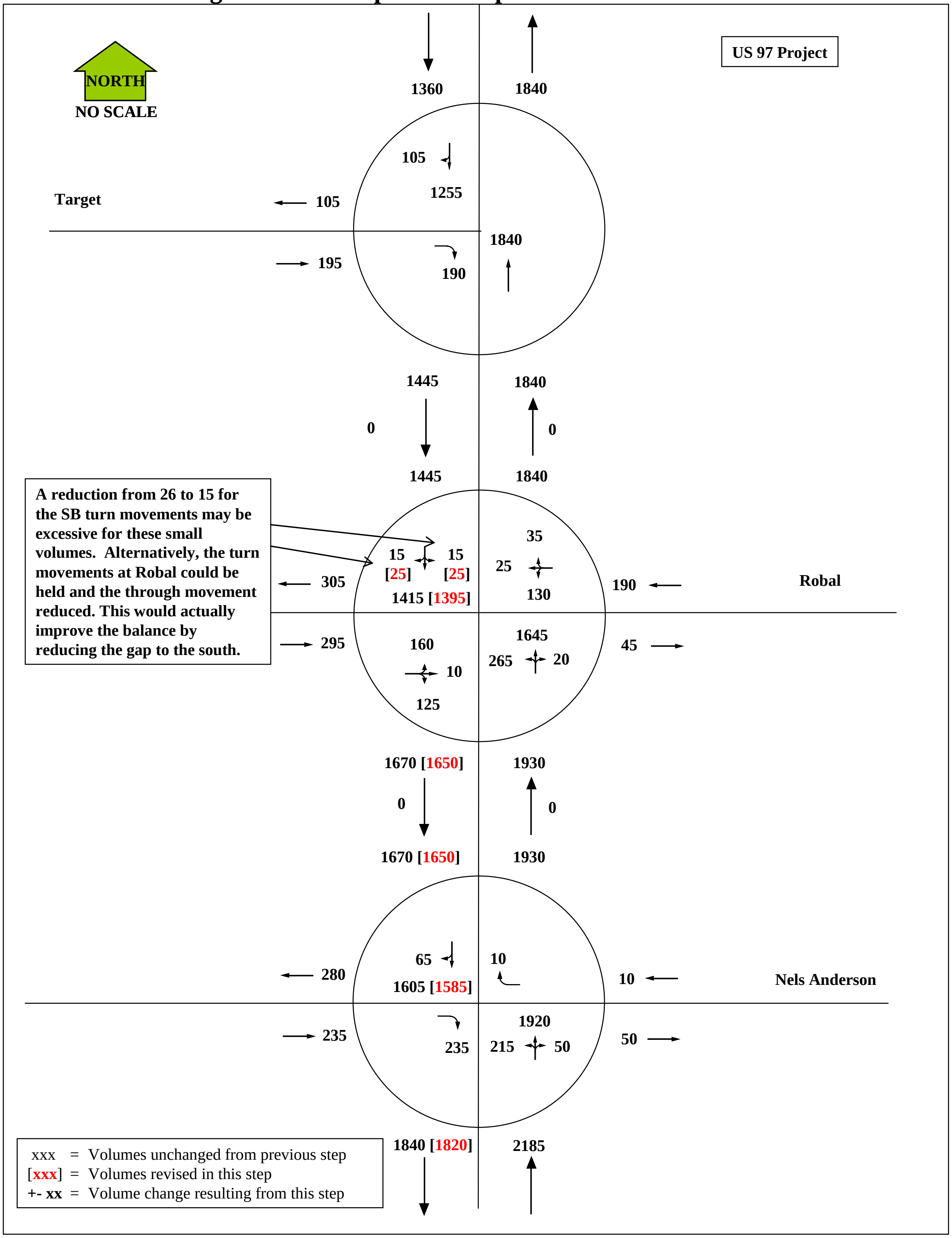
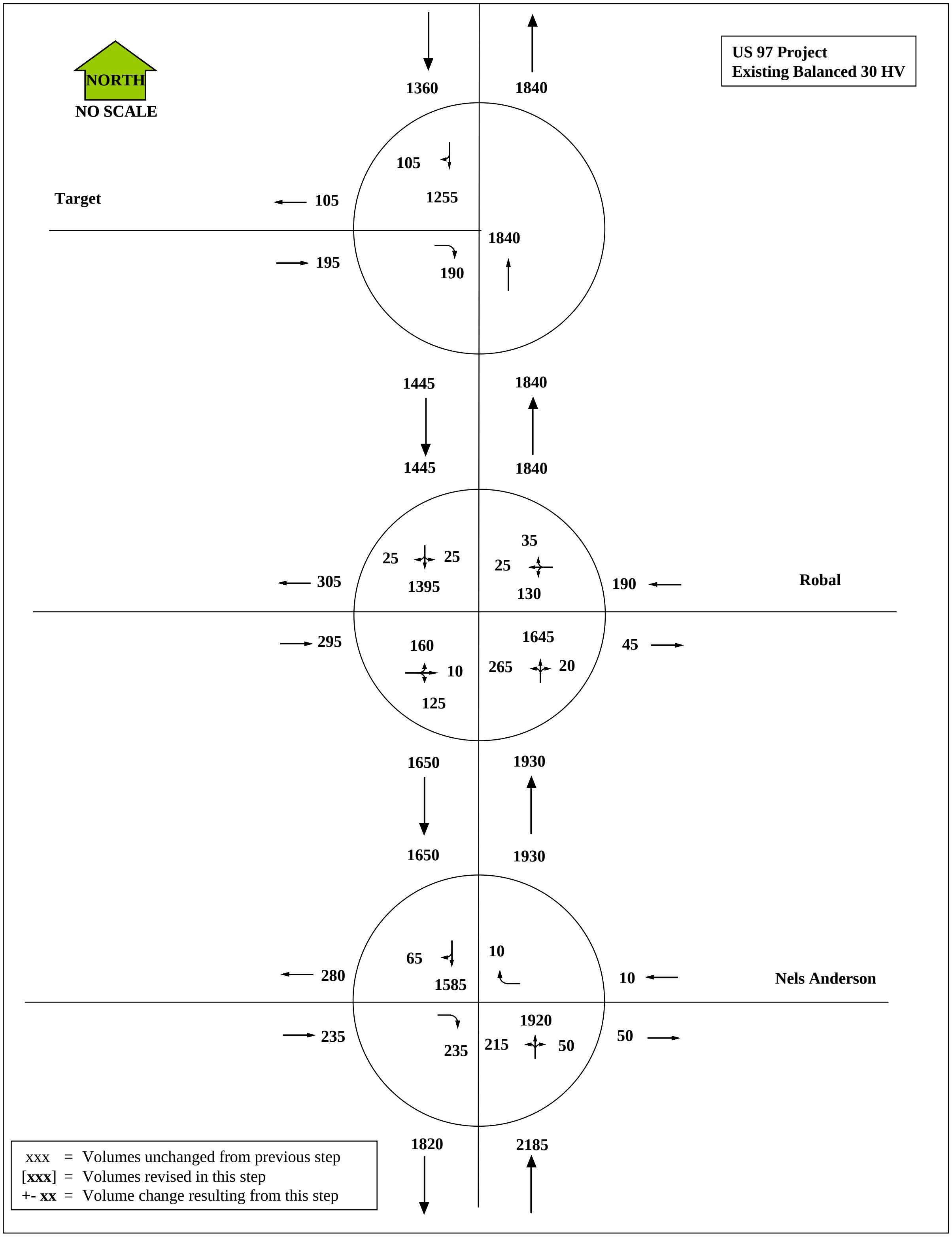


Exhibit 5-5 Final Balanced 30HV



5.6.2 Rounding

The final 30HV needs to be rounded. The traffic volumes are not that precise to go down to one vehicle with the normal errors that occur within the counting and adjusting processes. For reduction of systematic error, balancing should occur before rounding of values. However, balancing the network can be easier if the network is not down to the individual vehicle. The analyst can choose a methodology, but the important thing is to stick with one method consistently through the entire project. Volumes should be rounded to the nearest five for the existing or base year. Volumes less than five vehicles should use the "<5" symbol instead of using zero.

In calculations of seasonal, historical and other factors, intermediate values should not be rounded. Only final reported results should be rounded. Significant digits should be kept in mind and based on the value that has the least significant digits. For example, 34 x 2.538 should be rounded to no decimal places.

5.6.3 Documentation

It is critical that after every step in the 30HV volume development process that all of the assumptions and factors are carefully documented, preferably on the graphical figures themselves. Count specifics, raw and system peak hours used, ATRs used, seasonal adjustments, ATR 30HV adjustments, and yearly growth factors are some of the items that need to be documented. If all is clearly documented, then anyone can easily review the work or pick up on it quickly without questioning the assumptions made. The documentation figures will eventually end up in the final report or in the technical appendix. The volume documentation should include:

- Map-based figure showing (See Exhibit 5-19):
    - Roadway network and intersection lane configurations and control type
    - Raw traffic volumes showing raw peak hour
    - For each intersection, count date, duration, type, and raw peak hour time
- Map-based figure showing (See Exhibit 5-20):
    - Selected system peak hour
    - Raw count data by movement for the selected system peak hour
- Map-based figure showing (See Exhibit 5-21):
    - Yearly growth and seasonal factors used for each intersection
    - Unbalanced factored 30HV
- Map-based figure showing (See Exhibit 5-22) balanced 30HV (this figure is used in an existing year analysis technical memorandum).

Exhibit 5-6 Raw Traffic Counts

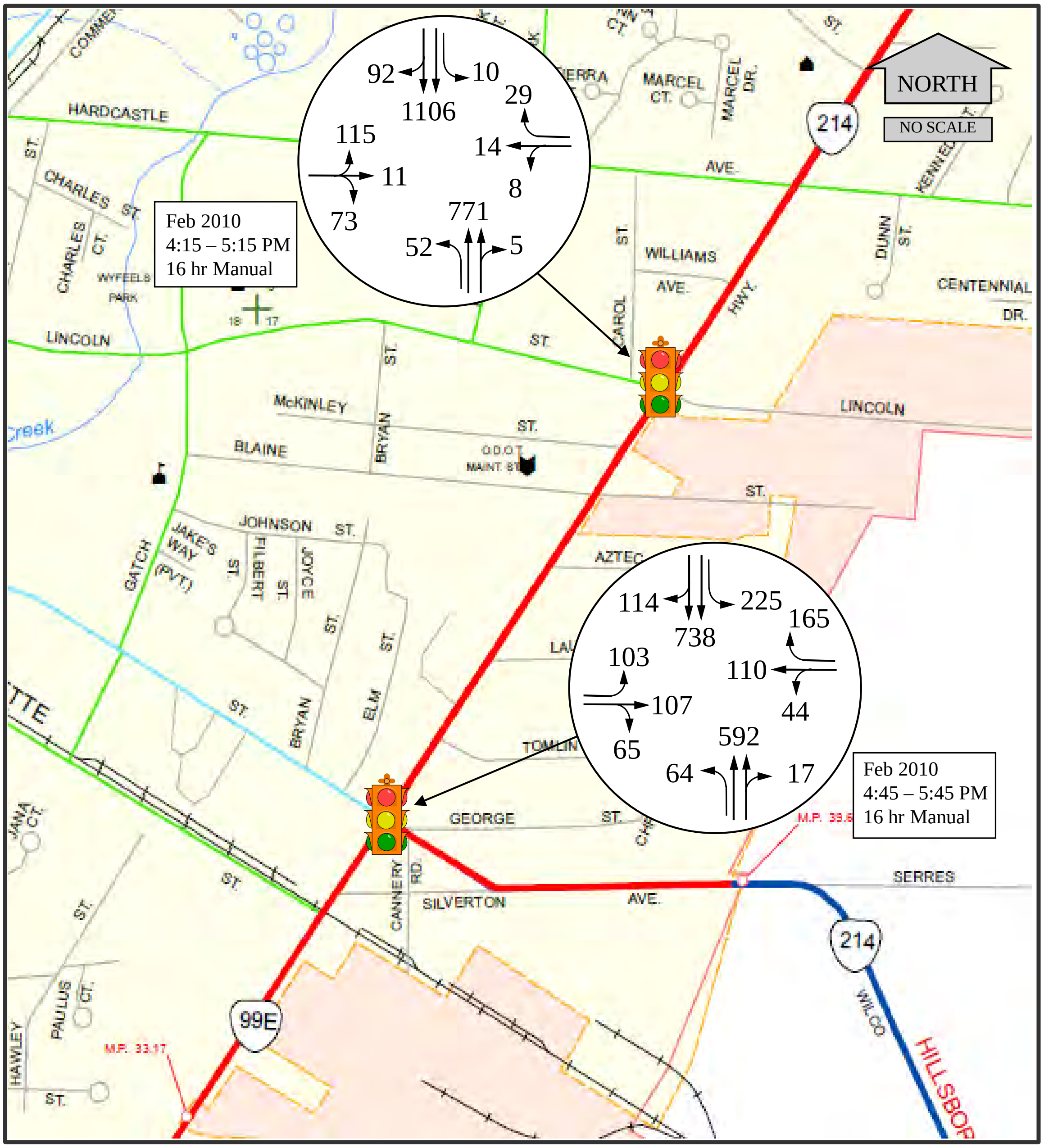


Exhibit 5-20 System Peak Hour

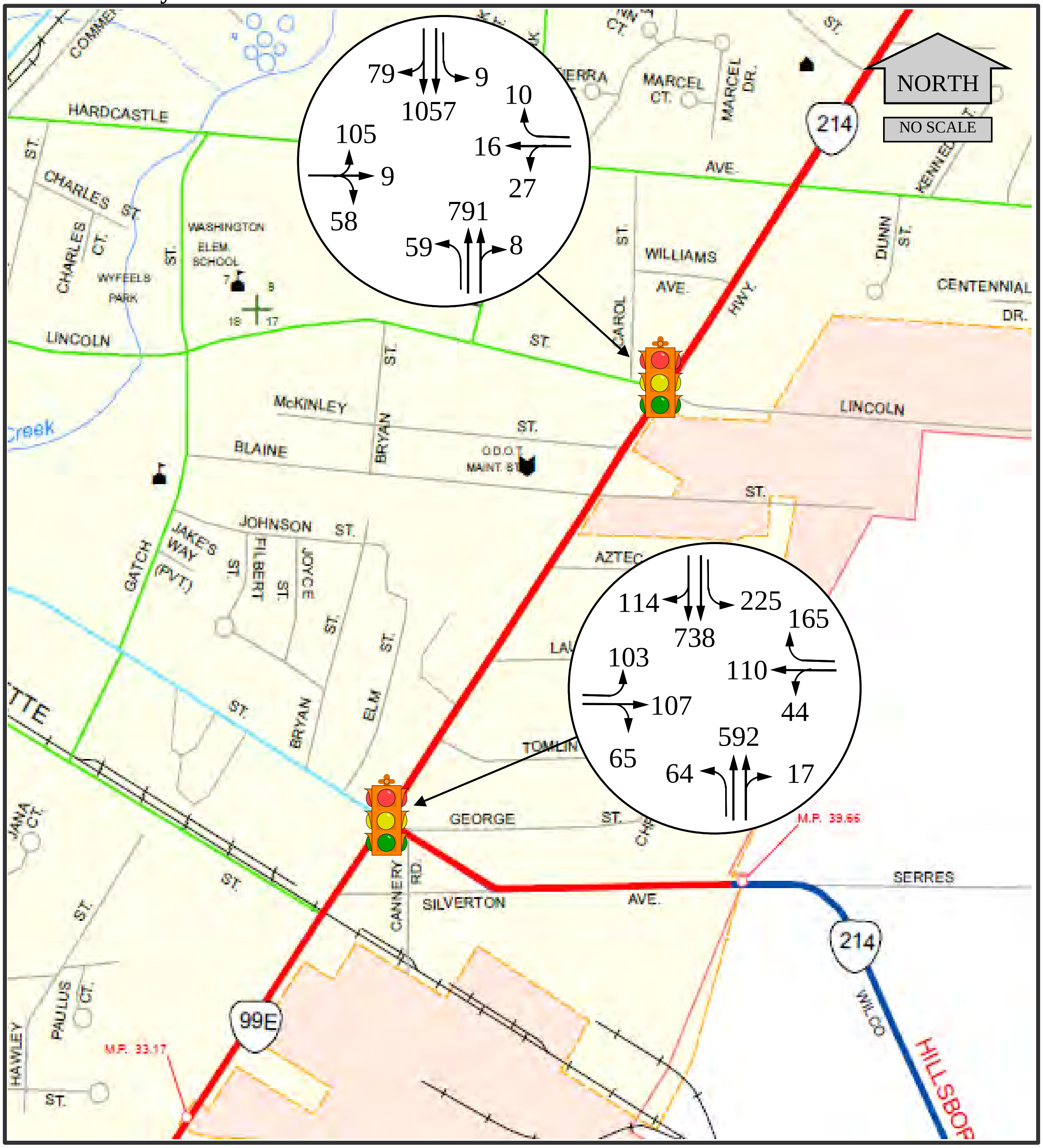


Exhibit 5-21 Seasonal and Historical Growth

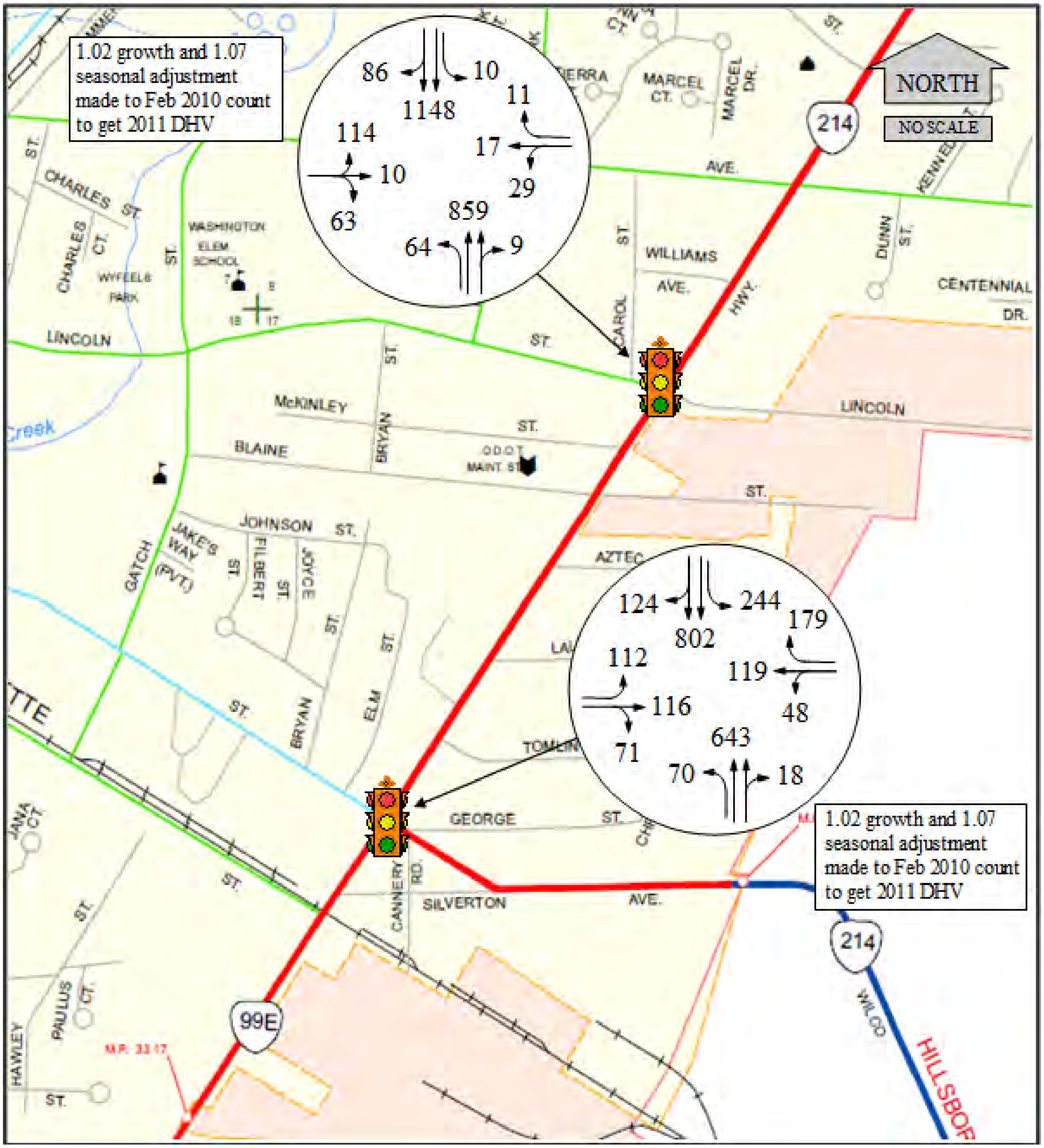
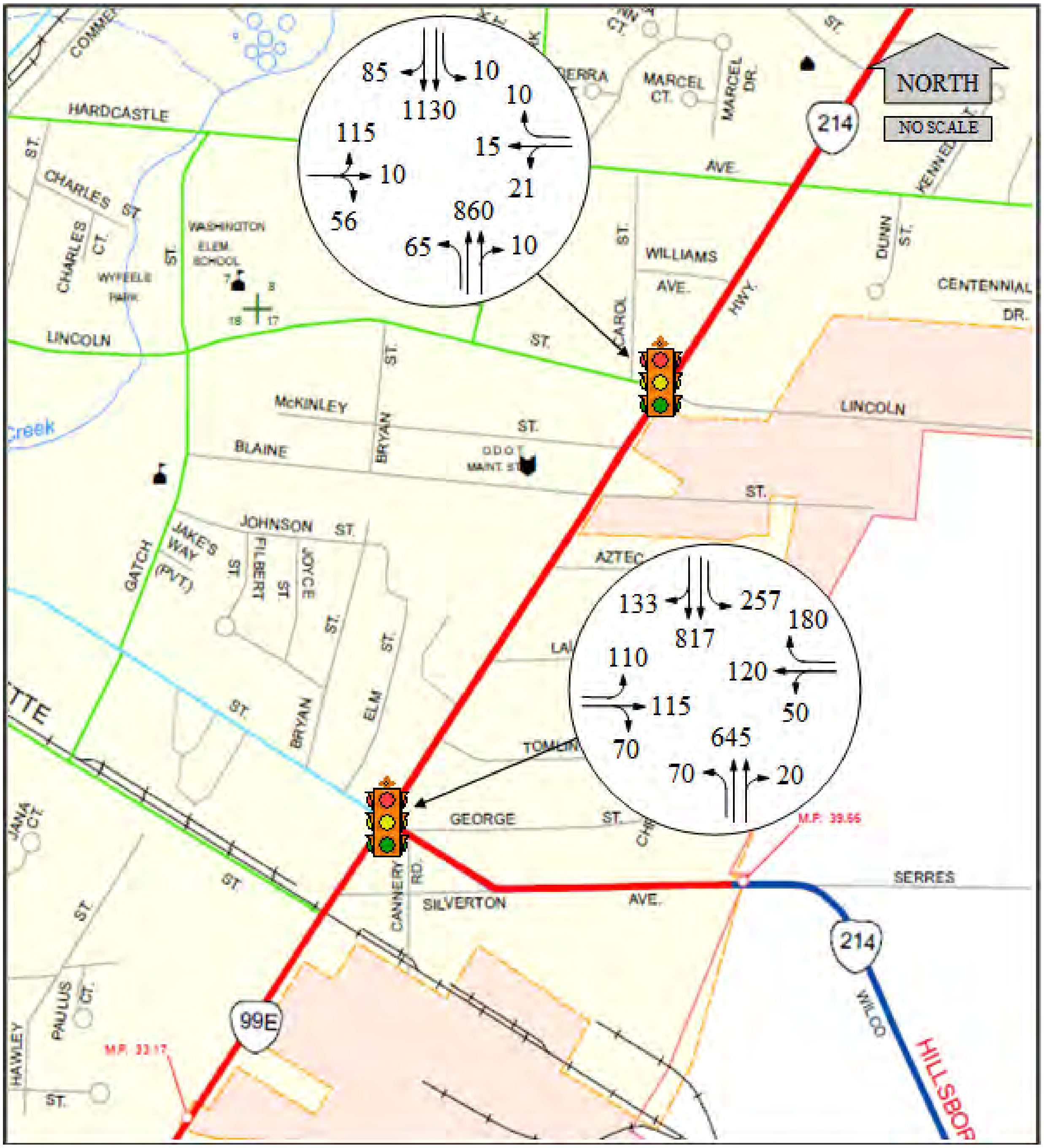


Exhibit 5-22 Balanced Network



5.7 K-30 Factor Development

For certain planning studies, such as a county TSP analysis, sketch planning level analysis of highway segments may be appropriate, for all or portions of the study. To develop planning-level design hour volumes, TVT AADT volumes can be used. The TVT AADT volumes are used to derive the 30HV by multiplying by the K-30 factor. A K factor is the ratio between the peak hour and the ADT. The K-30 factor is the ratio of the 30th highest hour to the AADT and should be used for this purpose. Short term count K factors should not be used for this purpose. A background report on this topic is available on the APM website[1] under Supplemental Materials.

K-30 factors are derived from ATRs and can be found in the TVT section on "Summary of Trends at ATR Stations". They are listed under Historical Traffic Data, Percent of ADT for the 30th Hour. A representative ATR needs to be identified following the procedures described in Section 5.4.

Example 5-8 Converting AADT to DHV using the (K_{30}) factor

This example illustrates how to calculate and apply the K-30 Factor.

Find the 30HV for sketch planning analysis for a segment of Kings Valley Highway No. 191 (OR 223) at MP 28.00.

- **Step 1: Transportation Volume Table** – The TVT is used to find the AADT for this segment.

The 2019 AADT from the Transportation Volume Table is found to be 820.

- **Step 2: ATR Trend Summary** – ATR 02-005, located on Kings Valley Highway at MP 26.40, can be used for this location. The ATR number corresponds to a table in the last half of the TVT that contains yearly summaries for each ATR. From the column titled "Percent of AADT" the 30th Hour percent of AADT should be recorded for the past five years. The highest and lowest percentages should be eliminated to account for construction activity that may have occurred in the vicinity of the ATR during the five-year period. An average percentage of AADT is then calculated for the remaining three years.

K-30 Factors Using ATR #02-005

	2019	2018	2017	2016	2015
K-30	12.2%	12.5%	13.5%	13.5%	12.1%

Note: Shaded values dropped from average calculation.

As shown in Example 5-1 Seasonal Factor – On-Site ATR, the percentage of AADT values listed during June and September for the past five years are reviewed to calculate the average. The

[1] Use of Short-Term Interval Counts to Determine K Factors, Don R. Crownover, P.E., ODOT Transportation Systems Monitoring (TSM) Unit, August, 2006.

highest and lowest values, shown as shaded, are dropped from this calculation. The average K-30 factor is determined as follows:

- The average K-30 factor is: (13.5% + 12.5% + 12.2%) / 3 = 12.7%.

Calculate the two-way 30HV:

30HV = (AADT) x (Average K-30 Factor) = 820 x 0.127 = 104 vph.

Obtain the directional split for the 30^{th} highest hour as defined by the D-30 factor from the OTMS "Volume/Annual High Hour" report or request the EDW (Enterprise Data Warehouse) direct pull of data from TSM unit. Internal ODOT employees can pull data from the ATRShar server drive. (see Appendix 3C: Oregon Traffic Monitoring System (OTMS) Count Report Guide).

From the 2019 Annual High Hour Report for ATR 02-005
D-30 NB = 0.64 and D-30 SB = 0.0.36

Calculate the directional 30HV:

30HV NB = (30HV) x (D-30 factor) = 169 x 0.64 = 108 vph.
30HV SB = (30HV) x (1-D-30 factor) = 169 x 0.36 = 61 vph

5.8 Average Daily Traffic (ADT) Development

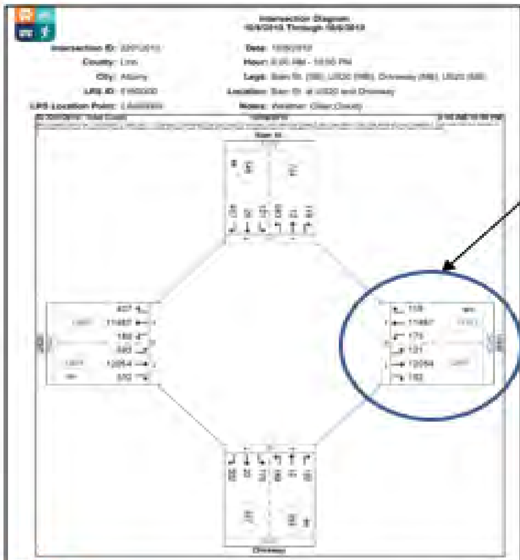
A number of operational analysis processes need to use daily traffic volumes instead of 30HV. Some of these are the safety analyses in Chapter 4, the preliminary signal warrants in Chapter 12 and air/noise data in Chapter 16. Many sketch planning and the HCM Planning analysis methods use ADT. ADT is also more perceivable by the public and is frequently used in meetings, environmental documents, and other traffic-related efforts. ADT should be thought of as a "peak month" ADT as it is developed using 30HV. This is not the same as annual average daily traffic (AADT) as AADT is developed using ATR sites which count all days of the year so seasonal trends can be accounted for.

ADT can be created directly on a link basis from 24-hr counts but these counts are unlikely to be done comprehensively over a study area or may be just limited to tube counts which do not catch turning movements. More typically, ADT is created from the 30HV or peak hour volumes.

The ADT conversion process requires the use of counts that can support ADT calculations (12 hours or more). In most cases, ADT cannot be created from peak hour or peak period counts. Use of shorter counts may be acceptable for use in preliminary signal warrants and safety-based analyses. K30 factors are not to be used in ADT development regardless of count duration as those are too general for a project analysis. K-factors and D (directional) factors need to come straight from an ADT-capable count or be derived from counts on other study area intersections/roadways with similar characteristics. Exhibit 5-23 shows an example of the calculation.

Exhibit 5-23 K & D – Factors from Counts

K and D-Factors from Counts



K-Factor

Westbound $\frac{941}{11761} = 0.080 \text{ or } 8.0\%$

Eastbound $\frac{1253}{12979} = 0.097 \text{ or } 9.7\%$

D-Factor

Westbound $\frac{11761}{25681} = 0.46 \text{ or } 46\%$

Eastbound $\frac{12979}{25681} = 0.51 \text{ or } 51\%$

Summary of Turning Movements
10/8/2010 Through 10/8/2010

Intersection ID: 22012010
County: Linn
City: Albany
LRS ID: 01600I00
LRS Location Point: 0.64
Date: 10/8/2010
Hour: 6:00 AM - 10:00 PM
Legs: Bain St. (SB), US20 (WB), Driveway (NB), US20 (EB)
Location: Bain St. at US20 and Driveway
Notes: Weather: Clear;Cloudy

System Peak Hour

24-hour volume ADT

	Summary By Movements													Entering Volumes			
Time of Day	N-E	N-S	N-W	E-N	E-S	E-W	S-N	S-E	S-W	W-N	W-E	W-S	Total	North	East	South	West
15:45	2	0	8	1	2	216	0	1	3	13	251	7	504	10	219	4	271
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4	250	1	4	4	8	276	5	550	16	236	9	289
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2	216	0	3	5	9	248	13	513	15	220	8	270
16:30	3	1	7	2	3	218	1	6	4	9	271	5	530	11	223	11	285
16:45	3	1	8	1	3	253	0	4	1	13	228	7	520	11	257	4	248
17:00	3	0	10	0	3	249	0	7	2	6	307	13	590	13	246	5	326
17:15	0	0	7	3	2	232	1	4	2	18	308	4	581	7	237	7	330
17:30	3	0	4	1	3	225	0	3	2	11	267	6	525	7	229	5	284
17:45	0	0	11	2	1	226	0	3	3	10	298	5	559	11	229	6	313
18:00	2	1	3	3	3	220	0	6	2	10	238	6	494	6	226	8	254
18:15	1	0	3	3	2	217	0	5	1	10	233	3	478	4	222	6	246
18:30	0	0	3	3	1	214	0	5	1	10	231	3	471	3	218	6	244
18:45	0	0	1	2	1	211	0	4	0	10	231	3	463	1	214	4	244
19:00	2	1	6	3	2	145	0	4	2	5	171	3	342	9	148	6	179
19:15	2	0	5	2	2	142	0	1	1	5	167	2	329	7	146	2	174
19:30	0	0	5	1	1	141	0	1	1	5	165	1	321	5	143	2	171
19:45	0	0	5	0	0	139	0	1	1	4	165	1	314	5	139	2	168
20:00	2	0	7	1	1	82	0	2	2	9	111	2	219	9	84	4	122
20:15	2	0	7	1	1	80	0	1	1	9	106	2	210	9	82	2	117
20:30	2	0	5	0	1	78	0	1	0	7	105	1	200	7	79	1	113
20:45	0	0	5	0	0	77	0	0	0	7	108	1	193	5	77	0	111
21:00	[illegible]	0	11	[illegible]	1	69	0	1	2	3	86	3	182	16	71	3	92
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	67	0	1	1	3	82	2	171	15	67	2	87
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	64	0	0	0	3	80	0	161	14	64	0	83
21:45	3	0	8	0	0	61	0	0	0	2	78	0	152	11	61	0	80
Total Count	121	20	407	119	175	11467	12	192	189	593	12054	332	25681	548	11761	393	12979

When the full 24-hour volume is not available, use the count expansion factors in Exhibit 5-24 to estimate from shorter duration counts.

Exhibit 5-7 Count Expansion Factors

Count Length (hours)	Expansion Factor
12	1.25
14	1.18
16	1.10
24	1.00

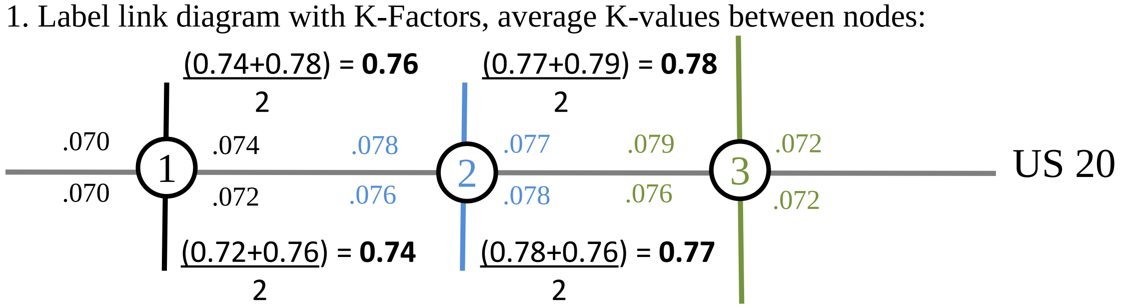
The D factor is calculated by dividing the highest 24-hour one-way volume by the two-way 24-hour volume.

Draw a link diagram of the study area. Label the K factors in one diagram and D factors in another diagram. Calculate average K factors between intersections. Links missing K or D factors (such as those using peak period counts) can be obtained (estimated) by averaging links with similar characteristics together in the study area. Divide the 30 HVs by the K factors to obtain the ADT on each directional link. Now sum the directional link ADTs to obtain two-way link ADTs. Now multiply the two-way ADTs by the D factor to obtain directional ADTs. The resulting splits may differ between the 30HV and ADT volumes.

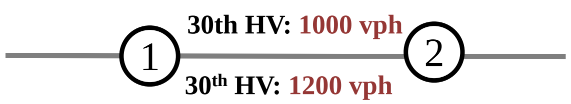
ADTs should be documented on a map-based figure on a link approach basis rather than showing individual turn movements.

Example 5-9 ADT Calculations

1. Label link diagram with K-Factors, average K-values between nodes:



2. Divide 30 HV by average link K-value (Step 1) to calculate ADT, sum ADT:

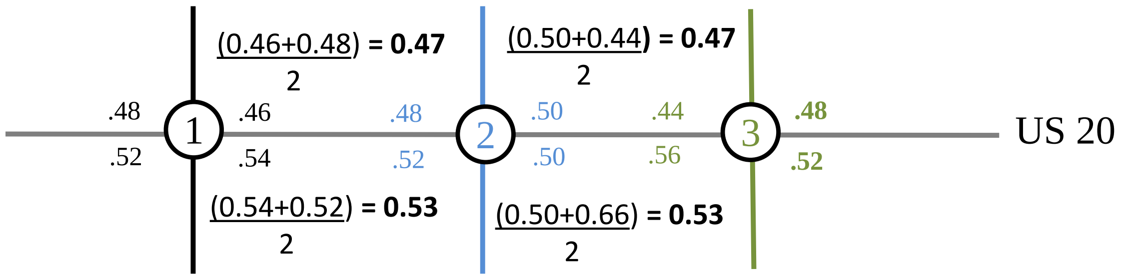


1000/**.076** = 13,158 ADT

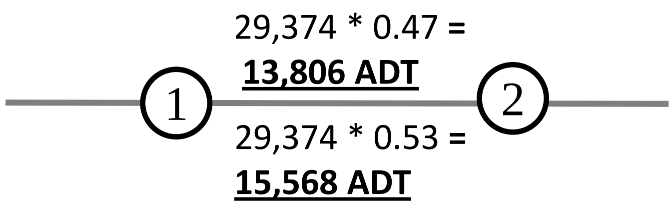
1200/**.074** = 16,216 ADT

∑ 29,374

3. Label link diagram with count D-Factors, average D-values between nodes:



4. Multiply ADT link sum (Step 2) by average D-value to calculate directional ADT:

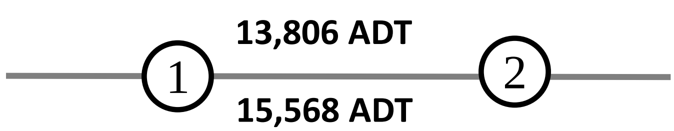


5.8.1 Converting ADT to AADT

ADT can be converted to AADT by applying the seasonal factor (See Section 5.4) straight instead of using the inverse in the 30HV process. This will convert the ADT to an average value (May/October). Example 5-9 ADT to AADT Conversions shows the typical calculations.

Example 5-10 ADT to AADT Conversions

Counts used to create the ADT values from the previous Example 5-8 ADT Calculations, were taken October 8th in a location characterized by a summer trend. After reviewing the ATR Characteristic Table it is determined that there are no characteristic ATR's that can be applied therefore, the Seasonal Trend Table must be used to convert the ADT to AADT.

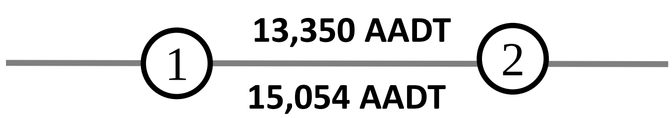


1. Using the Seasonal Trend Table under Summer Trend, averaging the October 1st and 15th column, the factor is 0.9670.

TREND	15-Feb	1-Mar	15-Mar	1-Apr	15-Apr	1-Oct	15-Oct	1-Nov	15-Nov	1-Dec	15-Dec
INTERSTATE URBANIZED	1.0390	1.0322	1.0254	1.0050	0.9845	0.9849	0.9968	1.0130	1.0292	1.0414	1.0536
INTERSTATE NONURBANIZED	1.2265	1.1538	1.0811	1.0579	1.0347	0.9551	0.9989	1.0247	1.0506	1.1208	1.1910
COMMUTER	1.0352	1.0428	1.0503	1.0158	0.9813	0.9720	0.9803	1.0074	1.0346	1.0547	1.0749
COASTAL DESTINATION	1.0873	1.0993	1.1114	1.0737	1.0359	0.9635	1.0370	1.0906	1.1441	1.1672	1.1903
COASTAL DESTINATION ROUTE	1.2552	1.2561	1.2571	1.1881	1.1191	0.9389	1.0669	1.1473	1.2276	1.3028	1.3779
AGRICULTURE	1.2653	1.2511	1.2369	1.1493	1.0617	0.8622	0.9263	0.9984	1.0705	1.2224	1.3742
RECREATIONAL SUMMER	1.5130	1.4824	1.4519	1.3336	1.2152	0.9005	1.0231	1.1707	1.3183	1.4751	1.6319
RECREATIONAL SUMMER WINTER	0.9771	1.0169	1.0567	1.1357	1.2146	1.2207	1.4874	1.5592	1.6311	1.2701	0.9091
RECREATIONAL WINTER	0.6663	0.7094	0.7526	0.9446	1.1365	1.4499	1.7607	2.2112	2.6618	1.6508	0.6398
SUMMER	1.1778	1.1604	1.1430	1.0881	1.0332	0.9436	0.9904	1.0445	1.0985	1.1455	1.1925
SUMMER < 2500	1.2418	1.2093	1.1768	1.1022	1.0276	0.9126	0.9590	1.0366	1.1142	1.1979	1.2816

*Seasonal Trend Table factors are based on previous year ATR data. The table is updated yearly.
*Grey shading indicates months were seasonal factor is greater than 30%

2. Multiply the factor times by ADT to convert to AADT



5.9 <u>Development of Volume-based Analysis Inputs</u>

5.9.1 Existing Peak Hour Factors

Peak hour factors (PHF) are used to account for the non-uniformity of traffic flow within the peak hour by converting hourly volumes to peak flow rates associated with a selected interval of time within the peak hour. The most common interval of time selected for traffic analysis is the peak 15 minutes. In areas near capacity the peak 15-minute flow can cause up to several hours of congestion. This typically happens when the demand exceeds the available capacity of the transportation system resulting in “peak hour spreading”, which is the extension of the peak period caused by a system breakdown. Therefore, it is often essential that the transportation system be designed to accommodate the peak 15 minutes of the peak hour.

Peak hour factors should be applied in most capacity analyses in accordance with the HCM, which selected 15-minute flow rates as the basis for most of its procedures. It is especially critical to examine the peak 15-minute period when potential queue lengths may become an issue, and at locations with sharp peaking characteristics such as employment sites and locations with low peak hour factors (less than 0.90). Some alternate mobility standards do not use PHFs where the volume-to-capacity ratio is equal to or greater than 1. Alternate mobility standards procedures are discussed further in Chapter 10.

5.9.2 Calculation

The PHF is typically calculated using data from traffic counts. It is the traffic volume during the peak 60-minute period divided by four times the volume during the peak 15-minute period.

$$\text{PHF} = \frac{\text{Volume During Peak 60-Minute Period}}{4 \times (\text{Volume During Peak 15-Minute Period})}$$

Typical PHF values range between 0.80 and 0.98. Factors greater than 0.95 are indicative of high traffic volumes, while factors less than 0.80 occur in locations with high peak demand (i.e. schools, factories with shift changes, or venues with scheduled events). PHFs calculated from actual traffic count data should always be used for analysis of existing conditions. For all applications other than sketch planning-level analysis, traffic count data should be obtained in 15-minute intervals. In calculating PHF, the system peak hour is first selected, and PHFs are calculated within that hour.

For segment analysis at a sketch planning-level, where traffic counts are not provided in 15-minute intervals, the HCM suggests the following PHF defaults:

- 0.95 for urban freeways;
- 0.92 for interrupted flow facilities;
- 0.88 for rural freeways, multilane highways and two-lane highways

Intersection PHF
The analysis method for intersections uses an intersection PHF to estimate peak 15-minute period equivalent hourly flow rates from the peak 60-minute period volumes. The peak 15-minute period with the highest intersection total entering volume (TEV) should be used to determine the PHF for each intersection. The application of global PHFs is generally not appropriate when count data are available. The intersection PHF is calculated as follows.

- **Step 1**: Determine the peak 15-minute period that has the highest intersection total entering volume (TEV).
- **Step 2**: Calculate the intersection PHF based on the time period determined in Step 1, by dividing the peak 60-minute TEV by four times the TEV occurring during the peak 15 minutes.
- **Step 3**: In the analysis software, apply the intersection PHF from Step 2 for all movements at the intersection (flow rates are then usually calculated by the analysis software).

Movement PHF should not be used because using the individual peak 15-minute periods by movement creates a situation that overestimates the volumes and does not exist in reality akin to the reasoning behind using a system peak hour. Conversely, using a single 15-minute period in essence creates a need to force this on all intersections which would create a “false precision” level of detail in the analysis (a system 15-min is too detailed), not to mention the difficulties in choosing the proper interval which may result in analyzing multiple intervals. In circumstances of unusually pronounced peaking on an intersection approach where an alternative method is desired, a written explanation detailing the analysis area context and proposed alternative methodology must be submitted and approved by TPAU and Region Traffic prior to use.

For intersection analysis at a sketch planning-level, where traffic counts are not provided in 15-minute intervals, the following intersection PHF defaults should be used:

- 0.95 for major arterial-major arterial;
- 0.92 for major arterial-minor arterial;
- 0.90 for minor arterial-minor arterial;
- 0.88 for minor arterial-collector;
- 0.85 for collector-collector or lower classification

5.9.3 Future Conditions PHF

Because traffic flow patterns may change over time and future conditions cannot be directly measured, analysis of future years should incorporate the sketch planning default values for PHF listed previously.

Engineering judgment must be used in the selection of PHFs for future years. In cases where the existing PHF is higher than the default value for the future PHF, it may be appropriate to retain the existing value for the future year, as PHFs do not typically decrease as traffic volumes and congestion increase. Likewise for areas that have low existing peak hour factors, using the future PHF default values could produce results that would underestimate the future traffic conditions. For areas with aggressive traffic demand management strategies contained in an adopted plan, a different PHF (to reflect spreading of the demand) may be used for future year analysis if agreed to by ODOT during the scoping process. For areas with pronounced peaking characteristics, such as industrial sites and schools, PHFs lower than the default values listed above should be considered.

Revising PHF for Future Year
In areas where alternative mobility targets are in place and the volume to capacity ratios are at or exceed 1.0 (capacity), PHFs may be assumed to be equivalent to 1.0.

5.9.4 Truck Factor Development and Documentation

Most deterministic analyses will require truck percentages. These can be developed directly from counts. Truck percentages are made up of the bus, single unit, single trailer, and multiple trailer vehicle classifications. Exhibit 5-25 shows a typical intersection count with the necessary fields highlighted.

Exhibit 5-85 Truck Classifications to use in an intersection count

Traffic Count Summary By Movement
10/1/2012 Through 10/1/2012

Intersection ID: 19796
County: Douglas
City: Roseburg
LRS ID: 001HOI00
LRS Location Point: 123.80

Date: 10/1/2012
Hour: 6:00 AM - 10:00 PM
Legs: SW Bellows St. (SB), W Harvard Ave. (WB), I-5 s/b on/off ramps (NB), W Harvard Ave. (EB)
Location: SW Bellows St. at W Harvard Ave. and I-5 s/b on/off ramps
Notes: Weather: Clear

South => West					Single Unit Truck			Single Trailer Truck			Multi Trailer Truck					
Time of Day	Motor cycle	Car	Light Truck	Bus	2 Axle	3 Axle	4 or more Axle	4 or less Axle	5 Axle	6 or more Axle	5 or less Axle	6 Axle	7 or more Axle	Ped	Bicycle	Vehicles
12:00	0	28	23	1	1	0	0	0	0	0	0	0	0	0	0	53
12:15	0	28	22	0	1	0	0	0	0	0	0	0	0	0	0	51
12:30	0	28	22	0	1	0	0	0	0	0	0	0	0	0	0	51
12:45	0	27	22	0	1	0	0	0	0	0	0	0	0	0	0	50
13:00	1	33	23	1	1	0	0	0	1	0	0	0	0	0	0	60
13:15	1	32	23	0	0	0	0	0	0	0	0	0	0	0	0	56
13:30	1	32	23	0	0	0	0	0	0	0	0	0	0	0	0	56
13:45	0	32	23	0	0	0	0	0	0	0	0	0	0	0	0	55
14:00	1	43	34	2	1	0	0	1	0	0	0	0	0	0	0	82
14:15	0	42	34	1	0	0	0	0	0	0	0	0	0	0	0	77
14:30	0	42	34	1	0	0	0	0	0	0	0	0	0	0	0	77
14:45	0	42	34	1	0	0	0	0	0	0	0	0	0	0	0	77
15:00	0	40	16	0	0	0	0	0	0	0	0	0	0	0	0	56
15:15	0	36	15	0	0	0	0	0	0	0	0	0	0	0	0	51
15:30	0	32	11	0	0	0	0	0	0	0	0	0	0	0	0	43
15:45	0	34	12	0	0	0	0	0	0	0	0	0	0	0	0	46
16:00	0	33	10	0	0	0	0	0	0	0	0	0	0	0	0	43
16:15	0	34	9	0	0	0	0	0	0	0	0	0	0	0	0	43
16:30	1	36	9	0	1	0	0	0	0	0	0	0	0	0	0	47
16:45	0	26	7	0	0	0	0	0	0	0	0	0	0	0	0	33

This is the peak hour. This line will need to be "pulled" from each sheet for each direction and movement.

In the count shown above, truck volumes need to be added up for this movement (S-W) for the system peak hour. To calculate the truck percentage for the northbound approach from the south leg, sum the trucks from the S-W, S-N and S-E movements and divide by the total entering volume from the south leg. To calculate the percentage of trucks for each movement, divide the truck volume by the total volume for that movement.

Example calculation for Northbound Truck % on west leg (Note: Don't forget to include buses)

$$NB\ Truck\% = \sum \frac{S{\rightarrow}N_{truck} + S{\rightarrow}E_{trucks} + S{\rightarrow}W_{trucks}}{All\ NB_{veh}}$$

$$NB\ Truck\% = \sum \frac{(0) + (3+2+7+3) + (1+1+5)}{28+530+626}$$

$$NB\ Truck\% = \frac{22}{1184} \qquad NB\ Truck\% = 0.01858 = 1.86\%$$

Example calculation for S-W Truck %

$$S{\rightarrow}W\ Truck\% = \sum \frac{S{\rightarrow}W_{truck}}{All\ S{\rightarrow}W_{veh}}$$

$$S{\rightarrow}W\ Truck\% = \sum \frac{(1 + 1 + 5)}{626}$$

$$S{\rightarrow}W\ Truck\% = \frac{7}{626} = 0.0118 = 1.12\%$$

Some analysis tools have different splits than just a “percent trucks.” Highway Capacity Software (HCS) has bus and recreational vehicle splits. Buses can be obtained from the count directly, but the recreational vehicle split has to be ignored as ODOT does not specifically call out this vehicle type. Once the appropriate classifications to use are identified, the values are summed up by intersection approach (or movement if desired).

Truck percentages can also be determined from the Truck Summary OTMS reports shown in Exhibit 5-25 (if individual bus splits were not needed). The Truck Summary report splits out trucks into medium and heavy categories by direction. Medium trucks are two-axle single unit and buses while heavy trucks are the three-axle and greater single unit and all combination vehicles. Exhibit 5-26 can be used to help identify the truck and all-vehicle peak hours and the medium and heavy truck percentages assuming that the csv format is downloaded into Excel.

Exhibit 5-9 OTMS Truck Summary Vehicle Classification Report

Oregon Traffic Monitoring System

Truck Summary Report for 1/1/2019 - 12/31/2019
Criteria: Location ID = 10011

Location: PACIFIC HIGHWAY NO. 1
Location ID: 10011
Milepoint: 82.04
Type: I-SECTION

		Medium Trucks (Classes 4-5)		Heavy Trucks (Classes 6-13)		All Vehicles	
	Start Time	NB	SB	NB	SB	NB	SB
08/27/2019	07:00:00	28	37	110	139	384	401
	08:00:00	35	34	143	179	504	485
	09:00:00	49	72	134	185	584	618
	10:00:00	48	70	165	181	673	633
	11:00:00	64	87	173	284	683	959
	12:00:00	66	58	167	164	723	692
	13:00:00	55	62	233	193	711	720
	14:00:00	55	81	247	258	793	895
	15:00:00	50	62	253	183	794	723
	16:00:00	39	65	266	193	763	710
	17:00:00	30	58	241	189	684	691
	18:00:00	34	36	213	173	595	504
	19:00:00	31	23	188	129	471	393
	20:00:00	12	20	149	116	327	336
	21:00:00	11	6	125	86	264	231
	22:00:00	10	6	110	86	214	233
	23:00:00	6	6	87	74	158	173
08/28/2019	00:00:00	6	6	74	51	133	131
	01:00:00	4	2	68	41	103	95
	02:00:00	9	6	76	63	123	119
	03:00:00	3	2	60	71	98	117
	04:00:00	9	5	71	73	141	183
	05:00:00	10	2	91	95	212	205
	06:00:00	31	25	84	130	277	290

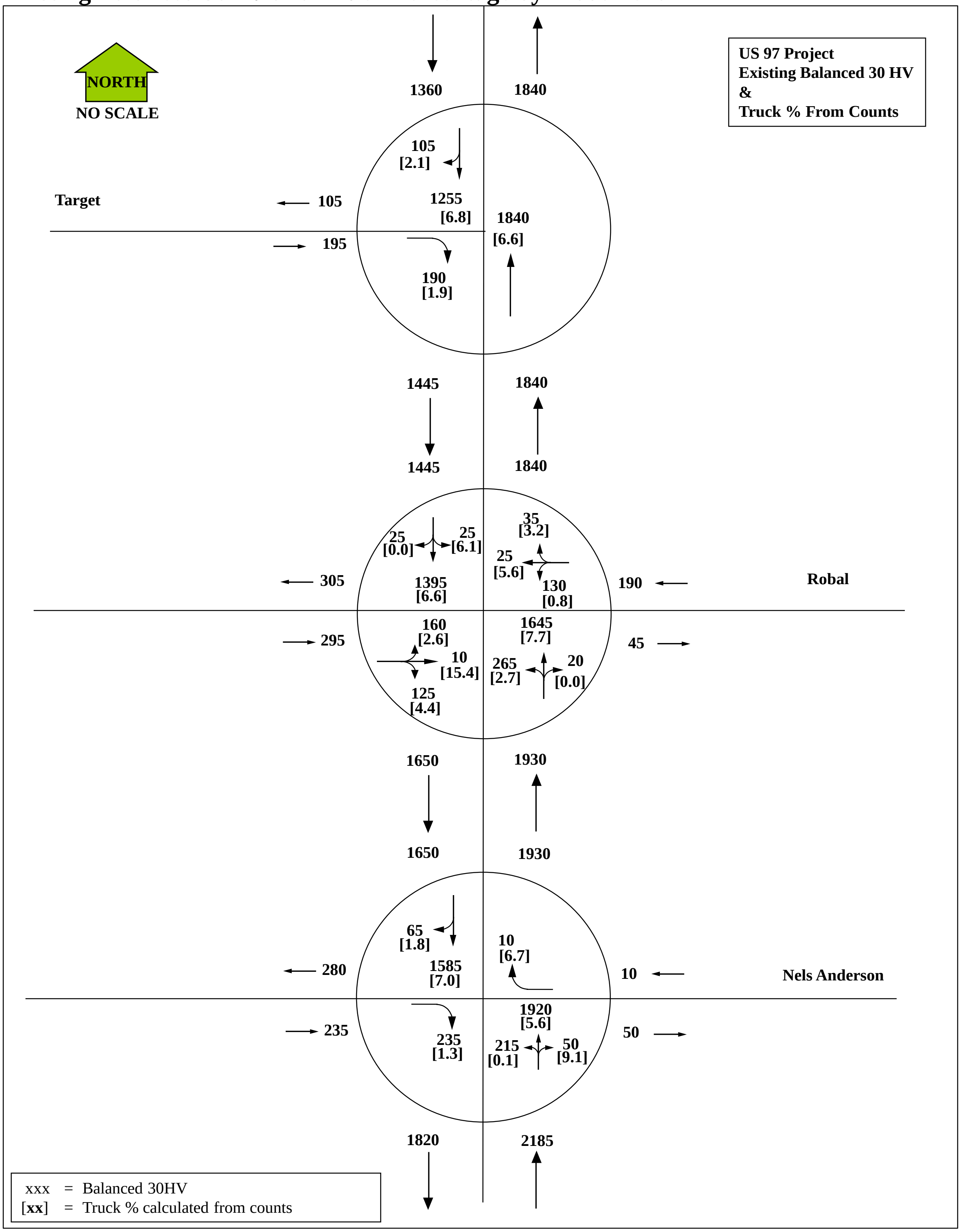
Generally, the truck percentages are entered into analysis software and used as is. However, in some projects and plans where truck movement is a large concern (or part of the project need) the truck percentages will need to be split out by movement and multiplied by the movement volume to determine the actual volume of trucks. This truck volume will need to be balanced as any other volume set and the actual truck percentages re-figured so the analysis is accurate.

Just using approach volumes would create areas where truck volumes would be inconsistent either by having trucks "disappear" or "appear" in the middle of a link or between links that would not make sense.

Example 5-11 Actual Link Truck Factors

When truck movements are of concern on a project, it may be necessary to balance the truck volumes. This requires the truck percentages from the counts as calculated above, as well as balanced 30 HV volumes (see Exhibit 5-18). Truck volumes are not rounded as the final outputs desired are percentages. The Figure below shows the truck percentages that were calculated from the intersection counts and the balanced base year volumes.

Existing Balanced 30HV and Truck Percentage by Movement



Step 1: Calculate the 30HV Truck volumes by multiplying the calculated truck percentage and

the 30HV balanced total volume. Sample calculations for the Target intersection are shown below.

Sample Truck Volume Calculations for the Target intersection

$$Truck\ Vol = Truck\ \% \times Movement_{Total\ Vol}$$
$$N \rightarrow S_{truck\ vol} = 0.068 \times 1255 = 85.3 = 85\ Trucks$$
$$N \rightarrow W_{truck\ vol} = 0.021 \times 105 = 2.2 = 2\ Trucks$$
$$S \rightarrow N_{truck\ vol} = 0.066 \times 1840 = 121.4 = 121\ Trucks$$
$$W \rightarrow S_{truck\ vol} = 0.019 \times 190 = 3.61 = 4\ Trucks$$

Step 2: After calculating all raw truck volumes, they must now be balanced. Balance the truck volumes using the same procedure used to balance a network. In this example, the difference was split between Nels Anderson and Robal Rd. Then holding Robal Rd constant the volumes at Target were proportionally adjusted to balance the remainder of the network.

Step 3: Once the truck volumes have been balanced, a new balanced truck percentage must be calculated by dividing the balanced truck volume by the total balanced volume for each movement. Sample calculations for the balanced truck percentages for the Target intersection are shown below.

Sample Calculated Truck Percentages

$$Truck\ \% = \frac{Balanced\ Truck\ Vol}{Movement_{BalancedTotal\ Vol}}$$

$$N \rightarrow S_{Truck\ \%} = \frac{95}{1255} = 0.0756 = 7.6\%$$

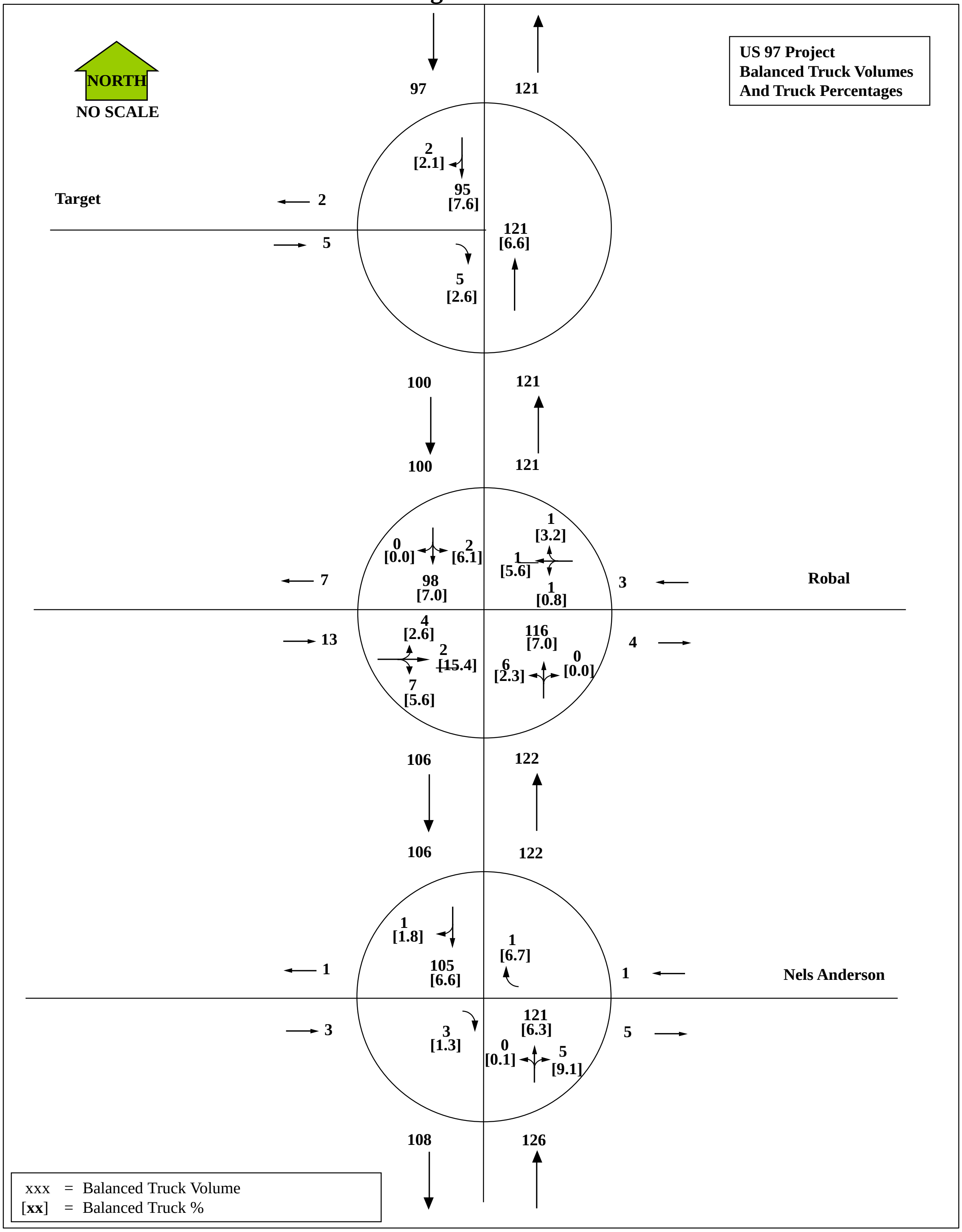
$$N \rightarrow W_{Truck\ \%} = \frac{2}{105} = 0.190 = 2.0\%$$

$$W \rightarrow S_{Truck\ \%} = \frac{5}{190} = 0.0263 = 2.6\%$$

$$S \rightarrow N_{Truck\ \%} = \frac{121}{1840} = 0.0657 = 6.6\%$$

The figure below shows the balanced truck volumes and the new calculated truck percentages. NOTE: The overall volumes do not change. The overall network is still balanced, no balanced volumes were changed. Only the truck volumes were changed and percentages adjusted to reflect the balancing of the truck volumes.

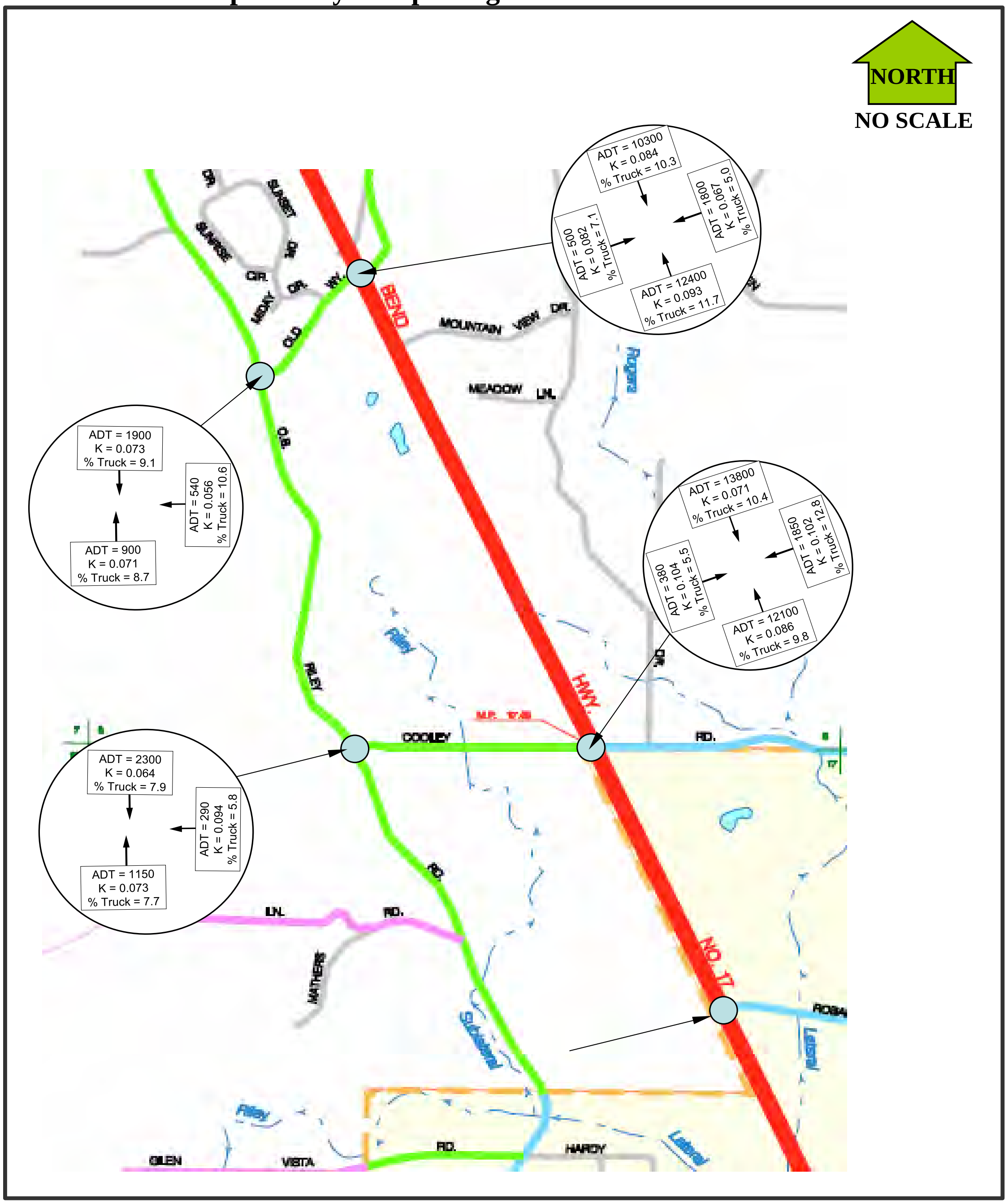
Balanced Truck Volumes and Percentages



5.9.5 Analysis Input Documentation

It is recommended that the truck percentages for each approach be put on a figure for future reference and for ease of entering into software packages. On the same figure, approach PHF's, and K-factors should also be added. This will streamline the entering of data plus make it an easy reference for the preliminary signal warrant process (which uses K-factors). See Exhibit 5-27 for a sample figure.

Exhibit 5-107 Sample Analysis Input Figure



6 FUTURE YEAR FORECASTING

6.1 Purpose

Design Hour Volumes (DHV) are used for ODOT planning and project level analyses. These are based on the existing year volumes developed in Chapter 5. The DHV is generally defined as the future year 30th highest hour (30 HV). Depending on scope and complexity of the analysis, different future methodologies are needed from simple historical trends to complex travel demand models. This chapter will outline the procedures for developing DHV and future Average Daily Traffic (ADT) used for ODOT planning and project level analysis. In addition, the processes for developing pavement design traffic volumes are also discussed. Future design hour volumes are a key input in following analysis steps and methodologies explored in later chapters. For more details on many of the methods in this chapter also refer to NCHRP Report 765.

6.2 General Considerations

The DHV typically controls the design of the project or represents a planning horizon year such as in a Transportation System Plan (TSP). These volumes can either be for no-build or build conditions. If a travel demand model is available for a study area, that is the preferred tool for future forecasting. If the study area is within a metropolitan area, then it is a federal requirement that a travel demand model is used.

6.2.1 Rounding

The DHV's need to be rounded before the network is balanced. The traffic volumes are not that precise to go down to one vehicle, especially considering projections that may be out 20 plus years. Balancing the network is easier if the network is not down to the individual vehicle. Future years five or ten years out should be rounded to the nearest five vehicles. Twenty-year future volumes can either be rounded to the nearest five or ten vehicles. Volumes less than five vehicles should use the "<5" symbol instead of using zero.

6.2.2 Need for Balancing

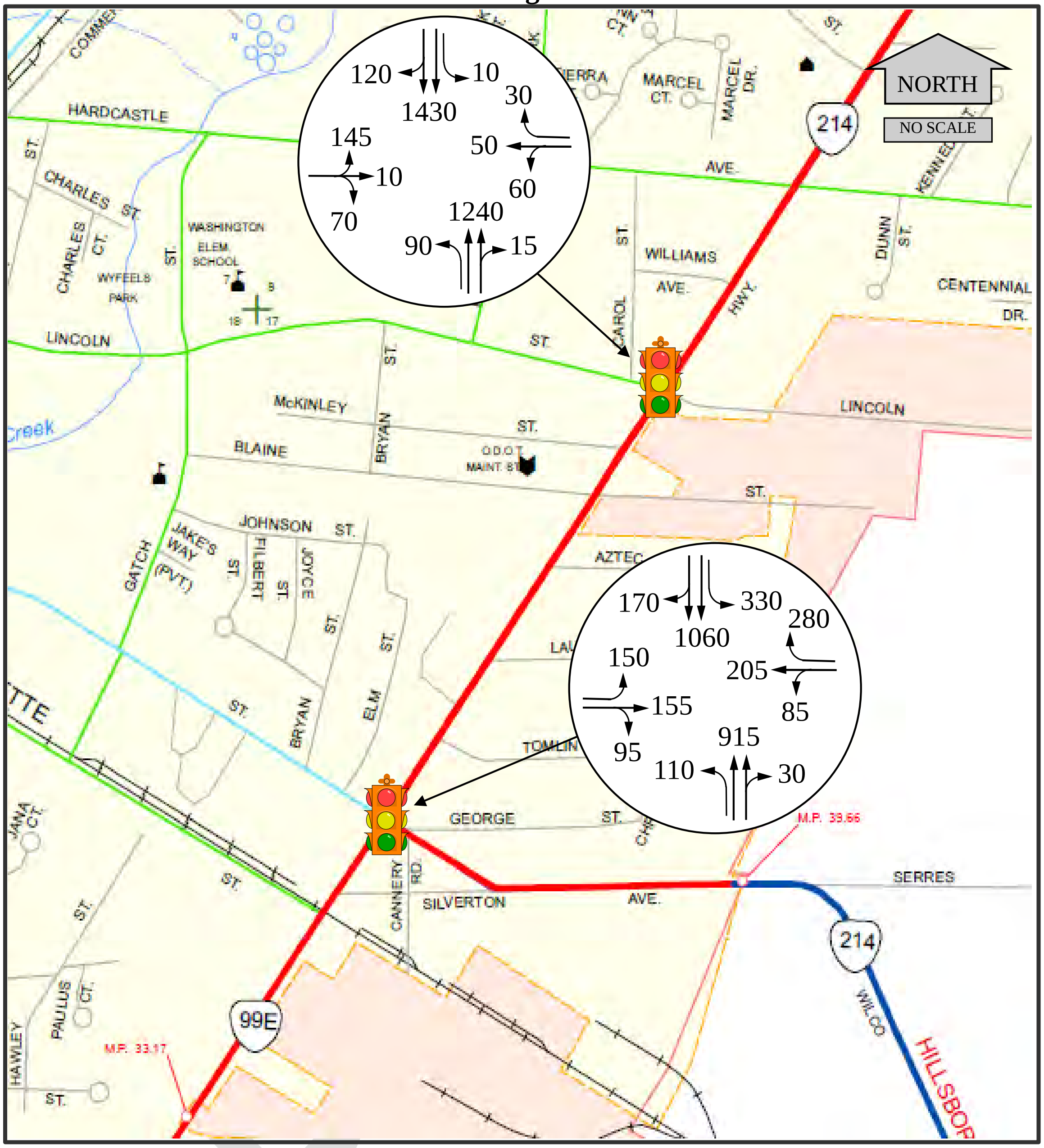
The DHV networks need to be balanced, even more so than with the existing conditions. Small differences in the existing year can become large differences in future years. Future planned changes in the network or land use remove the ability to use relationships between the obtained traffic counts as multiple growth rates are likely in effect. For areas that have travel demand models, balancing is critical to the success of post-processing. Refer to Section 5.3.2 for additional details on the balancing techniques.

6.2.3 Documentation

It is critical that after every step in the DHV process that all the assumptions and factors are carefully documented, preferably on the graphical figures themselves. While the existing year volume development is relatively similar across types of studies, the future year volume development can go in several different directions with varying amounts of documentation needed. Growth factors, trip generation, land use changes are some of the items that need to be documented. If all is documented, then anyone can easily review the work or pick up on it quickly without questioning what the assumptions were. The documentation figures will eventually end up in the final report or in the technical appendix. The volume documentation should include:

- Figures/spreadsheets showing starting volumes (30 HV)
- Figures/spreadsheets showing growth factors, cumulative analysis factors, or travel demand model post-processing.
- Figures/spreadsheets showing unbalanced DHV
- Figure(s) showing balanced future year DHV. See Exhibit 6-1
- Notes on how future volumes were developed:
    - If historical trends were used, cite the source.
    - If the cumulative method was used, include a land use map, information that documents trip generation, distribution, assignment, in-process trips, and through movement (or background) growth.
    - If a travel demand model was used, post-processing methods should be specified, model scenario assumptions described, and the base and future year model runs should be attached.

Exhibit 6-1 Balanced Future Year DHV Figure



6.3 Determining the Future Year(s)

The analyst should work with the region project leader/planner ideally during the scoping phase to determine the future year before beginning any future year forecasting. The future year determination is typically documented in a methodology and assumptions memorandum in addition to the overall scope of work/work plan.

The design hour that is used for many projects is 20 years after the year of project opening. It can be a considerable period between when the traffic analysis is completed and when the project is completed. Environmental documents (EA/EIS), Final design and approvals, permits, environmental clearances, right-of-way purchases, and funding availability/STIP programming, may add anywhere from two to five years before a project starts the construction phase. Depending on the project complexity, construction may take anywhere from one to multiple years with phases.

In planning, a 20-year horizon is typically used when evaluating transportation needs and solutions. Future horizon years should be, at a minimum, 20 years after the estimated plan completion/adoption year. This is typically two years after the plan starts. For refinement and other similar plans, the horizon year should be 25 to 30 years out, which would increase the life of the plan, especially if the project development process does not directly follow. Existing policies may determine the planning horizon such as the Transportation Planning Rule. Transportation Impact Analysis (or Study) (TIA/TIS) horizon year procedures are provided in the Development Review Guidelines.

Many times, additional future years are necessary beyond just the typical 20-year future. In projects, the year of opening (build year) plus interim future years may be needed to support project phases or environmental air/noise/energy analyses. For example, if a project base year is 2011, its build year could be 2014, a 10-year future year would be 2024, and the project future (design) year would be 2034. This would mean three separate future years to be developed by the analyst.

For TSP's, build years are not typically done, but there may be interim years to support Urban Growth Boundary (UGB) expansions or to maintain consistency with adopted plans. For example, a TSP refinement plan with an existing conditions year of 2011 may have a 2025 year to stay consistent with the earlier TSP but would still have a 2033 planning horizon year for a 20-year life beyond adoption.

Refinement plans that will be directly supporting later project development efforts will typically have the year of opening (build year) and future years. Planning efforts with three or more years between the plan and the start of a following project generally do not need build year analyses as the traffic analysis will likely need to be redone anyway.

Care should be taken to not extend the horizon year beyond the normal accuracy level. The limits of detailed analysis go out to about 30 years; anything beyond this is an estimation/approximation. If a travel demand model is used, five years is the limit of extrapolation beyond the model future year. If a project/plan requires more than five years, then the model needs to be updated, which typically involves creating a new future year (reference year) for the model. Contact TPAU or modeling staff in Metro for the Portland metropolitan area, Mid- Willamette Valley Council of Governments (MWVCOG) for the Salem-Keizer metropolitan area, and the Lane Council of Governments (LCOG) for the Eugene-Springfield metropolitan area immediately if a model update seems necessary. In some cases, if the travel demand model area is slow growing, the extrapolation limit can be longer.

To support planning for a future in which some motor vehicles are no longer human-driven, but instead are connected and automated vehicles (CAVs), Appendix 6B includes guidance on adjusting the future capacity of freeway segments and facilities, roundabouts, and signalized intersections in planning scenarios where CAVs are assumed to be part of the traffic stream. This guidance is based on methods presented in the HCM, which were developed by a multi-state pooled-fund study led by ODOT.

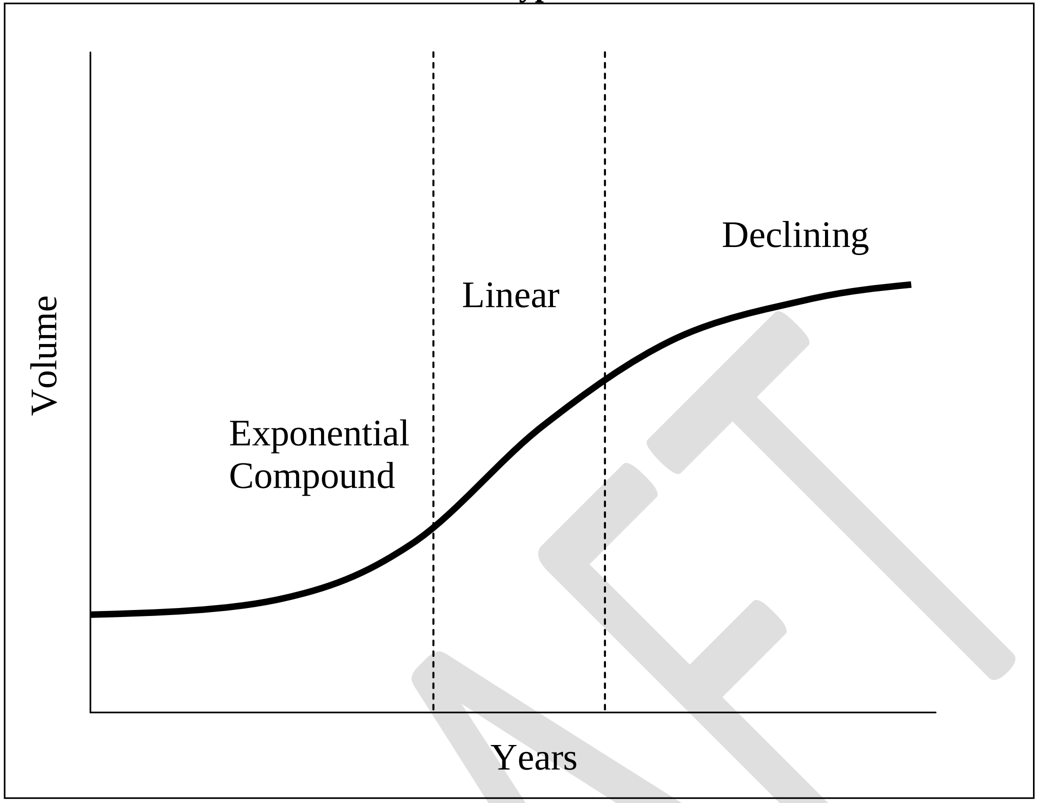
As of 2022, no vehicles were available commercially that met the definition of a CAV for the purposes of an HCM analysis (i.e., a vehicle with an operating cooperative adaptive cruise control system that is capable of communicating with other vehicles and driving without human intervention in any situation). The capacity adjustment process presented in Appendix 6B is intended for use only in longer-range planning analyses.

Because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in near-term analyses such as traffic impact studies.

6.4 Growth Patterns

Different growth pattern types can all be present in a study area. There can be areas of fast growth (i.e. next to an urban fringe interchange), steady growth, or slowing growth (infill). Growth can be negative over the short or long term (i.e. recessions, declining industries, migration, or competition with nearby areas). The analyst must have knowledge of the study area to make the proper future year assumptions. The typical long-term growth curve can be a combination of three conditions on the overall timeline (see Exhibit 6-2). For instance, for the first 5 years the growth is exponential (compound), the next 10 years is linear, but in the last 5 years the growth is declining. Growth curves can also be estimated by using a combination of differently sloped lines (piecewise, such as linear with different growth rates).

Exhibit 6-2 Traffic Volume Growth Types



- **Exponential Growth (Compound) –** An exponential increase in traffic volumes, typically associated with brand new growth in an area that has plenty of land and road capacity. Exponential growth predicts the future volume for a given year based on a percentage of growth from the previous year. This is typically limited to five years or less. Use of an exponential curve over a prolonged period can seriously overestimate future growth.

  Future Volume = Base Year Volume (1 + Growth Rate)$^{\text{Number of Years}}$
  $\text{Volume}_{FY} = \text{Volume}_{BY} \times (1 + Gr)^{(FY-BY)}$

  Where:
  Gr = Geometric growth rate
  FY = Future year
  BY = Base Year

  This method is not generally recommended unless it can be supported by data.

- **Linear Growth** - Linear increase in traffic volumes over time. This method assumes a constant amount of growth in each year and does not consider a capacity restraint. Areas that have or will likely have capacity constraints should use the declining growth curve shown below as long as there is sufficient evidence of a potential change. In many cases a linear growth rate is used since often there is insufficient data to support use of a more specific type of curve.

Future Volume = GF x Base Year Volume, or
$Volume_{FY}$ = GF x $Volume_{BY}$

Where:
GF = Growth Factor = 1 + (G x N)
G = Linear annual growth rate, expressed as a decimal, calculated per Section 6.5
N = Years beyond the base year
FY = Future year
BY = Base Year

- **Declining Growth (Logistic)** - Growth tapers off as land approaches built-out status and capacity of roadways. Future growth is mainly contributed by growth in background (through) traffic.

$$V=\frac{V_0 C}{V_0+(C-V_0)e^{-rt}}$$

Where:
t= time (years)
V= the volume at time t
V_0 = the initial volume at time 0
C= capacity (maximum sustainable volume)
r= rate of volume growth when volume is very small compared to the capacity.

C (capacity) is defined as service flow or saturation flow per lane. For interstates, multilane highways, and two-lane highways, it would be the maximum service flow at level of service (LOS) E. Signalized arterials would start with the ideal saturation flow which is reduced to the actual saturation flow with a few basic parameters as shown in the Highway Capacity Manual (HCM) . Alternatively, the capacity estimator in the Highway Design Manual (HDM) Section 1207 can be used and then use a characteristic K-factor to convert to an hourly value or can leave everything as ADTs (so C is an ADT-based value instead of hourly).

Once a "C" is found, then use the FVT/model to project linearly as a test to see if C is exceeded within the design/planning horizon or close (no more than 30-40 yrs.). If volumes exceed or get close to C then this curve would be used instead of linear. This would be the curve to use for future no-build. For the build, a determination would have to be made of the potential change in C. A significant increase in capacity may result in a linear relationship while a small change may affect it only a little in which the same curve could be used but with a higher C value. Use of this curve may indicate that there is peak spreading effects in play (see Chapter 8).

Estimates for rate (r) are done for the linear portion of the curve, so they can be based from the Future Volume Tables (see Section 6.5) or from a travel demand model. Growth rates have to be large enough over time (t) to be affected by the capacity value so the curve will flatten out. Small growth rates may never reach the capacity level so the curve will remain linear.

Exhibit 6-2 shows the three types of curves, starting off as exponential (compound), transitioning to linear, and then to declining growth.

6.5 Historical Trends

The historical trends method uses traffic volumes from previous years to project future volumes. This method assumes that the future growth trend will be similar to the historical trend. It is used mainly in rural or small urban areas where significant growth is not anticipated. Current and future year traffic volumes are available on the Future Volumes Table webpage. More detail on the Future Volume table structure is in Section 5.5. If desired, different growth curves can be used on historical trend data if the overall trend does not seem to be linear. Most of the time, the differences between alternate growth curves and the linear growth curve are small and not worth the effort to create the trend.

Certain areas may be in long-term decline with negative growth rates. It is not generally appropriate to forecast a long-term negative linear growth rate as many areas could result in a zero or an unrealistically low volume before the horizon year is reached. A conservative method is to assume no growth (base year = future year). It may be warranted to do a detailed investigation of the past trends to see if a different growth curve should be used. TPAU has data on past trends (20+ years) on the state highway system and can help with these types of issues.

Recessions/economic downturns can cause short-term dips in growth trends. It is possible to have a short-term low, none, or negative growth while still maintaining a long-term positive growth. Short- term recessions have little effect on a long-term analysis. Longer term recessions or slow growth periods need to be reviewed to make sure that the analysis is not starting from the low point (could underestimate volumes) or is projecting off the high point (could overestimate future volumes). These kinds of situations typically require a sensitivity or "bookend" type of analysis where both conditions are analyzed. While the terms, "growth rates" and "growth factors" appear to be interchangeable, they are not. Growth rates are decimal percentages versions of the yearly percentage growth. For example, if the growth rate is 2.5% per year, this would be 0.025. Growth factors are used in calculations and may represent one or more years (i.e. 1.025). Adding a "1" will convert a growth rate into a factor. It is important to remember when to convert or not. When converting multiple year growth factors into a single growth rate, make sure to remove the "1" before dividing (See Example 6-1). The basic growth rate and factor equations are shown below:

Growth Rate = (Growth Factor -1) / Number of Years

Growth Factor = 1 + (Growth Rate x Number of Years)

Example 6-1 Future Volumes Using Historic Trend

In this example, the forecast 20-year traffic volumes are developed based on historical counts.

For the Lava Butte ATR (#09-003) located on US 97 at MP 142.41, The following table shows the 1999 traffic volume, Year 2019 traffic volume and the R-squared value.

Example Future Volumes Table

Hwy#	DIR	MP	Description	1999	2019	RSQ
4	1	141.01	.01 miles S of Badger Rd	28400	47200	0.9212
4	1	141.52	.22 miles S of Murphy Rd	24000	41400	0.656
4	1	142.21	ATR 09-003 - Lava Butte	19600	32000	0.9338
4	1	143.47	.01 miles S of Galen Baker Rd	14200	23600	0.7328
4	1	153.09	.01 miles S South Century Dr	9600	11100	0.5788

RSQ = R-squared is an indication of data fitting to a line.

Based on the data above, the 20-year growth factor would be 1.63 (32,000/19,600). Assuming linear growth in the future, the annual growth rate would be (1.63 –1.0) /20 = 0.032, or 3.2%. The R-squared value of 0.9338 is acceptable, indicating a strong relationship. To convert the 1997 30 HV from this example to a 2019 DHV, the 1997 30 HV is multiplied by the 20-year growth factor, with an additional two years of growth added to this.

2019 DHV = 1997 DHV x (20-Year Growth Factor + 2 x Annual Growth Rate) =
112 vph x (1.63 + (2 x 0.032))
= 112 vph x 1.694
= 190 vph

When dividing the estimated future year volume by the most recent count volume it is important to note the numeric difference between the two years. In the example above, a 20-year growth rate was used between 1999 and 2019. Other highways may have been last counted in 1997 or 1998. This would mean that a 21- or 22-year growth rate should be applied. Dividing the total growth by 20 years would, in these cases, overestimate the growth rate.

For areas with calculated multiple growth factors, discard any with an R-squared value less than 0.75. Remaining growth factors that are within 1 or 2 percent can be averaged.

6.6 Trip Generation (For Traffic Impact Analysis & Zonal Cumulative)

Vehicle trips generated by future development are estimated using the Institute of Transportation Engineers (ITE) Trip Generation Manual (if manual trip calculations are used) or a travel demand model for larger studies (see Section 6.10, Chapter 17, and the [Modeling Procedures Manual for Land Use Changes (MPMLUC) - February 2012](). ITE trip generation rates are based on a database of trip generation studies conducted in the U.S. These studies collected data at existing land use generators including driveway vehicle counts and land use characteristics such as floor area size, number of parking spaces, etc. For each land use type, models were developed from the counts and the characteristics of the land uses, resulting in trip rates or equations for various time periods. ITE trip generation should not be confused with travel demand model trip generation, which is based on household surveys of person trips rather than vehicle counts. The following is a summary of ITE trip generation manual procedures. The manual itself should be consulted as part of any trip generation estimation process.

Trip generation rates are typically average values. There can be a large difference between the local trip generation potential and the national average values in the ITE manual. For common uses, such as single-family homes, gas stations, and shopping centers there have been plenty of studies done over the years that make up the research that is the basis for the trip generation rate or equation. Many land uses such as some commercial types have only a few studies, so the data should be used with caution. It is not uncommon for the standard deviation to exceed the averages if only a few studies exist. The ITE Trip Generation manual has guidance and cautions on the data limitations, what the assumptions/definitions for each land use are, procedures for use and conducting local trip generation studies if necessary.

Note that trip generation software such as Trip Generation by Trafficware is not standalone as it does not have the necessary background information on each land use like the manual does. The software is just a calculator tool that needs to be used in conjunction with the ITE Trip Generation Manual.

Trip generation may be composed of three basic types of trips: new/primary trips, pass-by trips, and diverted linked trips. New or primary trips are those trips that are new to the study area specifically attracted to or produced by a land use. Pass-by trips already exist along the roadway fronting the site but now access the site before continuing. Diverted linked trips exist in the study area but must divert onto local roads to access the site. Each of these types of trips is illustrated in the following Examples.

External versus Internal Trips

ITE Trip Generation rates are based primarily on single-use, free-standing sites. There are some exceptions such as shopping centers. Multi-use or mixed-use developments may have interactions such as where some trips may occur internal to the site, whether by motor vehicle or by walking. The total external trip generation of these sites may be less than the sum of the individual stand-alone trip rates calculated using the ITE trip generation rates. ITE Trip generation Manual provides a methodology to estimate this effect, called internal capture (Volume 1, Trip Generation Handbook, Chapter 7 Multi-

Use Development). This is a percentage reduction applied to the sum of the individual land use trip generation.

Other Considerations

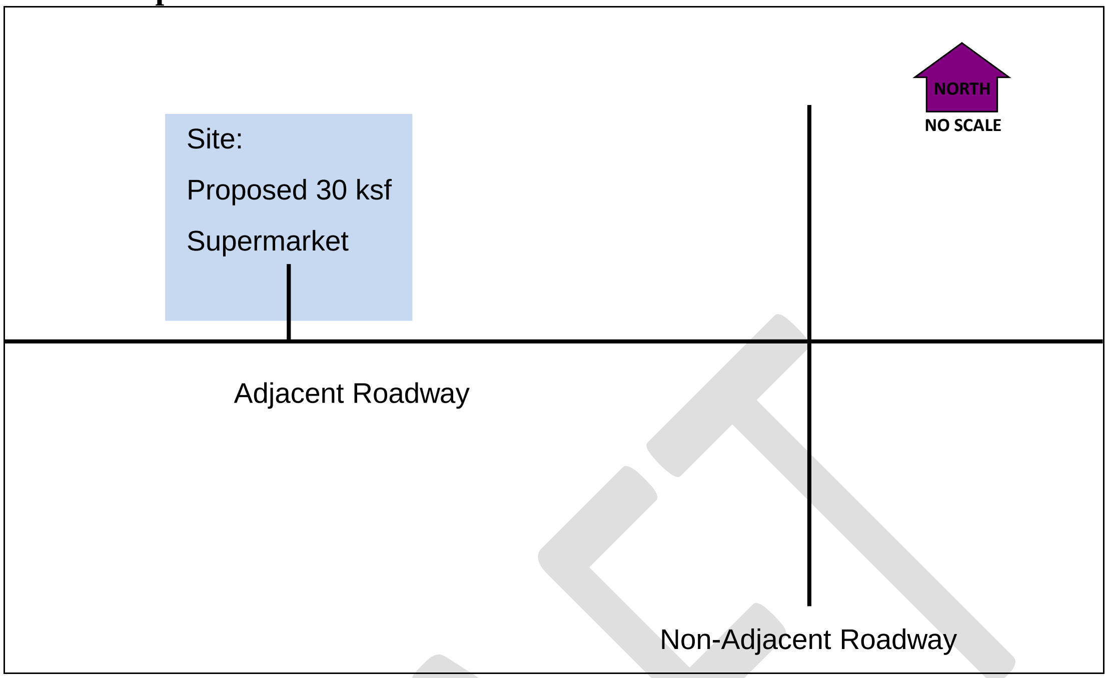
The ITE trip rates are based on data collected primarily at suburban locations with little or no transit service, nearby pedestrian amenities, or travel demand programs. The ITE Trip Generation Manual suggests the user may modify trip generation rates to reflect the presence of public transportation, Transportation Demand Management (TDM) measures, enhanced pedestrian and bicycle trip making opportunities, or other special characteristics of the site or study area. The Transportation Planning Rule (TPR) contains such a provision. OAR 660-012-0060 (6)(a) states that a ten percent reduction in ITE trip generation rates (that do not specifically account for mixed use effects) shall apply for proposed land use plan amendments which consist of mixed-use, pedestrian-friendly development and **prohibit** uses which rely solely on auto trips, such as gas stations, car washes, storage facilities, and motels.

ITE encourages users to supplement trip generation analysis with local data where practical. If local studies are conducted, they should follow the ITE Trip Generation Manual guidelines for conducting a trip generation study.

Example 6-2 Trip Generation

A supermarket with floor area of 30,000 square feet is proposed as a new development within a suburban area as shown below. The proposed use is adjacent to an east-west arterial and located to the west of a north-south arterial. It is desired to estimate the trip generation for the site.

Site Development Location



From the ITE Trip Generation Manual, 9th Edition, Volume 3, a supermarket is assigned Land Use Category 850. This is only a single land use so neither the internal trip capture reductions apply nor does the TPR mixed use reduction. The trip rates or equations, as appropriate, for this land use type are obtained from the manual and shown below. The manual provides trip generation data for certain days of the week such as the average weekday, Saturday and Sunday, as well as for specific hours of the day, such as the peak hour of the adjacent street traffic, or the peak hour of the generator. In this example the peak hour of adjacent street traffic is used. Average percentages of trips entering and exiting the site are also provided. Calculations are performed either in a spreadsheet or using commercial software. The resulting trips are typically summarized in a table as shown below. These represent all motor vehicle trips with origin or destination external to the site.

Site Trip Generation Summary

	ITE Land Use[1]	**Trip Rate or Equation[2]**	**Enter**	**Exit**	**Total**
AM[3]	850	3.40X	63 (62%)	39 (38%)	102
PM[3]	850	Ln(T)=0.74Ln(X) +3.25	163 (51%)	157 (49%)	320
Average Weekday	850	102.24X	1,534 (50%)	1,534 (50%)	3,067

[1]ITE Trip Generation 9th edition
[2]X = independent variable (thousand square feet of floor area) = 30.0 ksf
[3] Peak hour of adjacent street traffic

For commercial/retail types of land uses, peak hour site trips need to be further subdivided into new (primary), pass-by, and diverted linked trips. The ITE Trip Generation Handbook may have estimates of those percentages. These ITE estimates vary by land use size and other characteristics, so the chosen estimates should be close to the size of the proposed land use. Ideally, there will be more than one applicable estimate which can be averaged together. If not, other studies may be used or conducted, along with engineering judgment.

In this example, the ITE Trip Generation Handbook lists studies of average PM peak hour pass-by and diverted linked trip generation rates for supermarkets. Average rates are taken from data points where both pass-by and diverted linked rates are provided. Select those locations with size near that of the subject site. There are five study locations near 30,000 square feet in size. Of those, four locations provide both pass-by and diverted linked trip percentages. The average of these four sites is used. The average pass-by percentage is (32 + 44 + 19 + 28)/4 = 31%. The average diverted linked trip percentage is (20 + 27 + 45 + 32)/4 = 31%. The average primary trip percentage is (48 + 29 + 36 + 40)/4 = 38%. The number of PM peak hour trips by type are then calculated and displayed in a table as follows.

Site PM Peak Hour New, Pass-By, and Diverted Linked Trips[1]

	Percentage	Enter	Exit	Total
New	38%	62	60	122
Pass-by	31%	51	49	100
Diverted Linked	31%	51	49	100
TOTAL	100%	164	158	322

[1]ITE Trip Generation Handbook and Manual, 9th Edition

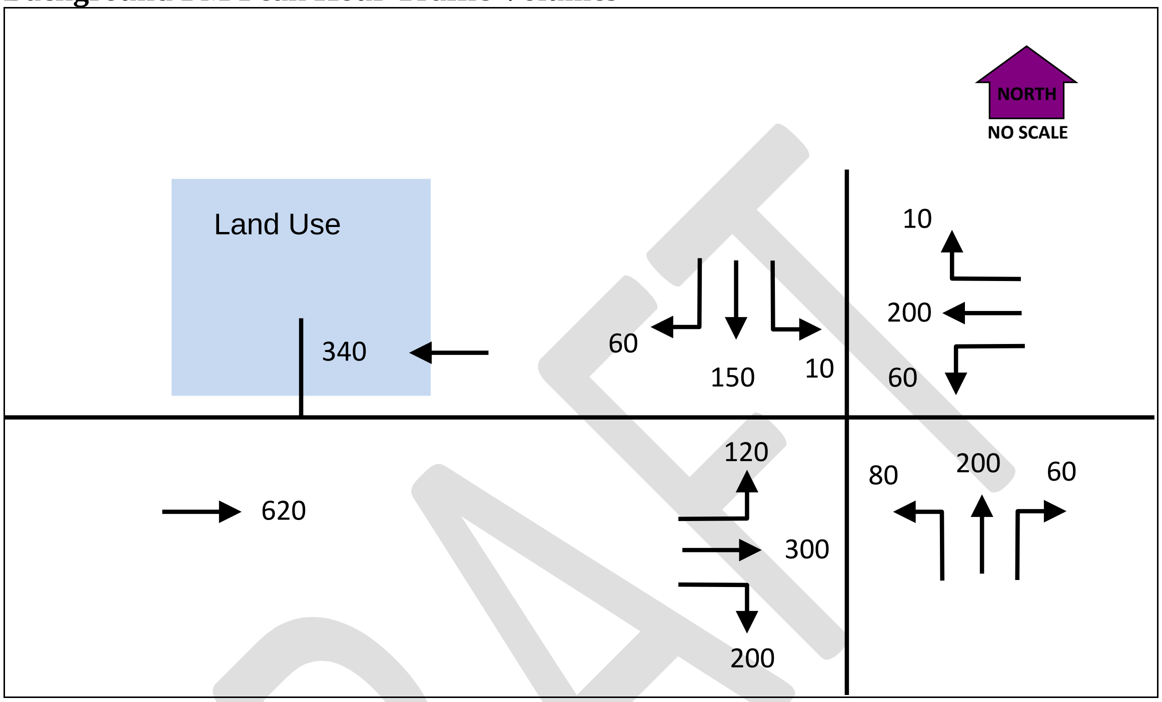
6.7 Trip Distribution (For Traffic Impact Analysis & Zonal Cumulative)

Traffic generated by a future development is distributed from the site based on existing origin-destination (O-D) study data if available, traffic count patterns for nearby similar land uses, local knowledge, or use of a travel demand model for larger studies (see Section 6.10, Chapter 17, and the Modeling Procedures Manual for Land Use Changes (MPMLUC) - February 2012), together with engineering judgment. Use of a travel demand model typically involves creation of the land use in a model scenario then having a select-zone analysis performed to determine in/out percentages from the site.

Example 6-3 Trip Distribution

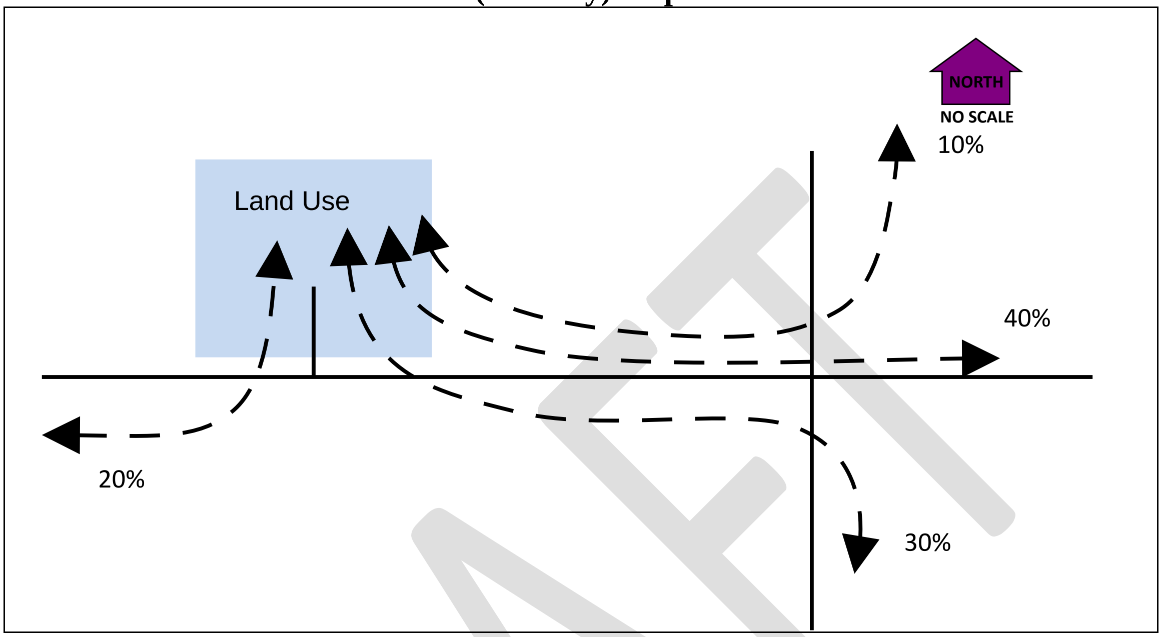
The existing PM peak hour volumes in the study area are shown below. All of the site trip adjustments will be added or subtracted from these values.

Background PM Peak Hour Traffic Volumes



Continuing from the previous example, the distribution of new trips to/from the proposed supermarket has been determined based on existing count patterns and engineering judgment and is shown in the diagram below.

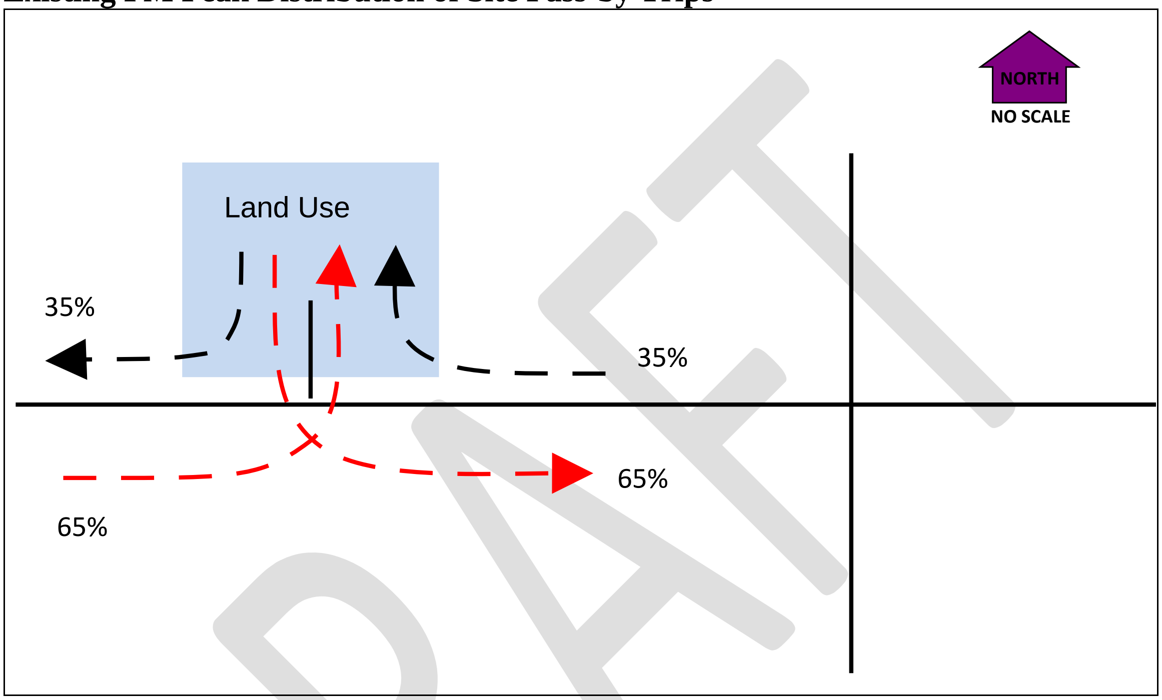
PM Peak Site Distribution of New (Primary) Trips



The distribution of pass-by and diverted linked trips should also be determined if present. Typically pass-by and diverted linked trips are based on traffic counts or travel demand models. In this example, the pass-by trip percentages are assumed to be proportional to the existing east-west PM peak hour directional volume of traffic on the adjacent roadway, as shown below.

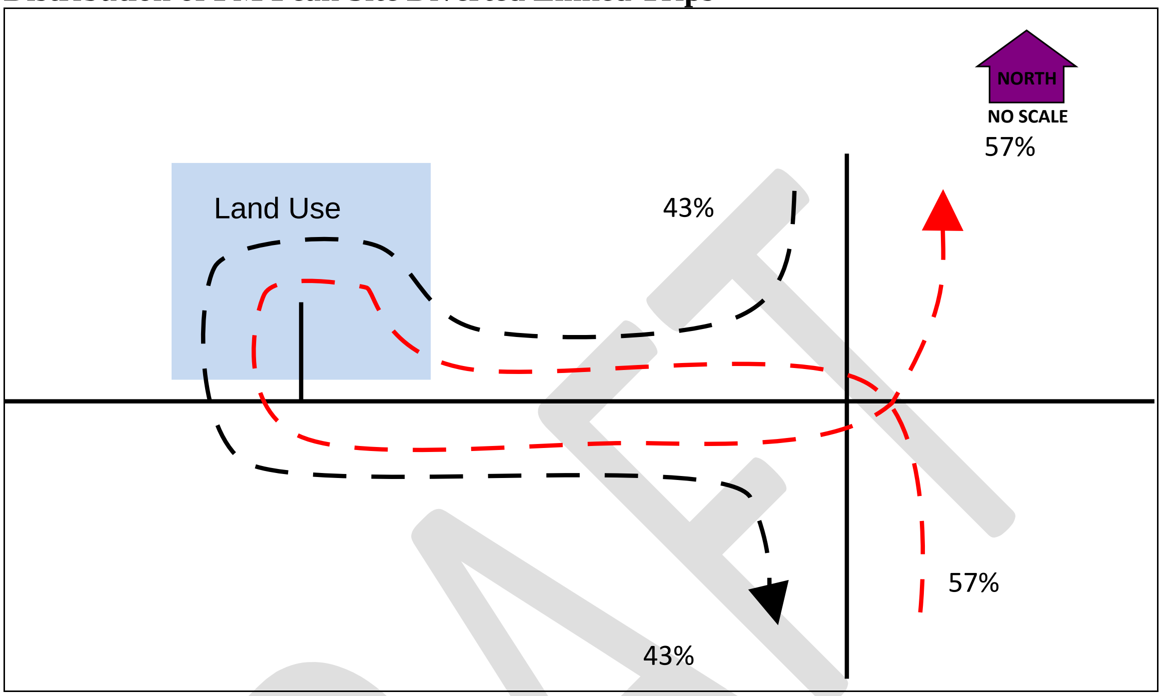
Adjacent roadway volume both directions = 340 + 620 = 960 vph.
Westbound proportion = 340 / 960 = 0.35
Eastbound proportion = 620 / 960 = 0.65

Existing PM Peak Distribution of Site Pass-by Trips



The diverted linked percentages in this example are assumed to be proportional to the north-south PM peak hour directional volume on the non-adjacent roadway, as shown in the next diagram.

Distribution of PM Peak Site Diverted Linked Trips



6.8 Trip Assignment (For Traffic Impact Analysis & Zonal Cumulative)

Traffic distribution to and from a future development is assigned to specific roadways either manually or using a travel demand model for larger studies (see Section 6.10, Chapter 17, and the Modeling Procedures Manual for Land Use Changes (MPMLUC) - February 2012). To create the trip assignment, apply the percentage distribution to the trip generation for the assignment of new, pass-by and diverted linked site trips. Summing up the background, new, pass-by and diverted linked trips results in the final total trip assignment.

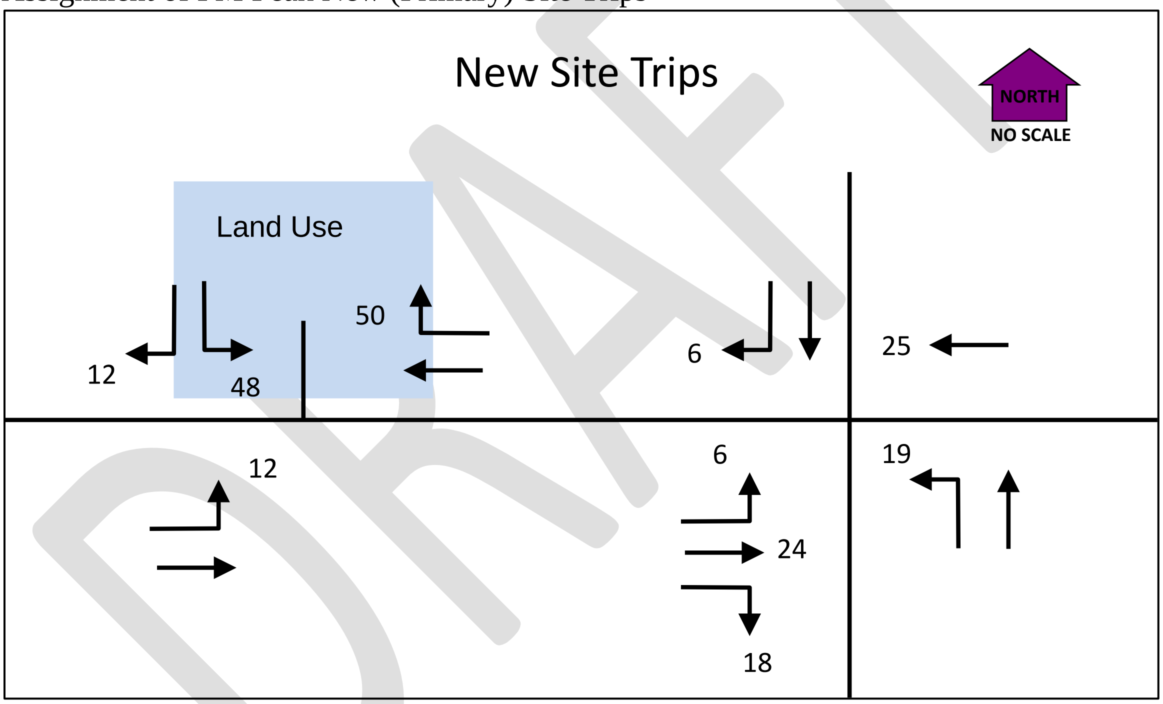
The assignment of each of the components of site trips (primary, pass-by, and diverted linked) should be calculated and displayed on separate flow diagrams. Calculations are typically performed in a spreadsheet or with a specific software application such as Vistro.

Example 6-4 Trip Assignment

The assignment of new (primary) trips for each turning movement is calculated by multiplying the previously determined number of directional new (primary) trips (trip generation) by the new (primary) distribution percentage applicable to that movement.

Sample calculation for eastbound to northbound left turn movement at the nearby intersection:
This movement is outbound from the site.
Trip generation of outbound new (primary) trips = 60
Trip distribution of outbound new (primary) trips = 10 percent
Trip assignment = 60 x 0.10 = 6 PM peak hour new (primary) site trips

Assignment of PM Peak New (Primary) Site Trips



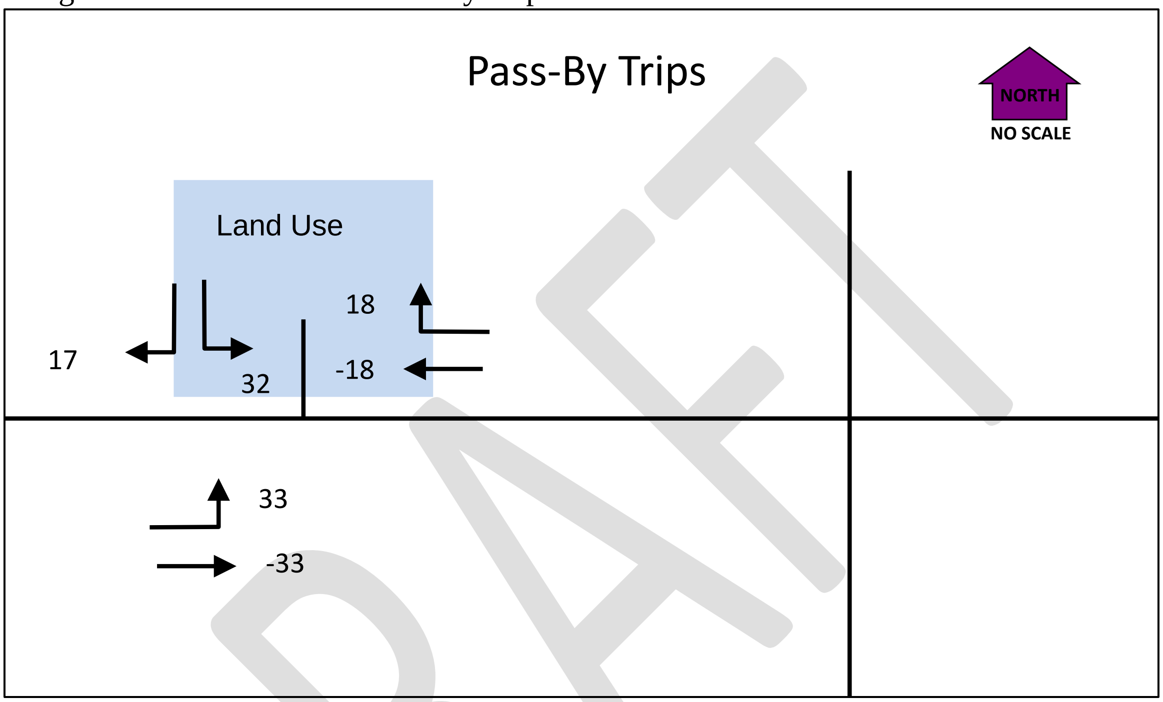
The assignment of pass-by trips for each turning movement is calculated by multiplying the previously determined number of directional pass-by trips (trip generation) by the pass-by distribution percentage applicable to that movement.

Sample calculation for westbound right turn movement from the adjacent roadway onto the site driveway:
This movement is inbound to the site.
Trip generation of inbound pass-by trips = 51
Trip distribution of inbound pass-by trips = 35 percent
Trip assignment = 51 x 0.35 = 18 PM peak hour pass-by site trips

Note that at the intersection of the adjacent roadway with the site driveway, through movement pass-by trips will have negative values, since they turn into the site instead of traveling through.

Also note that beyond the site driveway intersection, pass-by trips are zero. Pass-by trips do not add new trips to the system; they only change turning movement volumes at the site driveway(s).

Assignment of PM Peak Site Pass-by Trips

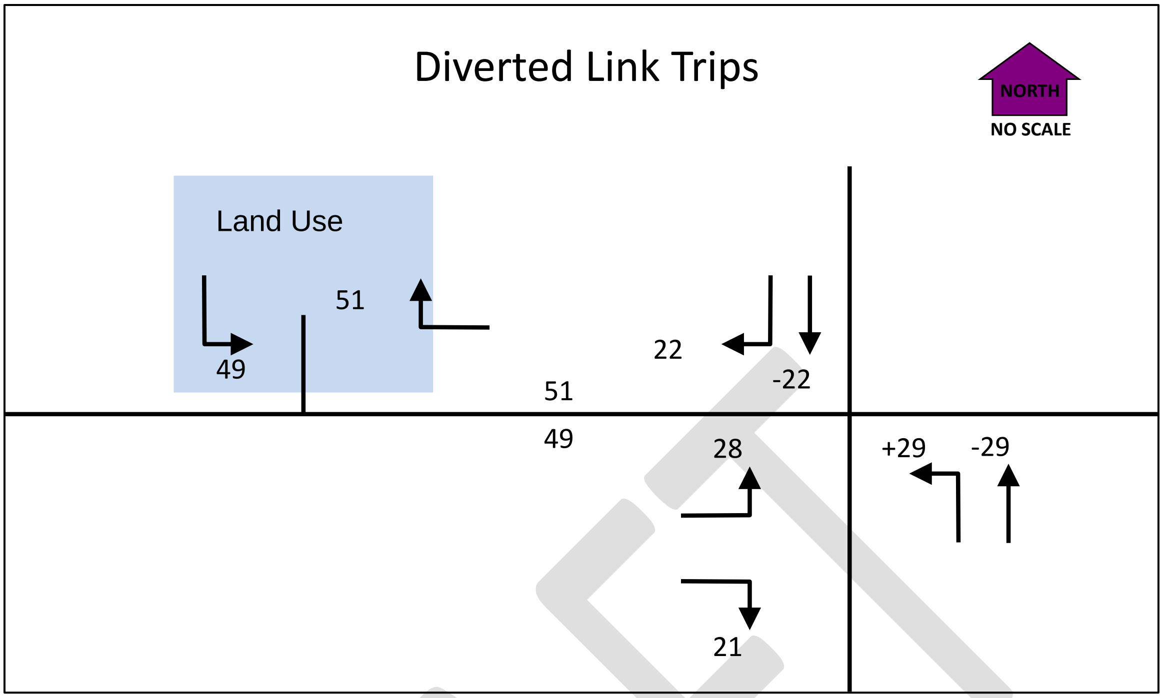


The assignment of diverted linked trips for each turning movement is calculated by multiplying the previously determined number of directional diverted linked trips (trip generation) by the diverted linked distribution percentage applicable to that movement.

Sample calculation for northbound to westbound left turn movement from the non-adjacent roadway to the adjacent roadway:
This movement is inbound to the site.
Trip generation of inbound diverted linked trips = 51
Trip distribution of northbound diverted linked trips = 57 percent
Trip assignment = 51 x 0.57 = 29 PM peak hour diverted linked site trips

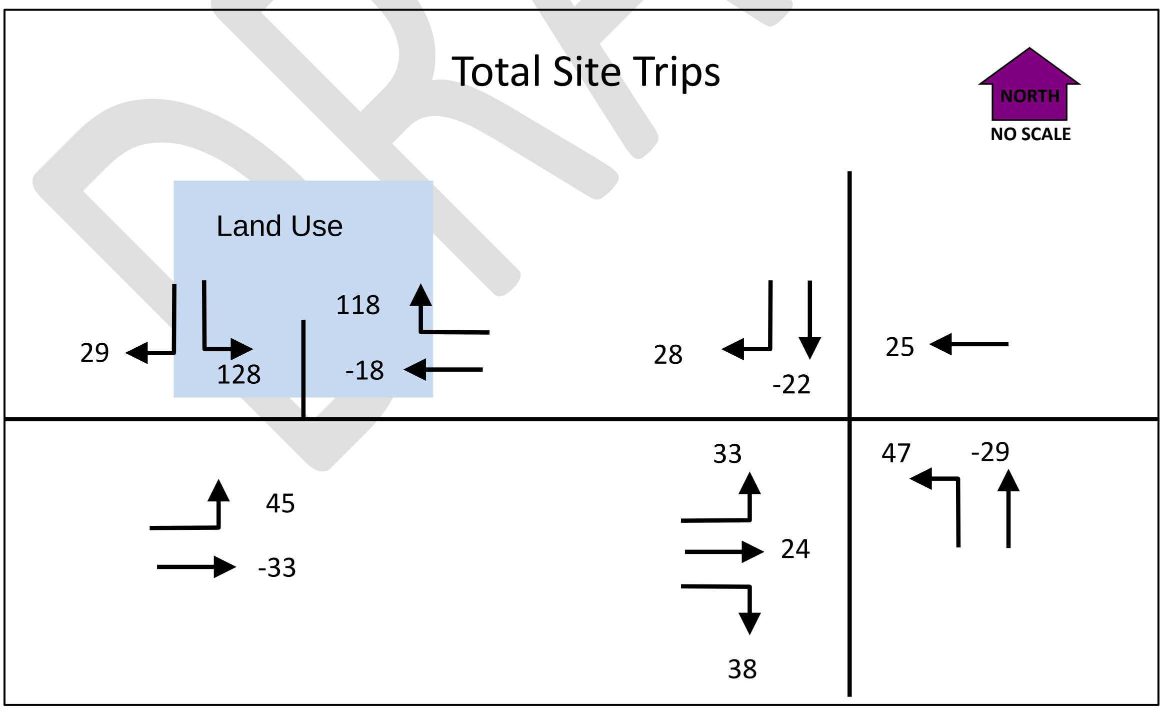
Note that at the intersection of the non-adjacent roadway with the adjacent roadway, northbound and southbound through movement diverted linked trips will have negative values, since they turn onto the adjacent roadway instead of traveling through.

Assignment of PM Peak Diverted Linked Trips



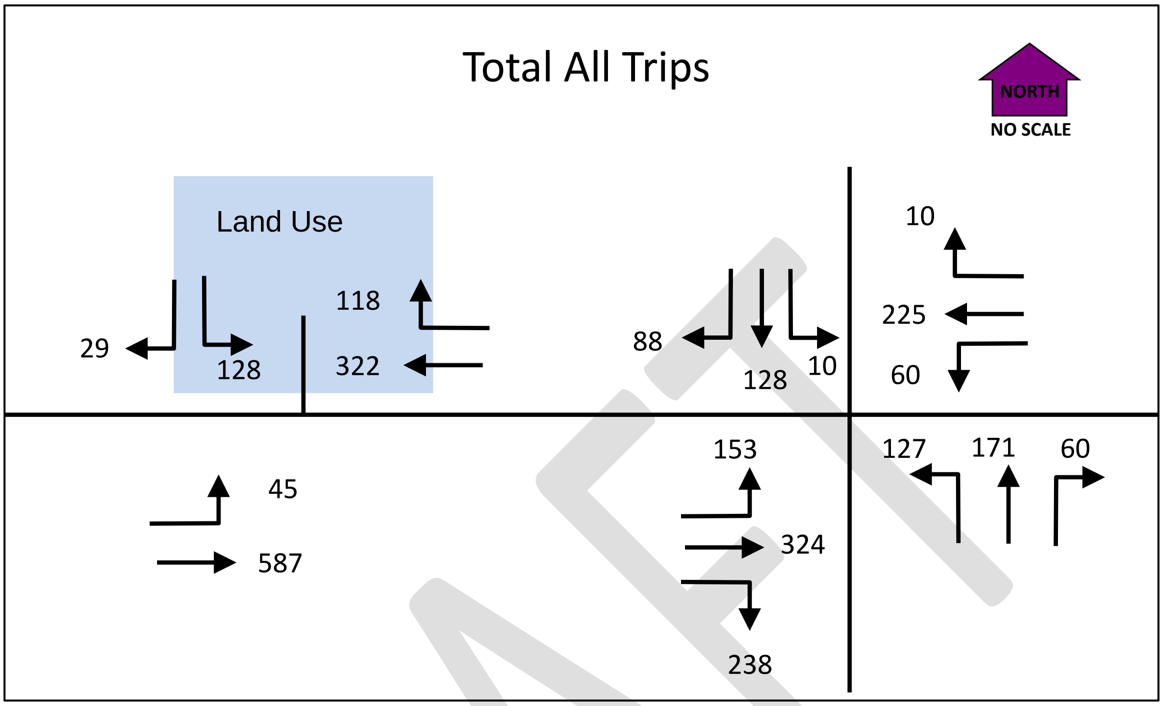
Each of the components of site trips are then summed for each vehicle movement and displayed in a flow diagram of total site trips as shown below.

PM Peak Total Site Trip Assignment



The total site trips are then combined with Background Traffic to create Background plus Site trips as shown below.

PM Peak Total Trips (Background + Site)



6.9 Travel time-based Trip Assignment (Optional; For any cumulative analysis)

Traffic distribution to and from a future development is assigned to specific roadways either manually or using a travel demand model for larger studies (see Section 6.10, Chapter 17, and the [Modeling Procedures Manual for Land Use Changes (MPMLUC) - February 2012]).

Future development trip assignment is often made based on the shortest path distance or travel time. Shortest path travel time in some cases may be approximated using segment lengths and posted speeds. However, this method does not account for intersection delays, which can be a significant component of total travel time in some situations. The following methodology identifies path travel times inclusive of intersection delay.

In this method, for a specified origin and destination, competing O-D paths are evaluated iteratively by assigning trips, calculating resulting nodal delay values, computing overall travel times, and repeating the process until the competing paths have roughly the same travel time. It should generally be assumed that if travel times are within 20%, there will be some split of the site trip assignment. If the travel time difference exceeds 20%, all the site trips can generally be assigned to the shortest travel time path. If the travel time difference is less than 10%, an equal split of site trips can be assumed. The result is the number of site trips assigned to each route.

Travel time on the paths is the sum of the travel time on links, based on link distance and posted speed, and control delay at intersections, based on deterministic analysis results such as using HCS or Synchro. Note that this method assumes relatively uncongested conditions.

Analysis Steps

1. Develop background traffic volumes.
2. For the initial site assignment, assign 100% of site trips to the shortest distance path.
3. Add the site trip assignment in Step 2 to the background volumes to obtain initial Total Traffic volumes.
4. For each alternative O-D path and direction (inbound or outbound from the site) being studied,
    a. Calculate segment travel times using posted speed and link lengths.
    b. Using initial Total Traffic volumes, apply HCM methodology to calculate delays for the movements along the path that pass through stop-controlled or signalized intersections.
    c. Sum the segment travel times and delays from Steps 2a and 2b to determine the total travel time for each path. Identify the path with the shortest travel time.
    d. If the shortest distance path travel time is significantly shorter than the next competing path, the site trip assignment is complete (100% of site trips are assigned to the shortest distance/travel time path).
    e. If the shortest distance path travel time is close to or greater than the travel time of the next competing path, further iterations are required.
5. Re-assign some proportion of the site trips to the shortest travel time path identified in step 2c. Using this site trip assignment, sum with Background Traffic to develop second iteration Total Traffic volumes.
6. Repeat Steps 2b and 2c using the revised iteration of Total Traffic volumes developed in Step 5b. Incrementally re-assign a proportion of site trips until each path is equal in travel time. The site trip assignment is then complete.

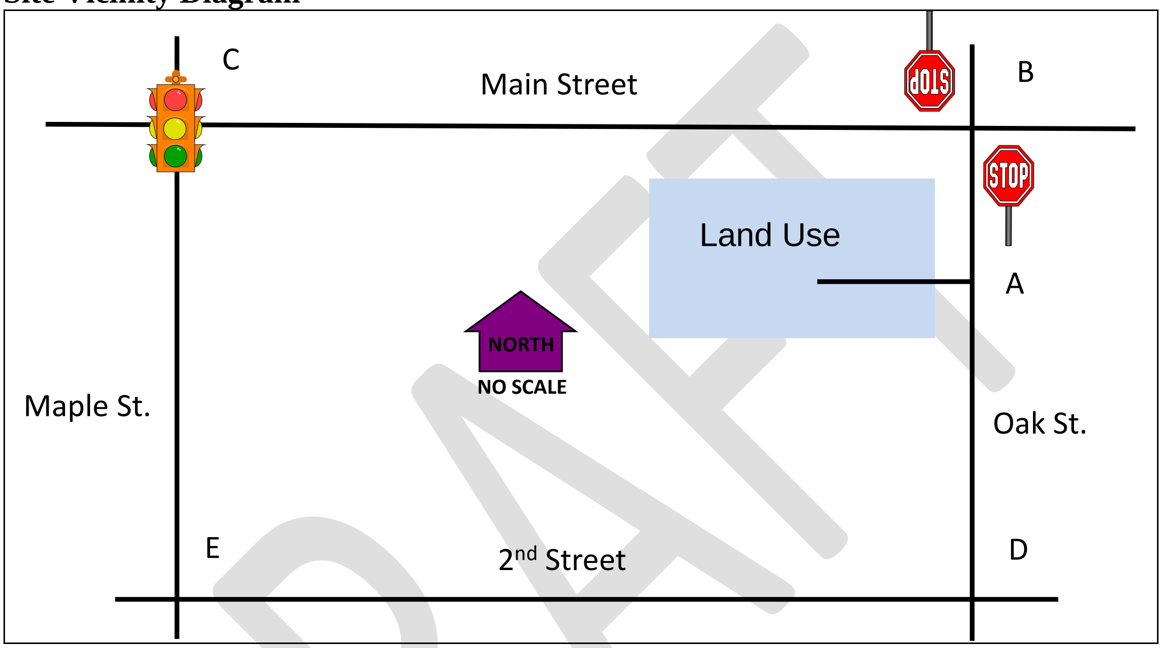
The following example illustrates this method.

Example 6-5 Estimating Trip Assignment Based on Travel Time

A land development is proposed near a state highway. The site is located near the unsignalized intersection of Main Street at Oak Street, as shown below. There are two potential paths for outbound site trips from the development onto the highway in the westbound direction. By inspection, the shortest distance path is ABC. This requires site trips to turn left at the intersection of Main Street at Oak Street, an unsignalized two-way stop controlled intersection. The stop-controlled left turn movement at this intersection experiences high delay. The alternate path, ADEC, is less direct. However, this path has less intersection delay where it accesses the highway at the signalized intersection of Maple Street and Main Street.

It is desired to assign the outbound site PM peak hour trips such that either path will provide an equal travel time from the development.

Site Vicinity Diagram



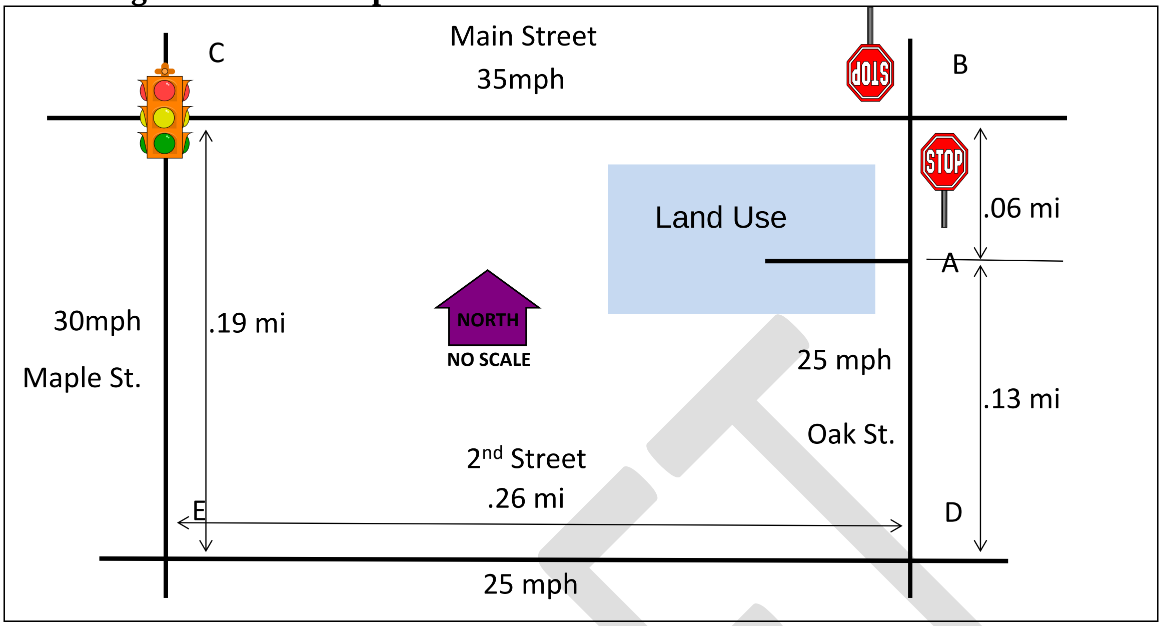
Step 1. For this exercise it is assumed that background traffic volumes have been previously developed.

Step 2. Site trips are initially assigned 100% to the shortest distance path, ABC.

Step 3. The initial site trip assignment from Step 2 is added to Background Traffic from Step 1 to create Initial Total Traffic volumes.

Step 4a. Link lengths and posted speeds are obtained and shown in the next figure below. Link lengths are typically measured from aerial photos and posted speeds collected from a field visit.

Link Lengths and Posted Speeds



Segment travel times are calculated as follows, using sample segment BC.

$$\text{Travel Time Segment BC} = \frac{0.26 \text{ mi}}{35 \text{ mi/hr}} \times \frac{3600 \text{ sec}}{\text{hr}} = 26.7 \sim 27 \text{ sec}$$

Travel times for each segment are summarized in a table:

Roadway	Segment	Segment Travel Time
Oak St	AB	9
Main St	BC	27
Oak St	AD	19
2nd St	DE	37
Maple St	EC	23

Origin-destination (O-D) path segment travel times are calculated by summing the segment travel times.

O-D Path ABC = AB + BC = 9 + 27 = 36 sec
O-D Path ADEC = AD + DE + EC = 19 + 37 + 23 = 79 sec

Step 4b. For this exercise, assume that HCM intersection analysis has been previously conducted for background volumes, resulting in the following intersection delays. Please note that the delay values in this example are for illustrative purposes only and are not actual HCM-calculated values. For this example, it is assumed that delays at other intersections along each path are negligible.

Background Traffic Movement Delay

Intersection	Movement	Movement Delay (s)
Main St at Oak St	South to West left turn	44
Main St at Maple St	Westbound through movement	14
Main St at Maple St	South to West left turn	21

Intersection movement delays are summed for each O-D path

O-D Path ABC = 44 + 14 = 58 sec
O-D Path ADEC = 21 sec

Step 4c. Segment travel times and delays are summed to determine the total travel time for each path.

O-D **Path**	**Segment Travel Time (s)**	**Intersection Delay (s)**	**Total Travel Time (s)**
ABC	36	58	94
ADEC	79	21	100

The O-D path with the shortest travel time is path ABC.

Step 3. Site trips are assigned 100% to the shortest path ABC.

Step 4. The site trip assignment in Step 3 is added to the background volumes to obtain initial Total Traffic volumes.

Step 5. HCM methodology is applied to re-calculate intersection delays based on Total Traffic volumes.

Total Traffic Movement Delay (First Iteration)

Intersection	Movement	Movement Delay (s)
Main St at Oak St	South to West left turn	58
Main St at Maple St	Westbound through movement	16
Main St at Maple St	South to West left turn	24

Intersection movement delays are summed for each O-D path

O-D Path ABC = 58 + 16 = 74 sec
O-D Path ADEC = 24 sec

Segment travel times and delays are summed to determine the total travel time for each path.

O-D **Path**	**Segment Travel Time (s)**	**Intersection Delay (s)**	**Total Travel Time (s)**
ABC	36	74	110
ADEC	79	24	103

At this point, the shortest path based on travel time is now path ADEC. Since path travel times are close, the next iteration would be to assign some of the site trips to path onto path ADEC. Step 5 is then repeated. When path travel times are approximately equal, the site trip assignment is complete.

6.10 Zonal Cumulative Analysis

In quick growing areas or for entire cities, where a more accurate analysis is needed of the impacts (differing growth and trip patterns across the study area), the zonal cumulative analysis process should be used rather than the first-level TIA-level cumulative analysis. The zonal cumulative analysis process is essentially a manually constructed travel demand model. The process includes the major three modeling steps (trip generation, trip distribution, and trip assignment). The major difference between the zonal cumulative and a travel demand model is that the zonal cumulative uses ITE trip generation instead of population and employment, and projects the incremental growth in trips rather than creating separate base year and future year assignments. Generally, this level of effort is too complex for a TIA.

Like with other cumulative analyses, the zonal cumulative method should be limited to cities of 10,000 or less population or a chunk of no more than 10,000 of a larger urban area. Areas larger than these can theoretically be done, but the number of zones and network size becomes overwhelming to do manually. In addition, multiple analyses (TSP, corridor or refinement plans, projects) are likely in the near (5-10 year) future then construction of a travel demand model should be considered (refer to Chapter 17). Re-purposing a zonal analysis for another project can be more time consuming than just building a travel demand model initially. These larger areas would require the enhanced zonal cumulative analysis process shown in Section 6.11 or use of a travel demand model as discussed in Section 6.12 and Chapter 17.

The basic steps for a zonal cumulative analysis are:
1. Identify the study area and divide into transportation analysis zones (TAZ)
2. Identify vacant lands, in-process developments, comprehensive plan allowed land uses/densities, and development rates.
3. Estimate future trip generation potential
4. Determining the through trip percentages (external – external, E-E) and E-E trips for the external stations
5. Determining the internal – external (I-E) and external –internal (E-I) trips at each external station (external zone)

6. Determining the trip distribution for the internal – external (I-E) and external – internal (E-I) trips for each internal TAZ.
7. Determining the trip distribution for internal-internal (I-I) trips
8. Calculating network link travel times
9. Assigning total trips to the network

The above process is set up for multiple traffic assignments based on a single land use scenario. Doing an additional land use scenario would require repeating Steps 3-9 or using the more-automated enhanced zonal cumulative analysis. Use of software such as Vistro may help streamline the accounting of generated trips and the distribution/assignment of them if the network and land uses assumed are not too complex.

6.10.1 Step 1 – Identification of Study Area and TAZ's

The study area should completely cover the project limits and all adjacent land areas that would affect the project area. The study area should be defined such that all relevant facilities are included, since there may be other roadways that could directly influence the traffic patterns on the facilities being analyzed. The location where each relevant roadway facility crosses the outside edge of the study area needs to be noted as an "external station." The external station is where traffic enters/exits the study area and can also be considered an external zone. The study area needs to be broken into homogenous land-use zones (i.e. residential, commercial, or industrial) called transportation analysis zones (TAZ). There generally is no need to break the land use into further sub-categories (i.e. low vs. medium density residential). If zones are not completely homogenous, there will need to be additional accounting of internal trips between residential and commercial for example as there will be trips generated that will never extend beyond the zone boundaries. It is likely that a Geographic Information Systems (GIS) land-use or tax-assessment file is available from the local city or county that will be of great assistance and streamlining of this and the following tasks.

Each TAZ boundary generally needs to follow physical or man-made boundaries. This would include rivers, railroads, and major roadways. Property lines generally break on these major features, and the GIS file can be used to help ensure that the TAZ boundaries do not cross or break up properties.

Example 6-6 Study Area and Zone Identification

This example is based on an actual zonal cumulative analysis done for the US97/South Century Drive project in the Sunriver resort area south of Bend. The overall project area is shown in the figure below. The study area reflects the area that would feed into South Century Drive and eventually to US97. Areas to the south of the project area would most likely feed into the next major road to the south (LaPine State Recreation Road) or into the City of LaPine.

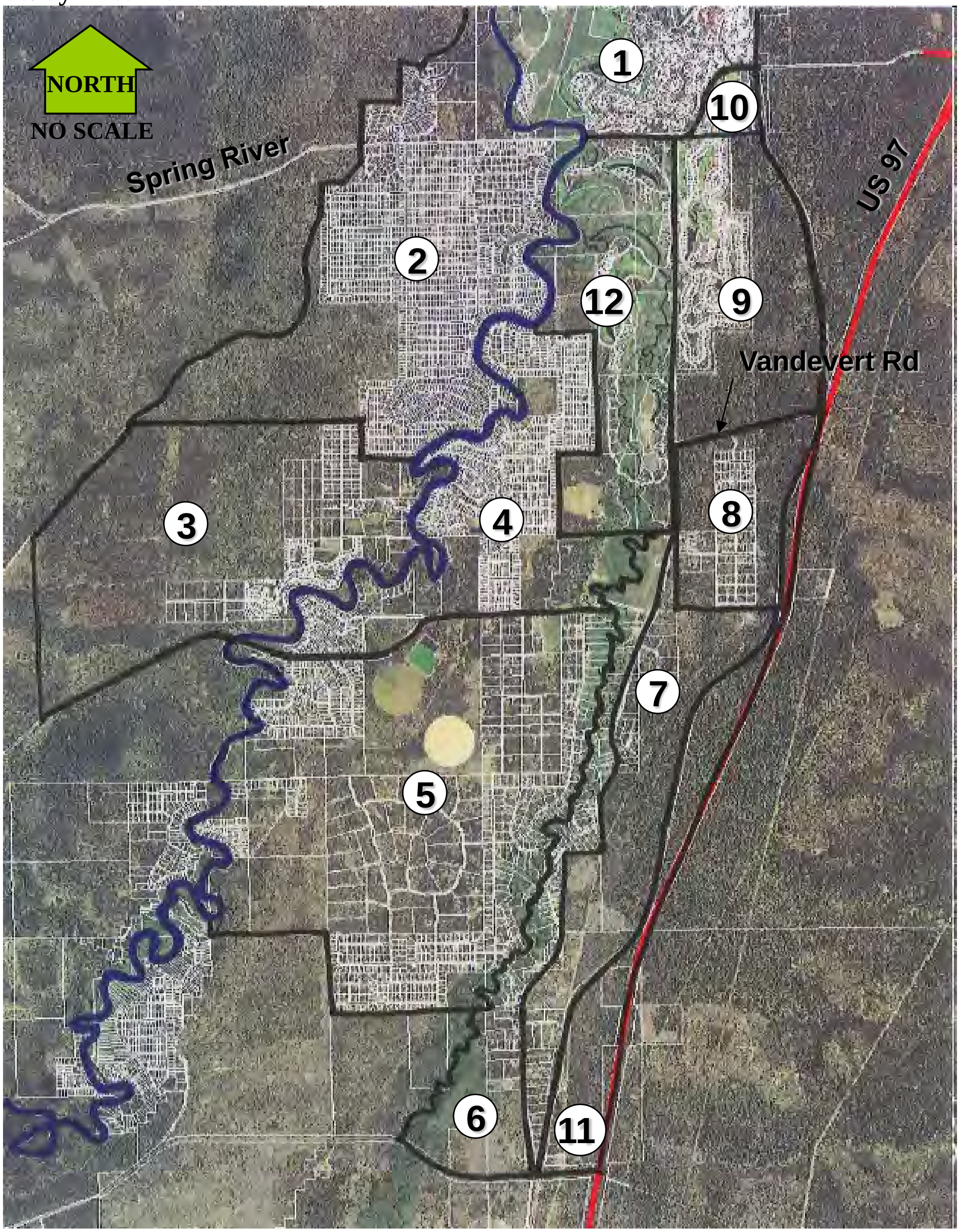
TAZ’s were split on land use or on natural or man-made boundaries. Examples for land use:

- Zone 1 is the current Sunriver resort development and is generally residential.
- Zone 2 is the existing Spring River residential subdivisions
- Zone 9 is vacant Forest Service forest lands (although zoned for resorts).
- Zone 10 is the Sunriver business district and is generally for commercial uses.
- Zone 12 is the Crosswater resort/golf course

TAZ zone boundary splits examples:

- The Deschutes River forms the boundaries between Zones 1 and 2 and Zones 3 and 4.
- The Little Deschutes River forms the boundary between Zones 5 and 6.
- The BNSF railroad tracks form the boundary between Zones 7 and 11.
- South Century Drive forms the boundaries between Zones 1 and 10, Zones 12 and 9, and between Zones 6 and 7.

Study Area Zones



6.10.2 Step 2 – Identification of Land Use Characteristics and Zoning

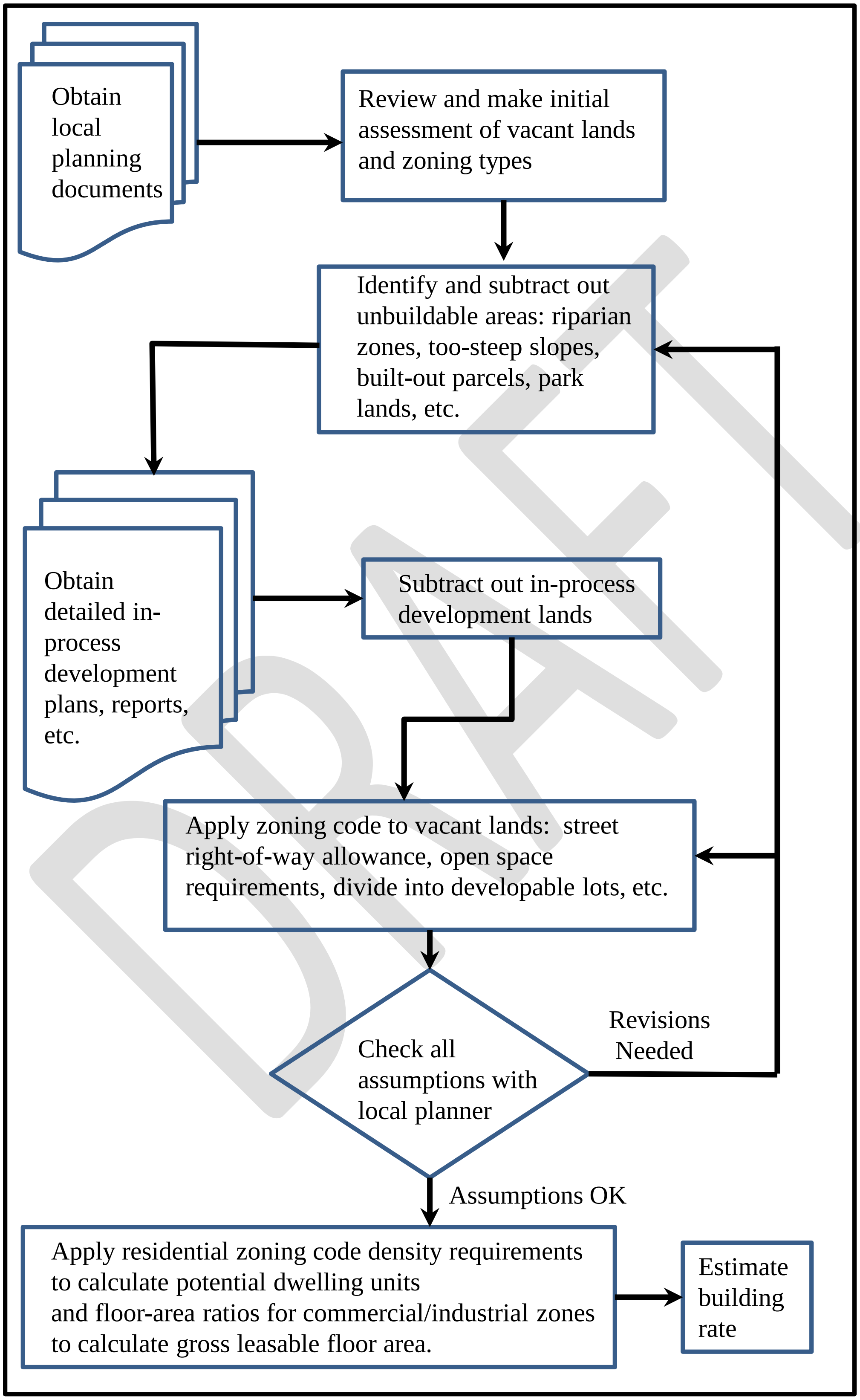
After the homogenous TAZ's have been developed, all the land use characteristics and related zoning needs to be identified. This will create the parameters necessary for estimating the future trip generation in Step 3. The zonal cumulative process assumes that the traffic counts (and the resulting 30 HV) obtained for the study area account for all built properties. The focus of the process is to determine the total change from the base to the future condition. Areas that are vacant or partially built will need additional data to help determine the future growth potential. Exhibit 6-3 shows the process for determining the TAZ land use data for vacant lands.

Vacant Lands

For each TAZ, any vacant buildable lands need to be identified. This can be facilitated by obtaining the following items:

- Buildable Lands Inventory (BLI) – Not every study area will have an applicable BLI available, but when available, they will limit the amount of data needed. The BLI will specifically identify buildable areas that can be coordinated directly with the study area TAZ's. Detail levels vary but at a minimum, the truly developable vacant lands can be determined. The BLI will help identify areas that are not suited for future development such as wetlands, riparian zones, flood plains, steep slopes, resource areas, etc. The BLI needs to be relatively recent (no more than 3-5 years old for normal growth areas but longer periods might be okay for slow growth areas).
- Comprehensive Plan, Zoning Map and Code – Obtaining the comprehensive plan, zoning map and zoning code is the minimum information needed to do a zonal cumulative analysis. The comprehensive plan and related zoning code (see Exhibits 6-4 and 6-5.) will tell what the allowed (permitted outright and through the conditional use permit process) uses are, and lot requirement details such as minimum/maximum sizes, development densities, building heights, floor-area-(coverage) ratios, green/open space requirements, etc. The zoning map allows for easy identification of where the different zoning is applied (see Exhibit 6-6). Generally, land uses should be limited to outright permitted uses only as there is no guarantee that a conditional use would be granted. Exceptions need to have concurrence with the local planning staff.
- GIS-based land-use/property/tax assessor's database – If available, a GIS-based property database will greatly streamline the identification of zoning and whether a property is vacant. These databases generally include location, tax-lot numbers, zoning, acreage, ownership, improvements (structures), and year of improvement (building permit issued). Use of GIS allows the properties to be grouped by TAZ.

Exhibit 6-3 TAZ Land Use Data Process



- Transportation System Plan (TSP) – Newer TSP's may show environmental baseline information such as wetlands, park lands, greenways, historic properties, etc. that can be used to further identify non-buildable areas (especially if a BLI is not available). The TSP will also show future road improvements that can be used to modify the road network.
- Other land-use plans – These cover a wide range of planning maps such as subdivision plats and development master plans that can identify in-process or future developments. Certain areas may not have BLI's so park plans, school district plans, wetland plans, park plans, and other plans of this type fall into this category. All of these can be used for identification of future growth potential. Contacting the local planning office is the best source for this information.
- Current aerial photos – If current aerial photos are available, identification of vacant lands can be easily done with a GIS database or comprehensive plan. However, use of aerials can be a potential pitfall if the aerial has not been verified to be recent. Please note that commercial aerial mapping such as Google Maps could be 3-5 years out of date and is not recommended to be used (unless extensively field-verified) for this process.

Exhibit 6-4 Urban Standard Residential Zone Excerpts

(1) Purpose. The RS Zone is intended to provide for the most common urban residential densities in places where community sewer services are or will be available and to encourage, accommodate, maintain and protect a suitable environment for family living.

(2) Permitted Uses. The following uses are permitted:

(a) Single-family dwelling.

(b) Agriculture, excluding the keeping of livestock.

(c) Rooming and boarding of not more than two persons.

(d) Home occupations subject to the provisions of Subsection (15) of Section 25.

(e) Park rehabilitation, minor betterment and repairs.

(f) Accessory dwelling in a subdivision or Planned Unit Development (PUD) approved after December 2, 1998, provided that overall density in subdivision or PUD does not exceed 7.3 dwelling units per gross acre.

(5) Lot Requirements. The following lot requirements shall be observed, provided that the approval authority may allow smaller lots of different housing types in a new subdivision or Planned Unit Development (PUD) approved pursuant to this ordinance and consistent with the Comprehensive Plan designations for preservation of areas of significant interest when these lots or housing types are internal to the subdivision or PUD.

(a) Lot Area: A lot in a subdivision or planned unit development approved after (date of adoption) shall have a minimum area of 4,000 square feet provided that the overall density does not exceed 7.3 dwellings gross per acre, and provided that where new subdivisions abut lots of 20,000 square feet or less the exterior lots shall be at least 75% of the abutting lot sizes and lot lines or be approved through a hearing process. All other lots shall have a minimum area of 6,000 square feet.

Exhibit 6-5 Mixed Employment Zone Excerpts

(1) Purpose. The zone is designed to provide for a mix of uses such as office, retail, services, light manufacturing and warehousing that offer a variety of employment opportunities in an aesthetic environment and having a minimal impact on surrounding uses.

(2) Permitted Uses. The following uses are permitted in the ME Zone subject to the provisions of Design Review and Site Plan review in Section 23 and Section 23 A.

(a) Automobile and truck repair and service, provided wholly within an enclosed building.

(b) Automobile, truck, and recreation vehicle sales of new vehicles, including service facilities and outdoor storage of vehicles.

(c) Bakery for wholesale and retail distribution.

(d) Banks, savings and loan institutions, credit unions and other financial institutions.

(4) Height Regulations. No building or structure shall be hereafter erected, enlarged or structurally altered to exceed a height of 45 feet, without a Conditional Use Permit approval.

(5) Lot Requirements. The following lot requirements shall be observed:

(a) Lot Area: Each lot shall have a minimum area of 6,000 square feet.

(b) Lot Width: No requirements.

(c) Lot Depth: Each lot shall have a minimum depth of 100 feet.

(d) Front Yard: The front yard setback area shall be a minimum of 10 feet.

(e) Side yard: None except a side yard setback area shall be a minimum of 15 feet when abutting a lot in a residential zone.

(f) Rear Yard: None except a rear yard setback area shall be a minimum of 15 feet when abutting a lot in a residential zone.

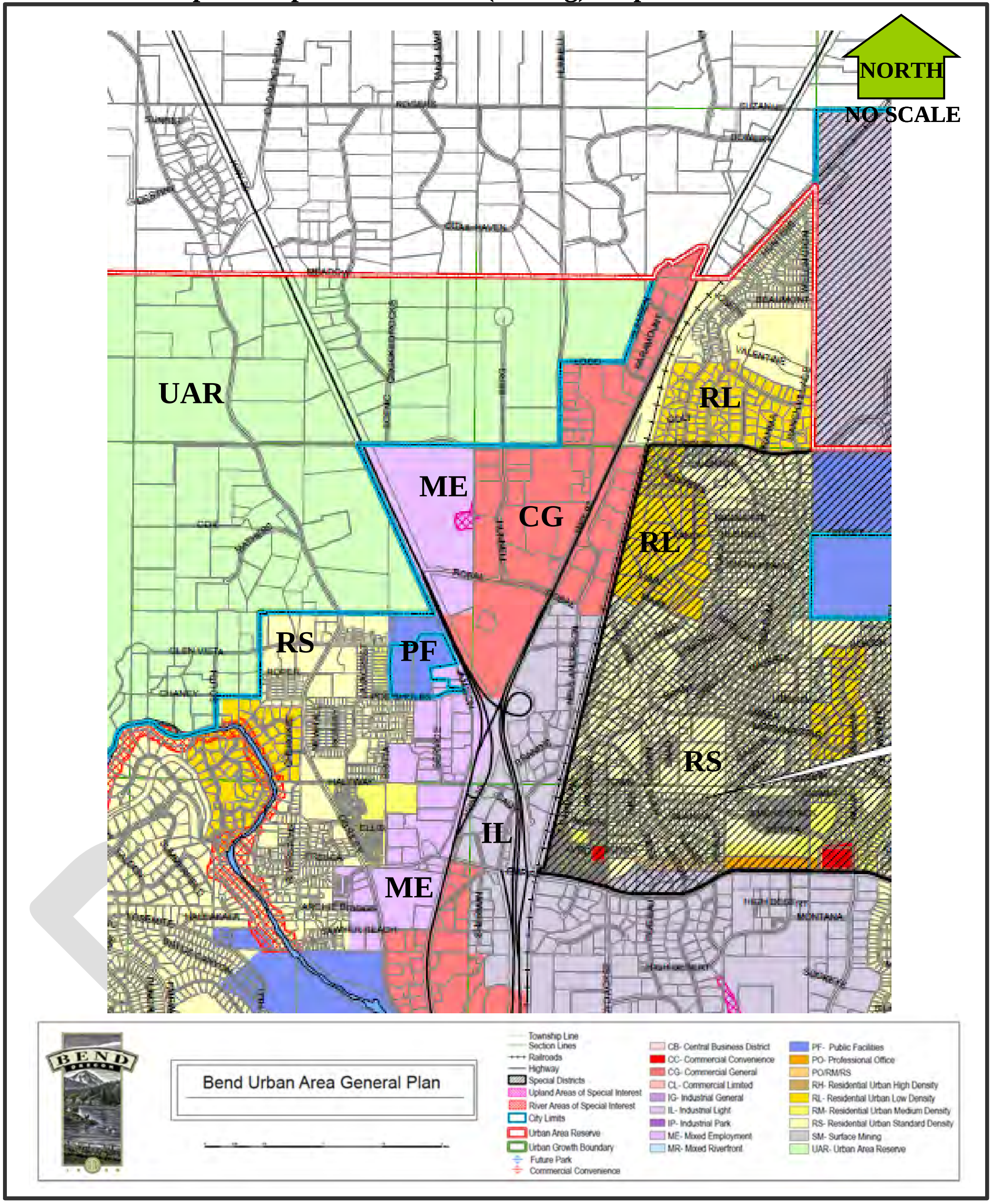
(g) Lot Coverage: The maximum lot coverage by buildings and structures shall be 50 percent of the total lot area.

(6) Off-Street Parking and Loading. Off-street parking and loading space shall be provided as required in Section 24.

The first step to obtaining any of the above is to get in contact and establish a good relationship with the local planning office or local planner. They will know all of the details, recent and past development history, insight on building rates and typical/likely growth patterns. You will need to have them review your development assumptions that you make, vacant lands, and growth potential to make sure that the analysis is reasonable before moving on to more detailed steps.

Once the unbuildable areas have been identified, then any in-process developments need to be identified. Contact the local planning office and ask about any approved but not yet built residential or commercial developments. Obtain documentation such as plats, site plans, or transportation impact analyses (TIA) describing the development. The total number of lots, units, or square footage needs to be noted for the later trip generation calculations. These developments need to be subtracted from the total vacant land to determine the truly vacant buildable land available for future growth.

Exhibit 6-6 Sample Comprehensive Plan (Zoning) Map



Estimating Potential Future Land Use

At this point, the remaining vacant land would be available for future development but is not planned as such. For each TAZ and/or parcel clumps within a TAZ, the zoning code requirements need to be applied.

For residential lands, there will be likely minimum lot sizes or minimum/maximum dwelling unit per acre requirements. A good way to judge what these might be is to take a GIS-based survey of adjacent similarly zoned single-family home subdivisions and figure out a typical lot size (this is one of those assumptions that the local planner should review for reasonability). About 24% of vacant residential lands should be reserved for local street rights-of-way. The zoning code may indicate additional specific values for green or open space requirements that will need to be accounted for. Buildable residential lands then would be calculated by subtracting the local street and green space requirements from the total land area and then dividing by the typical lot size to determine the number of single-family dwelling units for each TAZ.

Multi-family residential lands such as apartments, condominiums or townhouses need to be figured out on a dwelling units per acre basis. The zoning code should tell the range of acceptable densities regardless of building heights. GIS can be used to help determine a typical value by surveying similarly zoned areas in the local area. The local planning office should be able to give some guidance in this area. Once a typical dwelling unit per acre value is determined then this can be directly multiplied by the vacant buildable multi-family lands to determine the total amount of multi-family dwelling units. The final value may need to be summed from multiple parcels depending on how these lands are spread around in a single TAZ.

Non-residential (commercial/industrial) lands are generally figured out on a parcel basis as parcels are generally large. Zoning codes typically use floor-area-ratios (FAR) for determining maximum lot coverage. The typical FAR range is between 25 and 35% of the lot size which covers green/open space, access roads and parking. For denser areas or industrial areas, height requirements may allow more than a single story which would allow more square footage for the maximum lot coverage. However, if maximum square footage requirements are shown, then these apply to the total of all floors. Multiplying the lot size by the FAR or using the maximum square footage per parcel will determine the maximum potential gross leasable floor area. Commercial and industrial lands need to be kept separate as trip generation multipliers will be different.

Example 6-7 Estimating Future Buildable Land Use

For a particular TAZ, there are 100 acres of vacant buildable zoned RM (medium-density single family residential) land. The zoning code indicates that lot sizes are to be 5,000 square feet at a minimum. A survey of recently completed subdivisions in the area shows that the average lot size is about 6,000 square feet. This TAZ is covered by additional provisions that require 10% of the total development to be common open space. This TAZ also has 5 acres of neighborhood commercial at a maximum FAR of 35%.

Deduct the common open space from the total land area = 100 ac – 100 ac x 0.10 = 90 ac.

Subtract out the typical 24% allotment for local streets = 90 ac – 90 ac x 0.24 = 68.4 ac

Calculate the total number of potential dwelling units = (68.4 ac x 43,560 sq ft/ac) / 6,000 sq ft = 496.6 = 496 lots (need to round down).

Calculate the commercial maximum gross leasable floor area = 5 ac x 0.35 x 43,560 sq ft/ac = 76,230 sq ft = 76, 200 sq ft.

Building Rates

Typically at this point a spreadsheet is set up by TAZ showing the potential dwelling units for residential and square footage for commercial or industrial areas. These TAZ totals represent build-out which may or may not occur within the study horizon (future or design year). Some TAZ's will be currently mostly built out, so it may only take a few years to reach build-out while other TAZ's are vacant and build-out may be over 40 or more years. Faster growing areas (i.e. Central Oregon) will take less time than historically slower growing areas (South Coast).

For each land use type within each TAZ, a building rate needs to be calculated. This aspect of the zonal cumulative method can be one of the most difficult to obtain accurately. The best way to help determine this is to use a GIS-based tax assessor's database to group properties by TAZ and to indicate in which year an improvement occurred. Non-GIS spreadsheets/databases can also be used, but these will require visual matching of tax- lots with the TAZ's. An alternative source is to use issued building permits assuming that these are available for at least the last 10 years. These values can be plotted over the long-term to determine a curve or best-fit regression line to determine a rate of growth. The rate curve will be the best with a good amount of data points covering at least 10 or more years. Another method is to take the total amount of improved lots divided by the number of years those improvements represent to get an average historical building rate.

There may be areas with little or no historical growth, so these should be estimated by using nearby areas with similar characteristics. If an area is relatively homogenous (i.e. all residential) then the building rates could be simplified to a few or a single area-wide value. Commercial or industrial areas building rates could be simplified to a few or a single value depending on if the characteristics are similar. Keep in mind that if there are a lot of in-process developments, these may skew the historic building rate. Any of these results need to be tempered by the advice of the local planner on the historic or future building rates. At a minimum, there needs to be a single building rate for each land use type used. If a database or permits are not available or available for enough years, then the application of this method becomes difficult at best and the enhanced zonal cumulative method should be used. The enhanced zonal cumulative method uses explicit base and future models and is less concerned with a historical building rate.

Example 6-8 Historical Building Rate

Within a residential TAZ, there are several subdivisions that were ready for development in the year 2000. These subdivisions made up a total of 400 lots. The future year for this analysis is 2030. From the county tax assessor's database the following data were obtained from 2000 to 2010:

Year	Number of Permits Issued
2000	20
2001	27
2002	6
2003	4
2004	22
2005	42
2006	37
2007	9
2008	0
2009	0
2010	3

From a review of the data, it is apparent that growth was relatively unsteady and that a regressed trend would not give a good predicted rate. The residential area has the same general characteristics, so an average TAZ historical rate is probably the best method.

From 2000 to 2010 a total of 170 lots had building permits issued. This equates to 170 lots / 11 years = approximately 15 lots per year

This TAZ was not built out in 2010 with 170 of the 400 total built. There are 400 – 170 = 230 vacant lots remaining. With a 15 lots / yr rate, this TAZ would be built out in 230 lots/15 lots per yr = approximately 15 years or 2025. Since the future year is 2030, the maximum potential of 230 new lots in the analysis would be used.

6.10.3 Estimating Future Trip Generation

At this point, the land used for each TAZ have been calculated. Use the ITE Trip Generation manuals or Trip Generation software to compute the increase in peak hour trip generation potential for each TAZ as per Section 6.6. Use major ITE land uses (i.e. shopping center, office park, apartments, etc.) that have ideally equations or reasonable data points. For each land use, make sure to specify the split between the entering and exiting trips. Depending on zone structure and land uses, commercial trips should be primary (new trips on the system) trips, unless there is not enough background traffic to accommodate the potential pass-by trips. Trip generation documentation should be in a spreadsheet form.

Example 6-9 Projected Trip Generation

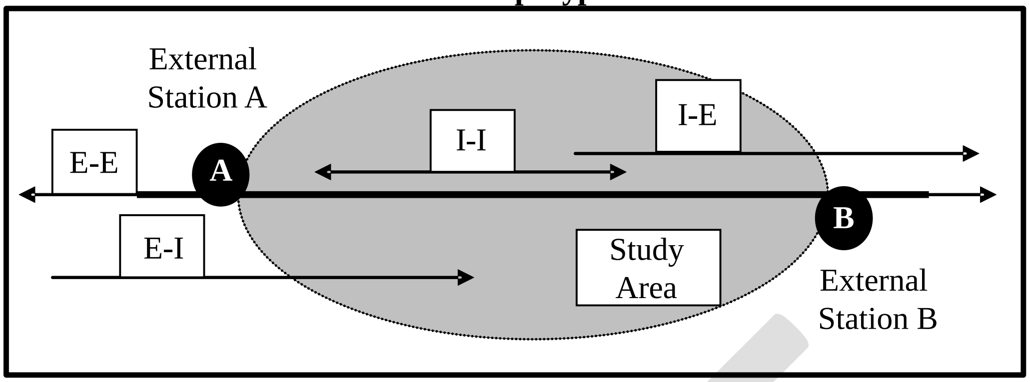
A zonal cumulative analysis was performed for a growing rural residential community along a state highway. This example study area will be used throughout the rest of this section. The study area was split into four zones (1-4). The number of vacant lots and the building rate were estimated following Section 6.10.2. The current year is 2002 with a project horizon year of 2027. All the zones except for Zone 2 reach build-out. The trip generation is based on the ITE Trip Generation PM peak single family home rates (Category 210) of 1.01 trips per dwelling unit in the peak hour.

(1) Zone	(2) Vacant Lots (2002)	(3) Building Rate (Homes/yr)	(4) New Homes built by 2027	(5) 2027 Trip Generation Total (Enter/Exit) (vph)
1	537	80	537	542 (349/193)
2	984	30	750	758 (488/270)
3	167	17	167	169 (109/60)
4	640	35	640	646 (416/230)

6.10.4 Trip Types

The total trips in the study area are made up of internal and external trips. External trips are those trips that have at least one end located outside of the study area, as defined by the study area boundary. Internal trips begin and end within the study area. There are four types of trips that make up the total trips that are the focus of the next three sections. The trip types are external-external (E-E), external-internal (E-I), internal-external (I-E), and internal-internal (I-I). See Exhibit 6-7.

Exhibit 6-7 External and Internal Trip Types



6.10.5 Estimating Through (External-External) Trips

External-external or E-E (through) trips are trips that have both ends (origin and destination) outside of the study area. The E-E trips are the first of the four basic trip types that need to be created from the overall study area volumes. The objective of this section is to calculate the proportion of E-E trips compared to the total trips and eventually the total E-E trips.

The point where each study area roadway crosses the study area boundary is known as an external station. This point represents the connection to the outside world. The external stations are shown as the black circled “A” and ”B” in Exhibit 6-7. External stations are treated essentially as additional zones in this methodology, so number them to not conflict with the TAZ number to avoid confusion. Examples could be using letters (A, B, C…) or a completely different and distinctive number series (500, 501, 502…).

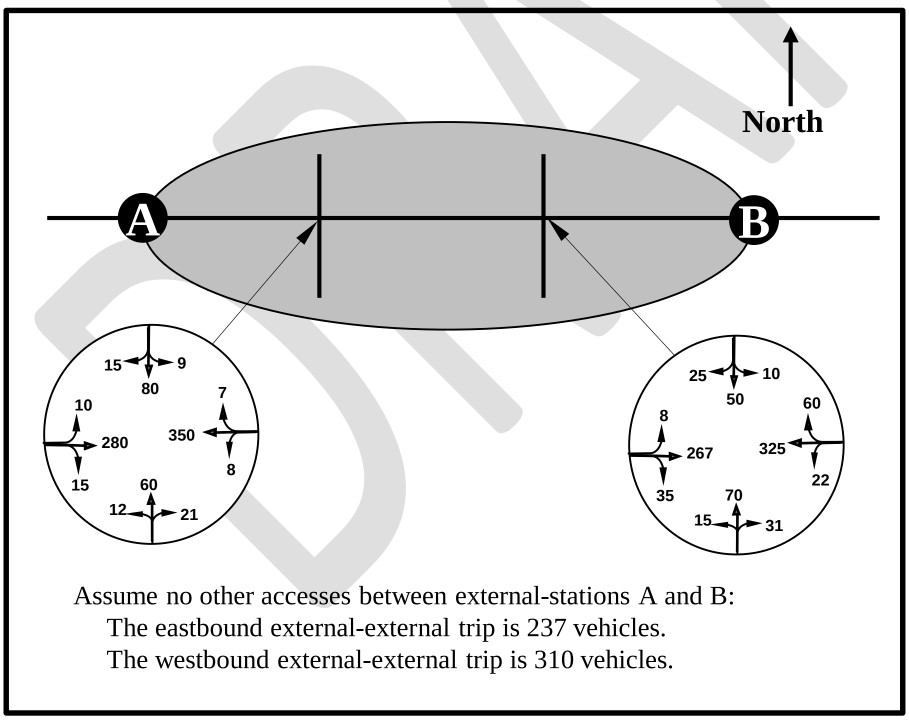
The E-E trips for the base and future years are calculated at each external station. Only roadways with significant volume in the study area should be counted as having external stations. While it is possible to have every local road study area boundary crossing an external station, the complexity of the analysis rapidly increases, the extra work may not be justified, and the necessary detail level needed may not be achievable. Use of origin-destination, license plate matching, or a Bluetooth MAC-address matching study are desired with areas having multiple external stations to simplify finding the E-E trip proportions.

At each external station, the proportion of E-E trips needs to be calculated. This can be obtained directly from an origin-destination or other matching study between each external station pair and for each direction of travel. In most cases, an approximation procedure is necessary to estimate the E-E proportion. Using the directional peak hour (30HV) volumes at an external station, hold that volume and proceed to the other external station by subtracting all turn volumes at each intersection downstream. The remaining volume is the E-E trip volume. This remaining volume is divided by the total directional volume at the external station to determine the percentage of E-E trips. The process is repeated in the other direction and for each external station and for each external station pair (i.e. three external stations: 1-2, 2-1, 1-3, 3-1, 2-3, 3-2).

It is possible to end up with a negative value upon reaching the other external station, especially if there are a large proportion of turning volumes compared to the non-turning volume. The turn movements inside the study area are made up of all the trip types so it is possible that large internal-internal trip patterns skew the turning movements to an extent, so a negative value is obtained. A negative value indicates a need to investigate the higher local turn movements to see where they are destined to and to make sure that other external trips are not mixed in. For areas with complex trip patterns or a good number of external stations, some sort of origin-destination study is recommended for all or part of the study area.

Example 6-10 E-E Approximation Process

In the figure below, there are two external stations, labeled A and B representing the roadway as it enters and leaves the study area. The volumes at intermediate study area intersections are shown below. The initial external station volume is determined from the given volumes. For example, at External Station A, the eastbound volume starts with 305 vph (280 EBT + 10 EBL + 15 EBR) entering the study area.



Starting with the first intermediate intersection and proceeding eastbound, the turning volumes are subtracted from the initial 305 vph value. The resulting value is the total E-E trips for the eastbound External Station A movement. This is repeated for the westbound direction.

Total EB E-E trips = 305 – 10 – 15 – 8 – 35 = 237 vph
Total WB E-E trips = 407 – 60 – 22 – 7 – 8 = 310 vph
At External Station A, the total EB trips are 10 + 280 + 15 = 305
At External Station A, the total WB trips are 12 + 350 + 15 = 377
At External Station B, the total EB trips are 31 + 267 +10 = 308
At External Station B, the total WB trips are 22 + 325 + 60 = 407

The External Station A E-E EB trip percentage = 237 / 305 = 0.77
The External Station A E-E WB trip percentage = 310 / 377 = 0.83
The External Station B E-E EB trip percentage = 237 / 308 = 0.77
The External Station B E-E WB trip percentage = 310 / 407 = 0.77

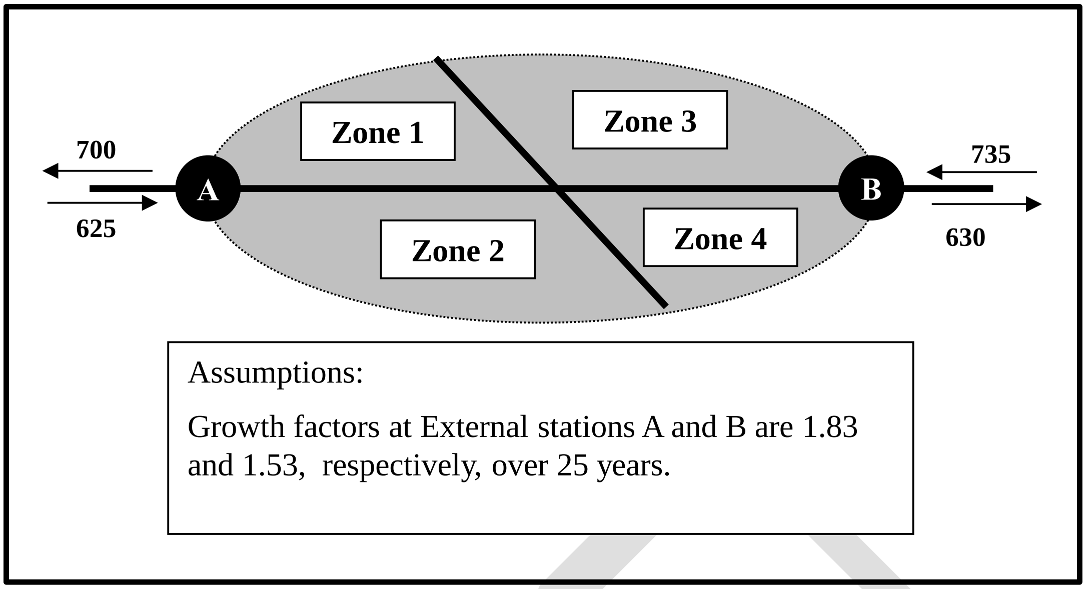
Therefore, there is a 77% chance that trips entering or leaving External Station A are E-E trips. There are 305 total trips on the EB approach at External Station A and it was calculated that 237 of them travel all the way to External station B above. So, 237 / 305 = 0.77 or 77%. In the WB direction at External Station A, there are 377 trips on the approach and it was determined that 310 trips travel from External Station B to A. So 310 / 377 = 0.83 or 83%.

Example 6-10 determined the base year E-E trips. In order to calculate the future year E-E trips, a growth factor needs to be calculated for each external station. This growth factor is computed from the Future Volume Tables for the road segment(s) in question. Non-state roadways can be estimated from historical local counts if available or using a state highway with similar volumes and characteristics as a surrogate. The growth factor is a multi-year factor depending on the number of years between the base and future year. For example, if the base year was 2002 and the future year was 2027, then a 25-year growth factor would need to be computed. See Section 6.5 for more information.

Example 6-11 Future Year E-E Calculations

Given in this example:

- Traffic volumes at the External Station A are 625 entering and 700 exiting.
- Traffic volumes at the External Station B are 735 entering and 630 exiting.



From the figure, the 25-year historical growth factor is 1.83 for External Station A and 1.53 for External Station B and shown in Column (2). The base year 2002 volumes shown in Column (1) are multiplied by the appropriate historical growth factor to obtain 2027 design hour volumes and are shown in Column (3).

The E-E trip proportions were calculated using the method in Example 6-10 for each direction at each external station and are shown in the below table in Column (4):

- External Station A – Entering = 0.83
- External Station A – Exiting = 0.86
- External Station B – Entering = 0.82
- External Station B – Exiting = 0.82

The total new E-E trip growth from 2002 and 2027 is calculated by multiplying the E-E proportions times the difference between the 2002 and 2027 DHV and is shown in Column (5).

Ext. Trip Table	**Direction**	**(1) 2002 DHV (vph)**	**(2) Growth Factor**	**(3) 2027 DHV =(1)*(2) (vph)**	**(4) E-E Trip Prob.**	**(5) 2027 E-E Trip Growth =(4)*((3)-(1)) (vph)**
External Station A	Enter	625	1.83	1144	0.83	431
	Exit	700	1.83	1281	0.86	500
External	Enter	735	1.53	1125	0.82	320

Ext. Trip Table	Direction	(1) 2002 DHV (vph)	(2) Growth Factor	(3) 2027 DHV =(1)*(2) (vph)	(4) E-E Trip Prob.	(5) 2027 E-E Trip Growth =(4)*((3)-(1)) (vph)
Station B	Exit	630	1.53	964	0.82	274

6.10.6 Estimating External-Internal (E-I) and Internal – External (I-E) Trips

Once the total E-E trips have been determined, the External-Internal (E-I) and Internal-External (I-E) trips can be determined. The E-I and I-E trips have one end of the trip in the study area and one end outside of the study area.

The E-I trip growth is the total external station growth entering the study area minus the directional E-E growth. Conversely, the I-E trip growth is the total external station growth exiting the study area minus the directional E-E growth.

Example 6-12 Future Year E-I and I-E Trip Calculation

From the information in Examples 6-10 and 6-11, the 25-year growth of E-I and I-E trips are calculated as shown in Column (6).

Ext. Trip Table	Direction	(1) 2002 DHV (vph)	(2) Growth Factor	(3) 2027 DHV =(1)*(2) (vph)	(4) E-E Trip Prob.	(5) 2027 E-E Trip Growth =(4)*((3)-(1)) (vph)	(6) 2027 E-I, I-E Trip Growth = (3)-(1)-(5) (vph)
External Station A	Enter	625	1.83	1144	0.83	431	88
	Exit	700	1.83	1281	0.86	500	81
External Station B	Enter	735	1.53	1125	0.82	320	70
	Exit	630	1.53	964	0.82	274	60

6.10.7 Trip Distribution of E-I and I-E trips

Trips entering a zone are also known as an “attracted” trip while trips leaving a zone are a “produced” trip. The zonal cumulative analysis uses a gravity-based method for determining a TAZ’s distribution of attractions and productions. Larger zones (on a trip generation basis, not geographical area) attract and produce more trips than smaller zones on a relative basis. This process is known as trip distribution.

After the external-external trip growth has been removed from the total external trip growth, the remaining trips are distributed to the internal zones according to the following procedure.

- Distribution of growth in external-internal trips:
    - Calculate the attraction probability of each zone’s new trip attractions by dividing its new trip attractions by the study area’s total new trip attractions.
    - Distribute the growth in external-internal trips for each external station by multiplying these trips by each zone’s attraction probability.
- Distribution of growth in internal-external trips:
    - Calculate the production probability of each zone’s new trip productions by dividing its new trip productions by the study area’s total new trip productions.
    - Distribute the growth in internal-external trips for each external station by multiplying these trips by each zone’s production probability.

When distributing trips, care needs to be taken considering the land use within each zone. For the typical afternoon peak hour analysis, trips will be traveling from employment zones (commercial/industrial) to residential zones; from residential zones to commercial zones (i.e. shopping/eating after work); and from commercial to other commercial zones (trip chaining). Residential to residential trip distribution should be avoided as it will likely cause unrealistic results.

Example 6-13 E-I and I-E Trip Distribution

Distribute the new external-internal and internal-external trips in Example 6-12 to the four zones shown in Example 6-11.

Solution:

The total new trips are obtained from the trip generation done in Example 6-9 (Column 5). All zones are summed up to determine the grand new trip total for the entire study area. The trip attractions and productions are the entering and exiting trips, respectively, also from Example 6-9. The attractions and productions are also summed up across all zones to determine the total attracted and produced trips.

The calculation of the attraction and production probabilities is shown in the external trip attraction and production probability table below. All trips are in vehicles per hour. For example, Zone 1's attraction probability is 349/1362 = 0.256 and its production probability is 193/753 = 0.256.

External Trip Attractions and Productions Probabilities

Zone	1	2	3	4	Total
Total New Trips (from Example 6-9)	542	758	169	646	2115
Trip Attractions (from Example 6-9)	349	488	109	416	1362
Attraction Probability	0.256	0.358	0.080	0.306	1.000
Trip Productions (from Example 6-9)	193	270	60	230	753
Production Probability	0.256	0.359	0.080	0.305	1.000

The distribution of new external-internal trips is shown in the E-I table below. For example, Zone 1's new external-internal trips at External Station A are 88*0.256 = 23 vph.

External-Internal Trip Attraction Distribution

External Station	New E-I Trips	Zone 1	Zone 2	Zone 3	Zone 4
A	88 (from Example 6-12)	23	31	7	27
B	70(from Example 6-12)	18	25	6	21

The distribution of new internal-external trips is shown in the I-E table below. For example, Zone 1's new internal-external trips at External Station A are 81*0.256 = 21 vph.

Internal-External Trip Production Distribution

External Station	New I-E Trips	Zone 1	Zone 2	Zone 3	Zone 4
A	81(from Example 6-12)	21	29	6	25
B	60(from Example 6-12)	15	22	5	18

6.10.8 Trip Distribution of Internal – Internal Trips

After the new external-internal and internal-external trips have been distributed for each zone, the remaining new attractions and productions are internal-internal trips. While these attractions and productions balance in the examples for simplification, typically they do not. The I-I table would need to be balanced using a Frater methodology (i.e. see NCHRP Report 765). External-External proportions or zone production/attraction probabilities may need to be slightly adjusted for balance to occur. Internal –internal (I-I) trips are trips that both start and end in the study area. The total I-I trips are determined by subtracting the total new E-I/I-E trips from the total new trips for each TAZ.

Example 6-14 I-I Total Trip Calculation

From the previous example, the total new attraction/production trips for each zone in vehicles per hour are shown on the first line of the trip attractions and productions probability table. The total E-I and I-E trips for each zone are summed up and subtracted from the total new trips.

For Zone 1, the total new trips are 542. The total E-I trips (from Example 6-13's trip attraction distribution) are 23 + 18 = 41 while the I-E trips are 21 + 15 = 36 (from Example 6-13's trip production distribution table).

Trip Type	Zone 1	Zone 2	Zone 3	Zone 4	Total
Total New Trips	542	758	169	646	2115
Total E-I trips	41	56	13	48	158
Total I-E trips	36	51	11	43	141
Total resulting I-I trips	**465**	**651**	**145**	**555**	1816

The total I-I trips are calculated by subtracting the total E-I and I-E from the total new trips: Total Zone 1 I-I trips = 542 – 41 – 36 = 465 trips
This is repeated for all remaining zones. Note that the table is balanced as the sum of the total new trips minus the total E-I and I-E trips equals the sum of the column totals. For the above table: 465 + 651 + 145 + 555 = 1816 and 2115 – 158 – 141 = 1816.

To distribute the internal-internal trips for each zone, use the same distribution process as described in Section 6.10.7. Once the total I-I trips are determined for each zone following the process shown in Example 6-13, the total internal attractions and productions need to be calculated. The total internal attractions are determined by subtracting the total external-internal attractions from the total attractions. Likewise, the total internal productions are determined from subtracting the total internal-external productions from the total productions. Attraction probabilities are calculated by dividing the total attractions for each zone by the total attractions. Production probabilities are done similarly.

Example 6-15 I-I Trip Distribution

For each zone, the total internal attractions and productions need to be calculated. For Zone 1, there were 464 total internal-internal trips determined in Example 6-14 and shown below in the internal-internal trip attraction/production probability table. In Example 6-13 the total external-internal trip attractions were 349 while the total external-internal trip productions were 193. The total external-internal trip attraction/distribution table has a total of 41 (23+18) trips for Zone 1. Likewise, the corresponding internal-external trip production table has a total of 36 (21 + 15) trips for Zone 1.

Total I-I trip attractions = 349 – 41 = 308 trips
Total I-I trip productions = 193 – 36 = 157 trips

The calculations are repeated for all zones and are shown in the table below. The total I-I trip attractions and productions are summed up across all zones for a grand attraction and production totals. Attraction and production probabilities are also calculated for all zones. For Zone 1 the 308 internal attracted trips are divided by the total 1204 attracted trips = 308 / 1204 = 0.256 Production probabilities for Zone 1 are 157 / 612 = 0.256.

Internal Trip Attractions and Productions Probabilities

	Zone 1	Zone 2	Zone 3	Zone 4	Total
Total Internal-Internal Trips	464	650	145	553	1812
Internal Attractions	308	432	96	368	1204
Attraction Probability	0.256	0.358	0.080	0.306	1.000
Internal Productions	156	218	49	185	608
Production Probability	0.257	0.358	0.081	0.304	1.000

The distribution of new internal-internal attractions is shown in the table below. For example, the attraction trips to Zone 1 from Zone 2 are 308*0.358 = 110.

Internal Trip Attraction Distribution

Zone	I-I Attraction	Zone 1	Zone 2	Zone 3	Zone 4
1	308	79	110	25	94
2	432	111	155	34	132
3	96	25	34	8	29
4	368	94	132	30	112

The distribution of new internal-internal productions is shown in the table below. For example, the production trips from Zone 1 from Zone 2 are 156*0.358 = 56.

Internal Trip Production Distribution

Zone	I-I Production	Zone 1	Zone 2	Zone 3	Zone 4
1	156	40	56	13	47
2	218	56	78	18	66
3	49	13	18	4	15
4	185	48	66	15	56

6.10.9 Initial Trip Assignment

Following trip distribution, the next step in the procedure is trip assignment, which involves placing the growth in trips on the road network. Trip assignment is the process used to estimate paths the trip will take, which ultimately results in traffic flow on the network. It assigns the trips to specific routes and establishes volumes on links, taking

into consideration network characteristics to find the shortest path between origins and destinations.
The trip assignment is based on the shortest path in the network by time. A travel time needs to be identified for each link on the network. At the minimum, each route should be traveled at least once in each direction during the analysis hour to help estimate a travel time for that direction on the link. The ideal method to determine travel times would be following the procedures in Chapter 3 for travel time studies. For roadways that do not exist, speeds should be estimated using distance and likely travel or posted speed.

The analyst has to establish the travel times for the four trip-types. For each production-attraction pair in the trip table, the likely paths need to be identified. There is likely to be more than one path for each pair, depending on the network. The link travel times are summed to create the total path travel time.

Where there are multiple paths for an individual production-attraction pair, the appropriate path needs to be selected. The selection should be based on the smallest travel time. If the path travel times are within 10% of each other, the trips can be assigned equally across those paths. When the times are more than 50% different then assign the trips all to the shortest path. When the difference is between 10 and 50 %, the trips should be assigned on a weighted average based on the inverse proportion. For example, assume that the paths were 4 minutes versus 3 minutes. Since the 4-minute path is 33% longer than the 3-minute path, the trips are assigned 33% to the longer 4-minute path and 67% to the shorter 3-minute path. Engineering judgment must be used when reviewing travel times of closely spaced parallel paths to assure that actual driver behavior is modeled and not just a mathematical difference.

The analyst must now assign the trips by trip types starting with E-E and progressing to E-I, then I-E and finally I-I as illustrated in the example.

6.10.10 Total Trip Assignment

Once all the trips have been assigned by trip type, the analyst must check for links that have been over-assigned based on capacity. The analyst should first review the base year analysis to determine whether certain links are below, near/at or over capacity. For links in the base year that are approaching or over capacity, a flag for possible trip reassignment is identified. Also, links with a large amount of growth in traffic should be identified as needing a check for over capacity conditions.

Any flagged linked may require that the assignment of the growth trips be reassigned to links with available capacity. The analyst needs to use an HCM segment methodology to determine the link capacity. The analyst will need to convert the base year intersection volumes into link volumes. The growth trips need to be added to the base year trips to determine total trips assigned to a link. The analyst then identifies any links that have trips assigned in excess of the capacity. For the over-capacity links, the excess trips above capacity from growth need to be reassigned to other logical paths. This iterative process should be continued until the excess trips have been assigned logically onto the network.

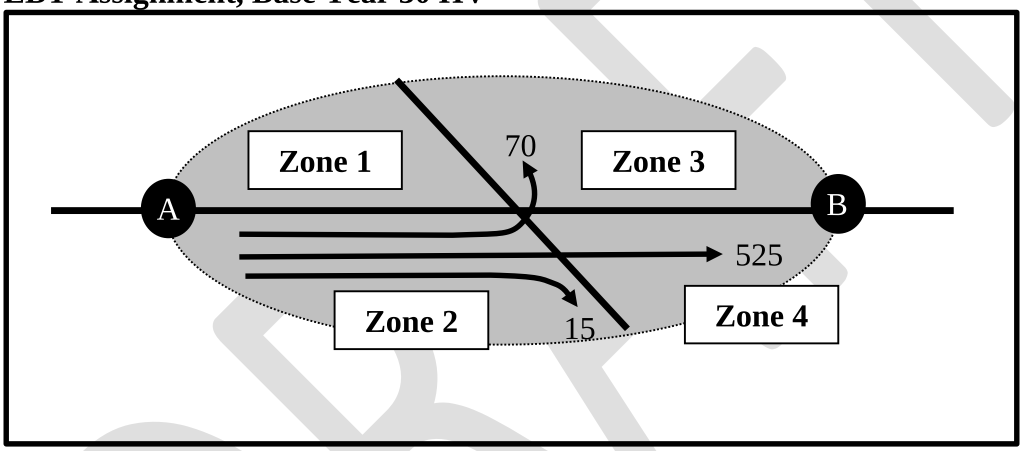
Depending on the network, there may be limited additional paths so the excess trips cannot always be completely reassigned.

Example 6-16 Trip Assignment for Eastbound Through

This example illustrates the aggregation of the individual trip types for the future for one movement on one approach at an intersection within the study area. This would need to be repeated for the other movements and other intersections. This example focuses on the eastbound through (EBT) movement.
Base Year Trips

The process starts with the existing year (base year) 30HV volume previously calculated using procedures from Chapter 5. From the figure below, there are 525 base-year trips to be assigned to the EBT movement.

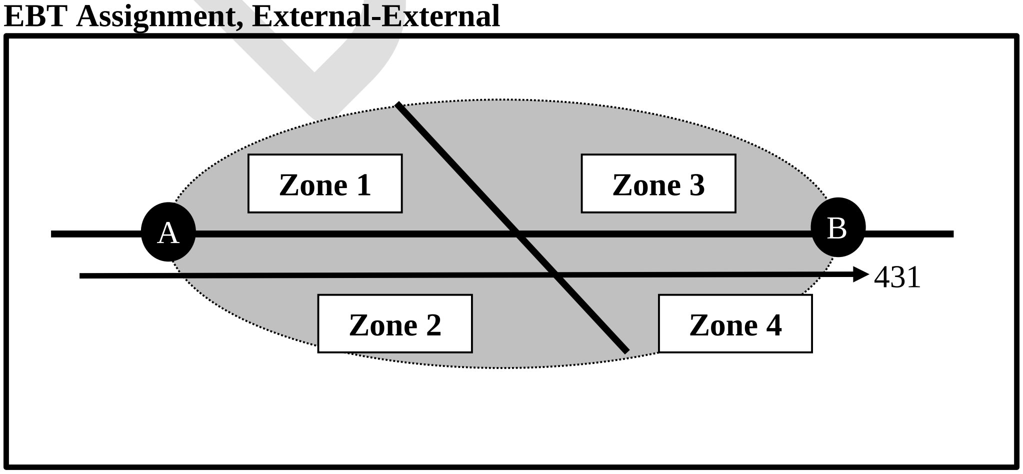
EBT Assignment, Base Year 30 HV



External-External Trips

Next, the external-external trips are added. From Example 6-11, the future year eastbound E-E trips are 431 and are shown in the next figure below.

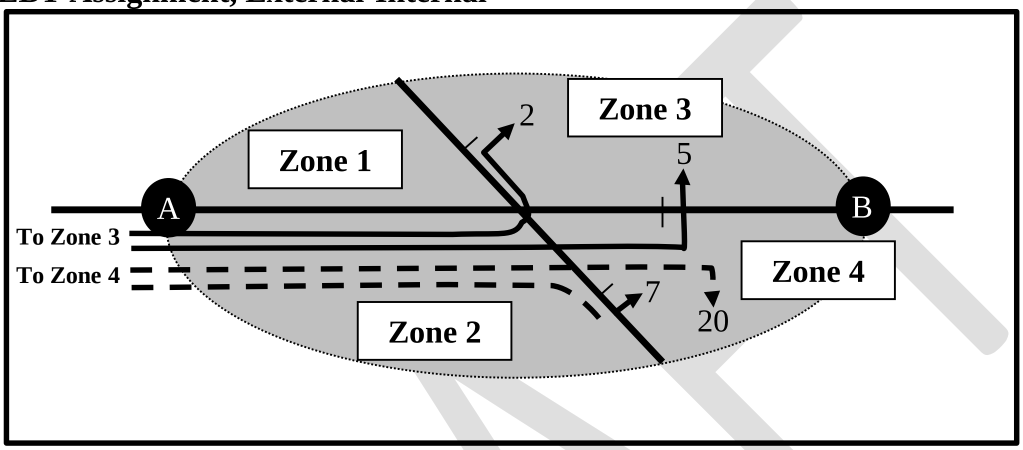
EBT Assignment, External-External



External-Internal Trips

From the figure below, note that Zone 1 and 2 E-I trips will leave the roadway before the intersection at access points or turning left or right at the side street. Both of these do not add to the assignment for the eastbound through (EBT) movement. However, they would be part of the assignments for the eastbound left and right.

EBT Assignment, External-Internal



There are seven total external-internal trips from External Station A to Zone 3. These seven trips can access to Zone 3 by turning left at the intersection or through downstream accesses via the mainline based on assignment assumptions. Two of the seven trips will travel to Zone 3 by turning left at the intersection and the remaining five trips will travel to Zone 3 through downstream accesses via the mainline.

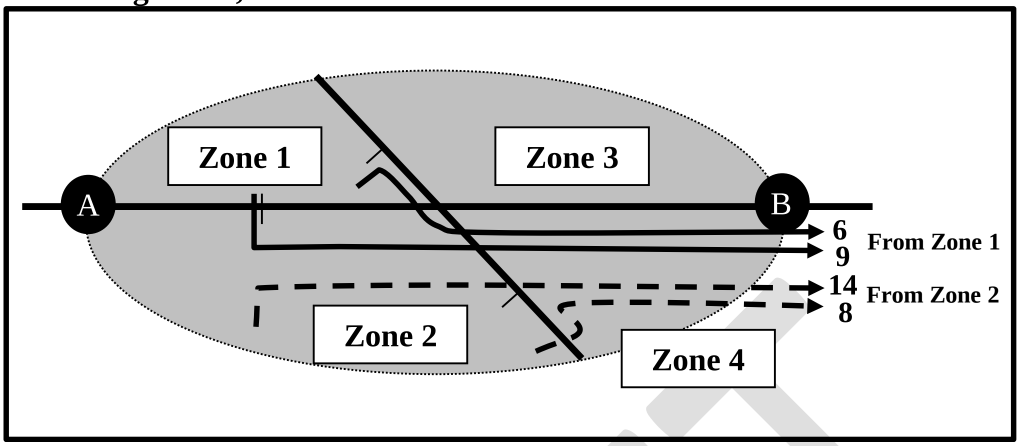
The same process is followed for trips from External Station A to Zone 4. The total external-internal trips from External Station A to Zone 4 are 27. The same accessibility assumptions from Zone 3 apply to Zone 4. Here, assuming 20 out of the 27 will access to Zone 4 by downstream accesses on the mainline and seven via the side street.

The resulting total E-I trips for the EBT movement is 25 (20 +5).

Internal-External Trips

From the figure below, note that Zone 3 and 4 I-E trips will turn onto the mainline roadway at the intersection or access it downstream of the intersection. These movements would not be counted into the EBT movement assignment. They would be part of the assignment for the northbound right and southbound left movements.

EBT Assignment, Internal-External



The total internal-external trips from Zone 1 to External Station B are 15. The assignment of these trips depends on the accessibility from Zone 1 to the external station B. Nine (9) of the 16 trips (depending on the accessibility from Zone 1 to the road network) will travel to External Station B by using upstream access points. The remaining six trips will travel to External Station B by turning left at the intersection.

The total I-E trips are 23 (9+14) for the EBT movement.

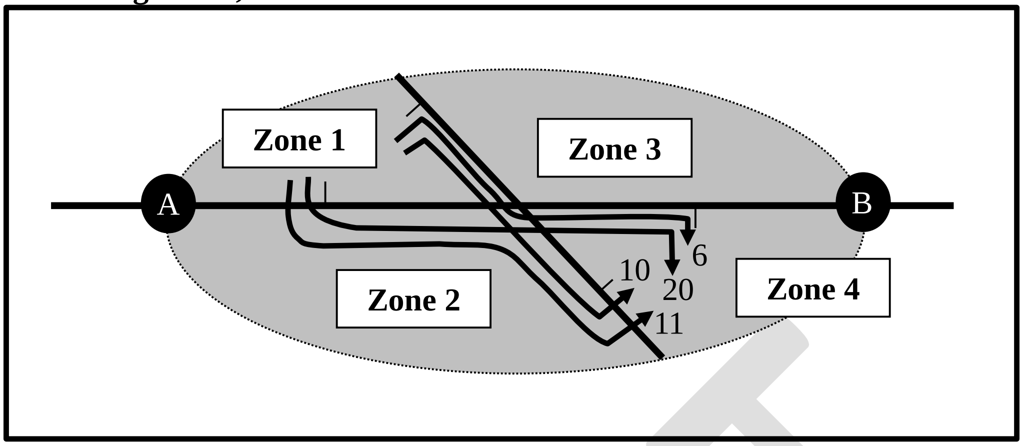
Internal-Internal Trips

There are internal-internal production trips from Zone 1 to Zones 2, 3 and 4, and from Zone 2 to Zones 1, 3 and 4. The internal-internal production trips from Zone 1 to Zones 2 and 3 and from Zone 2 to Zones 1 and 4 do not go through the subject intersection as they use other smaller connectors or accesses. The only applicable internal-internal production trips are between Zone 1 and 4 and Zones 2 and 3.

The total internal-internal production trips from Zone 1 to Zone 4 (See Example 6-11) are 47 trips and from Zone 2 to Zone 3 are 18 trips. The internal-internal trip assignment from Zone 1 to Zone 4 and from Zone 2 to Zone 3 is based on the network accessibility assumptions.

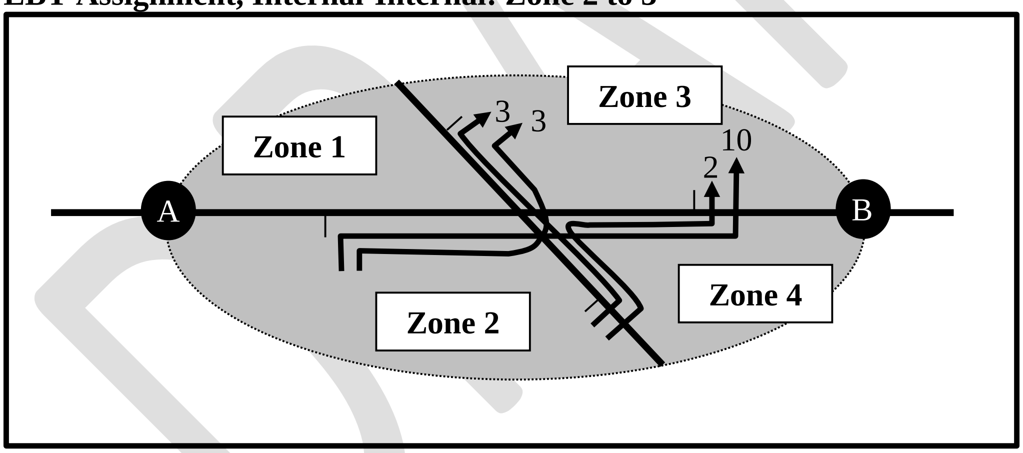
On the side street, I-I trips from Zone 1 to 4 will take a couple different paths by either traveling through the intersection southbound (10 trips) or turning left (6 trips) as seen in the figure below. On the mainline, the I-I trips from Zone 1 to 4 will access the mainline mid-block and travel through the intersection eastbound (20 trips) or turn right at the intersection (12 trips). Of the 48 total I-I trips from Zone 1 to 4, only 20 trips are assigned to the EBT movement.

EBT Assignment, Internal-Internal: Zone 1 to 4



Likewise, between Zone 2 and 3, the I-I trips will either travel between the two zones by the side street or the mainline. On the side street, I-I trips from Zone 2 to 3 will either travel northward through the intersection (3 trips) or turn right (2 trips) as seen in the figure below. On the mainline, I-I trips from Zone 2 to 3 will turn left at the intersection (3 trips) or travel through the intersection to downstream accesses (10 trips). Of the 18 total I-I trips between Zone 2 and 3, only 10 trips are assigned to the EBT movement.

EBT Assignment, Internal-Internal: Zone 2 to 3



The total I-I trips on the EBT movement are 30 (20 + 10) from both zones.

Total Trip Assignment

The resulting total trip assignment is created by summing up the base year trips, the external trips and the internal trips as seen in the table below.

Total Trip Assignment – EBT (vph)

Trip Type	Trips (vph)
Base Year 30HV	525
E-E	431
E-I	25

Trip Type	Trips (vph)
I-E	23
I-I	30
Total	**1034**

For Build Alternatives, the trip generation and distribution are fixed, so only the trip assignments need to be modified. The travel times would have to be determined for the new roadway network and then the trips assigned by the methods above.

6.11 Enhanced Zonal Cumulative Analysis

> *EZCA is not and shall not be considered the equivalent of a travel demand model, even though it uses similar techniques and tools. It should not be used for analysis other than the original use without coordination with TPAU to determine if significant re-evaluation and possible updating of information is required.*

The enhanced zonal cumulative analysis (EZCA) methods provide a series of enhancements to the typical application of zonal cumulative analysis methods. The purpose of the EZCA application is to enhance traffic circulation and operations analysis that can account for details that are otherwise not captured in the cumulative analysis approach. The full application of these enhancements has also been referred to as the small community forecast tool (SCFT) methodology. The EZCA methods can be used wherever the regular zonal cumulative analysis process is used, as well as larger areas (greater than 10,000) that do not have a travel demand model in place[1]. The development and application of the EZCA methods can be cumbersome and labor-intensive; however, the methods provide significant refinement and application beyond the typical zonal cumulative analysis. Larger areas will still require the manual spreadsheet-based setup, however, the enhanced zonal cumulative allows greater automation in trip assignment using VISUM (or other modeling assignment software). By using assignment software tools, the process can be streamlined to more closely resemble a travel demand model. The creation of an enhanced zonal cumulative analysis should be weighed against the creation of a full travel demand model (refer to Chapter 17). As with a travel demand model, analysts should coordinate with TPAU to determine if subsequent project application (i.e., use for project analysis beyond the original study) is appropriate for the scope and development of the EZCA. Appendix 6A provides a sample application of the EZCA process that was performed for Canby.[2]

[1] While larger areas may trigger development of travel demand model, in some cases full model development may not be feasible given project application needs and schedule. In these areas, EZCA may still provide an alternative tool for forecasting traffic volumes or other project application. An example of a larger area with EZCA application is Canby, Oregon, which had a population over 15,000 in 2010.
[2] Technical Memorandum #3: Canby TSP – Future Forecasting, DKS Associates, March 31, 2010.

6.11.1 Process

The general process for developing an EZCA tool follows a similar process to the zonal cumulative analysis, while adding enhancements that address some of the key limitations. Primary enhancements include:

- Scope of land use and trip assignment - The major difference (enhancement) is to include an estimation of base year and future year total land use (and trips), instead of only using a growth increment. This allows redistribution and reassignment of existing trips that may occur due to adjacent changes in land use or the transportation network. For example, an existing trip from a residence to a grocery store may use a different route (assignment change) if a new street connection is build. Similarly, if a new grocery store is built and is located closer, the driver may decide to travel to this store instead of the other store (distribution change). This fundamental difference provides additional flexibility and options for project application.
- Trip Matrix – All trips in the network are typically assigned using a trip matrix, including existing trips. Fixed trips (such as driveway pass-by trips) can be added, but doing so removes trip assignment flexibility and routing effects due to network condition and changes. Existing count data are used for calibration purposes, but is not considered "background" trips.
- Trip assignment approach - The EZCA utilizes an equilibrium assignment procedure that automates the routing choices with improved accuracy over manual assignment. Depending on the assignment software tool used, more detail such as link and/or nodal delay can be added to better represent the effects of congestion. This will allow a more responsive, accurate and comprehensive analysis of the base and future conditions. This approach provides two key benefits:
    - Fundamentally, assignment and routing is automated based on a comprehensive, analytical check of available routes and travel times.
    - Network detail (connector loading, street system, traffic control, etc.) can be scaled by project need, as described in Chapter 8 (Mesoscopic Analysis)[3].
- An iterative process of checks, back-checks, and balances – The use of spreadsheets (land use summaries and matrix development) and assignment software (trip assignment) allows for a semi-automated process that can be used to perform iterative checks and testing.
- Calibration of a base year "tool" – Since all internal trips (rather than just the growth increment) are being estimated through land use, trip generation, trip distribution, and trip assignment, the network coding and analysis assumptions (including land use and trip generation) can be validated in a way that is not feasible with the standard TIA level and zonal cumulative analysis methods.
- Various other enhancements that may be performed to refine a standard zonal cumulative analysis application, such as data to refine land use development

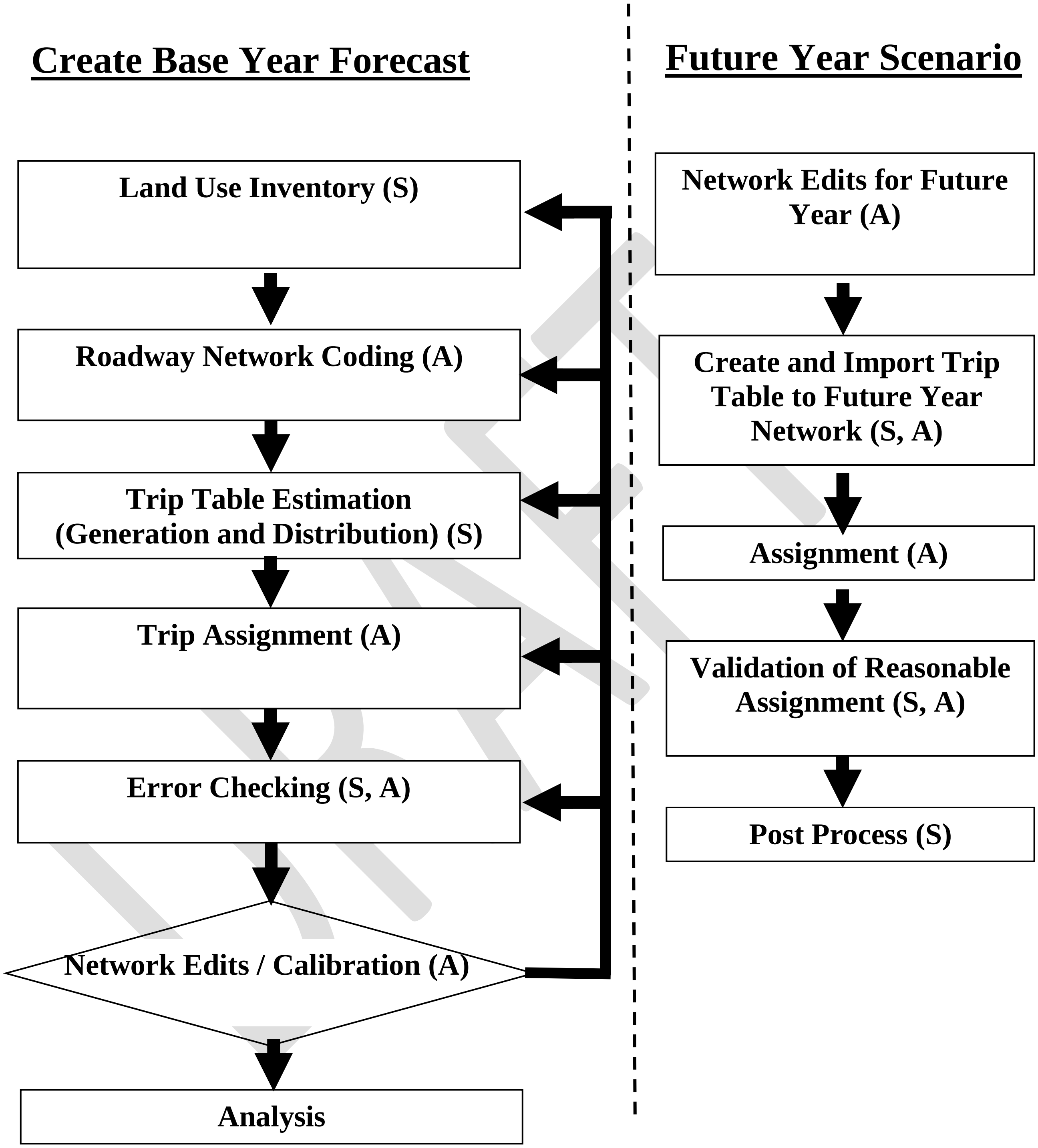
[3] Typical applications of EZCA will include an all-street system reflecting comprehensive traffic control, as described in Windowed Area Models.

(Section 6.8.2) collection of Bluetooth data (Section 6.8.5) to inform external trip patterns.

The overall EZCA Process (Exhibit 6-8) is generally like the typical zonal cumulative analysis method. However, subtle differences exist as listed in Exhibit 6-8.

The analysis results are impacted by the combined effects of the integrated EZCA components (land use, trip rates, loading locations, network coding, etc.). For this reason, it is important to balance the level of detail and effort that is put into each component to minimize compounded errors. For instance, trip generation relies on both land use and trip rates. A perfect inventory of land uses will not provide true enhancements to this procedure unless trip rates are also appropriate.

Exhibit 6-8 EZCA Process



Note: Typical tools for the above process include spreadsheets and the traffic assignment software. Steps noted with (S) are typically conducted in a spreadsheet, while those marked with (A) are typically performed in the assignment software.

Exhibit 6-9 Comparison of Standard Zonal Cumulative Analysis and EZCA

Zonal Cumulative Analysis Step (Standard Approach)	Enhanced Zonal Cumulative Analysis (EZCA)
General Set Up	
1- Identify the study area and divide into transportation analysis zones (TAZ)	Study area typically whole community or large subarea
(Step not used)	Set up the transportation network coding
Land Use & Trip Generation	
2- Identify vacant lands, in-process developments, comprehensive plan allowed land uses/densities, and development rates.	Determine all existing and future land use totals (not just growth)
3- Estimate future trip generation potential	Estimate existing trips using ITE average trip rates as the initial assumption.
Trip Distribution	
4- Determining the through trip percentages (external – external, E-E) and E-E trips for the external stations	Approximation by subtraction procedure is difficult due to larger network size, and typically need OD data such as Bluetooth.
5- Determining the internal – external (I-E) and external –internal (E-I) trips at each external station (external zone)	Completed for all trips, not just growth increment.
6- Determining the trip distribution for the internal – external (I-E) and external – internal (E-I) trips for each internal TAZ.	Complete for all trips, not just growth increment.
7- Determining the trip distribution for internal-internal (I-I) trips	Complete for all trips, not just growth.
Trip Assignment	
8- Calculating network link travel times	Not done manually. Assignment software uses the "free flow" travel time as the initial assumption (updated through an iterative assignment process) based on link and/or nodal travel times. Link delay may be needed to account for congestion on free-flow facilities.
9- Assigning total trips to the network	Not done manually. Assignment software is used.
Additional Steps for EZCA	
(Step not used)	Calibration of tool estimation of traffic volumes compared to counts

Although this process is not a travel demand model, many of the elements are similar so general background information can be obtained from Chapter 17 and the Modeling

Procedures Manual for Land Use Changes (MPMLUC). See Chapter 8 for related information for developing traffic networks for subarea models.

The following sections provide details for each component of the ECZA process, with a focus on methods that differ from the standard zonal cumulative analysis approach. Unless otherwise noted, the approach generally follows Section 6.8.

6.11.2 Land Use

Land use inventories for EZCA are required similar to the standard zonal cumulative analysis approach (typically collected at the parcel level and aggregated to the zonal level), with the following differences:

- Inventories include all land use, not just the growth increment. For this reason, both base year and future year inventories are developed for the study area.
    - Base year inventories should be consistent with traffic data that was collected for calibration.
    - Future year inventories include not only development, but potential redevelopment as well.
- The study area is typically comprised of a large area and may even represent an entire community. Larger study areas that encompass the entire community provide the ability to compare to coordinated population projections that may not provide as much value to a smaller area.
- The land use data are typically grouped into five main categories – households (which may be split between single family and multifamily uses), retail employment, service employment, education employment, and other employment. However, these categories may vary by application depending on the community, allowed zoning uses, and context. These land uses should generally correlate with ITE categories or combinations of ITE categories based on related uses that exist in the community. For instance, a community may have the following uses, grouped as shown:
    - Single Family Residential Household
    - Apartment/Condo
    - Retail Employment (Gas station, Fast Food, Bank, Shopping Center)
    - Service Employment (Strip mall offices, auto shops, and specialty shops that do not generate as many trips as active retail uses)
    - Education Employment
    - Other Employment (Office Park, Industrial Uses)
- Land use inventories are aggregated to transportation analysis zones, as shown in Exhibit 6-10.
- For future conditions, the future population projection from the analysis must be consistent with the county-coordinated population forecast for the city. The future employment needs to be at a level that can be supported by the population in the study area[4]. The future employment and population must be consistent with

[4] The *Modeling Procedures Manual for Land Use Changes*, ODOT Transportation Planning Analysis Unit, February 2012 identifies employment ratios that are reasonable.

the local jurisdiction's comprehensive plan. The analyst will need to consult with the local jurisdiction to determine where future growth is to be allocated. The local jurisdiction will also need to review the final allocations for consistency. The modular integration of the EZCA elements allows the ability to easily test different land use scenarios (update trip generation and copy resulting trip matrix to assignment software).

Exhibit 6-10 Sample Land Use Totals (Households and Employees) by TAZ

TAZ	HH_BASE	RETL_BASE	SERV_BASE	EDUC_BASE	OTHR_BASE
101	26	0	0	0	0
102	9	8	0	0	0
103	295	0	0	0	0
104	68	0	0	0	40
105	0	0	0	0	25
106	149	0	0	0	0
107	175	0	0	0	0
108	162	0	0	0	0
109	163	0	0	0	0
110	131	0	0	0	0
111	70	0	0	0	0
112	109	0	0	0	0
113	66	0	0	0	0
114	164	0	0	0	0
115	140	0	0	0	0
116	85	0	0	0	0
117	63	0	0	0	0
118	124	0	0	0	0
119	121	0	0	0	0
120	105	0	0	0	0
121	45	0	12	0	29
122	214	0	1	0	62
123	0	0	0	0	0
124	134	0	0	0	0
125	1	0	25	0	300
126	133	0	0	0	0
127	213	0	0	74.9	0
128	193	0	0	0	0
129	111	0	0	0	0
130	75	30	105	0	13

6.11.3 Roadway Network

The standard application of zonal cumulative analysis can require rigorous manual calculations and checks to determine appropriate assumptions about travel time and vehicle routing. As an enhancement, assignment software can be used to provide a platform for assigning the trip matrix in place of manual assignment methods. This enhancement provides a technical basis (such as travel time or other factors) for assumptions related to trip routing. For this reason, the roadway network needs additional detail that is not typically included in a standard travel demand model (See network details in Chapter 8 regarding mesoscopic modeling and subarea models).

The network should include, at a minimum, arterial and collectors within the study area. Local streets may also be desired. Additional details can be added, which include posted speeds, traffic control, lane geometries and the number of travel lanes. The analyst in consultation with the local jurisdiction must determine whether committed (funded), financially constrained and/or all planned projects are to be added to the future network. Elements of the transportation network should include:

- Link (Street) Detail
    - Location – the more roadways included the more flexibility of assignments is included, specifically roads that are subject to cut-through traffic and any roads that are parallel to major routes. Some software allows importing GIS shapefiles from sources such as local agency inventories, or the Open Street Network which has much of this information. However, the amount and accuracy of these data need to be (field) verified or edited. More links in the network do not necessarily increase the accuracy of the assignment results, and care should be taken with calibration.
    - Speed – free flow speeds initially based on posted speeds. In some cases the addition of a volume-delay-function (vdf) may be needed to account for delays accrued on uncontrolled facilities.
    - Lanes – number of lanes based on field conditions. This input affects the number of fully developed approach lanes at intersections and is utilized for intersection geometry and delay.

- Node (Intersection) Detail
    - Control – type is specified based on field inventory and is used to calculate the turn-based travel delay using HCM methodology for the given control type. Calibration may require modifying the control (i.e. yield to stop) to better match field operations.
    - Lane Configuration – (turn lanes) based on field inventory to determine the intersection turn delay

- Zone Detail
    - Centroid connectors represent the non-modeled minor streets and accesses within a zone to load trips from a TAZ centroid onto the network. Extra

connectors should be added, as needed to represent a particular network scenario, to distribute trips from the TAZ to match the loading of the trips onto the network such as from new roads, minor streets and off-street parking.

6.11.4 Trip Table Estimation

Trip table estimation is a component that appears generally similar yet is fundamentally different between the zonal cumulative analysis and EZCA. One notable deviation from the zonal cumulative analysis is including all trips in the forecast rather than just the growth component. This develops volumes for the base year that can be used to calibrate the analysis tool, as well as providing more reasonable trip distributions. The approach of distributing only new growth does not allow for the redistribution of existing trips. For this reason, all trips, including those that are currently part of the roadway system, are estimated in the trip table as E-E, E-I, I,E, or I-I trips (see Section 6.8 for additional information about trip types and distribution).

Trip Generation

External Trip Ends

- There are three steps to prepare for the distribution of external trips. First, the number of external trip ends is estimated based on design hour volumes at external nodes. Second, these external trip ends are separated into external-external (E-E), external-internal (E-I), and internal-external (I-E) trips. Third, projected future growth rates are applied (to determine future trips levels). The base year external trip ends are based on 30th HV design volumes at key gateway locations (Design Hour Volumes at External Nodes).
- Traffic counts should be collected at major external gateways
- The impact of low volume external gateways (typically minor rural roads serving less than 50 vehicles during a peak hour[5]) that do not serve as major regional connections may be important to the overall estimation of the trip table, but it may not appropriate to consume data resources collecting detailed traffic data at each location. The total number of external gateways needs to be coordinated and agreed upon with TPAU. For example, many communities have rural roads that extend outside the UGB that connect to outlying homes but do not provide regional connections to other communities. The traffic volumes at these gateways may be initially estimated based on agreement and coordination with TPAU and a combination of the following resources, as available:
    - An inventory of the land uses (typically the number of homes) that are likely to be served.
    - Adjacent traffic counts with an assumption made about the difference in demand that would result. While traffic counts may not be collected at these specific gateways, there should be other internal network counts that are collected at key junctions that can be used to estimate if the inventory

[5] This general threshold may vary based on the context of the area and system connectivity.

is reasonable. Counts that are too far away with many intersecting roads may result in overestimation at the gateway and a tube count should be obtained at these locations.

- Future external trip ends are estimated based on forecasted growth at each of the external gateways. The FVT growth projections are utilized to estimate the growth rate at the external gateways.

Internal Trip Ends

- The number of internal trip ends is determined using land use trip generation methodology by applying ITE trip rates to the land uses described in Section 6.9.2. In some cases, combinations or “blended” rates that represent multiple ITE uses, and trip rates may be used depending on the grouping of land use categories. For instance, if a single category is used for households and both single family and multifamily homes are present, a blended rate that combines both the single-family rate and multifamily rate may be used. For these cases, combinations of trip rates should be used that are representative of the land uses occurring in the community and reflected in each general land use category.
- National average ITE trip rate data may not reflect the unique characteristics of the study area. When sufficient data are available, trip rates may need to be adjusted during the calibration process. Any adjustments to the ITE trip rates need to be documented and reviewed by TPAU. (See calibration section for additional details). Exhibit 6-11 shows a summary of land use and trips (inbound) for each zone.

Exhibit 6-11 Sample Internal Trip Generation (Trips Inbound) by Zone and Land Use

	Land Use (HH, EMP)						Trips (Inbound)					
TAZ	HH_BASE	RETL_BASE	SERV_BASE	EDUC_BASE	OTHR_BASE		HH_BASE trips	RETL_BASE trips	SERV_BASE trips	EDUC_BASE trips	OTHR_BASE trips	Total Trips
101	26	0	0	0	0		12	0	0	0	0	**12**
102	9	8	0	0	0		4	15	0	0	0	**19**
103	295	0	0	0	0		139	0	0	0	0	**139**
104	68	0	0	0	40		32	0	0	0	2	**34**
105	0	0	0	0	25		0	0	0	0	1	**1**
106	149	0	0	0	0		70	0	0	0	0	**70**
107	175	0	0	0	0		83	0	0	0	0	**83**
108	162	0	0	0	0		77	0	0	0	0	**77**
109	163	0	0	0	0		77	0	0	0	0	**77**
110	131	0	0	0	0		62	0	0	0	0	**62**
111	70	0	0	0	0		33	0	0	0	0	**33**
112	109	0	0	0	0		52	0	0	0	0	**52**
113	66	0	0	0	0		31	0	0	0	0	**31**
114	164	0	0	0	0		77	0	0	0	0	**77**
115	140	0	0	0	0		66	0	0	0	0	**66**
116	85	0	0	0	0		40	0	0	0	0	**40**
117	63	0	0	0	0		30	0	0	0	0	**30**
118	124	0	0	0	0		59	0	0	0	0	**59**
119	121	0	0	0	0		57	0	0	0	0	**57**
120	105	0	0	0	0		50	0	0	0	0	**50**
121	45	0	12	0	29		21	0	11	0	1	**35**
122	214	0	1	0	62		101	0	1	0	3	**105**
123	0	0	0	0	0		0	0	0	0	0	**0**
124	134	0	0	0	0		63	0	0	0	0	**63**
125	1	0	25	0	300		0	0	24	0	14	**38**
126	133	0	0	0	0		63	0	0	0	0	**63**
127	213	0	0	74.9	0		101	0	0	59	0	**160**
128	193	0	0	0	0		91	0	0	0	0	**91**
129	111	0	0	0	0		52	0	0	0	0	**52**
130	75	30	105	0	13		35	58	102	0	1	**196**

Trip Distribution

Trip distribution estimates the percentage of trips from one zone to another. In this analysis, trip distribution and generation are used to define the vehicle trip tables that are incorporated into the existing and future tools.

Distribution is based on the trip ends generated as either productions or attractions for each zone. The productions and attractions are used to determine an attraction probability and production probability for each zone relative to other zones in the network.

Trips are distributed between TAZs using two trip tables:

1. Production Trip Table – Each zone's productions are multiplied by the other zone's attraction probabilities resulting in a set of trips from one TAZ to every other TAZ.
2. Attraction Trip Table – Each zone's attractions are multiplied by the other zone's production probabilities resulting in a different set of trips from each TAZ to every other TAZ.

The distribution procedure includes the following steps:

1. Distribute E-E trips based on data collected from origin-destination study (such as Bluetooth data collection)
2. Allocate remaining external trips to E-I and I-E, based on directionality, as applied to zonal cumulative analysis.
3. Allocate remaining internal trips to I-I based on the attraction and production probabilities as done in the zonal cumulative analysis, resulting in a productions trip table and an attractions trip table. These steps need to be performed for trips generated by household land uses and then for trips generated by employment land uses. Household-based trips should use only employment land uses to calculate attraction and production probabilities.
4. Unlike a zonal cumulative analysis, this method requires the creation of a single I-I trip distribution table. To balance the trip productions and attractions and avoid double counting (since the trip generation process identifies trip ends, and every trip has two trip ends), the production and attraction trip tables must be averaged to result in a final I-I trip table.
5. The I-I trip table is combined with the I-E and E-I trip tables to address all identified internal growth. The E-E trips are then added to complete the trip table for all growth. The resulting trip table will be used as the input into the trip assignment tool. This procedure was followed for both the base and future years.

6.11.5 Trip Assignment

The use of Visum (or other assignment software) facilitates this trip assignment process by incorporating intersection delay which allows for the higher level of detail for approach delay. Route selection is set by the shortest path based on travel times between locations. The Visum travel times include both link (street) and node (intersection) components. The intersection delay in Visum is based on Highway Capacity Manual (HCM) methods. Including the concept of intersection turn delay allows for a greater level of detail than zonal cumulative analysis and allows for comparing future network alternatives. Because this process has more HCM intersection level-detail, less calibration of and post-processing adjustments of turn movements are needed. In some cases link delay (in addition to free flow travel time) may also be considered using a volume delay function (vdf) to account for delays on uncontrolled facilities.

Other link-based tools like EMME can be used in place of a turn-based delay component but require more detail in the vdfs than is typically included. The route selection is also set by the shortest path based on travel times between locations. Because this is a link-based delay, this method will take more calibration to match field conditions. Traffic assignments are applied in an iterative process until equilibrium is reached, like a travel demand model.

6.11.6 Calibration

The calibration process requires an iterative approach that assesses how well the EZCA tool is performing relative to real-world data (such as traffic counts). The integrated enhancement components allow for feedback in the form of "model error" if the tool is not performing well. Such error could be a result of many individual or combined factors, so it is important to have a systematic approach for calibrating and validating the tool prior to project application.

The calibration process includes three tiers, which should be approached *in the following order*:

1. Error Checking – Check for indication that an entry error has been made. Potential errors may include an allowable turn being closed in the travel network, incorrect stop sign orientation, or missing zone connectors, etc.
2. System Issues – Check for issues related to overall system assumptions. These issues may include the system-wide trip generation rate used for all households in the system.
3. Spot Location Issues – Issues that are localized and do not have a greater system effect (such as local circulation patterns in the assignment that that do not match turn movement data and local circulation patterns).

Calibration Measures

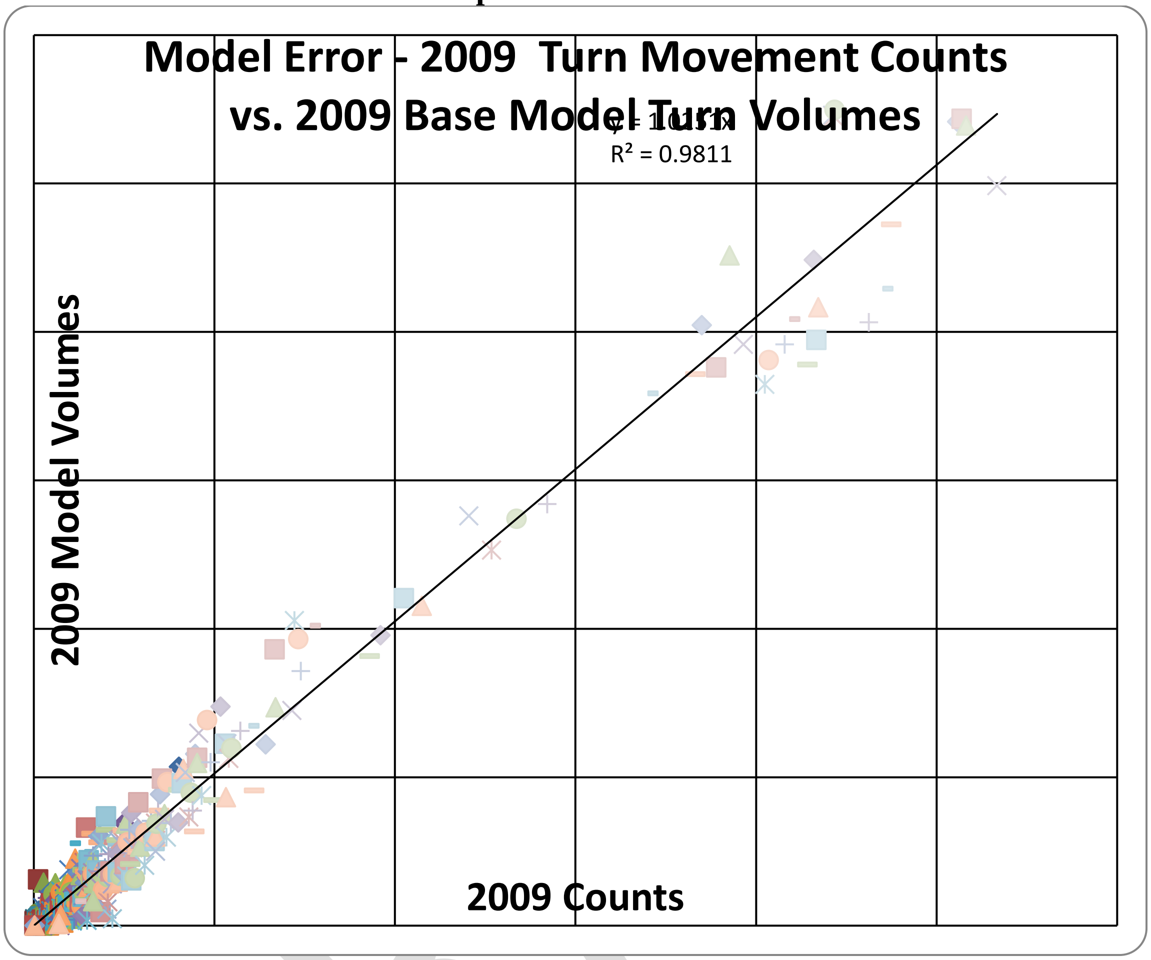
Qualitative calibration checks outlined in Chapter 8 (Mesoscopic Analysis) should be used depending on the scale of the traffic assignment network. Typically, a windowed subarea-model will provide a similar approach. However, since the EZCA is not based on a travel demand model, there are additional checks and considerations.

The following series of checks should be used through the calibration process.

- Do assigned traffic patterns match general routes that are observed?
- Do traffic counts match data for roadway segments and the specified intersection turn movements? Calibration compares assigned volumes to the base year 30th Highest Hourly Volumes (30th HV) on links and nodes. Typically the volumes should be within 10% of each other. Major outliers should be investigated and remedied, as feasible.
- Do systematic comparisons of model error indicate that the tool is generally reflecting count data? A plot comparing the existing traffic counts and the base year tool volumes for all study intersection turn movements should be analyzed to evaluate the accuracy of the tool. In addition to a visual verification of the goodness of fit, the following quantitative measures should also be considered:
    - The slope of the fitted curve should be close to 1.0. This value is generally indicative of how the overall magnitude of traffic estimated by trip generation compares to the traffic counts. A value of 1.2 would indicate that the estimated volumes are generally skewed 20 percent higher than the existing counts.
    - The R^2 value should also be close to 1.0. This value generally indicates how well the estimated volumes fit the observed, target volumes.

Exhibit 6-12 demonstrates a plot of the estimated turn movement volumes with the observed (counted) volumes. The slope of the fitted curve is 1.025, indicating that the model volumes are generally only two percent higher than the existing counts and that the trip generation is appropriate and does not require further refinement. Furthermore, the R^2 value of 0.976 indicates that the model volumes are consistent with the target volumes.

Exhibit 6-12 Calibration Plot Comparison of Observed and Estimated Turns



Both system metrics are equally important and should be considered. An R2 value of 1.0 with a slope of 1.5 does not indicate a calibrated tool (since volumes are consistently being overestimated by 50%).

Calibration processes may include the adjustments to account for the following items. The resulting impacts of adjustments to these parameters can easily be measured using the calibration measures to determine if the adjustment is beneficial to the overall calibration of the tool. Model calibrations are performed in an iterative process to determine the relative effect of each calibration measure.

- Pedestrian Conflicts - The pedestrians crossing a roadway influences the capacity and travel time of the corridor. The additional delay attributed to vehicles stopping for pedestrians can be emulated using “dummy traffic signals” at midblock locations.
- Street Speeds – Speeds on some links may need to be adjusted to account for driver behavior, such as increasing the speed on a downtown link to have it remain as a primary attractive route. Conversely, lowering the speed on a link will make it less attractive, so trips use other routes.

- Centroid Connectors – These connections may be needed to be added, deleted or changed to match the field operations within the study area. It may be determined that an initial approximation for centroid loading did not provide adequate detail and additional refinements (such as additional spreading of demand within the zone) may be needed.
- Trip Generation Rates – It may be that an individual land uses (such as residential or industrial uses) are isolated in the network with limited access, in a way that allows for determining how many actual trips (from count data) are being generated by the use. If the land use is sufficiently intense, it may provide adequate data to determine that a modification or reconsideration of the trip generation rates are needed. Any modifications of rates or uses need to be reviewed by TPAU. The following adjustments may need to be explored, in the following order:
    - Consideration for a "special trip generator" in unique cases that a location exists that isn't reflected in the overall land use and trip generation estimates. These uses could include major landmarks, recreational areas, or tourist destinations that have more traffic than would be otherwise estimated using typical land use categories and rates. An example could be a waterfront area or park that is regionally popular. Sufficient traffic count data should be available to estimate the trips generated by such uses. The application of such uses and rates would generally be limited to a single zone or small set of zones, relative to the overall number of zones in the network.
    - For more comprehensive or system-wide trip rate adjustments, the assumptions about representative ITE uses and rates (Section 6.9.4) may need to be reexamined. Such assumptions could include both the selected type of representative land uses as well as the overall share or weight within the category.
    - Finally, it may be determined that a local trip generation rate is a better fit than the national average rate[6]. Adjustments to account for local trip rates that differ from the ITE national average should be taken with care and only performed in cases where there is sufficient traffic data to support the adjustment. Such traffic data would typically include a combination of a network-wide calibration measure (slope) that indicates an overall trend of traffic estimation being too high/low and specific traffic count locations that serve to somewhat isolate a homogenous set of land use (for example a gated community or other large development, residential or commercial, with limited, consolidated access points). Adjusted rates should typically stay within 25% above/below the average ITE rate and include data that was used for the basis of the adjustment.
- Zone Trips (Land Use or Trip Rates) – Origin-Demand Matrix Estimation (ODME) can be used as a general tool to indicate zones or O-D pairs that are skewing the calibration of the model. While the raw results of the ODME should

[6] Adjustments made to trip rates should be made on the basis of traffic count data and taken with care. Areas that are forecasted to have larger amounts of land use growth have a higher sensitivity to adjustments that are made to trip generation rates.

not be applied directly to the model, running ODME can be used to discover zones that may require additional investigation into assumed land uses and/or trip rates for those land uses.

Because the EZCA process uses travel demand assignment software, the resulting volumes must be appropriately post-processed like other travel demand model outputs for formal traffic alternatives analysis[7]. However, depending on the amount of data and calibration success, turn-based post-processing may be feasible. See Section 6.12.1 for additional details on post-processing traffic volumes.

[7] Some preliminary alternative testing may be conducted within the forecast tool to inform a set of alternatives for further evaluation.

6.12 Urban Travel Demand Models

Chapter 17 provides an overview of travel demand models and their use in Oregon. The following procedures address how to apply traffic assignments from travel demand models in calculating future volumes for facility level analysis. If a travel demand model is available for the study area, then that is the preferred tool for future forecasting. If the study area is within a metropolitan area, then it is a federal requirement that the corresponding regional travel demand model is used.

6.12.1 Post Processing Methods – Future No-Build Link Volumes

This section presents the methodology to create future, no-build volumes from existing year counts. This methodology differs from the future, build alternative volume development methodology that is presented in Section 6.12.3.

Basic Steps – Future No-Build

The basic steps to developing Future No-Build link volumes are outlined below. A template spreadsheet tool is available to implement these steps on the Technical Tools page.

1. Start with balanced existing year DHV directional link volume set (i.e., volumes for both directions and not two-way link volumes).
2. Set up study area intersection street names and directions in a post-processing tool. It is recommended to include associated model "from node" and "to node" for each link if it is desired to use lookup tables for volumes and avoid manual entry of link volumes.
3. Acquire the directional link volumes from a travel demand model run (e.g., VISUM trip assignment) from a base year and from a future year model.
4. Document the project's existing year and design year as well as the model base year and future year.
5. If the project existing year is not the same as the model base year and/or if the project design year is not the same as the model future year, then the raw model link volumes need to be increased or decreased to the correct project years (see Example 6-17).
6. Apply growth, difference, and weighted growth equations between Base year and Future No Build year volumes. For each directional link, select growth, difference, weighted growth, or modified average methods following the guidance provided in the following sub-sections. This will result in preliminary directional link volumes (see Examples 6-18, 6-19, and 6-20).
7. Calculate the inflows and outflows at each intersection based on link volumes. Determine the difference between the flows. The inflows and outflows must be equal for each intersection. Split the difference (typically 50-50) to each of the flows by increasing or decreasing link volumes in proportion to the total flow volume. This can most easily be achieved by making use of the 'Inflow&Outflow' tab in the linked spreadsheet tool.

8. Determine intersection turn movements using the NCHRP 765[8] methodology with a tool such the Excel-baed [post-processing tool]() available on the APM website.
9. Re-balance network and round volumes to obtain Future No-Build DHVs.

Model Year Adjustment

If the project existing year is not the same as the model base year and/or if the project design year is not the same as the model future year, then the raw model base year and/or future year link volumes need to be adjusted (if the years align no adjustment is necessary). This adjustment is needed prior to selecting a method (growth, difference, or weighted growth). To adjust the volumes, calculate an annual growth rate for each link using the model base year and future year volumes. Then apply this growth rate to adjust the model volumes up or down as appropriate to align with the project years.

Note that some models may have official interim years that can also be employed for more extreme cases. Use the closest model year (either base, reference, interim, or future) to the desired project year (existing, build, design etc.) in order to maintain consistency.

Use the following formulas:

Annual Growth Rate = ((Future Year Model Volume / Base Year Model Volume) – 1) / (Model Future Year – Model Base Year)

Existing Year Model Volume = Base Year Model Volume * (1+ Annual Growth Rate * (Project Existing Year - Model Base Year))

Project Design Year Model Volume = Future Year Model Volume * (1+Annual Growth Rate * (Project Design Year – Model Future Year))

Example 6-17 Adjust Model Link Volumes to Match Project Existing and Design Years

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- Example Data:
    - Existing Year = 2020
    - Project Design Year = 2040
    - Model Base Year = 2019
    - Model Future Year = 2043
    - Raw Model Base Year Link Volume = 1,186
    - Raw Model Future Year Link Volume = 1,426

Annual Growth Rate = ((1,426/1,186) – 1) / (2043 – 2019) = 0.01

[8]NCHRP Report 765 may be found here: https://nap.nationalacademies.org/catalog/22366/analytical-travel-forecasting-approaches-for-project-level-planning-and-design

Model Base Year Link Volume Adjusted to 2020 = 1,186 * (1 + 0.01 * (2020 – 2019)) = 1,198
Model Future Year Link Volume Adjusted to 2040 = 1,426 * (1 + 0.01 * (2040 – 2043)) = 1,383

Growth Method

The Growth Method uses growth equations to calculate future design hour volumes. Caution should be used with this method as it may severely overestimate growth on links that have little volume in the base year and significant volume in the future year. The basic form of the growth method (per NCHRP Report 765, the replacement for the old Report 255) is:

Future DHV = (Future model year vol / Base model year vol) * Exiting Year 30 HV

Example 6-18 Post-Processing – Growth Method, No Build

- Example: 2040 DHV = (2040 Model Vol / 2020 Model Vol) * 2020 30 HV
- If the 2040 Model had 1,390 vph, the 2020 Model had 1,195 vph and the 2020 30 HV had 1,690 vph, the growth method using a simple linear method would be:

2040 DHV = (1,390/1,195) * 1,690 = 1,965 vph

Note that the future model year link volume is 1.16 times that of the base model year. Large differences like this will result in a volume overestimation and indicate that another method should be used. See the Selection of Method to Use Section below for further guidance.

Difference (Incremental) Method

The Difference (Incremental) Method should be used in areas where large differences (>25%) in base and future model link volumes exist. This method is preferred in NCHRP Report 765. The basic form of the difference method is:

Future DHV = Existing 30 HV + (Future model year – Base model year)

Example 6-19 Post-Processing – Difference Method, No Build

2040 DHV = 2020 30 HV + (2040 Model -2020 model)

Using the same data from the growth method above, using the difference method would be: 2040 DHV = 1,690 + (1,390 – 1,195) = 1,885 vph.

Comparing the results between this and the above growth method example show that the difference method creates a smaller volume.

Weighted Growth Method

The Weighted Growth Method is a modified Growth Method which incorporates both the Growth Method and the Difference Method. It is used when the difference between the base and future model link volumes are greater than 5%. Use the following equations:

Model Ratio (MR) = Future Year Model Volume / Base Year Model Volume

Future DHV = ((MR – 1) * Difference Method DHV + Growth Method DHV) / MR

Example 6-20 Post-Processing – Weighted Growth Method, No Build

- Example Data (using data from Growth and Difference Methods above):
  - Future Year Model Volume = 1,390
  - Base Year Model Volume = 1,195
  - Difference Method DHV = 1,885
  - Growth Method DHV = 1,965

MR = 1,390/1,195 = 1.16

2040 DHV = ((1.16 – 1) * 1,185 + 1,965) / 1.16 = 1,953

Modified Average Method

While NCHRP Report 255 discussed averaging the results from the ratio and difference methods, the updated NCHRP Report 765 no longer recommends this. However, NCHRP Report 765, Section 6.1 and Table 6-2 include a method which averages the Weighted Growth Method and the Difference Method. To avoid confusion this averaging method will be differentiated from the old method by being referred to as the Modified Average Method. The Modified Average Method should be used when there is a large (e.g. greater than 10%) difference between the Growth Method and the Difference Method.

Selection of Method to Use

All four methods should be calculated in a spreadsheet on a directional link basis using percent and absolute difference. The spreadsheet tool provided on the Technical Tools page has built-in formulas to help with choosing the appropriate method. The decision tree is as follows.

1. If the Model Ratio (Future Year Model Volume / Base Year Model Volume) is **greater than 25%**, then use the Difference Method. If the Model Ratio (Future Year Model Volume / Base Year Model Volume) is **less than 25%**, then continue to decision #2.

2. Compare the link DHV of the Difference Method with the Growth Method. If the absolute difference between these two DHV results is **less than an absolute difference of +/-10%,** then use the Growth Method.
3. If based on decision #2 the Growth Method is used, then determine if Growth Method or Weighted Growth Method will be used. If the Model Ratio (Future Year Model Volume / Base Year Model Volume) is less **than 5%**, then use the Growth Method. If the Model Ratio is greater **than 5%**, then use the Weighted Growth Method.
4. If the Model Ratio (Future Year Model Volume / Base Year Model Volume) is **less than 25%** and the absolute difference between Difference Method and the Growth Method DHV results is **greater than an absolute difference of +/-10%,** then use the Modified Average Method.

Note that the decision tree guidance above is not a definitive rule, but rather reliable guidance. NCHRP Report 765 suggests that the analyst should ultimately evaluate the results from all the methods and then select a preferred method based on the context of the existing year DHVs and turn movement volumes and knowledge of future growth in the area. However, the guidance above is a good place to start and then modifications may be made as necessary based upon engineering judgement.

The three previous examples show the potential difference between growth, weighted growth, and difference methods where the two growth methods may overestimate the future 2040 volume when the base year model volume is much smaller than the future model year volume. In the examples provided, the Weighted Growth Method would be used based upon the parameters outlined in the decision tree.

If the 2020 Model had 500 vph instead of 1,195 vph, the growth method would result in 4,700 vph, the weighted growth method would result in 3,345 vph, and the difference method would result in 2,580 vph. When the base and future models are much different, the results from the three methods are much different. In this case, the difference method would be used.

Adjacent link growth rates can be averaged together to reduce the number of calculations and/or adjustments necessary if they are close together (less than 10%).

Spreadsheet Application

A spreadsheet is necessary to track the post-processing calculations, especially on larger networks. It is advised that the Post-Processing spreadsheet tool be used. A sample spreadsheet area is shown in Exhibit 6-13. Directional links are represented as rows. Columns represent the post-processing calculations through Step 4. In this case, since the model base and future years do not match the project existing and future years, the volumes must be adjusted as described above.

The growth, difference, weighted growth, and modified average methods are then applied. In the spreadsheet, conditional formatting was used to help identify links where the difference method was selected. These are shown with red font and pink fill and indicate that the ratio of the future year to base year model link volumes exceeds 25 percent. If the ratio is less than or equal to 25 percent, then the growth or the weighted growth method will be used. The growth method is used where the ratio of the future year to base year model link volumes are within 5 percent. If the difference between the future year volumes calculated using the difference and growth methods exceeds 10 percent, then the modified average is used, and conditional formatting highlights these cells yellow.

Exhibit 6-13 Post-processing Spreadsheet, No Build

Existing Year	2020		Enter data in grey shaded cells
Project Design Year	2040		
Model Base Year	2019		
Model Future Year	2043		

Post-Processing - Future Year 2040 OR 99W Directional Link Volumes

*Use Diff when percent>25% (RED fill) **Use Grwth when model link vol w/in 5% ***Use Avg if percent diff>10% (YELLOW fill)

Link Identification				Balanced Existing Yr 30HV Counts	Model Volumes: Model Assignment (*enter manually if no access to Visum file*)		Model Volumes: Adjustment (model years to align w/ project years)			Post Processed Project Design Year Volumes 2040							
					2019	2043		2020	2040								
Route	Direction	From	To		Raw Model Base Year	Raw Model Future No-build	Annual Growth Rate (red when negative)	Model Base Adj. to Prj. Existing Yr	Model Future Adj. to Prj. Design Yr	Future/Base Model Ratio (red when >25%)	Percent Difference (yellow when >10%)	Difference Method*	Growth Method**	Weighted Growth Method	Modified Average*** Diff & Wted Grwth	Method Suggested	Final Future No-build 30HV Volumes
OR99W	SB	RRFB by Co-op	Crystal Lake	1691	1186	1426	0.008	500	1390	2.78	58.22	2581	4700	3343	2962	diff	2581
OR99W	NB	Crystal Lake Dr	RRFB	1287	801	932	0.007	807	913	1.13	4.44	1393	1457	1449	1421	wt grwth	1449
OR99W	SB	Crystal Lake Dr	Hopkins Ave	1620	1211	1535	0.011	1224	1484	1.21	4.37	1879	1963	1949	1914	wt grwth	1949
OR99W	NB	Hopkins Ave	Crystal Lake	1173	795	944	0.008	801	922	1.15	4.24	1294	1350	1343	1318	wt grwth	1343
OR99W	SB	Mayberry Ave	Alexander Ave	1308	1127	1469	0.013	1141	1414	1.24	2.49	1581	1621	1613	1597	wt grwth	1613
OR99W	NB	Alexander Ave	Mayberry Ave	996	728	880	0.009	735	857	1.17	3.82	1119	1162	1156	1137	wt grwth	1156
OR99W	SB	Alexander Ave	Viewmont Ave	1357	970	1343	0.016	986	1278	1.30	6.46	1650	1760	1735	1692	diff	1650
OR99W	NB	Viewmont Ave	Alexander Ave	954	638	790	0.010	644	767	1.19	5.32	1076	1135	1126	1101	wt grwth	1126
SW Avery Ave	EB	West extent	OR99W	360	182	238	0.013	184	229	1.24	10.10	405	448	440	422	avg	422
SW Avery Ave	WB	OR99W	West extent	175	148	126	-0.006	147	128	0.87	-2.36	155	152	151	153	wt grwth	151
Crystal Lake Dr	EB	OR99W	East extent	307	71	82	0.006	71	80	1.12	8.90	316	346	343	329	wt grwth	343
Crystal Lake Dr	WB	East extent	OR99W	165	68	67	-0.001	68	67	0.98	-1.36	163	161	161	162	grwth	161
Alexander Ave	EB	OR99W	East extent	94	143	112	-0.009	141	115	0.81	12.56	67	76	78	73	avg	73
Alexander Ave	WB	East extent	OR99W	72	92	89	-0.002	92	89	0.97	0.88	70	70	70	70	grwth	70
Alexander Ave	EB	West extent	OR99W	13	36	45	0.010	37	44	1.20	-25.28	20	16	17	19	avg	19
Alexander Ave	WB	OR99W	West extent	23	52	59	0.006	52	58	1.11	-12.51	29	25	26	27	avg	27

6.12.2 Assessing Induced & Latent Demand Effects

The methodology for post processing future build alternative volumes from future no-build volumes has steps which differ from post processing future no-build alternative volumes from the existing year counts (see Section 6.12.1 for guidance on creating future no-build volumes). Clear guidance for post-processing future build volumes is provided in Section 6.12.3. However, prior to calculating the future build volumes induced and latent demand effects must be assessed. This section presents the methodology for this assessment.

All forecasts involve uncertainty. When sources of uncertainty are known, it is important to evaluate potential impacts. For traffic forecasts this is especially important in order to avoid unexpected outcomes and ensure facility designs meet future goals and objectives. Demand effects are one of the biggest uncertainty issues surrounding today's current projects, especially in congested metropolitan areas. These come up frequently when analyses, project reports, or environmental documents are reviewed by a wide range of participants from federal agencies to the public. Definitions are continually confused or misused, and these can frequently become "hot-button" issues which can wreak havoc on project schedules. A consistent and clear definition of all types of demand, including induced and latent demand, is critical for the analyst to evaluate and communicate their potential effects. This section explains the different definitions of travel demand, the relevant context and background surrounding them, and how to analyze the effects of various projects on demand.

This section applies to all projects especially those with applicable National Environmental Policy Act (NEPA) classifications (i.e. Categorical Exclusion, Environmental Assessment and Environmental Impact Statement). It also applies to transportation system plans and ODOT corridor, facility and refinement plans. Transportation Planning Rule (TPR) plan amendment application is optional, however, anything that is deemed to have a significant effect and triggering TPR requirements should consider it as induced and latent demand effect questions may arise in the approval process.

Definitions

- **Forecasted (planned) demand –** represents expected demand given forecast land use, economic growth, and the available transportation network, which is based on city and county comprehensive plans and reflected in the zoning code. Assumptions that underlie project alternatives need to be consistent with comprehensive plans. Travel demand models use comprehensive plan land use and transportation availability assumptions to forecast travel demand, which provides housing, economic development, and urban land supply context within the travel model.

- **Latent Demand** - this is demand for transportation that consumers do not utilize because they cannot afford the cost, or it is not currently available. Latent demand responses are typically associated with network limitations, such as capacity constraints, incomplete networks for non-auto modes, sub-optimal operational performance, travel time costs and reliability on a specific route. Network investments that resolve limitations impact the cost of using a route and generate potential change in system use – shifting routes, time of day, mode choice, and destinations. System investment that reduces travel time and cost may result in users changing their use of the system. For example, trips that have shifted the time of day that they travel may move back into the peak hour. Trips that are diverted to other travel routes may shift back to the improved route. Latent demand is also referred to as *pent-up demand* in the field of economics. Latent demand does not include induced demand.

- **Diverted demand** – this is a common term that defines one aspect of latent demand. These terms may be used to describe the same phenomenon. This is existing system demand diverting from typical patterns related to routes and time of day, and day of week. This is the largest aspect of what typically makes up latent demand effects in Oregon and typically occurs over a short time span as users readjust their travel patterns.

- **Induced demand** – new demand for travel that did not exist prior to the build scenario. This is above and beyond forecasted and latent demand associated with planned land use, it is demand that is the result of changes in land use (zone changes) or economic conditions that create new trips. Induced demand can be identified on a regional scale when there is an increase in demand that cannot be attributed to other types of demand, such as demand diverting from one facility to another. Induced demand typically occurs over a long time span and may relate to land use changes outside of the study area or a reduction in the cost of accessing destinations previously too costly to access.

When reviewing and considering these definitions it is critical to keep in mind that they apply to any travel mode. Most users may typically think of how the auto mode reacts to new transportation infrastructure, but similarly, the walk, bike, transit, or freight modes apply within these definitions.

For example, if an improved bicycle facility is added and bike trips redirect or a mode shift occurs, then that is latent (pent-up) demand for that bicycle infrastructure. If a freight bottleneck is eliminated and brings back diverted freight trips, then that is also latent demand. If a large light rail project is built that spurs new transit-oriented-development (TOD) land use changes beyond what was originally planned, then that is induced demand for transit ridership and the other modal trips that come along with that new land use.

The key statement here is that latent and induced are mode-neutral terms. The key difference is whether a land use shift is occurring that is different than previously projected (planned) land use growth due to the transportation investment. That transportation investment can be for any mode.

Demand Context

The land-use/transportation/economic relationship is dynamic. Land too expensive to develop now may hit a trigger point in the future that spurs development earlier than expected. Investment (or lack of) in one transportation facility may impact system use region-wide in unexpected ways. Changes in travel behavior can result in users choosing new routes, new times of the day to travel, reducing or adding new trips.

It is important to understand why demand exists and its relationship to the transportation system. The purpose of the transportation system is to accommodate movement of people and goods for household and business activity. Investment decisions must take into consideration many statewide goals and objectives, while meeting federal mandates, efficient use of scarce resources and minimizing negative outcomes.

Transportation users represent a large set of individual decision makers with their own goals and objectives related to personal travel, business travel and freight movement. Households choose where to live to meet accessibility needs, such as access to jobs, schools, shopping and recreational activity within their budgets for time and money. Businesses locate to access goods and services they need for their business, acquire workers, and reach customers within their budgets for time and money. Meeting these economic needs results in demand for transportation locally, regionally, nationally and internationally.

In general, Oregon faces low risk related to induced demand because of the state's strong land use laws, which exist to prevent sprawl. Changes to land use must be approved by local jurisdictions, so a facility project cannot induce demand just by itself. However, it should be noted that the presence of a new facility regardless of jurisdiction could increase pressure on the land use and or economic trigger/tipping points. Local jurisdictions could approve future zone changes above the forecasted demand in response to economic development pressures, for example, so most of the backstop against future demand effects will fall to local governments. Dense land use developments built in response to improved multimodal facilities (e.g. high frequency/capacity transit) could end up producing more vehicle trips (as not all new residents would use transit) than in the original planned uses.

Land use planning in Oregon consists of a system of laws and government collaboration after voters approved the overall framework in 1973. This statewide coordination process now preserves vast areas of land for farm and forest production, protects habitat, conserves natural resources, and protects air and water, all while continuing to allow development of land for homes and businesses. Comprehensive land use planning in Oregon is based on 19 statewide land use goals, including transportation.

Travel demand models are designed to represent behavioral changes in response to infrastructure investments within the context of forecasted/planned land use, because travel involves many different users with different transportation needs. Changes to the system via plans and projects that affect user cost (time and money) will impact demand. The challenge is to predict what the net impact will be on the system and determine whether mitigation to manage negative impacts is called for.

It is important to assess whether there is forecast risk associated with induced and latent demand. Measuring the two individually is not actually necessary, because the objective is to identify unexpected future demand with potential to impact the project design needed to accommodate future traffic volumes. However, measuring them individually may be necessary to answer specific questions or project-area concerns or to be proactive against future review. Generically, this is based on comparing model assignments between the future build alternative and the future no-build scenario, within the context of the same land use forecast from local comprehensive plans.

Analyzing for Induced Demand

Induced demand is typically land-use-based and occurs over a long time span, making it more difficult to assess its potential impacts. Uncertainty associated with land use comes from multiple areas. Some impacts may arise from development occurring outside of the study area generating new trips through the study area. While the roadway jurisdiction has influence over some factors (e.g. roadway network improvements or lack of) that could contribute to induced demand, local agencies have greater influence through land use planning, economic development, and local transportation investments. As the state economy continues to grow, more people and businesses will lead to increased demand on the transportation system. How those trips are made, and the overall uncertainty surrounding them, will be governed by geography, land use, and the economy as well as the cost, convenience, and availability of different modes.

There are some considerations that should be assessed on a project/plan and/or alternative basis:

- Areas that are under development pressure from nearby larger urbanized areas;
- Areas within the study area close to development "trigger points" where changes to local market forces or economic factors (rents, development costs, energy costs, regulations) may induce development earlier than expected;
- Land-use zoning changes may attract development of a different form than expected with the original zoning, potentially inducing new travel demand;
- Evidence of systemic congestion with relatively high (12+) Average Daily Traffic to (hourly) Capacity (ADT/C) ratios (see Chapter 9) but with limited potential for diverted demand

A screening-level prediction of induced demand impacts can be done by running an unconstrained (i.e. infinite capacity) model assignment. This will indicate the paths, routes, and mode choices that drivers would want to use if capacity was not an issue. More information on unconstrained demand model runs is in Chapter 17 (Section 17.4). A related topic that usually comes up when discussing demand is determining what trips are local versus what is regional or long-distance through trips. This can be done using a series of select-link analyses and zones grouped into districts (see Sections 10.8.2 and 17.4).

This unconstrained assignment would be compared with the regular capacity-constrained assignment by comparing the total amount of volumes on key links or across screenlines summing across multiple facilities surrounding the project study area. This could indicate an induced demand potential if the unconstrained assignment was 10% or higher over the constrained scenario at those locations. This would show the potential for attracting additional volume into the study area if capacity was improved. There could be latent demand effects mixed in so actual project induced demand potential for reporting out would require a more specific analysis using the Oregon Statewide Integrated Model (SWIM).

SWIM can be used to evaluate the induced demand potential for changing land use and related economic conditions over time based on either a no-build or a build alternative scenario. Any build scenarios should be constructed to be as consistent as possible to the urban travel demand model scenario, keeping in mind that some models will have more detail than is possible in SWIM. Minor features and/or roadways may not be capable of being included within a SWIM scenario; however, the absence will likely be insignificant compared to the regional aspect of this analysis. SWIM can also be used to evaluate for induced demand effects in urban areas not covered by a travel demand model. See Sections 7.4 and 17.2.4 for more information on SWIM. Section 7.4.5 contains an example of an application that was done for determining the potential for any induced demand effects on the roadway system, land use or the economy for Newberg-Dundee Bypass.

STOP *There are some available high-level induced demand calculators available such as from the National Center for Sustainable Transportation (University of California at Davis) or variants developed for other areas. These are simple calculators that use a proportional relationship between empirical gathered lane miles and vehicle-miles traveled for different roadway types and geographies along with an elasticity constant. The use of these calculator types shall not be used to estimate induced and latent demand effects on ODOT-funded projects as they are inappropriate for use in Oregon.*

These do not take land use, location within the regional network, economics, population/demographic changes, route shifting, trip purpose, extra space needed to accommodate complex merging and weavings, or any specific improvement details into account nor do they consider the impacts of Oregon's land use planning laws to prevent sprawl effects. Latent demand is mixed into these and called "induced" in attempts to simplify. As such, it is unlikely that the results will be consistent with approved evaluation tools such as SWIM and will be potentially misleading.

Analyzing for Latent Demand

Latent demand can occur where the future no-build demand has reached or exceeded capacity, and a portion has shifted to other routes, destinations, modes, or time periods to avoid congestion. Once the facility is at capacity, peak hour volumes no longer increase over time, while latent demand may continue to grow.

As was done with screening locations of induced demand potential, the same may be done for latent demand using an unconstrained model assignment. This will indicate the paths and routes that drivers would want to use if capacity was not an issue. If large shifts in volume are noted from going from one facility to another, then the destination facility, if improved by a build alternative, would likely be a location impacted by latent demand. The scope of the build alternative will determine how large the latent demand shift could be. This scoping test could be used to help size alternatives if latent demand shifts are an issue as it is possible to add too much improvement which shifts too many or the wrong type of trips (e.g. short local trips moving onto the highway facility meant for through or regional trips).

When a build alternative alleviates congestion, a portion of the shifted demand may return, which is reflected by an increase in the future build volumes. Alternatives that add capacity to a system that is at or over capacity, especially in an urban area, should be checked for the potential for latent demand. Areas with persistent or systemic congestion (indicated by ADT/C ratios of 12 or more) could be also checked across the peak and off-peak periods if the model used was compatible.

Determining any latent demand effects may be facilitated by requesting a plot from a small urban or regional travel demand model that compares the build and no-build scenarios (e.g. showing the absolute differences or relative percentages) for each directional link. If the volume difference on each link is not significant, i.e., less than 10%, then the future no-build DHVs may be assumed to be the future build DHVs. If the difference is 10% or higher (meaning there appears to be a latent demand effect), the appropriate growth or difference post-processing method should be determined and applied to the future no-build DHVs as shown in either of the two processes shown below.

Documentation Needs

If analysis is done to analyze for induced and/or latent demand, then this needs to be discussed in the future volume development section of the memorandum or report narrative. The results of the analysis whether it meets the significance thresholds or not must have the determination stated and clearly explained. This is especially important for documentation that will be eventually added to environmental assessments and environmental impact statements as this will be an area that federal, state, and local agencies as well as members of the public, and advocacy groups will cover closely in their review. Supporting calculations, plots, etc. should be available in appendices or in the technical records for future reference.

There should be, at the very least, a qualitative discussion with regional planners of the potential for induced and latent demand impacts in the appropriate documentation, if no model runs are performed. As a reminder, any federal or state transportation planning efforts in Oregon need to adhere to the urban growth boundary and current comprehensive plan zoning when considering land-use effects. It would also help to include language in summary statements such as, "changes to future land use beyond the comprehensive plan assumptions used could cause the future year volumes to be realized earlier which may impact the overall design life of the project alternative", which infers this uncertainty in the forecast.

6.12.3 Post Processing Methods – Future Build Alternatives Link Volumes

This section presents the methodology to create future build alternative volumes from future no-build DHVs. Prior to post-processing future build alternative volumes, two steps must be taken:

1. Post process future no-build volumes (Section 6.12.1).
2. Assess induced and latent demand effects (Section 6.12.2).

After these two steps have been completed, future build volumes may be post-processed.

Basic Steps - Build Alternatives

The basic steps to developing future Build link volumes links are outlined below.

1. Start with Future No-Build DHV directional link volume set.
2. Review the Build Alternative volume assignment plots to see whether any manual reassignments are needed. For example, manual reassignment may be necessary when there are parallel network issues and/or when zero volume is assigned to a link which may not make sense in the actual study area.
3. Compare the model future No-Build and Build scenarios for latent demand effects. [*Note that No-Build as used here may not be the exact same scenario as the typical Model No-Build that does not have any projects included. The requested output needs to have the appropriate projects identified and included in the scenario run.*] To facilitate comparisons, volume difference plots (between Build and No-Build scenarios) may be obtained. An unconstrained model run (infinite link capacity) should be obtained to screen for induced or latent demand effects. If the link volume differences are less than 10 percent, no further adjustments are needed and the No-Build DHVs may be treated as equivalent to the Build DHVs. However, Build DHVs should still be developed for documentation.
    a. If the link volume differences are greater than or equal to 10 percent, then the Build scenario should be assessed for induced demand in the Statewide Integrated Model (SWIM). Apply any percentage adjustments to applicable facilities for potential induced demand effects.
4. Adjust model year volumes to match project future year. The model future year must be adjusted up or down to match the project future year.

5. Apply growth, difference, and weighted growth equations between No Build and Build scenarios (see examples below). For each directional link, select growth, difference, or weighted growth methods following the guidance provided previously. This will result in preliminary directional link volumes.
6. Use screenlines (see Section 6.12.4) to determine volumes on any links that are not in both No-build and Build model scenarios
7. Calculate the inflows and outflows at each intersection based on link volumes. Determine the difference between the flows. The inflows and outflows must be equal for each intersection. Split the difference (typically 50-50) to each of the flows by increasing or decreasing link volumes in proportion to the total flow volume.
8. Determine intersection turn movements using a tool such the Excel post-processing tool available on the APM website.
9. Re-balance network and round volumes to obtain Future Build DHVs.

Growth Method:

1. Divide the Future Year Build model scenario volume by the Future Year No-Build model scenario volumes to derive the factor.
2. This factor is then multiplied by the Future Year No-Build Design Hour Volume to arrive at the Future Year Build Design Hour Volume (DHV).

Example 6-21 Post-Processing – Growth Method, Build

- Example: 2040 Build DHV = (2040 Model Build / 2040 Model No-Build) * 2040 No-Build DHV
- If the 2040 Model No-Build had 800 vph, the 2040 Model Build had 1,000 vph and the 2040 No-Build DHV had 1,600 vph, the growth method using a simple linear method would be:

2040 Build DHV = (1,000/800) * 1,600 = 2,000 vph

Note, for this link, the 2040 Model Build scenario was 25% greater than the 2040 No-Build scenario. This means that separate build volumes must be developed. Use of the no-build volumes for the build volumes is not permitted if the change is greater than 10% (note that best practice is to always develop build volumes even with a change of less than 10%. In other words, it is not required but it is best practice to execute the post-processing methodology and develop build DHVs even with a change less than 10%). The difference may be due to a latent demand shift if it is above and beyond any pure re-routing impacts because of physical network changes (new or removed roadways).

Difference Method:

1. Subtract the Future Year No-Build model scenario volume from the Future Year Build model scenarios volumes *(Note: this change in volume can be either*

positive or negative).

2. Add the difference to the Future Year No-Build Design Hour Volumes (DHV).

Example 6-22 Post-Processing – Difference Method, Build

2040 Build DHV = 2040 No-Build DHV + (2040 Model Build - 2040 Model No-Build)

Using the same numbers from the growth method above, using the difference method would be: 2040 Build DHV = 1600 + (1000 – 800) = 1,800 vph.

Like with Example 6-19, the build volume is greater than a 10% change over the no-build volumes which could indicate a potential impact from latent demand.

Weighted Growth Method:

1. Divide the Future Year Build model scenario volume by the Future Year No-Build model scenario volumes to derive the ratio.
2. Subtract 1 from the ratio.
3. Multiply the (ratio -1) by the difference method result to create a weighted difference.
4. Add the weighted difference to the growth method result and divide by the ratio created in step 1.

Example 6-23 Post-Processing – Weighted Growth Method, Build

2040 Build DHV = {[((2040 Model Build / 2040 Model No-Build) – 1) *(2040 Difference Method DHV)] + [2040 Growth Method DHV]} / {2040 Model Build / 2040 Model No-Build}

Using the same numbers from the growth method and difference method above, using the weighted growth method would be:
2040 Build DHV = {[((1,000 / 800) – 1) *(1,800)] + [2,000]} / {1,000 / 800} = 1,960 vph

Note that the weighted growth method DHV result is between the growth method and the difference method results. This may be preferrable as a less extreme result but will depend on knowledge of the study area characteristics and expected impacts of projects included in the build alternatives.

6.12.4 Screenlines

Screenlines can be used for calculating overall growth rates or used for calculating volumes on new links (links that only exist in one of the scenarios that you are comparing). This could occur either as new routes added in the future no-build when compared to the base year or as new routes added to the future build when compared to the future no-build.

Screenlines are useful where there are significant differences in growth within the study area. Screenlines should be strategically placed to cross the major links of the different growth areas. Screenlines are drawn the same in both the base model year and the future model year on the model volume plots. The link volumes crossing each screenline are summed. The summation of each future screenline is divided by the corresponding base screenline summation. This provides the growth rate for the different areas cut by the screenlines.

New Links

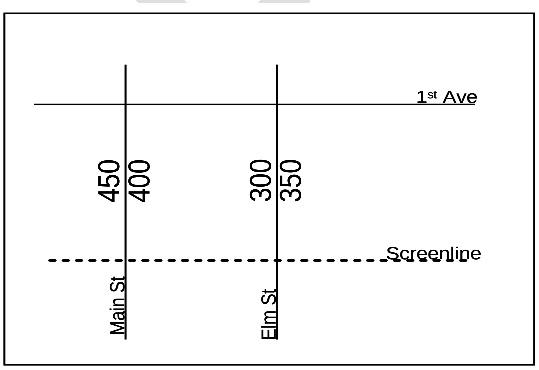
The main use of screenlines is to determine the future design hour volume of links that exist only in one scenario. This comes up when a new route is added to a scenario. This can occur with both future no-build and build alternative scenarios.

Example 6-24 Post-Processing – Use of Screenlines

For example, assuming a roadway network in the future no-build year had two north-south links at Main Street and Elm Street, but a build alternative added a new north-south link at Oak Street for a total of three north-south links. The analyst has the model outputs for the scenarios with and without the new connection and the future no-build DHV diagram (without the connection).

1. Draw a screenline across Main and Elm Streets in the future no-build model scenario and sum up the future no-build model volumes for each street as well as the total north-south future model volume.

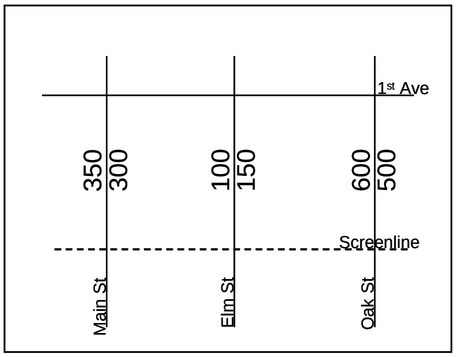
Future No-Build Model Scenario



Main Street Total = 450 + 400 = 850 vph
Elm Street Total = 300 + 350 = 650 vph
Total North-South Volume = 850 + 650 = 1500 vph

2. Draw a screenline across Main, Elm and (new) Oak Streets in the build alternative scenario and sum up the build alternative model volumes for each street as well as the total north-south build alternative model volume. Calculate the street (link) splits and directional (northbound and southbound) splits for all three streets.

Build Alternative Model Scenario



Main Street Total = 350 + 300 = 650 vph
Elm Street Total = 100 + 150 = 250 vph
Oak Street Total = 600 + 500 = 1100 vph
Total North-South Volume = 650 + 250 = 2000 vph

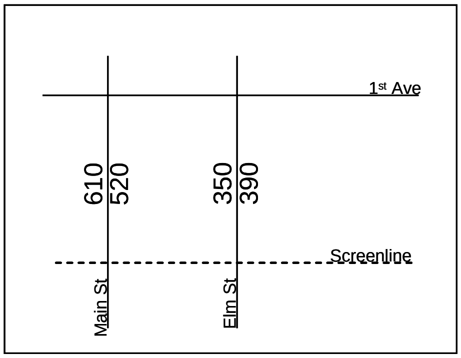
Note: in this example, the build alternative scenario pulls in 33% more traffic (2000 vph vs. 1500 vph) than the future no-build model scenario. This is a result of previously diverting traffic returning to the route that it wants to use. In this case the analyst must use the build alternative model scenario to create the future design hour volumes. If the difference was less than 10%, then the analyst could use the future no-build volume distributed on the build network.

Link Split Calculation:
Main Street split = 650/2000 = 0.325 (32.5%)
Elm Street split = 250/2000 = 0.125 (12.5%)
Oak Street split = 1100/2000 = 0.55 (55%)

Directional Splits Calculation:
Northbound Main Street = 300/650 = 0.46
Southbound Main Street = 1 – 0.46 = 0.54
Northbound Elm Street = 150/250 = 0.60
Southbound Elm Street = 1 – 0.60 = 0.40
Northbound Oak Street = 500/1100 = 0.45
Southbound Oak Street = 1 – 0.45 = 0.55

3. Draw a screenline across Main and Elm Streets in the future DHV diagram and sum up the total future DHV volumes for each street as well as the total future north-south DHV volume.

Future No-Build DHV



Main Street Total = 610 + 520 = 1130 vph
Elm Street Total = 350 + 390 = 740 vph
Total North-South Volume = 1870 vph

4. Calculate the total north-south build alternative DHV by creating a ratio by dividing the build alternative screenline grand total (Main/Elm/Oak Streets) by the future screenline grand total (Main/Elm Streets) and then multiplying this ratio with the future DHV screenline total (Main/Elm Streets).

General equation form:

$$\frac{Build\ \ Alternative\ Model}{Future\ \ No-Build\ \ Model} = \frac{Build\ \ Alternative\ DHV}{Future\ \ No-Build\ \ DHV}$$

Modified equation to solve for the total Build Alternative DHV:

$$Build\ \ Alternative\ DHV = \frac{Build\ \ Alternative\ Model}{Future\ \ No-Build\ \ Model} \times Future\ \ No-Build\ \ DHV$$

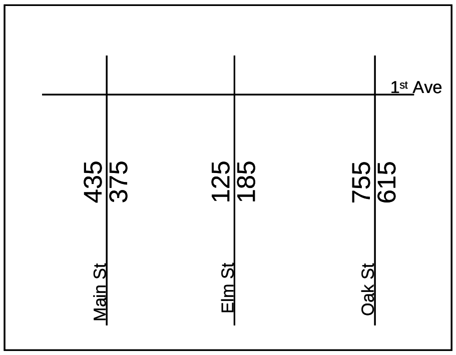
Build Alternative DHV = (2000/1500) x 1870 = 2493 vph

5. Using the Build Alternative DHV (from Step 4) and the link splits (from Step 2), determine the volumes for each of the streets. Oak Street would be calculated by:
Build Alternative DHV (Oak) = Oak Street split (from Step 2) x Build Alternative DHV total (from Step 4)

Build Alternative DHV (Oak) = 0.55 x 2493 = 1371 vph
Build Alternative DHV (Main) = 0.325 x 2493 = 810 vph
Build Alternative DHV (Elm) = 0.125 x 2493 = 312 vph

6. Compute the directional DHV for each street by applying the directional splits from Step 2 to the DHV totals for each street calculated in Step 5.

Future Build DHV



Main Street DHV (NB) = 0.46 x 810 = 373 vph (rounded to 375)
Main Street DHV (SB) = 0.54 x 810 = 437 vph (rounded to 435)

Elm Street DHV (NB) = 0.60 x 312 = 187 vph (rounded to 185)
Elm Street DHV (SB = 0.40 x 312 = 125 vph

Oak Street DHV (NB) = 0.45 x 1371 = 617 vph (rounded to 615)
Oak Street DHV (SB) = 0.55 x 1371 = 754 vph (rounded to 755)

6.12.5 Turn Movements

Once all the link volumes have been determined from the standard or screenline post-processing methods for either no-build or build scenarios, turn movements need to be calculated for each intersection.

Intersection volumes are represented at the turning movement level. It is important to note that the model is built and calibrated at the link level, not the turn level. Turn movement outputs from the model shall not be used directly in software for analysis. TPAU will not provide turning movement volumes for model requests because they are typically misused to directly post-process turn movement volumes instead of following the link based post-processing procedure. Turn volumes should be developed from post-processed link volumes according to the procedures below.

Future No-Build Turn Volumes

The volume of traffic entering an intersection can be unequal to the volume of traffic leaving an intersection due to the directional forecasting of each link. The total volume entering an intersection must balance with the volume leaving the intersection. If an imbalance exists, then inflow/outflow volumes will need to be balanced proportionally.

Typically, the future no-build turning movement volumes make up the same proportion of the overall approach volume as the existing (base) year movements. In most cases the volume of a future turning movement can be initially estimated using the percentage corresponding to the existing turning movement relative to the approach. However, if there are new links, or other significant network differences, then it is best to use the no-build model turn percentages from VISUM flow bundles instead of the existing turning movement proportions. Impacts from other projects should show up in the future no-build model assignment and would impact the growth rate or methodology used to calculate the future no-build design volumes.

It may be needed to manually adjust the turning movement proportions based on the impact of projects. The turning movement percentages provided by the Future Year No-Build model VISUM flow bundles can help point out some of these impacts. *(Note: A capacity constrained demand model should indicate the shift in travel patterns and the directional link volumes from the model should be used as a starting point to arrive at a future DHV. Arriving at a post-processed set of volumes requires a method such as described above or in NCHRP 765).* The origin and destination matrix can also be a helpful tool to obtain the distribution of trips between zones. Model runs with and without committed/STIP projects can be run to determine the impacts, if any, from nearby financially constrained future projects. After turn movements have been developed, the network needs to be re-balanced, and volumes rounded to obtain Future No Build DHVs.

Future Build Turn Volumes

At this point the directional design hour volumes have been calculated for each link, so the intersection approach entering and exiting volumes are known. The traffic volume entering each intersection should balance with the traffic leaving the intersection. If an imbalance exists, then inflow/outflow volumes will need to be balanced proportionally.

Build turning movements are a combination of the known travel pattern changes and the existing turning movements. Model flow bundles are a model attribute which identifies link volume turn movements. Model flow bundles can be used as a starting place. Flow bundle plots will need to be requested to verify or modify the turning percentages (or can be created on the fly if access to the VISUM file is available). In some cases, developing the build turning movements is not a straightforward process. It may involve reviewing all the turning movements and using engineering judgment and knowledge of the area. After turn movements have been developed, the network needs to be re-balanced, and volumes rounded to obtain future Build DHVs. Balancing adjustments to a turn movement may result in changes to other movements on the entering or exiting leg. These changes are necessary to preserve the link volumes. The post-processed link volumes need to be held constant as much as possible to preserve the future growth ratios and/or differences.

Matrix-based programs are the preferred method to assign turning volumes based on the link volume. Tools should reflect the NCHRP Report 765 Chapter 5 (Fratar volume balancing method) turning movement process. The intersection approach link volumes and the exiting link volumes are entered (after link post-processing is complete). ODOT's spreadsheet tool on the Technical Tools page goes through an iterative process to closely match the data that were entered. It is important that inflows equal outflows for each intersection. If an imbalance exists, then inflow/outflow volumes will need to be balanced proportionally.

Turn Movement Volume Development Matrix Iteration Tool

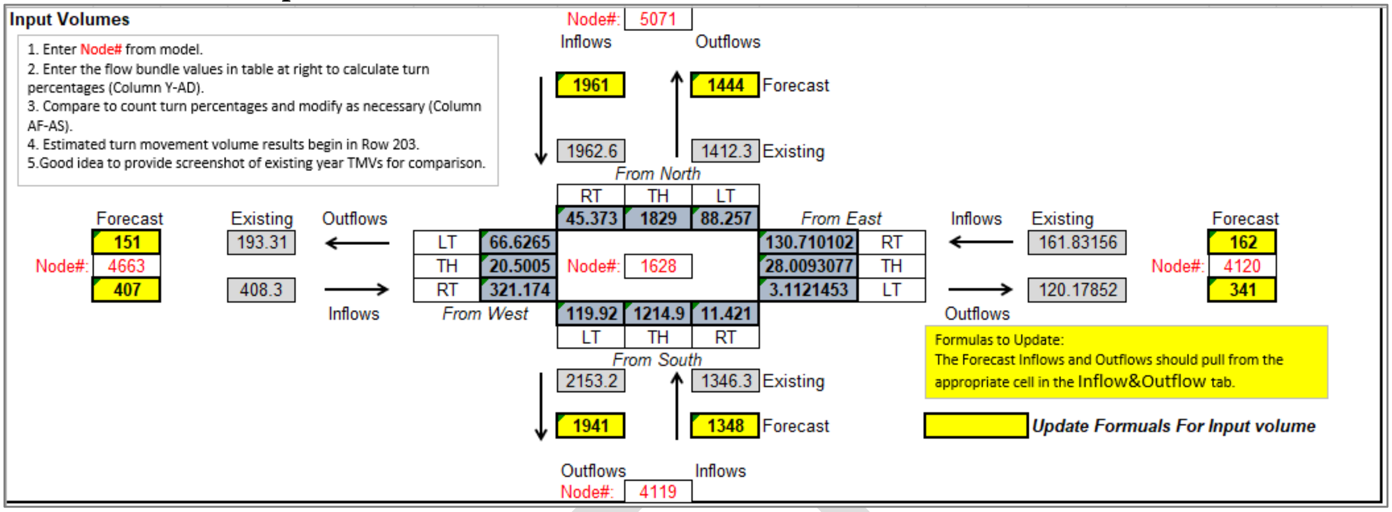
This tool is an Excel-based spreadsheet tool which implements a turning movement matrix and steps through balancing iterations to land on estimated turning movements. This tool is based on an Excel-based NCHRP 255 turn movement calculator using an iterative method which has been modified to reflect NCHRP Report 765 updates. Prior to use, flows in and out of the intersection must be balanced so that inflows equal outflows.

The turn volume development spreadsheet is depicted in Exhibits 6-14, 6-15, 6-16, and 6-17. Exhibits 6-14 and 6-15 depict the section of the spreadsheet where the necessary data is entered for the turn movement development. The data required include:

- Node numbers,
- Link volume inflow and outflow (post-processed and inflow/outflow balanced), and
- Turn percentages from flow bundles or existing year counts (as appropriate).

Preliminary future link volumes with balanced inflows and outflows are populated in the yellow cells by pulling data from the Inflow&Outflow tab in the spreadsheet tool. Do not use unbalanced inflows and outflows.

Exhibit 6-14 Data Input for Turn Movement Iteration



The turn percentage values used as the seed for the Turn Movement Volume (TMV) iterations should be gleaned from the VISUM flow bundles. The existing year count turn percentages should be referenced to assure flow bundles make sense based on projects implemented (future build) and/or study area knowledge. These values may be entered in the table to the right of the data input section depicted in Exhibit 6-14. This is illustrated in Exhibit 6-15.

Exhibit 6-15 Turn Percentage Input for Turn Movement Iteration

Enter Flow Bundle Link Value

Flow Bundle Turn Percentages

Leg	Inflow	Movement	Outflow	Future No-Build Turn %	Existing Count Turn %
		SBL	62	0.043	0.128
North	1426	SBT	1333	0.935	0.843
		SBR	33	0.023	0.029
		WBL	1	0.015	0.061
East	67	WBT	10	0.149	0.121
		WBR	42	0.627	0.818
		NBL	84	0.089	0.091
South	944	NBT	851	0.901	0.895
		NBR	8	0.008	0.014
		EBL	39	0.164	0.283
West	238	EBT	12	0.050	0.206
		EBR	188	0.790	0.511

Below the data input section, the spreadsheet tool conducts six row and column iterations to produce estimated turning movements. An example of the first iteration is shown in Exhibit 6-16. Check that the percent change in the final iteration is less than 10%. If it is greater than 10% more iterations should be added.

Exhibit 6-16 First Iteration for TMV Calculation

Initial Turning Movement Matrix, Tijb

	Forecasts		East Leg	North Leg	West Leg	South Leg	
			341	***1444***	***151***	***1941***	***Djf***
		Counts	120.18	1412.27912	193.30597	2153.237	Djb
East Leg	***162***	161.831555	0	130.710102	28.009308	3.112145	
North Leg	***1961***	1962.58045	88.257	0	45.372849	1828.951	
West Leg	***407***	408.30116	20.5	66.6265491	0	321.1741	
South Leg	***1348***	1346.2876	11.421	1214.94246	119.92382	0	
	Oif	Oib					

outflows, *j*

inflows, *i*

first row iteration

	Djf*			
	119	1413	193	2150
161.831555	0	131	28	3
1960.65707	88	0	45	1827
406.592787	20	66	0	320
1347.71526	11	1216	120	0

compare

	Djf*	Djf	change
j=1	119	341.311677	-65.0%
j=2	1413	1443.80517	-2.0%
j=3	193	150.524957	28.0%
j=4	2150	1941.15487	11.0%
Totals	3875	3876.79667	

first column iteration

	341.31	1443.80517	150.52496	1941.155
159	0	134	22	3
1937	252	0	35	1650
413	57	67	0	289
1369	32	1243	94	0
Oif*				

compare

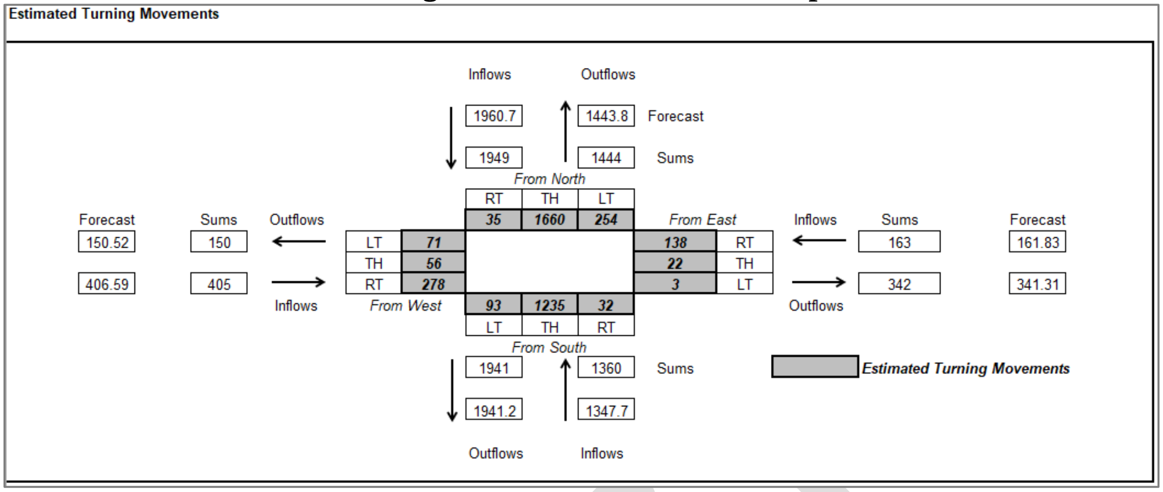
	Oif*	Oif	change
i=1	159	161.831555	-2.00%
i=2	1937	1960.65707	-1.00%
i=3	413	406.592787	2.00%
i=4	1369	1347.71526	2.00%
Totals	3878	3876.79667	

second row iteration

	343	1426	150	1958
161.831555	0	136	22	3

After iterating, the bottom rows of the spreadsheet tool display the preliminary future year turn movements as shown in Exhibit 6-17. These turn movements need the final step, which is to re-balance the network while holding the model link growth values as much as feasible.

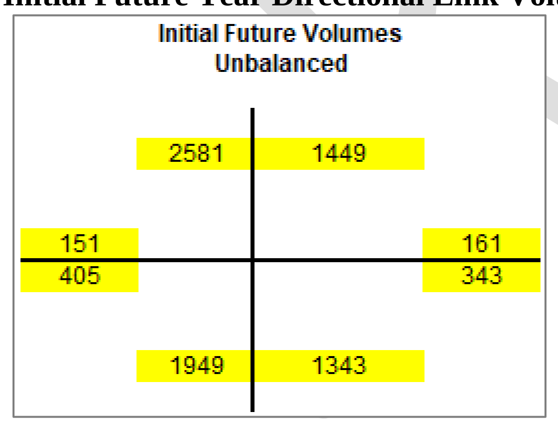
Exhibit 6-17 Estimated Turning Movements at Bottom of Spreadsheet Tool



Example 6-25 Post-Processor Tool Turning Movements

This example is based on the previous post-processing spreadsheet application shown in Exhibit 6-13. It is desired to determine Future Year turn movement volumes at the intersection of Crystal Lake Drive at OR 99W. In this example, Crystal Lake Drive at OR 99W is a standard signalized four leg intersection. Future year directional link volumes at the intersection have been previously determined and are shown in the intersection diagram below.

Initial Future Year Directional Link Volumes



The first step is to ensure that the directional link volumes produce equal intersection inflows and outflows at the intersection, and to adjust them if they do not. In this case, the inflows are greater than the outflows. As shown in the table below, the inflows total 4,490 and the outflows total 3,891. Typically, an average of the two values is used.

In this example, the average of the inflows and outflows is 4,191. Individual directional link volumes are then adjusted as follows for the EB outflow.

Initial EB outflow = 343
Proportion of total outflows = 343 / 3,891 = 0.09
Reduction needed = 3,891 – 4,191 = 300 vph
Adjustment = 0.09 x 300 = 27 vph
Adjusted outflow = 343 + 27 = 370

The adjusted intersection inflows and outflows are shown in the table below.

Initial Future Volume			Adjusted	
	Inflow	**Outflow**	**Inflow**	**Outflow**
EB	405	343	378	370
WB	161	151	150	163
NB	1343	1449	1253	1561
SB	2581	1949	2409	2098
Total	4490	3891	4191	4191
Average	4191			
Difference	-299	299		

The next step is to determine the initial turn movement percentages. In this example, the future no-build flow bundles are used to calculate the initial forecasted volume and the Base Year 2020 turn movement percentages are documented for reference. These turn percentages are summarized in the table below.

Enter Flow Bundle Link Value

Leg	Inflow	Movement	Outflow	Flow Bundle Future No-Build Turn %	2020 Existing Count Turn %
North		SBL	62	4%	13%
	1426	SBT	1333	93%	84%
		SBR	33	2%	3%
East		WBL	1	1%	6%
	67	WBT	10	15%	12%
		WBR	42	63%	82%
South		NBL	84	9%	9%
	944	NBT	851	90%	90%
		NBR	8	1%	1%
West		EBL	39	16%	28%
	238	EBT	12	5%	21%
		EBR	188	79%	51%

The final step is to scroll through the six iteration tables and assure the convergence change percentage is within 10%. After a sufficient iteration convergence is achieved, the estimated turning movement volumes can be found in the bottom rows of the spreadsheet tool. The analyst may need to make further adjustments to turn movements as part of re-balancing of the network.

6.12.6 Automated Post-Processing

Certain software tools perform post-processing internally. However, no tool can substitute for the need to manually check results for reasonableness. The analyst should have a good understanding of the internal methodology used in the software.

VISUM has an internal post-processing module. TPAU evaluated VISUM 2024 by comparing its outputs to the manual method. It was found to be inconsistent with ODOT's manual methodology for the following reasons. VISUM's built in post-processor:

- Uses NCHRP 255 instead of 765
- Has the basic assumption that base year counts equal base year model volumes; this requires multiple user-defined attributes and extra process to approximate ODOT/NCHRP 765 methodologies
- Will end up black-boxing the post-processing and will require VISUM to review (which is generally not in most ODOT regions)
- Will complicate review and make the process less transparent (critical for reviewers)
- It appears that methodology is only close for the future no-build (and this assumes that there were no changes in the study area outside of the subject project) as the flow-bundle method does not work for future build scenarios as differences are substantial (this is due to the basic assumption noted above).

For these reasons TPAU's post-processing spreadsheet tool should be used (or something similar/consistent to NCHRP 765) for link post-processing.

There are other proprietary products from consultants (JRHMoves from JRH Engineering and Furness from CH2MHill) that have been evaluated and found to be consistent with NCHRP 765 for links and turn moves.

6.13 <u>Forecasting Truck Volumes for Pavement Design</u>

Because pavement design is based on the number of axle-loadings experienced on the roadway, heavy vehicle information needs to be furnished in a specific format. The pavement designer will contact the project traffic analyst for this truck information. The Pavement Services Unit (PSU) does all of the pavement designs for all ODOT projects, unless consulted out. Most of these projects are small operational improvements, pavement preservation or bridge replacement projects.

Pavement design volume requests are either at a scoping or project-specific level. All scoping-level requests are done internally by the PSU. A project-level request will need to be done by the project traffic analyst who could be a region project analyst, a consultant, or a TPAU analyst. These requests should be routed through the Transportation Systems Monitoring Unit (TSMU). TSMU will then forward the request to the appropriate project leader and then from there to the project analyst.

Requests for heavy vehicle volumes are e-mailed by PSU. These requests should include the project name, project limits, project type (scoping or project-level) name, and route number. They should also include any special needs such as breakouts on one-way facilities. In addition, the analyst should discuss factors that may influence the output and therefore the design with the pavement design staff involving the pavement designer. The discussions can help clarify the level and precision of the data required which may streamline the process.

Responses are typically transmitted via e-mails with a spreadsheet-formatted attachment showing: current and 20-year future Average Daily Traffic (ADT), a 20-year growth factor, and the current heavy vehicle volume by class (see Exhibit 6-15). If one-way ADT is requested, state direction and intersection leg, if applicable.

General information and special considerations

General truck growth rates are assumed to mirror the overall growth rate on a segment calculated by methods described previously in this chapter. Vehicle classification specific growth rates are not available due to the lack of commodity and routing information. The trucking industry introduces new products, with different axle configurations, which have insufficient highway count data.

Before reporting out any data, the analyst should verify that there are no special considerations that need to be accounted for such as having count information that may reflect an average and not a typical vehicle mix. Sometimes a specific generator may not be directly accounted for that might require splitting a segment. Other considerations might include:

- Variations in vehicle classifications that are not accounted for such as vehicles with drop-axles that change the vehicle classification or the different configuration for a logging truck – 5-axle loaded logging trucks versus a 3-axle when the empty trailer is piggy-backed.
- Having a count taken in the summer on a route that services a local school, so it is missing the school bus volumes.
- Heavy generators such as mills, industrial areas, distribution centers, truck stops, or intermodal transfer facilities
- Periodic users, such as material/construction sites, logging or agricultural

The Federal Vehicle Classifications are shown in Exhibit 6-14. All vehicles are classified into one of the 13 categories. For example, class 6 is described as 3-axle, single unit which can be reported out as shown in Exhibit 6-15 with the information including the number of axles and trailers of each class. The truck volumes are also shown.

Exhibit 6-14 Federal Vehicle Classifications

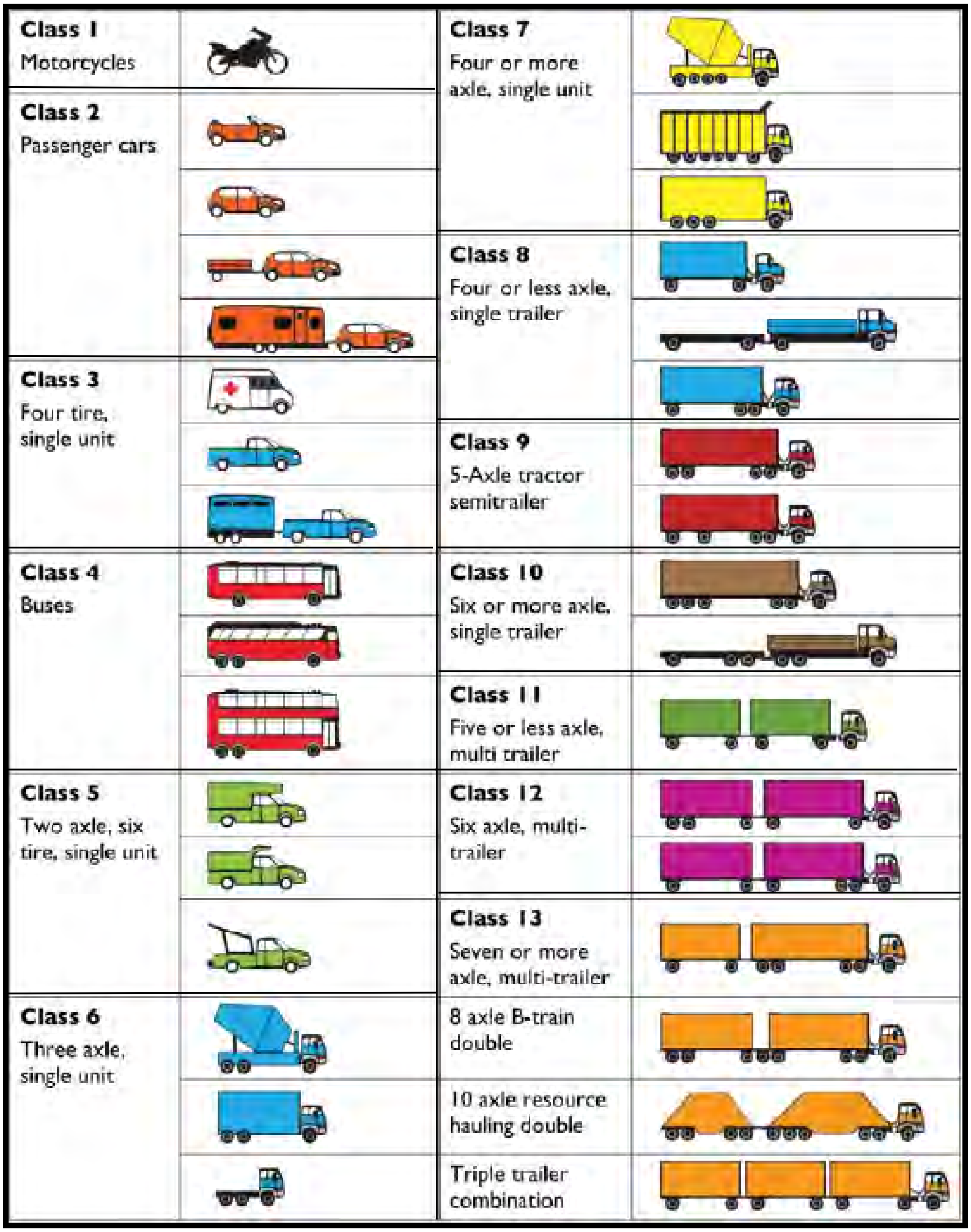


Exhibit 6-15 Example Response Spreadsheet Attachment

Project	Hwy 500 (OR 795) MP 77 - MP 83			
Class	Descript.	# Axles	# Trailers	**Count** 2010 Vol
4	buses			18
5	2ax	2		40
6	3ax	3		25
7	4ax	4		2
8	4ax trl	4	1	0
9	5ax trl	5	1	12
10	6ax trl	6	1	4
11	5ax dbl trl	5	2	2
12	6ax dbl trl	6	2	0
13	7ax dbl trl	7	2	0
			Total Trucks	103
Future Volume Tables 2010 AADT = 3000, 2030 AADT = 200				
				% Trucks 3.43% 20-year growth factor = 1.4

The percent truck calculation is the total number of trucks divided by the base year AADT. For this exhibit this would be 103 / 3000 = 3.43%. The 20-year growth factor is the future AADT divided by the base year AADT or 4200 / 3000 = 1.4.

<u>Process</u>

To prepare the data, knowledge of the highway characteristics in the area is helpful. While field investigation is likely not practical, use of the ODOT video log, Transviewer GIS or other mapping aide should be used. There is more information readily available for use on state highway segments than on local jurisdiction roadways. The general state highway process takes advantage of currently available formatted outputs. The scoping-level process is used for scoping level requests or simpler projects. The project specific process is more detailed. These requests include intersection, or straightaway classification counts and may involve local road segments or state highway segments that need special considerations are applied. A spreadsheet calculator, Heavy Vehicle Pavement Design Spreadsheet, has been developed to streamline the process for both scoping and project level request.

6.13.1 Scoping Level

Using the state highway number, milepoint and other locational data, refer to the Traffic Volumes and Vehicle Classification Report available at:

https://highway.odot.state.or.us/cf/highwayreports/traffic_parms.cfm

This report provides overall and vehicle AADT volumes and vehicle classification by the specified segment. The segments are determined by major intersections and other characteristics. Using this information, enter the following into the output spreadsheet (see Example 6-26) for each identified segment:

- Highway/road name
- Beginning mile point
- Effective date of data
- AADT
- AADT 20-YR
- Volumes for Class 4-13

Example 6-26 Scoping Level Project Forecast

As part of a pavement preservation scoping project process, the section of Highway #7 (US20) from M.P. 40 to M.P. 48, heavy vehicle counts were downloaded from https://highway.odot.state.or.us/cf/highwayreports/traffic_parms.cfm. The AADT Volume, AADT 20-year Volume, Beginning M.P., and the volumes for Classes 4 through 13 were copied into the Heavy Vehicle Pavement Design Spreadsheet in the Scoping Level tab. The data are shown in the next table and the future forecasted results are shown in the following table.

Project	Hwy 7 (US20) M.P. 40-48			
				Count
Class	Descript.	# Axles	# Trailers	2013 Volume
4	buses	2		22
5	2ax	2		123
6	3ax	3		2
7	4ax	4		0
8	4ax trl	4	1	111
9	5ax trl	5	1	132
10	6ax trl	6	1	29
11	5ax dbl trl	5	2	10
12	6ax dbl trl	6	2	9
13	7ax dbl trl	7 +	2 +	33
			Total Trucks	471
AADTs 2013 ADT = 1200, 2033 ADT = 1300				
20-year growth factor = 1.08				

Project	Hwy 7 (US20) M.P. 40-48			
Class	Descript.	# Axles	# Trailers	**Forecast** 2033 Volume
4	buses	2		24
5	2ax	2		134
6	3ax	3		2
7	4ax	4		0
8	4ax trl	4	1	121
9	5ax trl	5	1	144
10	6ax trl	6	1	32
11	5ax dbl trl	5	2	11
12	6ax dbl trl	6	2	10
13	7ax dbl trl	7 +	2 +	36
			Total Trucks	514

The following example is a relatively simple project level analysis that does not involve local roads or projects with special considerations.

> *While the following example used future forecasts from the Future Volume Tables for the purpose of illustration, the volumes could have just as likely been derived from a cumulative analysis or a post-processed travel demand model. Choice of future forecasting methods is dependent on the study area, the project requirements, and the tools available. See earlier sections in this chapter.*

Example 6-24 Simple Project Level Forecast

As part of a pavement redesign project, a new pavement structure needs to be designed. The pavement engineer on the project first needs to know the heavy vehicle volumes that need to be designed for. The engineer had a 24-hour traffic count available in 2011 at the project site and recorded the following data from it. The project base year is 2014 with a 2034 future year. The engineer also recorded the 20-year ADT from the Future Volume Tables.

Project	Hwy 2 (US730) M.P. 190 - 195			
Class	Descript.	# Axles	# Trailers	**Count** 2011 Vol
4	buses	2		96
5	2ax	2		475
6	3ax	3		57
7	4ax	4		8
8	4ax trl	4	1	316
9	5ax trl	5	1	724
10	6ax trl	6	1	442
11	5ax dbl trl	5	2	74
12	6ax dbl trl	6	2	38
13	7ax dbl trl	7 +	2 +	245
			Total Trucks	2475
Future Volume Tables 2011 ADT = 2500, 2031 ADT = 2600				
				% Trucks 42.3% 20-year growth factor = 1.04

The count information and Future Volume Tables' information were entered into the Heavy Vehicle Pavement Design Spreadsheet and the results found recorded from the spreadsheet into the table below.

Project	Hwy 2 (US730) M.P. 190-195			
Class	Descript.	# Axles	# Trailers	**Forecast** 2034 Vol
4	buses	2		101
5	2ax	2		497
6	3ax	3		59
7	4ax	4		8
8	4ax trl	4	1	331
9	5ax trl	5	1	757
10	6ax trl	6	1	463
11	5ax dbl trl	5	2	77
12	6ax dbl trl	6	2	40
13	7ax dbl trl	7 +	2 +	256
			Total Trucks	2589

The Heavy Vehicle Pavement Design Spreadsheet calculator used in this example is shown below.

Heavy Vehicle Forecast for Pavement Design Spreadsheet: Project Level Analysis				
Project	Name	APM Example		
	Description	Cold Springs Automatic Traffic Recorder,		
	Additional Identification	Sta. 30-002, Hwy No. 2 (US730): 0.24 mile east of Pendleton-Cold Springs		
	Hwy #	36		
	M.P.	193.70		
	Base Year	2014		
	Forecast Year	2034		
Count	TCM # (If available)	480		
	Duration	24 hr		
	Factor	1.00		
	Year	2011		
Future Volume Table	Base Year / Future Year	2011		2033
	Base Year AADT	2500		
	Future Year AADT	2600		
	20-yr Growth Rate (per yr.)	0.18%		

Class	Description	# Axles	# Trailers	2011 24 hr Count Volume	2011 24 hr Count Volume	2014 Base Year Volume	2034 Forecasted Volume
4	buses	2		96	96	97	101
5	2ax	2		475	475	478	495
6	3ax	3		57	57	57	59
7	4ax	4		8	8	8	8
8	4ax trl	4	1	316	316	318	330
9	5ax trl	5	1	724	724	728	754
10	6ax trl	6	1	442	442	444	460
11	5ax dbl trl	5	2	74	74	74	77
12	6ax dbl trl	6	2	38	38	38	39
13	7ax dbl trl	7	2	245	245	246	255
13	triples	7+	2+	0	0	0	0
Total				2475	2475	2488	2578

6.13.2 Detailed Process for Local Roads or Highway Segments with Special Considerations

A more comprehensive process is required when pavement traffic data are needed for non-state facilities or when special considerations exist. This process should also be used when recent classification counts are available. This process is based on intersection or straightaway classification counts.

Note: Additional classification counts may be necessary for longer preservation projects (or where there are no ATR/AVC's on a particular highway) as counts are not typically obtained for this project type.

Identifying Segments

To determine segments on state highways, use the Traffic Volumes and Vehicle Classification Report available at:

https://highway.odot.state.or.us/cf/highwayreports/traffic_parms.cfm.

Break roadway into segments if AADT volumes change by more than twenty percent (20%). These initial segments need to be reviewed to see if the truck classifications (4-13) change by more than ten percent (10%). The higher classifications have more impacts on the road surface (since these are the heavier vehicles and/or have multiple axles). Note that the review needs to consider where an absolute number versus the percentage change is significant. If there are questions whether a change is significant, the analyst should work with the pavement designer to determine if the segment break is necessary. The analyst needs to put greater importance on changes to the higher classifications. For example, an increase of five triple-trailer trucks (class 13) could cause a 50-100% increase with greater impacts than a 50% increase in delivery/ panel trucks (class 5). When this occurs, the analyst may need to subdivide the segments to better identify the truck volumes.

For local roads, using the count/classification available, determine volumes at different points in the project. Break into sections if volumes change by more than twenty percent (although this changes on a case-by-case basis) or the project includes more than one highway. While the project may involve only one road, there may be differences in heavy vehicle volumes on each side of a major intersection or urban/rural environment change. A change, such as road classification, may not be a reason to split a section, but may be a good place to look for a difference in heavy vehicle volumes. When a count is not available on a specific roadway, counts from similar facilities that serve the same areas may be used as a basis for an educated assumption of the truck percentages.

Count Data

To forecast truck volumes, obtain a 24-hour (less than three years old) full classification count(s) within or near the project. Since truck volumes typically increase overnight as a percentage of total vehicles, every attempt should be made to obtain a 24-hour count. If the project has an Automatic Vehicle Classifier (AVC), then those data can be used as a count. Internal staff may look in OTMS. Consultants should contact the TSM staff. If a recent count is not available, and time permits, a full classification count could be requested. Counts on interchange ramps do not have classification data unless specifically requested (such as a freeway-to-freeway interchange where there are not any ramp terminal intersections), so this is generally obtained from a ramp terminal intersection count.

Determining Classification Volumes

For each intersection count, total the volumes by class for each leg that data are desired. A full classification count is shown in Exhibit 6-16.

Exhibit 6-16 Traffic Count Sheet

Axle Factor Report
10/13/2009 Through 10/13/2009

Intersection ID:	22032009	**Date:**	10/13/2009
County:	Linn	**Hour:**	6:00 AM - 10:00 PM
City:	Albany	**Legs:**	Burkhart St. (SB), OR99E(Paciifc Blvd.) (WB), Burkhart St. (NB), OR99E(Pacific Blvd.) (EB)
LRS ID:	05800I00	**Location:**	Burkhart St. at OR99E(Paciifc Blvd.) and OR99E(Pacific Blvd.)
LRS Location Point	1.24	**Notes:**	Weather: Clear

						Single Unit Truck			Single Trailer Truck			Multi Trailer Truck			
Leg	**From To**	**Motorcycle**	**Car**	**Lt Truck**	**Bus**	**2 Axles**	**3 Axles**	**4+ Axles**	**4- Axles**	**5 Axles**	**6+ Axles**	**5- Axles**	**6 Axles**	**7+ Axles**	**Total All**
East	East-North	1	1254	291	7	8	1		6	3	1			3	1575
	East-South		341	160	1	10			1	1					514
	East-West	10	6727	1999	59	158	38	2	28	94	68	2		15	9200
	North-East	1	931	299	3	13	1		5	5					1258
	South-East		349	146		6				1					502
	West-East	9	6503	1941	57	145	38	2	38	111	74		1	10	8929
	Total	21	16105	4836	127	340	78	4	78	215	143	2	1	28	21978
	Axle Factor	1	1	1	1	1	1.5	2	2	2.5	3	2.5	3	3.5	0.965
	Vehicle Over	21	16105	4836	127	340	117	8	156	538	429	5	3	98	22783
North	East-North	1	1254	291	7	8	1		6	3	1			3	1575
	North-East	1	931	299	3	13	1		5	5					1258
	North-South	2	1138	427		9	4			1				4	1585
	North-West		34	7											41
	South-North	1	948	367	4	12	1		2	3	1				1339
	West-North		61	25	1	3	1								91
	Total	5	4366	1416	15	45	8	0	13	12	2	0	0	7	5889
	Axle Factor	1	1	1	1	1	1.5	2	2	2.5	3	2.5	3	3.5	0.991
	Vehicle Over	5	4366	1416	15	45	12	0	26	30	6	0	0	24	5945
South	East-South		341	160	1	10			1	1					514
	North-South	2	1138	427		9	4			1				4	1585
	South-East		349	146		6				1					502
	South-North	1	948	367	4	12	1		2	3	1				1339
	South-West		115	36		1									152
	West-South		195	86	1	4									286
	Total	3	3086	1222	6	42	5	0	3	6	1	0	0	4	4378
	Axle Factor	1	1	1	1	1	1.5	2	2	2.5	3	2.5	3	3.5	0.994
	Vehicle Over	3	3086	1222	6	42	8	0	6	15	3	0	0	14	4405
West	East-West	10	6727	1999	59	158	38	2	28	94	68	2		15	9200
	North-West		34	7											41
	South-West		115	36		1									152
	West-East	9	6503	1941	57	145	38	2	38	111	74		1	10	8929
	West-North		61	25	1	3	1								91
	West-South		195	86	1	4									286
	Total	19	13635	4094	118	311	77	4	66	205	142	2	1	25	18699
	Axle Factor	1	1	1	1	1	1.5	2	2	2.5	3	2.5	3	3.5	0.961
	Vehicle Over	19	13635	4094	118	311	116	8	132	512	426	5	3	88	19467

For each leg, there should be a total for each classification (4-13). Note this needs to include both directions (all the "to" and "from") on the approach. This means that at a 4-legged intersection there are six movements to be summed. Once the table is filled out, the columns must be totaled to report the number of trucks by classification. If the manual count is less than 24 hours, an expansion factor (see Exhibit 5-19) needs to be applied (typically a 1.1 converts a 16-hour count to 24-hour volumes). These classification subtotals are then combined to report the total truck volume(s). The truck total is then divided by the ADT to give the percent trucks. The Heavy Volume Pavement Design Spreadsheet can be used to streamline the individual segment calculations.

Growth Rate

For state highways, record each year and its volume for the road segment from the Future Volume Table (FVT) into the calculation spreadsheet which will compute the growth factor. The analyst will need to review the FVT segments to determine which have appropriate r-squared values and can be averaged together to arrive at a segment growth rate (see Chapter 5).

Off-system growth rates would need to be determined with county or city data (off system data source), such as historical tube counts, travel demand model growth rates, similar facilities or other project information in the area (see Chapter 5). If historical data, state highways with similar characteristics, or a travel demand model does not exist for the particular site, use 2% for rural areas and 1% for urban areas per year.

Appendix 6A – Sample Application of Enhanced Zonal Cumulative Analysis (EZCA)

Appendix 6B – Considering Connected and Automated Vehicles in Future-Year Roadway Capacity Forecasts

Appendix 6C – Supplemental Guidance on Connected and Automated Vehicle Analyses

7 SYSTEM PLANNING ANALYSIS

7.1 Purpose

The purpose of this chapter is to illustrate the different types of system planning analysis and related tools, applications, limitations, and data needs. These methods are recommended for use in larger scale planning studies.

- System Planning Analysis
- Statewide Integrated Model (SWIM)
- Travel Demand Models
- Regional Strategic Planning Model (RSPM)
- Land Use Scenario Tools

In a future update, RSPM will be updated and replaced by VisionEval which Is the current tool for strategic planning assessments.

7.2 System Planning Analysis

The following is a list of the different kinds of system planning from a low to a high level of detail.

7.2.1 Strategic Planning

Strategic Planning is a way to understand the first order effects of a broad array of policies with less required input detail (e.g., doubling transit service miles), to understand the tradeoffs of different futures (e.g., operational strategies vs. transit investment). Because these models may be less detailed, they run quickly and thus are able to make lots of runs to test plan resilience under a variety of future uncertainties (e.g. changing fuel price and income forecast). Strategic planning level of detail is limited to a state or regional scope. The tools for strategic planning analysis typically are SWIM and RSPM.

7.2.2 Statewide Systems

Statewide system planning generally is policy or economic based, such as relating to the Oregon Transportation Plan and/or state modal plans such as the Oregon Highway Plan. Statewide system planning is conducted to explore alternative futures related to greenhouse gas (GHG) emissions, land use development, population demographics and economic forecasts as they relate to use of the transportation system. Statewide system planning is used to develop investment strategies associated with different budget options, policy goals and legislative concepts evaluating the best options to meet statewide objectives. Tools used for statewide system analysis include SWIM and RSPM.

7.2.3 Regional Systems

Regional system planning generally focuses on specific areas like Metropolitan Planning Organizations (MPO), individual cities, or unincorporated or rural areas. These will involve creation or analysis of Regional Transportation Plans (RTP) or Transportation

System Plans (TSP). Typical tools used could be regional or urban travel demand models.

7.2.4 Corridor Systems

Corridor system planning can involve an individual route which can be made up of one or many different highways. This also can be just a small segment in a regional or urban area.

Exhibit 7-1 shows the typical tool applications for each type of system planning analysis.

Exhibit 7-1: System Planning Analysis Tool Applications

	Strategic	Statewide	Regional	Corridor
SWIM		X	X	X
Travel Demand Models			X	X
RSPM	X	X	X	
Land Use Scenario Tools			X	

System planning analysis may be a tool for considering the impacts of emerging trends and technologies, including connected and automated vehicles (CAVs). Although no CAVs are currently available commercially, it is expected that CAVs will start to become available within the 20- to 50-year planning horizons of transportation system plans and other long-range transportation studies. There are many potential impacts of CAVs on safety, operational efficiency, travel behavior, transportation accessibility, transportation equity, environmental impacts, and more. Appendix 6B provides more information on CAVs and their potential effects, focused on the methods in the HCM 7th Edition (HCM7) for adjusting roadway capacity for the presence of CAVs in the traffic stream on freeways and at signalized intersections and roundabouts.

Oregon has begun to assess the potential range of impacts of CAVs using travel demand models, with more information available in the Final Report for the project *Scenario Guidance for Travel Demand Modeling*, available at https://www.oregon.gov/odot/planning/pages/apm.aspx under “Supplemental Materials." While the Task 11 Final Report provides the most concrete guidance on capacity changes to make in a TDM for CAVs, it also provides guidance on other potential changes to make and use of TDMs to understand impacts beyond capacity, e.g. land use and urban form.

7.3 Statewide Integrated Model (SWIM2)

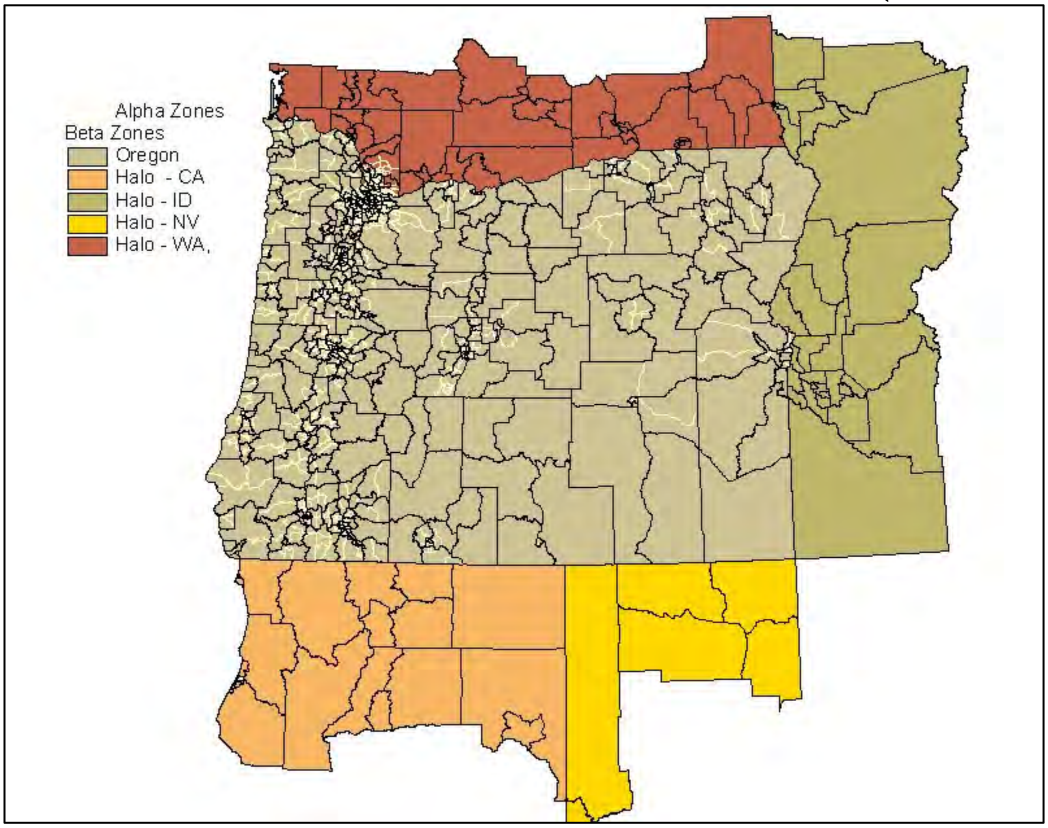
7.3.1 Introduction/Purpose

Transportation, land use and economics are all interwoven. Oregon's SWIM2 model allows regional and statewide policies to be tested to inform decision-makers on the complex interactions between land use, the transportation network, and the economy. SWIM2 has been used to examine a variety of transportation and land use policy actions, investments, and their interactions through time. It is designed to answer questions at a larger scale than the typical regional or small urban travel demand model. Unlike typical travel demand models where land use is the major input, the SWIM2 model uses the economy in terms of gross domestic product (GDP) and models land use and its impact on transportation.

7.3.2 Geography, Zone Size and Network Level of Detail

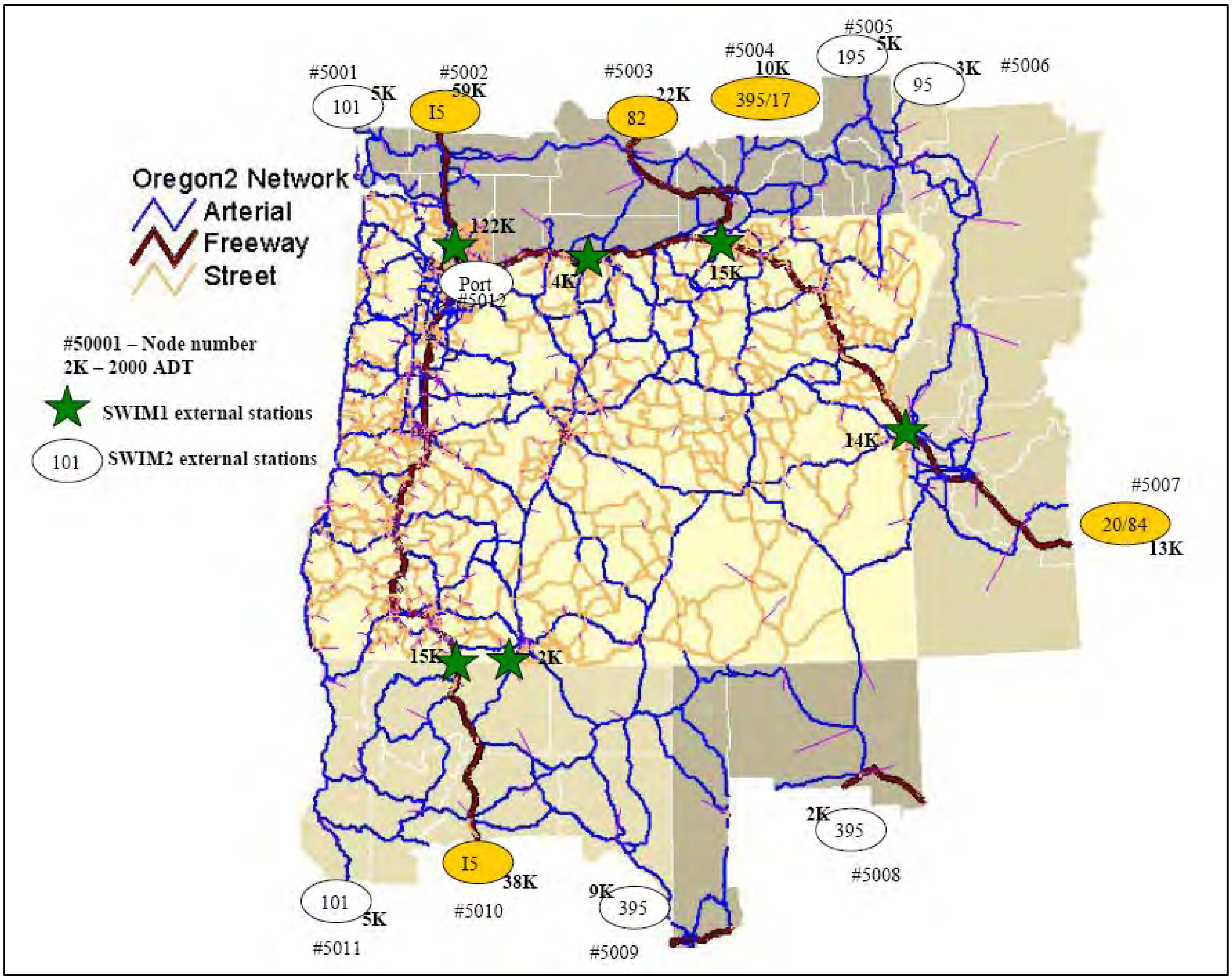
SWIM2 operates at two geographic levels within the model area (Exhibit 7-18). Both levels encompass all 36 Oregon and 39 (Halo) adjacent state counties. The halo encompasses a roughly 50-mile buffer around Oregon. A system of alpha zones used for trip assignment (light and dark lines in Exhibit 7-18) has the finest level of detail. A system of larger beta zones is used for land use allocations. The External Stations (Exhibit 7-19) serve as model area entry/exit points or gateways to World Market zones.

Exhibit 7-2: Current SWIM2 Model Extent and Zone Structure (October 2010)



Source: 2nd Generation StateWide Integrated Model (SWIM) Model Description - Model Build Documentation November 2010

Exhibit 7-3: Map of SWIM2 External Stations



Source: 2nd Generation StateWide Integrated Model (SWIM) Model Description - Model Build Documentation November 2010

7.3.3 SWIM2 Structure

The SWIM2 model is comprised of the following individually calibrated modules that represent the behavior of the land use, economy, and transport system in the sState of Oregon. Because the structure is modular, it allows for updates and improvements to be made with minimal disruption to the full model.

- **ED –** The Economics and Demographics module determines model- wide production activity levels, employment, and imports/exports.
- **SPG –** The Synthetic Population Generator module samples household and person demographic attributes (SPG1) and assigns a household to an alpha zone (used for trip assignment) (SPG2).

- **ALD –** The Aggregate Land Development module allocates model-wide land development decisions among study area alpha zones considering floor space prices and vacancy rates.
- **PI –** The Production allocations and Interactions module determines commodity (goods, services, floor space, labor) quantity and price in all exchange zones to clear markets, including the location of business and households by beta zone (used for land use).
- **PT –** The Person Travel module generates activity-based person trips for each study area person in the synthetic population, during a typical weekday.
- **CT –** The Commercial Transport module generates mode split for goods movement flows and generates truck trips, combining shipments and possible trans-shipment locations, for a typical weekday.
- **ET –** The External Transport module generates truck trips from input origin-destination trip matrices representing import, export (within 75 miles) and through movements based on PI and external station growth rates.
- **TS –** The Transport Supply module assigns vehicle, truck, and transit trips (separately) to paths on the congested transport network for a 24-hour period, generating time and distance skims for AM and off-peak periods.

The PI module operates on the less-detailed beta zone system (dark lines in Exhibit 7-18) where the external stations are replaced by world markets. The beta zones consolidate (aggregate) the alpha zones, with a focus on the small urban zones. In other urban areas, zones were consolidated based on a sliding population scale (approximately 25,000 persons per zone), respecting similar employment clusters and transportation commute-sheds. In rural areas, homogenous public lands (e.g., BLM, National Forests) were consolidated, while retaining most county and all ACT[1] boundaries.

The world markets assume that goods transport by truck and rail is limited to the US (except Hawaii), Canada and Mexico. Imports and exports to other regions in the world are shipped by barge, either from the Port of Portland or other US east or southeast marine ports.

The ED module estimates production activity, imports/exports, and employment exclusively at the model-wide level. Due to data limitations, ED uses an aggregated set of general industry sectors such as Wholesale Trade, Lumber and Wood Products, and Education[2]. ED outputs are disaggregated using fixed relationships into the industry categories used in the SPG and PI modules. These fixed relationships rely on employment and economic data.

[1]Area Commissions on Transportation (ACTs), used in Oregon transportation planning, provide a convenient way to divide the State into 12 areas.
[2]For a complete list of Industries and commodities please refer to the Model Build Documentation - 2nd Generation StateWide Integrated Model (SWIM) Model Description

7.3.4 Scenario Development

SWIM is intended to respond to large (regional or statewide) projects and policy questions, and is not suitable for fine-grained questions, such as specific land use changes (i.e., a new shopping center) or small network projects (i.e., widening of a 1-mile section of urban road). These kinds of smaller requests need to use the appropriate MPO or small urban model. However, SWIM can be used on smaller projects to help inform on trends such as trip distributions or verification of future growth rates where other rural information is unavailable or spotty.

Typical inputs would be to make any network modifications like adding a new highway corridor or significant bridge crossing that would affect the regional economy. The inputs are all integrated and provide feedback for each other (i.e. the transport system can affect land use which, in turn, affects the economy). The SWIM2 model network is primarily state highways. City networks are not as detailed as a MPO regional model. Land use inputs involve defining the allowable zoning and capability for each zone type. Economic inputs are based on the GDP by sector at a state level.

Because of its complexity and statewide application, ODOT staff and resources use SWIM2 to develop scenarios. SWIM2 is not to be requested using the standard model request form. Any potential SWIM application requests need to be routed to the TPAU unit manager. Although it is useful for developing and analyzing a wide range of policy alternatives and options, it can require several weeks to run the model to respond to the input changes. SWIM2 outputs are on an annual basis rather than a typical 20-year planning horizon for most travel demand models. Outputs are not intended for post-processing as the model is not to the typical link level of detail. There also is no “official” future no-build to compare to as there is no consistent statewide vision of a future network and zoning. Every SWIM module has generated outputs such as dollars traded by sectors from the PI module or population by zone from the SPG module. These outputs require analysis to be able to “tell the story” of the impacts of a particular scenario.

7.3.5 SWIM Applications

Estimated Economic Impact Analysis Due to Failure of the Transportation Infrastructure in the Event of a 9.0 Cascadia Subduction Zone Earthquake

The purpose of this analysis was to provide high- level estimates of avoidable economic impacts caused by damage to the transportation system from a major seismic event (a 9.0 Cascadia Subduction Zone Earthquake, where the fault breaks along the entire subduction zone – a worst-case earthquake scenario). Four alternative scenarios were used to evaluate the impacts of pre-emptive mitigation. This analysis was prepared for the ODOT Bridge Engineering Section, which is evaluating risks and identifying strategies to mitigate seismic vulnerabilities of the state highway system. The scenario approach was designed to provide a general sense of the magnitude and direction of avoidable economic impacts to Oregon from damage occurring on the highway/street transportation system alone (non-transportation losses were not accounted for). This analysis focused on the western portion of the state, defined as the area to the west of the Oregon Cascade Range.

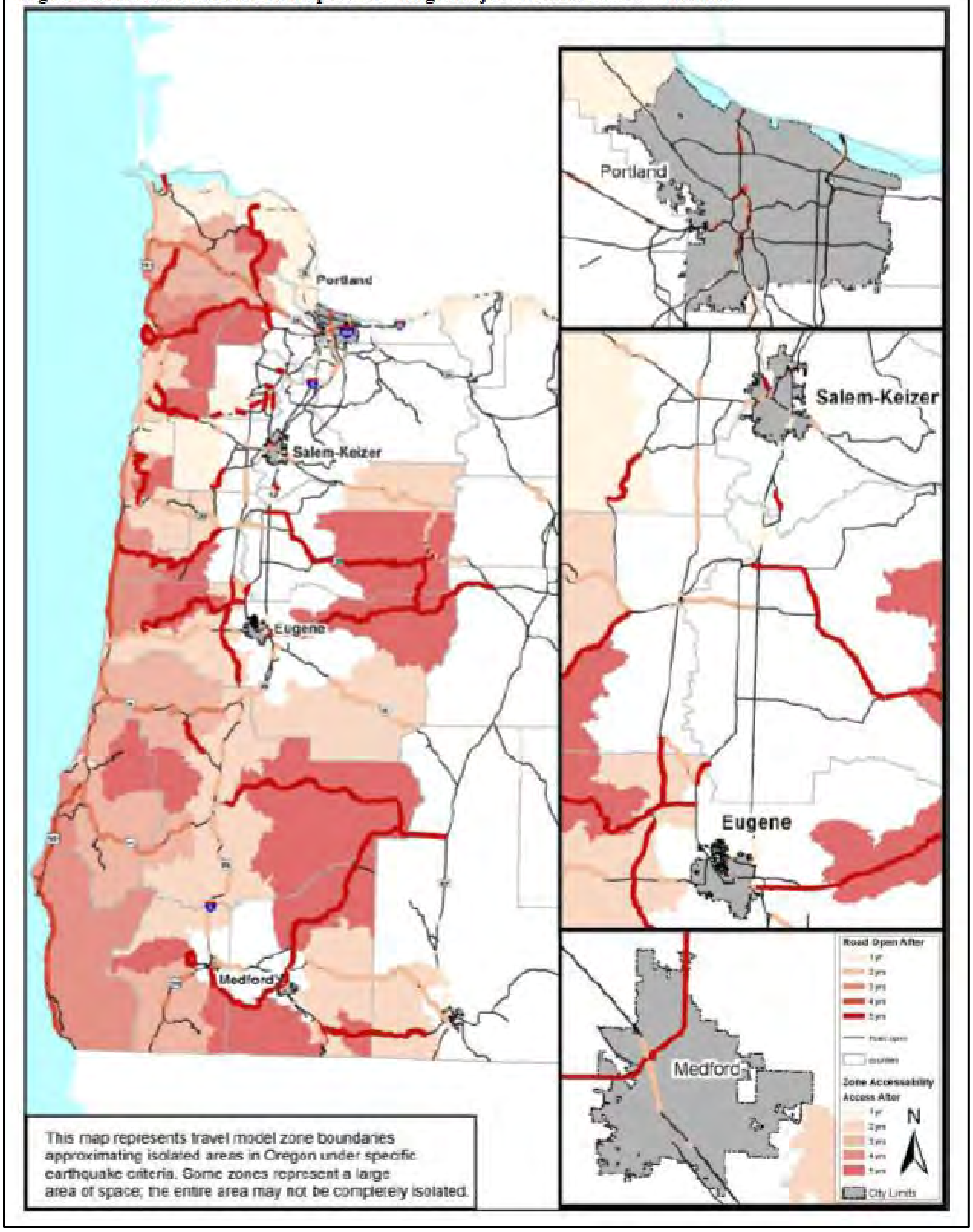
Results of this analysis indicate strengthening corridors before a major seismic event will enable the state to avoid a significant amount of economic loss. Significant economic losses in production activity can be avoided by preparing for a major earthquake ahead of time. With no preparation ahead of time, Oregon could lose up to $355 billion in gross state product in the 8 to 10-year period after the event. Proactive investment in bridge strengthening and landslide mitigation reduces this loss between 10% and 24% over the course of the eight years simulated for this analysis.

The analysis was conducted using the Oregon Statewide Integrated Model (SWIM). Only the roadway network was altered for the modeled scenarios. Corridors expected to experience damage from a major seismic event were represented as “failing.” The points of failure were identified by the ODOT Bridge Engineering Section for high-use state-owned facilities. For lower use corridors and non-state-owned facilities in the SWIM network, adjacent parallel routes within these corridors were altered to maintain consistency in network coding. Nearby facilities with similar proximity and characteristics of those identified to fail were represented to fail in the same manner. The purpose of this analysis was to evaluate the effects of impacts of transportation on economic activity separately, therefore building loss, damage to utilities, other damage or loss of life resulting from an earthquake was outside the scope.

Exhibit 7-20 shows the sections of highways affected by failures and areas of isolation. The roadway network is color- coded to illustrate when corridors would be repaired and returned to pre-earthquake conditions. Areas coded with the lightest color regain access to the highway system within one year, where the darkest red areas remain isolated for the full five- year repair period. Isolation means severely limited [day(s) of travel]) access to markets for the local economy, causing delay in economic recovery.

Exhibit 7-4

Figure 3. Isolated Zones and Repair Phasing: Major Seismic Event Scenarios



Estimated Economic Impact Analysis Due to Failure of the Transportation Infrastructure in the Event of a 9.0 Cascadia Subduction Zone Earthquake Technical Memorandum, ODOT/TPAU, January 2013.

Eastern Oregon Freeway Alternatives

The 1999 Legislature asked ODOT to look at the results of designating a north-south freeway in Central or Eastern Oregon, from the Washington to California borders. The objectives of House Bill 3090 were to:

- Define a better north-south connection to I-82 in Eastern Oregon
- Increase growth of Central/Eastern Oregon
- Decrease growth in the Willamette Valley
- Decrease travel and congestion on I-5 in the Willamette Valley

The basic approach of this study was to use SWIM to evaluate several alternative freeway scenarios. The alternative scenarios were modeled over a long-time horizon because of the amount of time required to build such a freeway and the time would take for land use effects to occur afterward. For the purposes of this study, completion of the freeway was set at 2020 and 2025. Since significant land use effects of major transportation changes take decades to occur, the modeling time horizon was established as 2050. Data for several evaluation measures were extracted from the model outputs to determine whether the objectives of the freeway would be accomplished. The objectives and measures are summarized in Exhibit 7-21.

Exhibit 7-5: House Bill 3090 Study Objectives and Evaluation Measures

Objective	Evaluation Measures
• Decrease travel time in Central & Eastern	• Average travel for Central and Eastern Oregon (minutes per passenger mile and minutes per ton mile)
• Increase the amount of travel occurring in Central & Eastern Oregon • Decrease travel and congestion on I-5 in the Willamette Valley	• VMT by region of the state • Average travel time for the Willamette Valley • Traffic growth on I-5 and other selected highways
• Increase growth of Central/Eastern Oregon and decrease in Willamette Valley	• Percent of households by region • Percent of jobs by region

Study of Eastern Oregon Freeway Alternatives Pursuant to House Bill 3090, ODOT/TPAU, April 2001

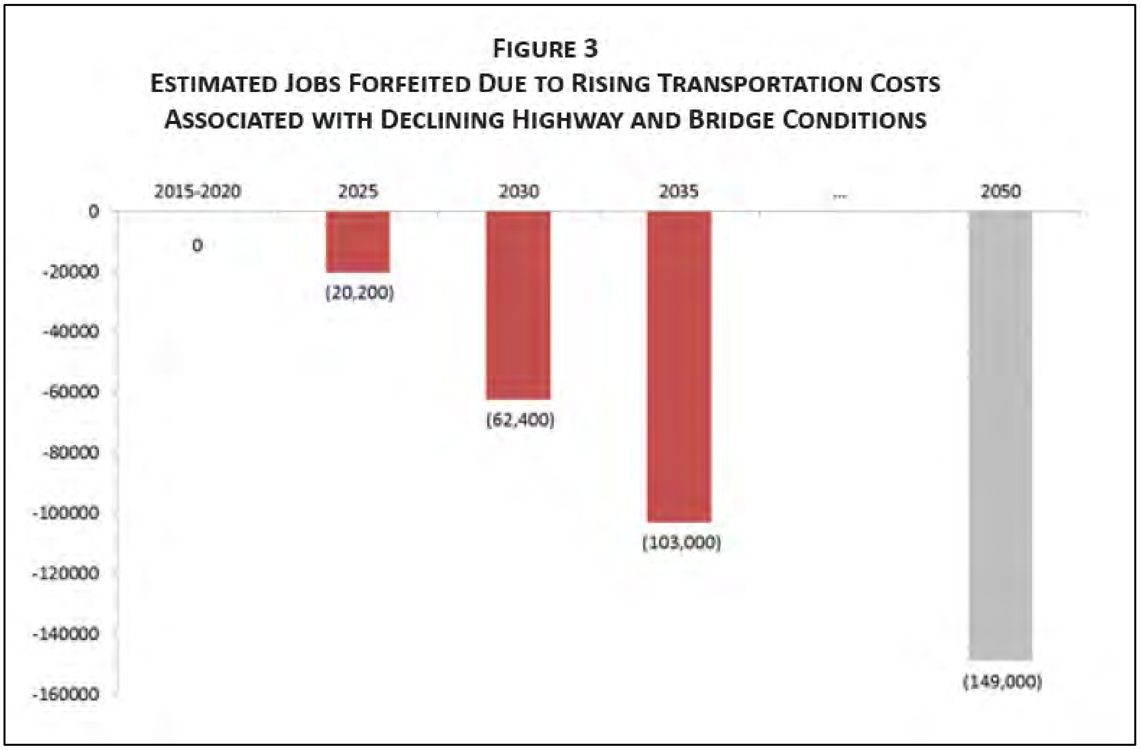
The results of the study found that building a new freeway connecting I-82 with California or Nevada to the south would significantly reduce travel time from border to border but would have little effect on the growth of Central or Eastern Oregon or the Willamette Valley. It would also have little effect on diverting traffic away from the Willamette Valley.

Rough Roads Ahead

The purpose of this analysis is to prepare a high- level strategic comparison between the current forecast budget and an alternative budget designed to preserve current conditions of state highways, roads and bridges. Two funding scenarios were developed for this high- level comparative analysis. The *Current Revenue/Deterioration Scenario* represents the current 20-year ODOT budget forecast for state highway spending. The *Maintain Current Conditions Scenario* represents a 20-year forecast for highway spending designed to preserve current highway conditions. The second-generation Oregon Statewide Integrated Model (SWIM2) is used for the scenario analysis.

One of the results were the estimated number of jobs forfeited due to higher transportation costs imposed in Oregon by declining highway and bridge conditions shown in Exhibit 7-22. Impacts to transportation costs start out small but increase rapidly; within 20 years there is significant impact on the growth of Oregon jobs. The number of estimated jobs lost increases over time. Between year 2025 and 2030 the number triples. By 2035 the number rises another 65 percent.

Exhibit 7-6: Jobs Forfeited



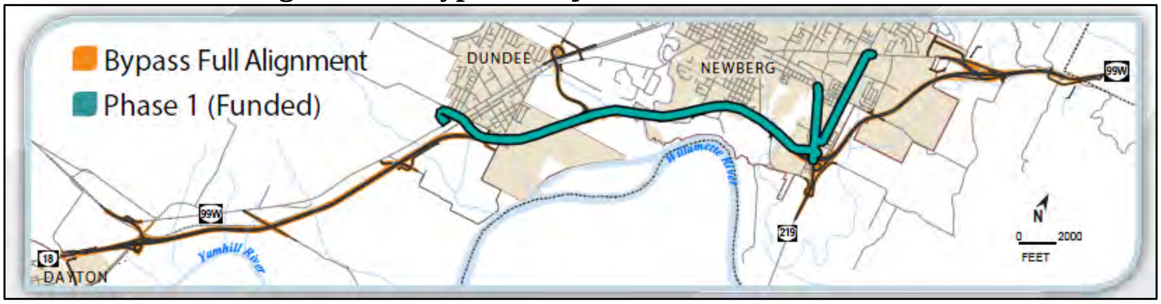
Rough Roads Ahead: The Cost of Poor Highway Conditions to Oregon's Economy, ODOT, 2014

The results of the study showed that deteriorating state highway conditions can be avoided. ODOT estimates that keeping the state highway system in its current good condition would cost an additional $405 million per year (constant dollars) compared to current budget levels. Given the expected economic losses and additional costs caused by a deteriorating system, the typical household will likely come out ahead with increased public investment in roads.

Newberg-Dundee Bypass

The Newberg-Dundee Bypass project was considered for funding under the Oregon Transportation Investment Act. The project, as shown in Exhibit 7-23, was modeled to assess potential land use, transportation and economic impacts of constructing or not constructing the project.

Exhibit 7-7: Newberg-Dundee Bypass Project



SWIM was used to model two scenarios: a Newberg-Dundee Bypass scenario, and a reference case or No-Action scenario. The distributions of households and jobs for the two scenarios were compared across external zones on the OR 99W/OR 18 corridor representing nearby communities in Yamhill County. Some of the conclusions from this modeling analysis effort were:

- The Newberg-Dundee Bypass will provide better access to McMinnville, which will help to stimulate the economic growth in the community.
- With the Bypass, there will be greater travel for all purposes between McMinnville and the Portland area consistent with the growth of population and jobs in McMinnville.
- Minimal effects will be seen in Newberg and other smaller communities in Yamhill County because of the Bypass.

Freight Plan

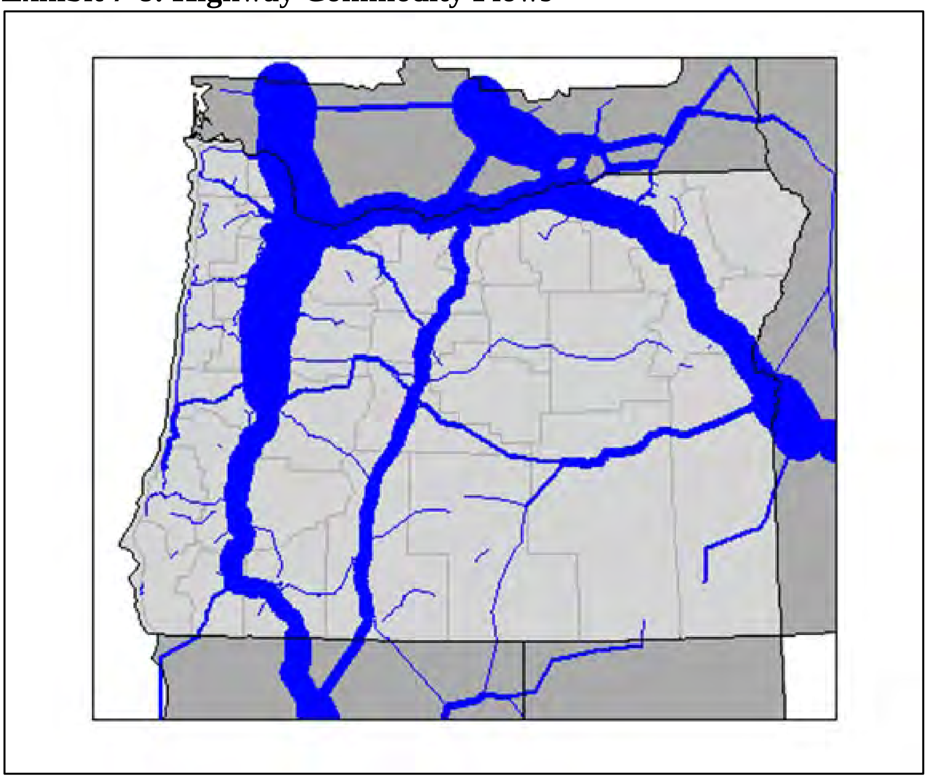
The purpose of the Oregon Freight Plan analysis was to gain an understanding of the spatial land use and transportation implications of different economic conditions. This analysis illustrated the variation in statewide and regional activity and commodity flow to help evaluate the risk associated with economic volatility on alternative Freight Plan strategies. As a result of the analysis, decision makers were better able to assess the robustness of freight strategies and avoid the creation of barriers that may prohibit the freight industry from reacting nimbly to economic change.

SWIM was used for this analysis. Four model scenarios were produced: business-as-usual Reference; Optimistic Economic Forecast; Pessimistic Economic Forecast; and High Transportation Cost. Highlights of the analysis findings include:

- Future demands on the freight system will be large, even if economic growth is muted. Economic inertia causes the dominant commodity mix and geographic flow patterns in Oregon to remain intact, with relatively small changes over time under various scenarios.

- Higher per-mile highway transportation costs result in less congestion, providing the impetus for shippers to increase the length of individual truck tours to increase operating efficiency. Higher transport costs result in reduced miles of travel and hours of travel statewide.
- Households relocate to reduce transport costs, causing urban density to rise and statewide auto miles of travel to fall.
- Commodities have unique and diverse patterns and logistics. Transportation services used to move these commodities are just as varied. Maintaining access to markets is key to economic competitiveness.
- The net results of thousands of shippers and buyers of goods and services are complex and, at times, counter intuitive. Modeling the dynamic nature of these forces provides valuable insight into the collective Oregon freight system needs.
- Assessing system performance and economic impacts is multifaceted. Attention must be given to regional issues, commodity characteristics, industry logistics, and employment patterns when evaluating alternative strategies.
- The largest commodity flows are on the I-5 and I-84 corridors, with significant flows on US-97 and US-20. Exhibit 7-24 shows the total commodity flows in the study area.

Exhibit 7-8: Highway Commodity Flows



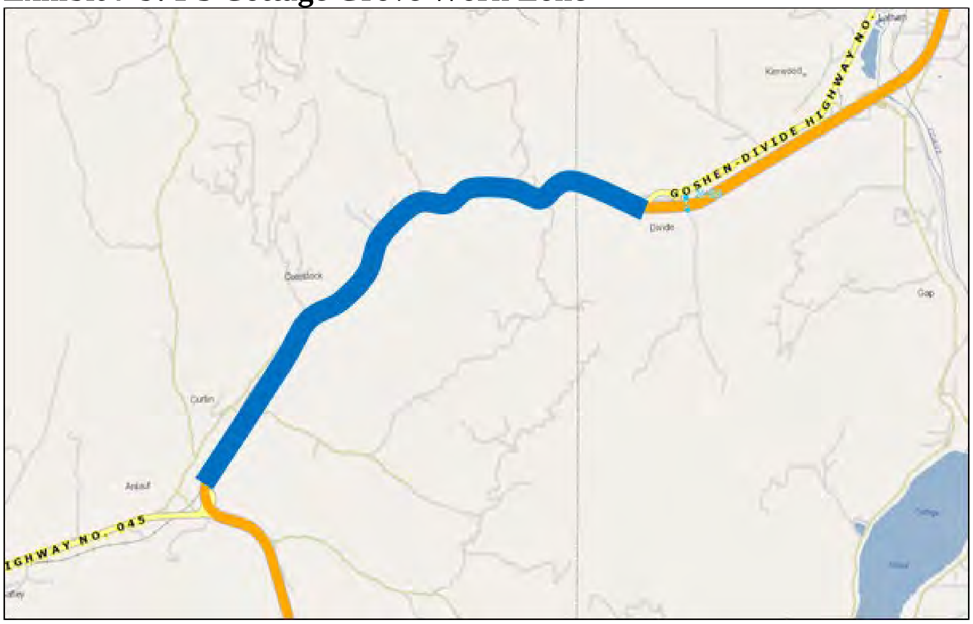
Oregon Freight Plan Modeling Analysis Technical Memorandum, ODOT/TPAU, August 2010

I-5 Cottage Grove Work Zone

The SWIM network was utilized to answer a question from Region 3 regarding an upcoming pavement replacement project. If VMS signs were in place in the Willamette

Valley and in California, what would be the diversion potential away from this section of I-5? Delay was added to the blue portion of I-5 in Exhibit 7-25 below and the model run to determine the potential traffic shifts.

Exhibit 7-9: I-5 Cottage Grove Work Zone



Signage in Eugene and Reedsport has the potential to encourage typical users of OR 38 (Umpqua Hwy), to use OR 126 (Florence- Eugene Hwy). Signs in Eugene and in California have the potential to encourage trucks and some autos to use OR 58 rather than I-5 when traveling North of Eugene or South into California. Using the statewide model, and engineering judgment it would be reasonable to estimate that with the additional signage there is a possibility to remove between 8%-10% of trucks and 5%-10% of autos from this section of I-5.

7.4 Travel Demand Models (Trip-based)

The intent of the travel demand model is to represent travel decisions that are consistent with the actual travel trends and patterns. The decisions are influenced by the available transportation system, the allocation of households and employment, household socioeconomics, and travel costs. Known Oregon travel behavior and relationships from household surveys are used to replicate the impacts on the actual transportation system.

Travel demand models can be used to predict future travel patterns and demands based on changes in the transportation system (i.e., new roads, wider roads with more capacity, closed roads, introduction of CAVs, etc.), changes in the land use (i.e., more residential development, a new industrial site, etc.), and changing demographics (i.e., more or less people in a specific area, access to a vehicle, aging population, etc.).

Travel demand forecasting can test the impacts of critical “what if” questions about proposed plans and policies. Model results can provide users with a variety of information on travel behavior and travel demand for a specified future time frame, such

as forecasted highway volumes for roadways, transit forecasts, and the effects of a proposed development or zoning change on the system. They allow planners to analyze the effects of latent demand and other unanticipated impacts to the system.
It is an important tool in planning future network enhancements and analyzing proposed projects and policies. Information from travel demand models is used by decision-makers to identify and evaluate different approaches to addressing transportation issues and to select policies and programs that most closely achieve a desired future vision. See Chapter 17 for more details on model structures, processes, and application elements.

7.4.1 RTP

Models can be used to quickly assess the entire MPO planning area which may contain multiple cities and the interactions between them. Use of demand- to- capacity ratios can indicate bottleneck areas or areas that potentially need improvements. Conceptual project scenarios can be added to test impacts on the overall network. These can be bundled into groups of projects for specific objectives (capital projects, multi-modal, mobility, etc.). Impacts of land use changes can also be tested, such as in a UGB expansion scenario, nodal development, neighborhood urban centers, etc. Transit and other multimodal benefits can be evaluated depending on the details of the individual networks (i.e. walk, bike and transit) and zone structure. If the model has enough detail, such as economic sensitivities, items like congestion pricing, parking pricing, and tolling can be evaluated. Models can also be used to create and evaluate accessibility, connectivity, and equity measures. Some operational strategies can be modeled such as TDM or ramp metering. Projects that come out of modeling are generally high level such as "Widen to four4 lanes", or "Add overcrossing", etc. which are consistent with the general level of detail available.

7.4.2 TSP-IAMP-Refinement Plans

TSPs, IAMPs and refinement plans typically deal with smaller areas or individual facilities or corridors. Like with the larger regional (MPO) areas, models can evaluate across a single city to determine capacity constraints to eventually determine project concepts. Modeling will be generally more specific such as adding or modifying roadway connections such as a new interchange. Individual facilities can be tested with different speeds or number of lanes or one-way/two-way directions to determine the impact on the city. Land use scenarios with differing levels of growth can be evaluated and compared with a baseline scenario from a localized zone to the whole city.

Some areas have air quality issues that require them to go through an air quality conformity basis which requires improvements on the system not to add more emissions than the specific target values. These can be for CO or particulate matter (PM). Trapped PM from woodstoves has been the focus of most Oregon AQ issues such as in Grants Pass and Klamath Falls. The overall roadway network including any improvements is based on VMT and run through the MOVES emission tool. Models streamline the process by allowing testing of multiple strategies with different mixes of projects. Certain projects could lessen VMT and emissions if trips are shortened or mode-shifted or allow travel at faster speeds. Conversely, some projects like a new interchange could encourage travel and increase VMT and emissions. It is this balance that needs to be obtained in the conformity process.

7.4.3 ABM

The Activity-Based Model (ABM) is a computer-based model used to estimate travel behavior and travel demand for a specific future time frame, based on several assumptions. It includes elements such as roadway and transit networks, a synthesized population and employment data, socio-economic characteristics, and travel costs. It deals with individual persons with a rich set of attributes that influence travel and linked trips or tours (i.e. home to shop to work) instead of groups of households and separate trips (i.e. home to shop and shop to work). This type of model can answer questions in finer detail. For example, demographics of individual users (i.e., low-income users) can be forecasted versus just a single number of trips by purpose from a zone. The ABM micro-simulates tours which are groups of linked trips (i.e. trip chaining) as that is how trips occur. This provides much more context for trips that do not begin or end at home (e.g., the mode for a lunch trip depends on the work commute mode) and allows household interactions for shared vehicle use. Microsimulation of households and persons over an entire day of travel enables the evaluation of pricing strategies in the context of a household budget. An ABM does everything the trip-based travel demand model does, but with considerably more behavioral content.

The ABM introduces two levels of zones with the typical transportation analysis zone (TAZ) created at the census block level and the micro-analysis zone (MAZ) at the parcel level. The non-auto modes are captured better because of the smaller MAZ structure which will make shorter trips more evident.

The ABM application is best used for providing the required detail for long range regional transportation plans (RTP) required by the Federal Highway Administration (FHWA) and Federal Transit Administration (FTA) for metropolitan areas. Currently, ABM is under development for some MPO areas in Oregon.

7.5 Regional Strategic Planning Model (RSPM) (aka GreenSTEP)

7.5.1 Introduction/Purpose

RSPM allows for strategic planning and testing of policy scenarios. Strategic Assessment is the first step in strategic planning. It assesses financially constrained adopted plans and does sensitivity tests of more ambitious plans and resilience to other future trends (e.g., fuel price). Scenario planning results in a preferred scenario. Metro & Lane Council of Governments (LCOG) were required to develop a preferred scenario by legislation. Metro also had to implement the preferred scenario in plans. CAMPO is to run several scenarios with RSPM that precede and inform local plans. These local plans will use more detailed traditional tools to implement the plans.

The strategic nature allows a broad view of policies associated with the development of land use, transportation, energy production, and economic development. These policies can be tested for resilience under uncertainties such as changing demographics, new untested technology solutions, and limited funding. The limited detail allows many high-level policy scenarios to be evaluated. RSPM captures policy interactions by micro-simulating reactions of individual households, primarily using relationships found in the National Household Transportation Survey. RSPM uses simplified inputs and relationships in order to facilitate quick run times for the policy tool. It sets the strategic components (e.g., doubling transit service miles), which complement more detailed traditional models (travel demand models, ABMs) that can be used to develop implementation details (e.g., new transit corridors and/or increased stop frequency).

RSPM produces high-level community outcomes (outputs) such as household travel (average daily VMT to all locations, congestion), health (active mode travel, air quality indicator),[3] environment (GHG emissions), economy (travel costs such as fuel, fees/taxes, and parking), etc. RSPM can comprehensively evaluate sets of local strategies, providing measures to help planners and decision-makers assemble programs to achieve the desired community vision/outcomes acceptable to policymakers. RSPM is sensitive to factors and new policies (i.e. car-sharing) that traditional travel demand models do not include. A key component of RSPM is that it models changes using a budget-based process that enables analysis of policies based on constraints where existing data are limited or does not exist. It also enables analysis of the travel response to pricing (e.g. pay-as-you-drive insurance). RSPM does not include an explicit roadway or transit network but instead uses supply and demand relationships by functional class to approximate congestion and Intelligent Transportation Systems (ITS) policy impacts.

The RSPM model estimates vehicle ownership, vehicle travel, fuel consumption, and GHG emissions at the individual household level. This structure accounts for the synergistic and antagonistic effects of multiple policies and factors (e.g. gas prices) on vehicle travel and emissions. For example, the battery range of electric vehicles (EVs) and plug-in hybrid electric vehicles (PHEVs) is less of an issue for households residing in compact mixed-use neighborhoods because those households tend to drive fewer miles each day. Modeling at the household level makes it possible to evaluate the relationships between travel, emissions and the characteristics of households, land use, transportation systems, vehicles, and other factors. In addition, household level analysis makes it possible to evaluate the equitability of the costs and benefits of different strategies.

[3] RSPM has been connected to the ITHIM (Integrated Transport and Health Impact Modeling) Tool in Portland and Eugene studies, allowing burden of disease (air quality and active mode travel) and safety outcomes.

General categories of RSPM inputs are shown in Exhibit 7-26.

Exhibit 7-10: General RSPM inputs

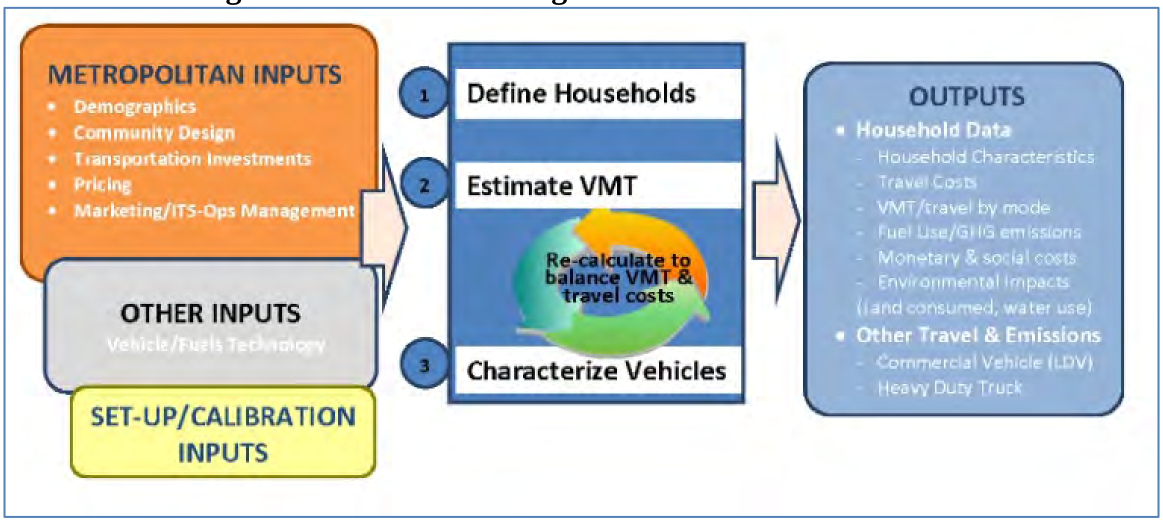
Regional Context	Local Actions		Collaborative Actions	
	Community Design	Marketing & Incentives	Fleet & Technology	Pricing
• Demographics • Income Growth • Fuel Price	• Future Housing (Single- & multi-family) • Parking Fees • Transit service • Biking • Roads	• TDM (home & work-based) • Car sharing • Education on Driving Efficiency • Intelligent Transportation Systems	• Vehicle Fuel economy (mpg) • Fuels • Commercial Fleets	• Pay as you drive insurance • Gas taxes • Road user fee

RSPM simulates how the following characteristics could impact the community outcomes:

- Demographics Trends – Household income, age mix, household size, university group quarters
- Community design
  - Urban characteristics, such as land use (density, mixed use), alternative modes (public transit, non-motorized transportation), and parking management.
  - Road characteristics, such as the supply of freeways and other arterials and the management of incident delay.
- Marketing & Incentives
  - Marketing characteristics, such as the deployment of employer-side and household-side travel demand management programs.
  - Efficiency education programs such as eco-driving, low-rolling resistance tires, and pay-as-you-drive insurance.
- Vehicles & Fuels – for personal and commercial service vehicles
  - Vehicle and fuels technology characteristics, such as fuel economy, proportions of electric vehicles, and fuel carbon intensity.
  - Vehicle fleet characteristics, such as the proportions of autos and light trucks and the age distribution of vehicles.
- Pricing
  - Prices, including fuel price, fuel taxes, mileage taxes (e.g., to cover road costs), congestion charges, and recovery of externalities or social costs, e.g., carbon taxes.

RSPM operates at the individual household level. The treatment of assumptions that determine travel characteristics is simplified, enabling the model to have a high degree of policy sensitivity and interactivity and yet easy to set up and run quickly. RSPM links a series of sub-models that forecast outputs, such as vehicle ownership and household daily VMT. The demand side of the model is disaggregated; it includes a synthetic population generator and an auto ownership model. The supply side is handled in an aggregate way without a detailed transportation infrastructure network. RSPM can be run at a state or MPO level. The MPO level uses census tract level districts to represent different neighborhoods, while the state tools use county zones to be responsive to regional differences. Exhibit 7-27 illustrates the RSPM process. The model distinguishes between households living in metropolitan, other urban and rural areas to reflect their different characteristics in terms of density, urban form, transportation system characteristics, and demand management programs. The environmental outputs of the model include fuel consumption by fuel type, electric power consumption by electric vehicles, and CO_2 equivalents for fuel and electric power consumed.

Exhibit 7-11: Regional Scenario Planning Model Process



RSPM is intended to be used in an environment where there are many unknown policy implementation details (e.g. doubling transit service miles) combined with uncertainty about factors that may or may not be controllable. RSPM is intended to be run at the statewide (at county resolution) or MPO (Census tract resolution) level. It could be run in smaller areas if data are available. RSPM requires construction of a base-year scenario using local data which are then calibrated with Census data (i.e. household size and income). RSPM is predicting household travel, so a conversion factor is necessary from a travel demand model or HPMS to create roadway-based travel within the MPO. Contact TPAU to inquire about use of RSPM for an application.

7.5.2 Outputs

The primary outputs of the RSPM are household travel, fuel, power consumption, and GHG emissions calculations, but other information is produced for households and commercial vehicles as well. The amount of commercial (light-duty) and freight (heavy-duty) travel is calculated as well as associated fuel, power consumption and GHG emissions for those vehicles. In addition, heavy vehicle travel, fuel and power consumption, and emissions are calculated.

Typically, RSPM scenario development includes many unknowns and potential combinations to explore the future uncertainties. This can result in hundreds if not thousands of individual runs. Interactive web-based visualization tools have been developed to effectively access many previously run scenarios allowing users to explore the tradeoffs and outcomes of various policy investment mixes.

7.5.3 Applications

The typical MPO application for RSPM is the strategic assessment. A strategic assessment uses the Regional Strategic Planning Model (RSPM) to estimate future GHG emissions and other outcomes based on state and local conditions. ODOT and DLCD staff work with MPO and local government staff to gather the data needed to develop the model inputs, and ODOT staff runs the model. ODOT and DLCD staff then work with the MPO staff to develop a report of the model outputs. The report also includes possible next steps for the region**.**

A strategic assessment evaluates the region's adopted plans and policies, assesses how far those plans help the region reach its goals over the next 20 years, and identifies alternative paths to achieving those goals. It also identifies the value of state-led actions such as newer clean vehicles and fuels. Largely a technical exercise, the assessment provides information that can help inform decisions about the future, helping communities to understand where the current path leads and what options exist for the region. This can inform plan updates and general decision-making. Additional work may be desired to help answer specific policy questions or to evaluate scenarios to formulate a vision for the region. If additional work is desired, support for scenario planning or additional analysis may be provided.

The purpose of the strategic assessment is to estimate travel and emissions likely to result if adopted plans are implemented and current trends continue. The assessment can provide information about:

- Household travel costs
- Transportation and energy costs
- Air quality
- Mixed-use development
- Health impacts
- VMT
- Travel delay
- Fuel consumed
- Walk trips and bike miles
- GHG emissions

The results of a strategic assessment can help the region determine whether current plans and trends are achieving the outcomes the region wants to see and identify potential actions to better meet the region's goals. The results of the assessment can also help local governments better understand issues and quantify the effect of adopted policies as they review and update the area's transportation plans and make investment decisions. It can also bolster collaboration on policies such as transit, parking, and state-led actions such as implementation of pay-as-you-drive insurance, by quantifying the value of such policies. The effort can inform the public of new policies and the tradeoffs of alternative paths to meet regional goals. In addition, the information provided in the assessment is intended to help local officials decide whether to pursue a more comprehensive analysis of land use and transportation options through formal scenario planning.

7.5.4 Examples

Statewide Applications

Statewide Transportation Strategy (STS): A 2050 Vision for Greenhouse Gas Emissions Reduction (OTC accepted in 2011)

RSPM (previously named GreenSTEP) was built for addressing legislative GHG reduction requirements for ground transportation. The STS process evaluated what it would take to achieve a 75 percent reduction from 1990 GHG emission levels by 2050 statewide. Many policy combinations were evaluated in cooperation with a stakeholder committee in three phases. The STS, through hundreds of runs of the RSPM tool, identified the most effective GHG emissions reduction strategies in transportation systems, vehicle and fuel technologies, and urban land use patterns to accommodate future growth and it showed how collaborative efforts in all areas were required to meet these goals. Beyond reducing GHG emissions, these strategies are expected to reap other benefits, including improved health, cleaner air, and a more efficient transportation system, as noted in the various RSPM outputs. The strategies resulting from the RSPM-based analysis and accepted by the OTC serve as guidance to help meet the state's GHG reduction goals while supporting other societal goals such as livable communities, economic vitality and public health. The STS points to promising approaches that should be further considered by policymakers at the state, regional, and local levels.

MPO Target Rule

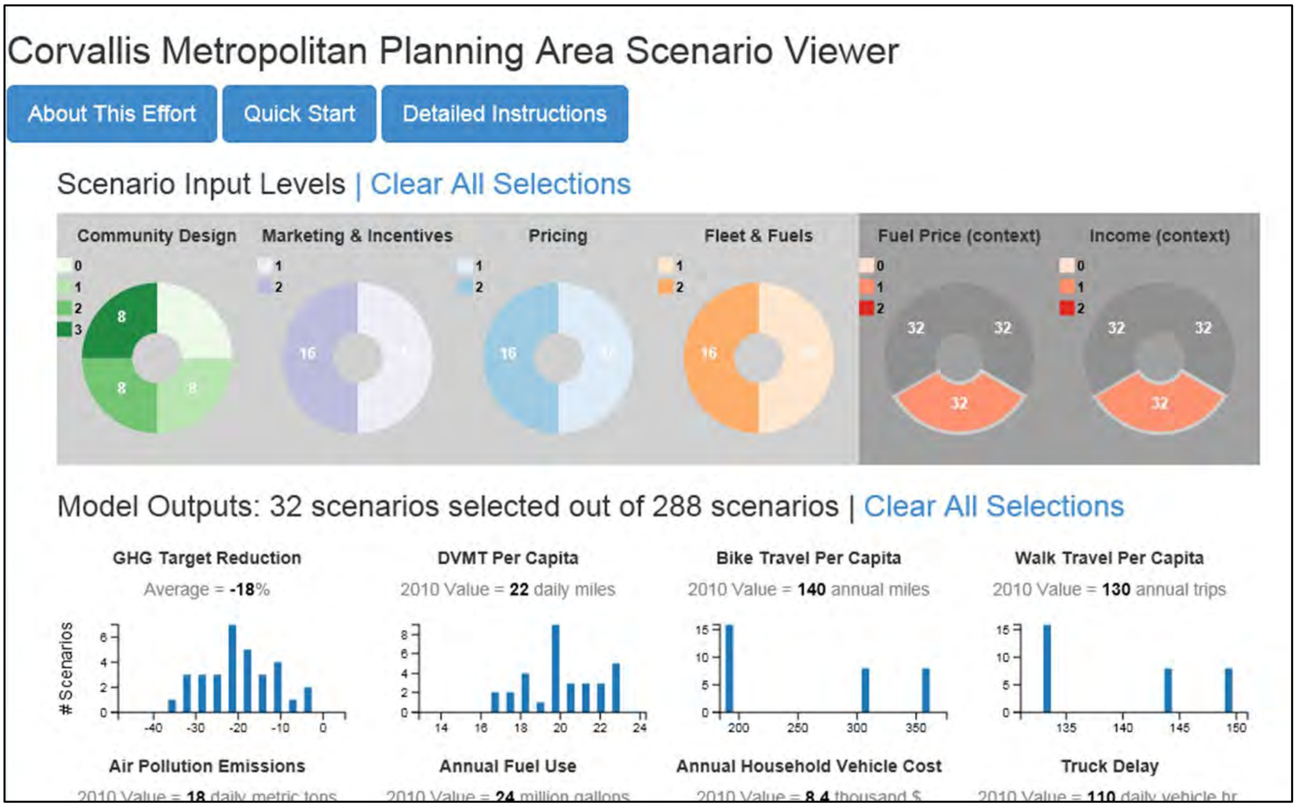
One action following from ODOT's STS is the establishment of GHG reduction targets for each MPO by 2035 by the Land Conservation and Development Commission (LCDC). Although the overall reduction target is set by the legislature, RSPM was used to evaluate the share of this target that could be achieved by state-led actions on vehicles and fuel programs, with the remainder the responsibility for GHG reduction attributed to the local MPOs. RSPM was critical in being able to assess the fleet turnover, multi-modal response to the cost of travel, and land use dependencies (e.g., EV range limitations) important in assessing the impact of new vehicle and fuel technologies across the state.

MPO Applications

AA Strategic Assessment, as supported by ODOT for mid-sized MPOs, is the first step in Scenario Planning as undertaken by Portland and Eugene-Springfield. Corvallis was the first to volunteer to use the RSPM model in a Strategic Assessment. It assessed their financially constrained adopted plans and performed sensitivity tests of more ambitious plans (e.g., more transit, alternative land use patterns) and resilience to future uncertainties (e.g., fuel price). The RSPM scenarios in this effort precede and inform local plans. The local plans (e.g., RTPs, TSPs) will use more detailed traditional tools to implement the strategic understanding resulting from the assessment.

As part of the Strategic Assessment, an interactive viewer was created to help simplify exploring the completed runs. Exhibit 7-28 shows a screenshot of the viewer created for the CAMPO Strategic Assessment project. The viewer shows how community outcomes (e.g., household travel costs, health, walk and biking travel, vehicle delays) change under adopted and more ambitious policies and investments (user input adjustments). Additionally, the user can identify the desired outcomes (e.g., meet GHG reduction targets, high bike trips), and be shown the policy combinations that reach those goals. The viewer can be customized for a particular area. The scenario data that forms the basis for the viewer can also be mined using data analysis software.

Exhibit 7-12: CAMPO Scenario Viewer



7.6 <u>Land Use Scenario Tools</u>

7.6.1 Introduction/Purpose

Objective: Develop sets of plausible future LU patterns and demographic consistent with various constraints, for local review. Travel impacts can be ascertained by combining the resulting land use inputs in a travel model, ABM, SWIM or RSPM.

Land Use Scenario Tools should:

- Allow the use of more thoughtful land use inputs. It is important to tie together the inter-relationships between land use inputs (e.g., dwelling unit type varies with density, income and household size will change with the development of a TOD, population and employment locations are driven by different criteria).

- Be a starting place for framing a land use conversation with local planners using their frame of reference and their resources (e.g. comprehensive plan, jobs-housing ratios).
- Serve as a check on the reasonability of local input variables. – the models are only as good as these key inputs.

Land use models are designed to predict the future pattern of population and employment, typically in an iterative fashion with a travel model. By connecting land use and transport models, land use can respond to market forces such as accessibility and congestion (e.g., locations with good accessibility are more likely to develop than remote locations). Travel can respond to market-driven development patterns (e.g., distributions of residents and employment locations determine activities and create demand for travel on the transportation system). The resulting land use forecast (population and employment by model zone) is a critical input to travel models, including JEMnR, OSUM, SWIM, and RSPM, which assess demand for travel against available infrastructure capacities.

The complexity of most land use models precludes widespread use by planning agencies. However, they are useful tools for forecasting land use inputs to transportation models and for analyzing the land use effects of transportation projects, such as fully considering how a development pattern will impact transportation, induced demand, and the cost of the resulting congestion.

Many tools have attempted various versions of these land use objectives in Oregon. No tool solves the issue fully for all purposes. This section will outline the various applications to date involving alternative versions of several tools and note future opportunities.

7.6.2 LUSDR (Land Use Scenario Developer in R)

LUSDR differs from most land use models in that it is designed to run quickly in order to create a large number (rather than just one) plausible future land use scenarios that meet zoning constraints and respond to market forces (when used iteratively with a travel demand model). This is important because the likely future development pattern can take many forms, the result of many factors that are not easy to forecast. By running many scenarios, one can understand the range of possible development patterns and the likelihood of development for a particular zone. The large number of plausible futures can be used to help evaluate how different possible development patterns will affect the transportation demand and resulting network performance. Using this information, local agencies can have a better sense for how future land use will improve or hinder traffic operation, which can be used to improve land use forecasts that help meet transportation objectives in modeling studies. LUSDR (or variations thereof) can be used to speed up development of land use inputs for travel models by creating a few “bookend scenarios” for local staff to pick from rather than trying to figure out where market forces will combine with available land capacities leading to likely locations for growth 20+ years in the future from now. By making it easier to develop alternative land use futures, LUSDR gives users the ability to test transportation networks under various future land use patterns. This enables testing transportation investment resilience to different futures, which is a risk assessment approach. It allows users to see how transportation infrastructure will perform under a range of possible future land use patterns, and it provides municipalities with a tool to assist with planning future growth.

LUSDR operates on a zonal (sub-regional) scale within an urban area but not at a parcel or urban block level. It requires substantial data and analytic resources to set up for a specific locality. LUSDR requires local zoning/comprehensive plan parameters (such as compatible development types and densities under the zoning designation) which are used to create a large series of plausible future land use development patterns. These development patterns can then be analyzed in a travel demand model, which provides an evaluation of travel resulting from each land use scenario. LUSDR and the travel demand model can be run iteratively through time, passing this information back and forth to simulate the effects of land use and transportation interactions (i.e., accessibility from the travel model is used to determine land development, while development from the land use model is used to determine travel demand). This process allows the testing of many future possible outcomes, to give local and regional agencies insight into how different land use patterns affect the transportation network. LUSDR can also help in other types of analyses, (e.g., GHG analysis in the RSPM model).

7.6.3 LUSDR Variations

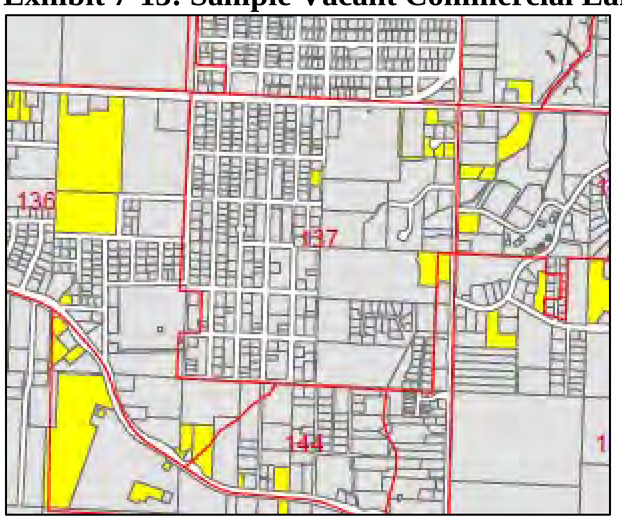
LUSDR-inspired land use scenario methods are now the norm for creating future land use inputs in a new or updated travel demand model scenario. Typically this involves using simplified LUSDR components and inputs. These methods provide an objective method that can simplify the land use development process at the local level by creating plausible scenarios that can be considered and modified rather than a review of TAZ by TAZ to assess the explicit future number of households and employees.

The earliest LUSDR variation was an application in Klamath Falls when a travel model update required new land use inputs. Local staff was challenged by the sheer number of possible future growth scenarios as the available vacant lands within the UGB was so large. Several household and employment growth scenarios were created and shown to local staff which later picked a couple to average together to create the final future scenario.

Another LUSDR variation was completed in both the Coos Bay/North Bend model update and a new model for The Dalles. The method resulted in a significantly shortened future scenario development process and increased understanding of the planned future on all sides. In this method, the local staff is asked to provide growth potential for households and employment which is later translated into TAZ values. This allows local staff to be more comfortable with planning- level terms instead of having to deal with TAZ-level detail. Small jurisdictions usually have little or no exposure to travel demand models, so limiting the technical details will make the process much smoother, especially if some introductory outreach is performed initially. The process ends with creating a single or small set of scenarios that the local staff can choose or modify as needed. The general process is as follows:

- Regional Land Use Control Totals - Use the base year land use total and other sources to determine total population and employment within the model area. Population sources include Oregon Office of Economic Analysis (OEA) and Portland State University Population Forecasts by County/Cities. Employment is typically scaled to achieve historic Census (or adjusted) jobs-household ratios.
- TAZ Land Acres by type - Based on the most current GIS parcel-level database available for the jurisdiction, extract the parcel-acres of the existing and vacant parcels by residential, commercial, industrial, and other property classes. Exhibit 7-29 shows part of the overall GIS plot for vacant developable commercial lands for the City of North Bend staff to which apply TAZ growth potentials to.

Exhibit 7-13: Sample Vacant Commercial Lands in the City of North Bend



- User-defined TAZ Growth Potential - Identify the growth potential by ranking the TAZs with 0, 1, 2, and 3 for no growth (0%), low (50%), medium (80%) and high (100%) with respective to land uses. Exhibit 7-30 shows part of the TAZ growth potential review that City of North Bend staff did as their part of the Coos Bay-North Bend model update.

Exhibit 7-14: Example Growth Potential Allocation from City of North Bend

COMMERCIAL			
	TAZ	DEVELOP. RATING	NOTES
	101	3	
	107		W. of Hamilton = 0; others = 2
	109		S. of Virginia = 2; others = 0
	110	2	
	111	2	
	112		E. of Hamilton = 1; others = 0
	113		S. of 15th & N. of 11th = 2; others = 0
	118	1	
	121	0	

- Available Residential Capacity - Calculate the current population density of residential land with each TAZ and residential land available for development based on the buildable land inventory and potential growth ranking;
- Allocate New Households - Allocate the future year population total in terms of household total into each TAZ according to the relative potential capacity for residential development;
- TAZ Accessibility - Use the existing base year model to figure out the accessibilities to each TAZ as one of the variables to determine the employment capacity;
- Available Employment Capacity - Calculate the employment capacity by retail, service, industrial and other sectors by TAZ according to the available vacant land (by commercial, industrial, and other category) and growth potential rankings; and
- Allocate New Employment - Apply the future total land use forecasts by sector by using the "Long's Model" methodology (a simplified technique that allocates employment growth to zones based on accessibility to potential customers) to allocate the potential employment growth to TAZs based on the buildable land capacity and potential growth rankings.
- Review/Sensitivity Tests - Adjustments can be done by having the local staff review the resulting TAZ plots to see if too much or too little growth by land use sector occurred. By changing the growth potential ranking up or down will re-allocate growth amongst the TAZ's by making certain ones attractive.

The Coos Bay-North Bend model update process ended up with a single land use scenario that was reviewed and slightly adjusted allowing the entire model update to complete on time. Since this process uses more planner-based terms it is important to keep the definitions consistent (i.e. the term vacant means no parcel development, not partially developed).

7.6.4 Place Types

Place Types can be helpful in visualizing and providing a common language for the land use conversation using any of the tools noted above. Adopted for Oregon from SHRP2 C16 RPAT (Rapid Policy Assessment Tool, formerly SmartGAP, a RSPM-derived modeling tool), Place Types provide a criteria-driven topology of land use patterns and allows for ways to visualize and map the different functions and roles of a community. Oregon Place Types are built on TAZ data, consistent with the travel demand model zones, and can be aggregated for use in other models. They use data on the 5Ds (development density, destination accessibility, design, diversity, and distance to transit) built environment of the area, building on TAZ household and employment data (e.g., density, mixed uses), as well as attributes representing urban design/walkability (i.e., link density) and transit accessibility. From these land use coverages, logic and threshold criteria are applied resulting in the following two Place Type dimensions:

- Regional Role (i.e., accessibility to regional job centers)
- Neighborhood Character (i.e., how well the pattern of development supports a multi-modal transportation system)

The full Place Type logic is summarized in Exhibit 7-31 with example outcomes for RVMPO noted in Exhibit 7-32. A visualizer has been developed to enable interactive viewing of the 5Ds and resulting Place Types. Place Types have been shown to be useful in quickly encapsulating the role of different community neighborhoods (e.g., job center, multi-modal main street), identifying locations to best support alternative mode investment (e.g., mixed use areas), and as a check on land use inputs (e.g., expected higher density highlights miscoding of employment data). The use of common Place Type criteria across the state enables useful comparisons for envisioning possible future development patterns (e.g., parts of Rogue Valley are planned to reach the density and multi-modal potential of Corvallis’s near-campus districts, with opportunities to support car-sharing and other modes).

In addition to facilitating the conversation and visualization of current and forecast land use patterns and opportunities for growth, Place Types will soon be utilized in RSPM to better model the effectiveness of TDM programs. Efforts to translate the method to census block-group coverage is underway and will allow stratifying out-of-state data by place type for use in Oregon tools that use Place Type land use classification.

Exhibit 7-15: Logic for Developing Oregon Place Type

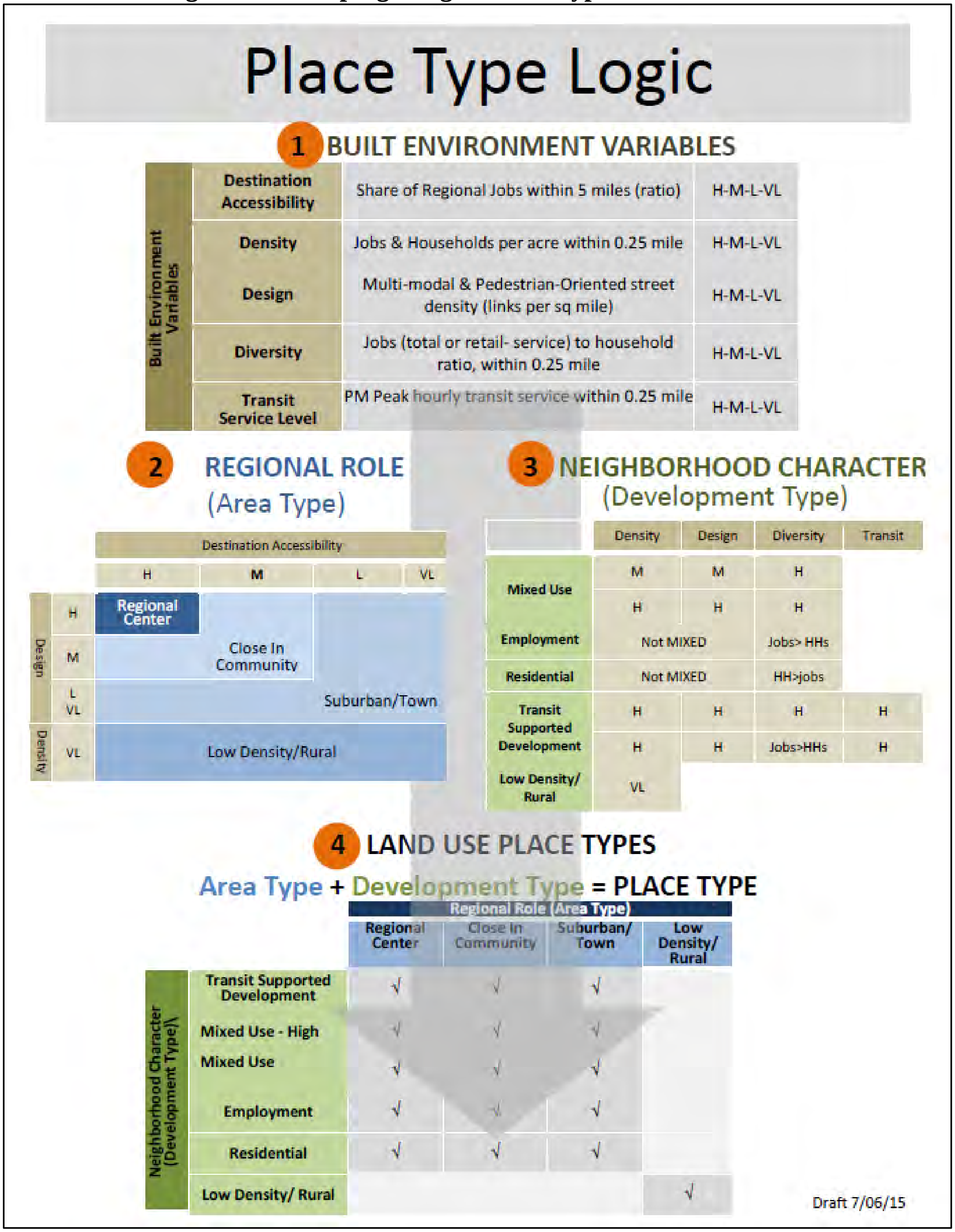
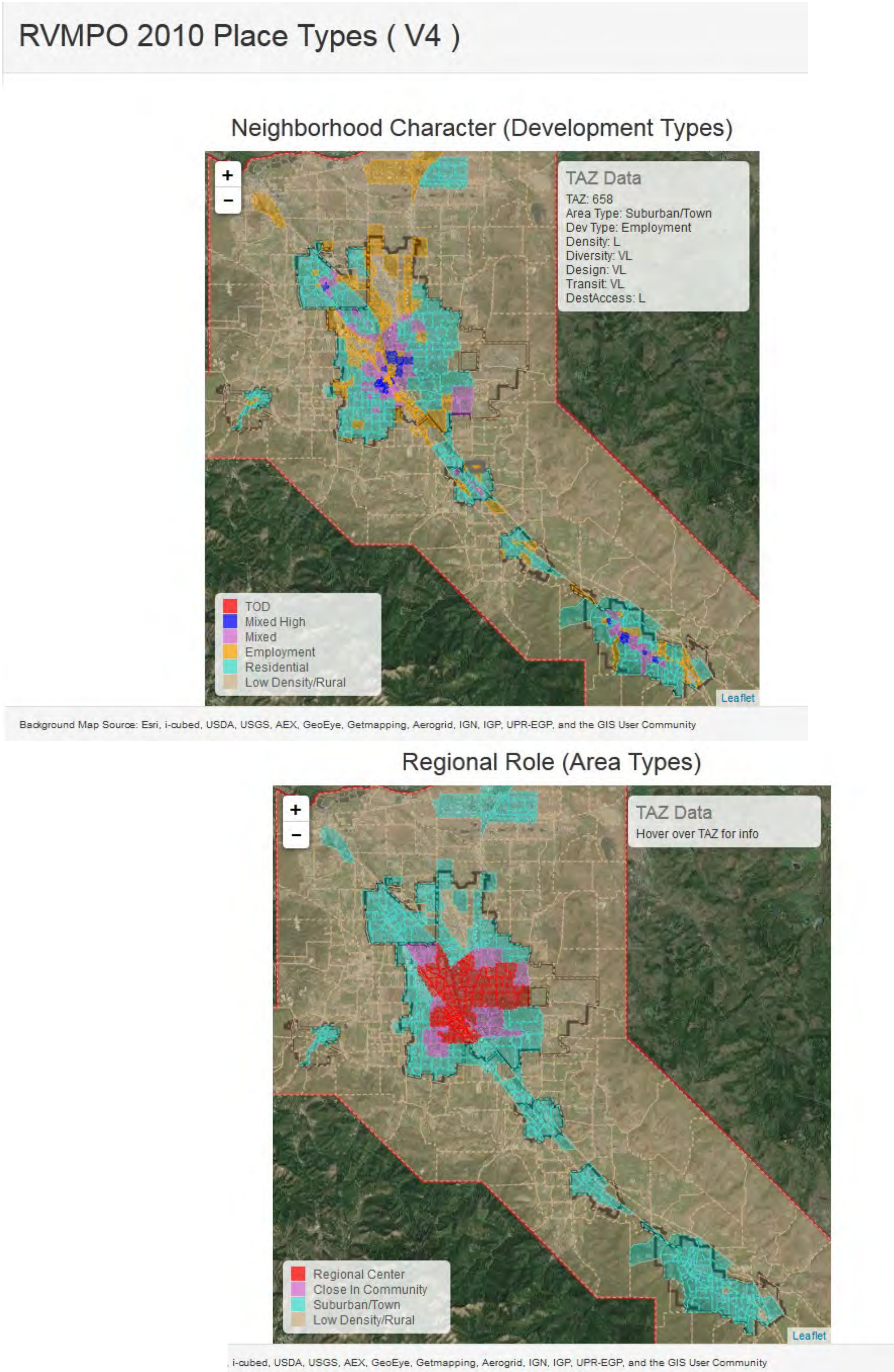


Exhibit 7-16: Example Place Type Maps for 2010 RVMPO



8 MESOSCOPIC ANALYSIS

8.1 Purpose

This APM chapter provides fundamental guidance and overview of an array of methods related to mesoscopic modeling. Mesoscopic methods are rapidly changing based on availability of new tools and data sources, such as the move towards activity based travel demand models. This chapter focuses on methods that have previously been applied in projects involving ODOT and is not intended to be comprehensive for all mesoscopic methods and tools. Other methods not documented in this chapter may be applied, if appropriate, through consultation with TPAU staff.

The purpose of this chapter is to provide an overview of methods and tools available to apply mesoscopic analysis. The information provided in this chapter is intended to provide the user with the information required to understand the general approach for scoping project methodology and understanding the differences between various methods along with limitations and advantages of using each method. For more details on many of the methods in this chapter also refer to NCHRP Report 765.

8.1.1 Overview of Chapter Sections

This chapter covers a broad array of topics related to mesoscopic analysis. Topics included cover:

- Scoping – How to identify approach, tools, and effort based on the analysis needs.
- Subarea Analysis – How to develop subarea models that increase the detail from an existing model.
    - Focusing – General procedures for adding detail and creating a focus area model within a regional model.
    - Windowing – General procedures for selecting an area to “cut” or “window” out into a separate model that can then have additional detail.
- Dynamic Traffic Assignment (DTA) – Analysis considerations and triggers that may lead to analysis that considers traffic routes and travel times that vary by time of day.
- Peak Spreading – General concepts and analysis considerations to identify how congestion spreads from peak periods to larger intervals and the impact on vehicle demand during shoulder periods.

Section 8.1.4 includes references to other chapters of the APM that provide material that may be related to mesoscopic methods covered in this chapter.

8.1.2 Key Definitions

Having a common understanding of the terms in this chapter is necessary for proper implementation of methods and tools. Definitions for terms included in this chapter are included in the Glossary. Selected terms that are needed for fundamental content are shown below.

- **Macroscopic Model–** Aggregate models that have a high-level view of the transportation system and do not include many transportation network details. Macroscopic travel demand models are generally large (potentially regional) in size and focus on general vehicle flows and route choice from one area to another. Streets may be approximated by the average number of lanes, a free-flow speed, and approximate vehicle capacity. Vehicle trips are routed through the network based on algorithms that select paths that minimize the travel time.

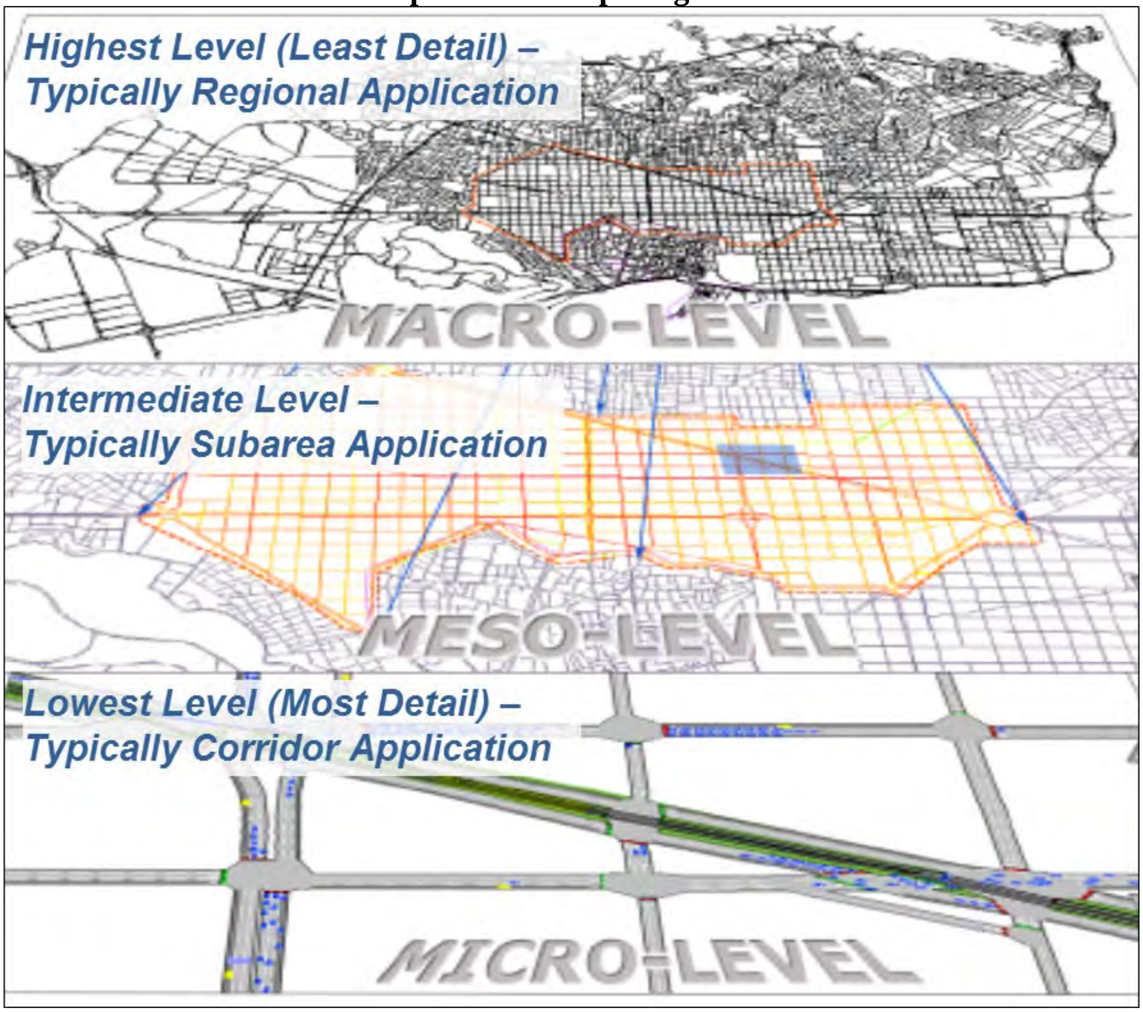
- **Mesoscopic Model** – A hybrid model that includes combinations or approximations of elements from both macroscopic and microscopic models. Mesoscopic models may include a routable network similar to a macroscopic model (with a supplementary origin-destination matrix), while also incorporating more detailed operational elements of the transportation network to better estimate travel time based on traffic operations similar to a microscopic model. Elements from either the macroscopic or microscopic models may be generalized or simplified.

- **Microscopic Model** – Detailed models that are at a fine scale and typically include all streets and components of the transportation system that impact travel. Such elements can include intersection control and striping, pedestrian crossings, transit stops, and even the inclusion of traffic calming measures. Microscopic models typically refer to simulation models that include randomized characteristics and behaviors of an array of drivers and vehicles as they traverse a network. The performance of these models is typically averaged over several "runs" to account for the randomized driver and vehicle characteristics. Unlike macroscopic models, traffic demand values are generally inputs and typically do not result from path choice within the model, therefore, there may not be a predetermined throughput. As a result, assigned traffic volumes at specific locations such as midblock or a turn movement may not match the input demand due to constraints on the network metering flow. For example, queues will build in a microscopic model and only vehicles that can make it through a bottleneck in a given time period will be observed.

- **Multi-Resolution** – The combined framework of an integrated series of models, each built or scaled for the appropriate level of "resolution" and detail given the context for project application and need. The individual model components (resolutions) each can be integrated for a particular project/analysis that benefits from the data analysis and output of the individual tool and level. General levels, or “resolutions”, that may be used to describe models or application that fit a general context include microscopic, mesoscopic, and macroscopic models as shown in Exhibit 8-1.

Appendix 8B provides a user guide for the PTV Vision Suite software which is an example of a multi-resolution tool. The guide is provided to help infrequent users get a quick start in building, importing and exporting networks for analysis.

Exhibit 8-1Multi-Resolution Spectrum Comparing Various Model Levels



8.1.3 Introduction to Mesoscopic Analysis

Transportation analysis methods have traditionally focused on two levels of detail, macroscopic and microscopic. ***Macroscopic*** analysis is concerned with system-wide travel movements; how much travel, of what types, when, how, and by what modes and major routes. Urban, regional, and statewide travel demand models are the primary tools used to do this level of analysis. These tools facilitate the evaluation of the effects of demographics, economics, land use patterns, transportation network configurations, and prices on travel patterns. These models assess how many trips are moving between areas (zones) and along which routes (links). Because they are not built for focused urban area studies they do not typically account for the influences on delay from turning lanes and signal or stop sign controls at intersections. Likewise, macroscopic models provide no information on the location and duration of queuing. Furthermore, they are not calibrated to the level of local streets and points of access to the network (e.g. parking locations).

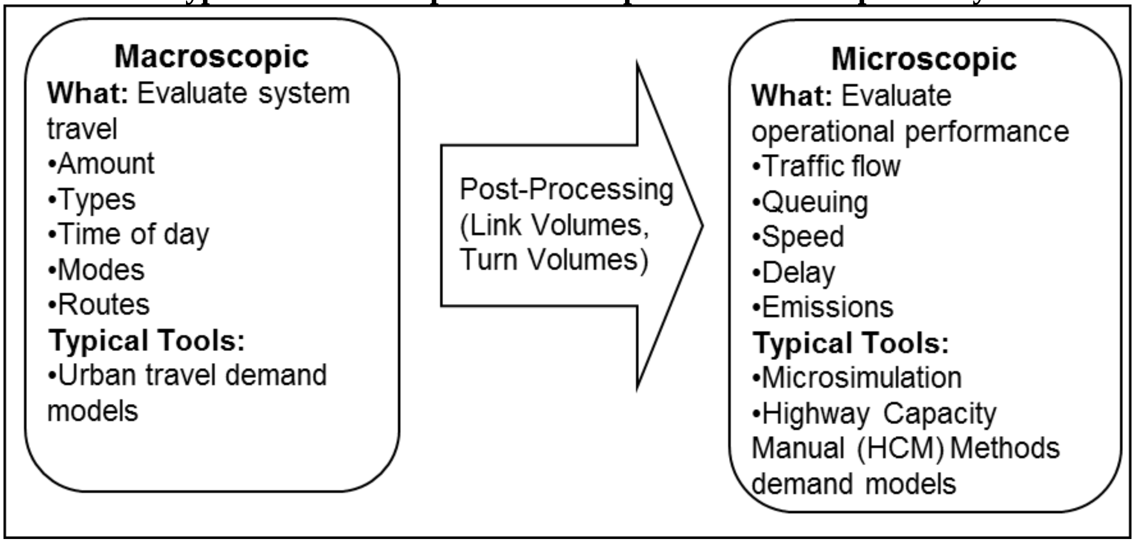
Microscopic analysis is concerned with the operational performance of transportation facilities; traffic flow rates, queuing, speed, and delay. This level of analysis uses micro-simulation models and highway capacity manual methods primarily. These tools facilitate the evaluation of the effects of localized land uses, roadway geometry, and traffic controls on traffic flow characteristics. However, most microsimulation models rely upon fixed post-processed traffic volume inputs from travel demand models to evaluate future year scenarios and are, therefore, only as good as the volumes put into them. Furthermore, project application of microsimulation models typically requires many hours devoted to model development due to the level of detail incorporated in the models. While these models provide a good estimation of traffic operations, they are often not practical to implement as a tool to evaluate or screen a large number of alternatives or a large analysis area.

One major difference between macroscopic and microscopic analysis is that macroscopic models use land use data as the primary input that dictates demand for travel, whereas microscopic use traffic volumes or vehicle trips as the primary input that dictates the demand for travel. The impacts of land use on travel demand are external to microscopic models and assumed to be already accounted for in the microscopic model’s traffic volume inputs. Likewise travel costs (e.g. fuel prices, the traveler’s value of time, transit fares) are direct inputs to macroscopic models dictating travel mode and route choices. These travel costs and decisions are external to microscopic models, and assumed to be already accounted for in the microscopic model’s traffic inputs.

These two levels of analysis (macroscopic and microscopic) are loosely coupled through the transmittal of link and turn volume data and the use of travel demand model post-processing methods. The flow of the data is from macroscopic to microscopic, and there is typically no feedback from microscopic to macroscopic (e.g. queuing calculations do not affect system travel calculations).

The following diagram illustrates these two levels of analysis and their connection.

Exhibit 8-2 Typical Relationship of Macroscopic and Microscopic Analysis



Increasing attention is being given to the combination between the macroscopic and microscopic modeling levels, often referred to as "***mesoscopic***." While the term "mesoscopic" can have various meanings for users in different fields and even to multiple users within the field of transportation, it generally is used to denote a hybrid of microscopic and macroscopic features. Exhibit 8-3 demonstrates the relationship between these three general fields.

Exhibit 8-3 Mesoscopic Overlap between (and Potentially Combining) Macroscopic and Microscopic Modeling Process

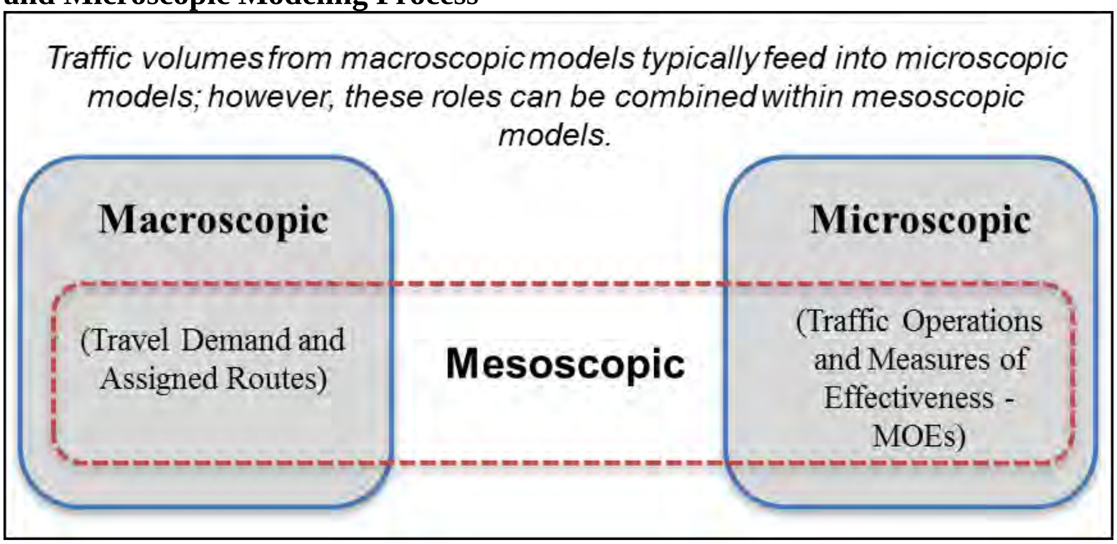


Exhibit 8-4 lists potential examples that summarize key comparisons among typical microscopic, mesoscopic, and microscopic models. These examples are provided for demonstrative purposes and actual characteristics of these models can differ.

Exhibit 8-4 Summary of Typical Differences among Microscopic, Mesoscopic, and Macroscopic Models

	Detail Present		
Model Element	**Macroscopic**	**Mesoscopic (Potential hybrid)**	**Microscopic**
Network Scale (Size)	Region-wide	Varies. Potentially region-wide, but may be smaller, depending on level of network detail and mesoscopic software	Typically a single corridor or small study area
Network Scale (Detail)	Regionally significant routes (generally collector and higher)	Varies. May include all public streets, but could include less depending on network size	All streets and major driveways
Intersection Detail	None (typically a simple node junction of streets without time-penalty and without geometric or control characteristics)	Generally includes types of attributes needed for HCM level analysis (intersection control, lane geometry, basic signal timing, etc.)	Full lane geometry and widths, turn bay lengths, traffic control and striping, signal timing detail for individual phases (if applicable)
Travel Time	Link-based travel times generally rely on volume-delay functions (vdf). Intersection delay is generally ignored or simplified.	Can have a combination of link and intersection travel time, though intersection delay may be less robust than microscopic models	Travel time is based on vehicle interaction and includes acceleration, deceleration, stopped delay, and other associated factors
"Outputs" - Measures of Effectiveness (MOE) / Performance Measures	Vehicle hours of delay, corridor travel time, average distance traveled by users on a link	Provides general MOEs possible in both macro and micro models, without full detail (e.g. queue lengths and impacts) of micro models.	Intersection turn delay, corridor travel time, 95^{th} percentile vehicle queue length, etc.
Routes / Assignment (Is route diversion possible?)	Yes (routes vary based on relative "cost" of all potential routes, typically travel times)	Yes	No (traffic volumes and route paths are typically a fixed input and route options/ diversion are not present)

Needs

Mesoscopic analysis capabilities can help meet emerging analysis needs and overcome limitations of the traditional process and tools (macroscopic forecasts fed into microscopic models). In particular, the following needs can be addressed:

1. Operational Impacts
    - Restricted funding and less interest in making major changes in the roadway system are shifting the focus of planning to system management and leveraging the effects of changes in operations and many small system improvements. In addition, there is increasing interest in modeling operational improvements that improve reliability and reduce incident-related congestion. Macroscopic models are typically not applied in a manner that is precise enough to model the effects of traffic operations or minor improvements. Microscopic models can analyze these effects, but cannot do so at a large-scale systems level. Moreover, microscopic models require a very large amount of data and are impractical to develop for an entire urban road system.
    - The focus on operational improvements and smaller system changes requires the use of more precise performance measures in order to distinguish the relative benefits of alternative choices. Macroscopic models use travel time and speed measures having limited precision (e.g. do not capture the vehicular delays upstream of system bottlenecks). These measures are necessary in order to predict travel patterns, but precision has been limited for reasons of computational tractability and because precision may not have been important for the regional decisions being made. Microscopic models can produce these measures, but only for a relatively small portion of the system.
    - The operational impacts of interactions with other modes (such as bus stops or rail crossings) can impact travel time along a route and thus route choice. For example, the impacts that transit vehicles have on auto travel times are ignored in most macroscopic models (e.g. additional traffic congestion caused by buses on mixed use roadways and vehicular delays occurring at rail crossings). These impacts can be explicitly modeled and evaluated using microscopic models.
2. Congestion Impacts
    - As traffic congestion severity, extent, and duration grow, the travel time and speeds estimated by macroscopic models become less able to reflect actual travel conditions. Macroscopic models have a limited ability to account for congestion on adjacent (downstream) links, as well as within (adjacent lanes) a given roadway segment. Consequently, macroscopic models are less able to account for the effect of traffic congestion on travel patterns. This has been a significant consideration in the Portland metropolitan area for some time, and is starting to become an issue in other areas of the state as well.

- The effects of severe congestion may not be reliably accounted for by microscopic models. How are the system constrained traffic volume and trip inputs for microscopic models developed for future year scenarios (as the severity, extent, and duration of traffic congestion grows) without the help of macroscopic travel models?

3. Sensitivity/Risk Testing/Alternatives Analysis
    - There is increasing interest in assessing the risks associated with uncertain futures (e.g. amount and distribution of land uses, fuel and other travel costs, government policies, regional economic and funding issues). The assessment of uncertainty and risk requires a more comprehensive analysis than is presently done. In the past, assumptions were made about many factors (such as land use, transportation network changes, etc.) in order to limit the number of alternatives needing to be analyzed. This has been necessary because of the amount of work required to develop models and process outputs from macroscopic models to be input into microscopic models and time required to adequately develop microscopic models.

Mesoscopic modeling has the potential for meeting these emerging needs and overcoming existing limitations by leveraging the strengths of both macroscopic and microscopic modeling:

- Operational performance can be calculated with more detailed metrics than is currently done by macroscopic models in order to account for the effects of smaller changes to the system and to distinguish smaller differences between alternative improvements. The calculations in mesoscopic models are less precise than those of microscopic models; this reduces the data needs and model run-times so that entire regional transportation systems or large portions of transportation systems can be modeled.
- Since mesoscopic models can make more detailed calculations of performance, in some cases they may be a substitute for microscopic models for the purposes of uncertainty and risk analysis. For larger systems with severe congestion issues, a mesoscopic model will allow for realistic results to be generated at a lower detail level but still meet the needs of the project development process. The calculation of the performance measures by a mesoscopic model would reduce the need/use of microscopic models for alternatives testing and greatly increase the number of scenarios that could be analyzed within a given amount of time and resources. Microscopic models would continue to be used for more detailed analysis of a limited number of scenarios.
- Mesoscopic models can provide a mechanism for feeding back better estimates of travel times reflecting very congested conditions to macroscopic modeling processes for forecasting travel demand for successive iterations in the macroscopic models. This might be done by incorporating the mesoscopic model into the macroscopic model, or by using the mesoscopic model as a post-processor of the macroscopic model.

When to Consider Mesoscopic Analysis

Many considerations exist that could lend mesoscopic procedures being applied for an analysis. Some of these considerations for mesoscopic application could include:

- Do a large number of system network alternatives need to be analyzed or screened at a system level? Is it not feasible/cost-effective (or appropriate) to model all alternatives in microscopic analysis? Does macroscopic analysis not provide adequate detail for providing relative comparisons among alternatives?
- Do network alternatives include operational impacts or improvements that may not be captured with a macroscopic model?
- Do network alternatives have the potential to impact system circulation and routing due to the outcome of the resulting traffic operations and flow?
- Does a level of congestion exist that may not be captured with a macroscopic model?

Any of the items listed above may be an indication that mesoscopic analysis would be beneficial for project application. While defined triggers do not exist, generally a mesoscopic approach will provide additional benefit in cases where both macroscopic traits (such as route choice) and microscopic traits (such as traffic operations, performance measures on duration and severity of congestion and queuing) are desired in a hybrid environment.

8.1.4 Related APM Chapters

Several other chapters provide related guidance. These chapters and the relation each has to this chapter are listed here.

- Version 2 Chapter 2: Scoping Projects – Guidance on scoping transportation analysis work. Includes comparisons of analysis tools and guidance on tool selection.
- Version 2 Chapter 3: Transportation System Inventory – Includes information about data collection that may be needed for application of the methods covered in this chapter.
- Version 2 Chapter 5: Developing Existing Year Volumes – Includes information about volume development that may be needed to estimate demand for some of the tools and methods covered in this chapter.
- Version 2 Chapter 6: Future Year Forecasting – Provides information about developing future forecasts and includes subarea assignment methods. The chapter also describes multiple forecasting methods, including use of travel demand models.
- Version 1 Chapter 8: Traffic Simulation Models – Procedures for microsimulation model development and calibration.
- Version 1 Chapter 10: Analyzing Alternatives – Includes information about developing sets of alternatives that may be analyzed or screened using the tools and methods covered in this chapter.

8.2 Subarea Analysis

This section provides an overview of types of subarea analysis and general considerations for application. Two types of subarea analysis, focusing and windowing, are covered in greater detail in following sub-sections.

8.2.1 General Considerations

The analyst has a number of tools and methods available for application. A critical component of any project is first selecting an appropriate tool and then determining how to best apply that tool. In many cases, the best tool available may not be adequately refined for the intended application. In the case of a regional travel demand model, the model may be constructed and calibrated to a regional scale. However, with the appropriate additional refinement, the model (tool) may be applicable to additional uses. Creating a subarea model is a common example of applying model refinement for more rigorous use beyond the original scale of the model.

This subsection describes general subarea refinement and considerations. A ***subarea*** is a specified area that is identified for refined analysis. This may require a model or tool that includes additional detail beyond what is used for areas outside the subarea in order to adequately capture the desired level of analysis. The subarea may be similar to a "study area" identified through the analysis as the general area included in the analysis, but often these areas differ based on the degree of analysis needs and tools present.[1]

Once an analyst is aware of pre-existing models, the decision about model applicability and potential to use the model for another purpose must be made. While this decision process should be coordinated with Transportation Planning Analysis Unit (TPAU) and documented, the following considerations may indicate the potential for applying subarea analysis:

Is the "base model" (agency or regional model) appropriate for further project use?

- Does the model boundary fully include the study area?
- Does the model consider the appropriate time period (hour of day, season, etc.)?
- If land use changes are being investigated, are the scale and type of uses appropriate for the model? Smaller magnitude areas, such as some traffic impact analysis (TIA) for the development review process, may not require land use adjustments in the model if it is being used to forecast background growth. However, larger magnitudes may require land use adjustments or even be beyond the scope of the model. Additional documentation on this matter is provided separately.[2]
- Does the model boundary include key locations outside of the study area that may influence operations within the study area (downstream interchanges, over capacity intersections, etc.)?

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[1] Considerations for identifying subarea boundaries are presented later in this section.
[2] Modeling Procedures Manual for Land Use Changes, TPAU, February 2012.

Is subarea refinement needed for model application?

- Does the model include all transportation facilities relevant to the study? This may include study intersections as well as parallel facilities or alternate routes, which may be needed for gauging traffic diversion.
- Is the zone structure detailed enough for the analysis and are centroid connectors placed in a way that will not impact the outcome of the results?
- Is the model sensitive enough to test the range of alternatives under consideration, such as intersection control and geometry or signal timing changes?
- Is a specific zone or unique land use not adequately captured due to the regional scale of the model?
- Is the zone and centroid connector structure detailed enough to perform Origin-Destination (O-D) matrix estimation procedures if needed?
- What will be the ultimate use of the model data? Do HCM procedures need to be performed on study intersections? Will the model be exported to microsimulation?
- Do other modes of travel need to be considered that are not included in the regional model? For instance, do a significant amount of trucks or heavy vehicles need to analyzed separately and/or account for restricted routes? Are there other modal network elements?
- Is a more realistic assignment needed due to extensive peak period congestion?

The considerations above are intended to serve as a general guide for determining model application potential. However, given project context, other considerations may exist and coordination with TPAU is needed. The following sections explore these general considerations in more detail.

8.2.2 Overview of Focusing and Windowing

Focusing and windowing are two general types of subarea models that are commonly applied. These approaches can be applied for both macroscopic and mesoscopic analysis models. While each of the two methods shares similarities, there are also distinct differences.

Focusing is the practice of adding additional refinement and detail to a model within the structure of that model. The additional resolution may be added to the supply (transportation network) or the demand (zone structure or loading). In either case, the full function of the model is maintained.

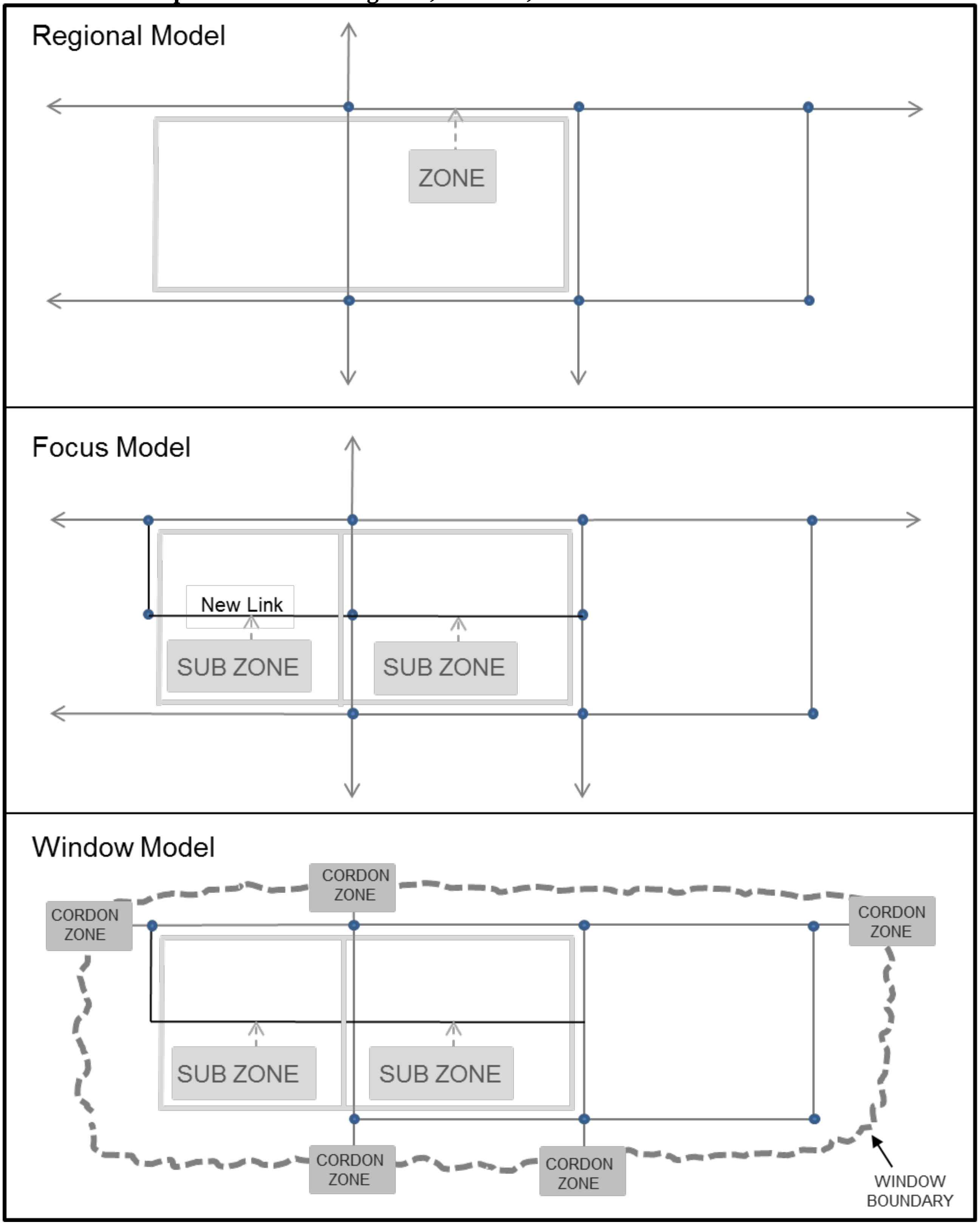
Windowing involves cutting out a portion or component of a model. Often, this will include a "window" of the transportation network in the subarea that then creates cordon (external) areas at the edge of the subarea. Windowing, like focusing, is applied to allow additional refinement or modification. However, because windowed models are separate from the original model, they are not held to the same requirements for consistency[3] and integration with the full model. This allows for testing changes from the original model (such as travel demand intensity or trip assignment technique). Results of the windowed model are specific to the window itself and are not necessarily relevant to the original model. In some cases, the entire model area may be windowed in order to test scenarios (or framework) that differs from the base regional model.

Refer to Appendix 8B Section 1 for guidance on windowing using PTV Visum software. If the model is in the Emme platform, see Section 4.1 for guidance on converting the model network to Visum.

Exhibit 8-5 demonstrates the general differences in network refinement that a focus and window subarea model would add to a base model network.

[3] Coordination with TPAU is necessary to determine if the level of adjustments are appropriate and reasonable for the windowed area model.

Exhibit 8-5 Comparison of Base/Regional, Focused, and Windowed Models



8.2.3 Scoping

The following sections provide an overview of scoping subarea modeling efforts, including general considerations for selecting windowing and focusing methods and level of effort for subarea application.

Tool Selection

In some instances either windowing or focusing may be an appropriate approach for refining the model area. However, in other cases one approach may be preferred due to limitations or flexibilities built into each method. Exhibit 8-6 highlights some situations to guide the analyst in selecting an appropriate methodology.

Exhibit 8-6 Considerations to Guide Selection of Subarea Model Methodology

Consideration	Focusing	Windowing
Alternate Traffic Routes	• If alternate (or parallel) traffic routes exist outside the subarea, they may be affected by non-uniform refinements made to the model within the focus area.	• Routes from the regional model will be fixed and constrained at the location of subarea boundary or cordon "cut" lines (outside the windowed area)
Assignment	• Constrained by base (regional) model	• Adjustments to assignment type and parameters allowed
Demand Intensity	• Constrained by base (regional) model	• Adjustments to demand intensity of zones allowed
Model Size	• Can be applied for any size model	• Applying a "cut" of the model and creating new cordon zones may be desired to reduce run time
Model consistency	• Edits require consistency within the framework of the regional model and running the full 4-step model	• Edits are allowed to deviate from regional model framework, however do not provide the ability to re-run the full 4-step model directly.
Future Work	• May be used for future projects	• Generally can only be used for the subject analysis
Intersection Control	• Typically not available in regional models	• Intersection control through nodal delay-based assignment
Level of Effort	• (Varies – see Exhibit 8-7)	• (Varies – see Exhibit 8-7)
Full 4-Step Model Runs	• Full model could be rerun with the additional detail of focus area	• Model generally reassignment only. A full model run (trip generation, distribution and mode choice) is not performed.
Transit	• Typically not affected in subarea focus model.	• Transit assignment and route functionality may break due to unique nature of coding.

Data

Types of data that are necessary to perform subarea analysis (such as focusing and windowing) may include:

- Traffic volumes – Needed to validate the model traffic volumes, particularly if new network is being added
  - Roadway tube counts
  - Intersection turn counts
  - Truck percentages
- Land use – Provides guidance for refining zone structure to allocate land use or trips to new zones.
  - Zoning map
  - Detailed land use metrics (in size of building and/or number of employees)
  - Aerial photos
- Network geometry – Characteristics needed to reflect infrastructure and control
  - Intersection and corridor geometry (lane use and channelization)
  - Posted speed
  - Capacity
  - Intersection control – type, orientation, signal timing plans, etc.
- Traffic operations – Existing traffic operations for model validation/calibration.
  - speed
  - travel time
- Traffic patterns – What is the distribution and what routes are being used?
  - origin-destination patterns
  - routes

Some data may only be needed depending on the detail of the subarea model (such as intersection control and geometry), while other data may only be needed if the subarea model will be later converted to microsimulation.

Resource Needs

The ability to scale the tool for the analysis is an important benefit of subarea modeling. The degree of effort needed to apply a subarea model can vary greatly based on the amount of detail that is put into refinement of the key model elements. Various model elements that may be refined for subarea analysis are listed in Exhibit 8-7 along with a sample of “low”, “medium”, and “high” levels of effort. For the purposes of this table, these levels are defined (as approximations) to be:

- Low Effort – Typically completed within a couple hours
- Medium Effort – Can be completed within a day
- High Effort – May require several days or more to complete

Exhibit 8-7 Subarea Scalability to Identify Resource Needs

	Range of Effort (Refinement and Application)		
Consideration	**Low**	**Medium**	**High**
Network Refinements	Add a link in study area or adjust speed/capacity	Review and refine all existing links and link properties for advanced street network detail. Add key missing links.	Review and refine all links in model and add any missing public streets (local, collector, arterial, etc.)
Zone System	Use existing zone system or split a single zone	Split zones (5 or less) in the immediate study area	Split zones (5+) over a broad area that may expand beyond the study area
Connectors	Use existing connectors, add connectors, or adjust placement to be loading to optimal roadway in study area	Review and refine placement and potentially weights of connector loading in general study area	Review and refine placement and weights of connector loading in broader area
Assignment Method (Window only)*	Use existing assignment method. May still require subarea “cut” of windowed area.	Modify assignment method to incorporate other considerations (such as fixed intersection delay by movement)	Implement detailed intersection operations elements that incorporate HCM intersection turn delay.
Intersection Control	No control setting, or simple node type of signalized/unsignalized	Control includes intersection geometry, specific control type and orientation	Control includes detailed type, geometry, and settings such as signal timing
Data Mining	Daily or peak hour link volumes	Peak hour/period link and turn volumes at key locations in study area	Peak hour/period link and turn volumes at locations in and beyond study area (and cordon zones if they exist)
Validation/ Calibration	Check/observe regional links in study area.	Check/observe regional links and additional subarea link network	Check/observe and plot the differences for all link and turn count locations to measure model calibration

Note: *Applies to Window models only since focus area models do not change assignment methods.

8.3 Focusing

The following sections provide an overview of how to apply focusing:

- Process – What steps are involved?
- Potential Issues – What are issues that may occur?
- Calibration – How to know when the model is “ready”?
- Application – When is this tool needed?
- Examples – When has this tool been used?

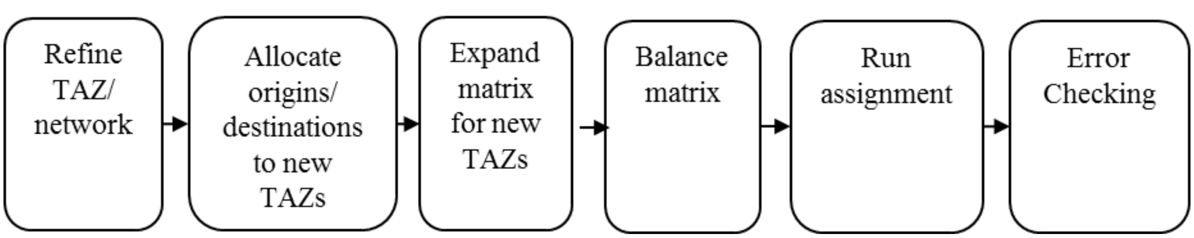
8.3.1 Process

While not explicitly listed as a repeated step below, continuous, on-going error checking and quality control are vital components of the model coding and application process. For purposes of simplification, this on-going component is not depicted.

The following steps, simplified in Exhibit 8-8, are used to apply focusing[4]:

1. **Refine Transportation Analysis Zone (TAZ)/network** - The standard TAZ system and network are refined in the subarea. This may include adding elements that were not included in the base model (such as additional roads), or adjusting elements that were in the base regional model (such as modifying link capacity).
2. **Allocate origins/destinations to new TAZs** - The origins and destinations for the original or “parent” TAZs from the model run are allocated to the disaggregated TAZs using one of several possible weighting schemes (Section 8.4.4).
3. **Expand matrix for new TAZs** - The assignment trip matrix from the model run is expanded to reflect the revised TAZ system.
4. **Balance matrix** - The expanded matrix is balanced using the disaggregated origins and destinations from Step 2.
5. **Run assignment** - A new trip assignment is run with the trip matrix from Step 4 and the refined network.
6. **Error checking –** Compare model output to check for consistency with base model.

Exhibit 8-8 Focusing Process



[4] *Modeling Procedures Manual for Land Use Changes*, ODOT Transportation Planning Analysis Unit, February 2012.

Potential Issues

The following potential issues may arise with a subarea model that uses focusing:

- Additional detail or street network may attract/detract travel demand from the study area. Creating additional transportation system links in the study area may increase the total capacity, thus attracting additional trips. Conversely, adding additional detail (nodes/intersections, traffic control, etc.) may disproportionately increase the travel time through the study area and detract traffic. These potential issues may indicate that windowing is more appropriate.
- Data may not be readily available to validate the traffic volumes on street network that is vital to the focused area.

8.3.2 Calibration (Model Checking)[5]

Analysis Procedure Manual Version 2 Chapter 6 includes additional information on model checks. It is assumed that the subarea models are based on regional travel demand models that have already been subject to a formal calibration and validation process. The following information is provided for calibration of subarea models for project application and may not be appropriate for regional models.

The calibration process is a critical step to ensure that the focused subarea model is compatible with the base model. If issues arise during the calibration process, it may be indicative that another method (such as windowing) may be more appropriate. The calibration process should generally include the following considerations, at minimum:

- Confirm the focus model is consistent with the base model
  - Is the overall demand matrix sum unchanged? There is risk of modification to the overall matrix demand when splitting the zone structure and creating new TAZ. Any change in overall demand (which may result when splitting zones with internal trips) should be minimal (typically less than one percent, unless documented otherwise).
  - Do links outside the focused area generally retain the same assigned traffic volumes? Significant shifts in traffic volumes may indicate incompatibility with the base model and a need to perform windowing as an alternate approach.
- Confirm the focus model is compatible with the base model
  - For a given screenline, do links within the focus area carry the same total assigned traffic as the base model? If the network was refined, traffic assignment may shift within the focus area, but the overall assigned volumes across a screenline should not significantly differ.
- Confirm the focus model is providing realistic results

[5] The following material includes components of both a validation and calibration process. It is assumed that the regional demand models would have been formally validated and calibrated during model development. The information in this section focuses on the calibration of subarea models for project application.

    - Do the sums of connector volumes for a zone match the total zone demand?
    - Does the routing for traffic into and out of new zones make sense? Common checks include a select zone analysis (Emme traffic assignment software) or a zonal flow bundle analysis (Visum traffic assignment software).
    - Do the routes, origins, and destinations for traffic using a new link make sense? Common checks include a select link analysis (Emme) or a link-based flow bundle analysis (Visum).
- Confirm the model reflects existing traffic data
    - Does the magnitude of resulting traffic demand match base year traffic count data? Differences in data are typically present due to:
        - Difference in model year/period and date of data collection (e.g., average weekday model for year 2010 compared to July 2013 count data).
        - Count data collected from various source and time periods. Assigned traffic volumes in the subarea model must balance (unless there is a connector between locations), so unbalanced count data can lead to incompatibility of the dataset and create more difficulty for the calibration process.
        - The amount of driveway trips related to pass-by and retail use, or the detail of street network included in the subarea model, may not be captured in the regional travel trends present in the regional travel demand model.
    - If assigning multiple vehicle classes, does the traffic demand match classification data?
    - Does the distribution of traffic match O-D patterns anticipated from license plate, Bluetooth, cell-phone, or other data survey means?

8.3.3 Key Considerations (Application and Issues)

> *In general, the level of detail and refinement in any particular element should generally reflect the detail of other model elements.*

The following questions and considerations are intended to help guide decisions regarding application of focus models. In general, the level of detail and refinement in any particular element should generally reflect the detail of other model elements. These considerations are not absolute but are intended to function as a general guidance when developing an approach and methodology.

Study Area Size/Boundary

When selecting an area of the model to focus, consider the following:

- Larger focus areas provide more opportunity to refine the model, but also require additional effort.

- Smaller focus areas may limit the effects of focusing.
- Having a constrained focus area with limited connectivity to surrounding areas helps to retain consistency with the base model assignment.
- A focus area should be adequately large that changes within the focus area do not impact areas outside of the focus area.

Network Link (Street) Refinements

- When refining the attributes of the street network, parameters should be consistent with assumptions made about the rest of the network. For example, streets within the focus area should have compatible capacity to similar streets outside the focus area.
- Is the structure of the focus area street network appropriate for the level of detail that is needed for the analysis? Are key streets missing from the model? These may include facilities that affect circulation as well as locations that require detailed traffic forecasts.
- Are the attributes that are used in the regional model appropriate for the focus area? Models are often calibrated to a regional scale and may not include unique details that reflect specific locations. Attributes such as speed or capacity may need to be refined to better reflect true conditions.

Network Zone Refinements

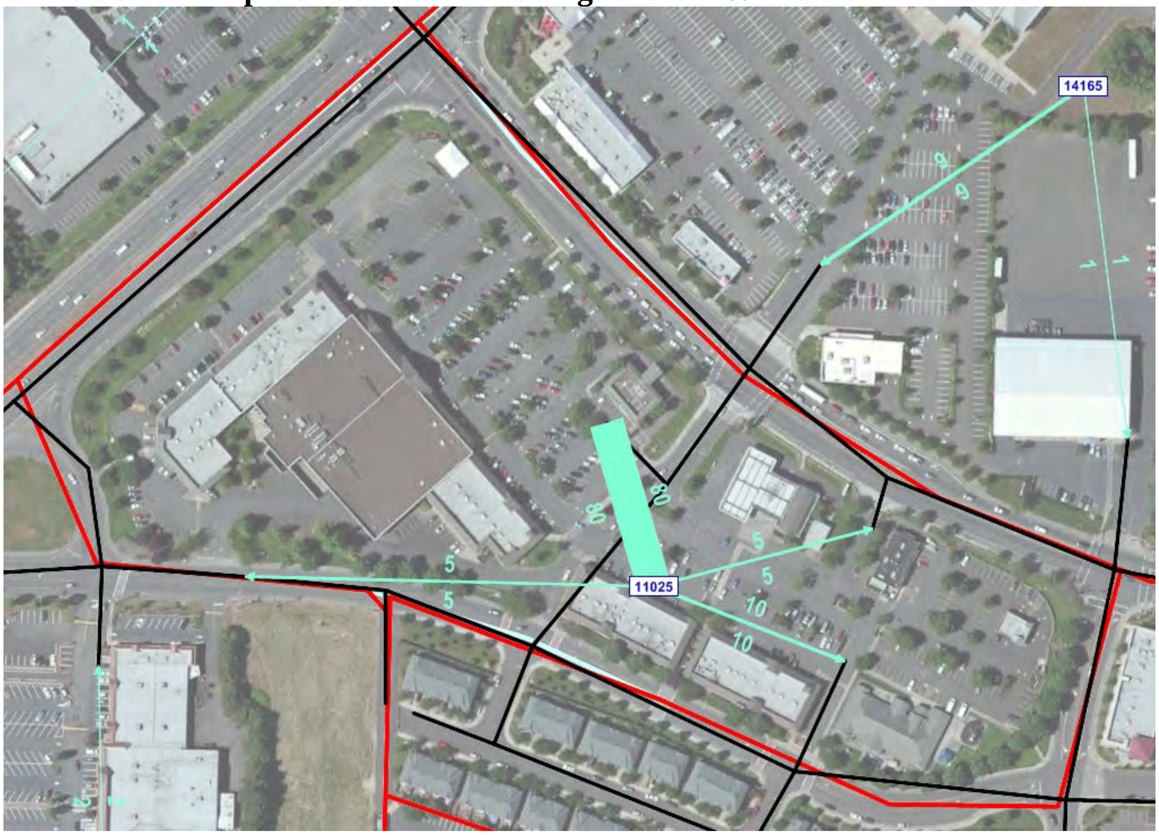
- Can a zone be split based on different land use types? This often can help to allocate the portion of trips from the original zone to the split zones.
- Are there constraints that affect connectivity (such as rail, topography, water, etc.) that drive the need to split a zone to better control loading?

Network Connector Refinements

Depending on the modeling platform, treatment of connectors may be used as a substitute for splitting zones. However, some modeling platforms may not provide the flexibility for multiple connectors per zone with different weighting allocations.

- Location of connector loading should best represent the real-world condition (location of internal streets, driveways, or parking) as is feasible given the model detail. Exhibit 8-9 provides an example of using connector loading to better represent actual network conditions. In this example, each connector for Zone 11025 is placed to represent approximate driveway locations and is given a relative weight based on the amount of trips that are served by the land uses at each driveway. In some cases additional link network and detail may be needed for realistic connector loading.

Exhibit 8-9 Example of Connector Loading for Real-World Condition



- The amount of connectors may need to be increased depending on the magnitude of zone trips and the uniformity of real-world loading throughout the zone. Commercial zones typically have less loading points due to the location of specific (high volume) driveways. Residential zones that may not require specific loading points that represent major driveways but instead multiple connectors that distribute these trips along the edge of the zone as local street circulation would provide.
- The weighting or portion of trips loading to each connector may be set based on the configuration and available data. A common case may be a commercial zone that includes multiple driveways. If traffic counts are available for the driveways, connectors can be weighted to replication these driveways and better reflect actual conditions. In other cases it may make sense to uniformly load (equal weight) connectors across the zone area to distribute demand more evenly – as in the case of a residential zone with uniform house coverage.
- What is the real-world circulation within the zone and should all connectors be used? Is internal circulation possible and is connector (driveway) use based on driveway delay or some other attribute? If so, movement within a zone may occur and trips (in some cases) may not use a given connector and/or connector weights may change. The level of internal circulation also may impact the amount of crossing driveway traffic (e.g., traffic from an eastern driveway heading to the west and traffic from a western driveway heading toward the east, versus all

western driveway traffic heading to the west and all eastern driveway traffic heading to the east).

Intersection Control

- What methods are applied to determine intersection capacity and impedance (delay) in the regional model? If none are applied, application of intersection control is likely beyond the scope of focusing and may be better suited for windowing (which provides more flexibility and deviation from the regional model).
- What level of detail is appropriate given the fidelity of other model elements? Models generally fit into one of the following levels of intersection control:
    - Level 1 – intersection control is ignored
    - Level 2 – some control types are coded (e.g., signalized intersections are flagged) with an equal delay given to all movements
    - Level 3 – all control types are coded and a look up delay is applied for all/some turns. This may consider influences of major and minor street approaches.
    - Level 4 – all control types (and potentially geometric details) are coded and delay is calculated at each intersection movement by a method such as HCM.

Section 8.5.4 (Windowing Key Considerations) includes additional information on application of intersection control and node delay.

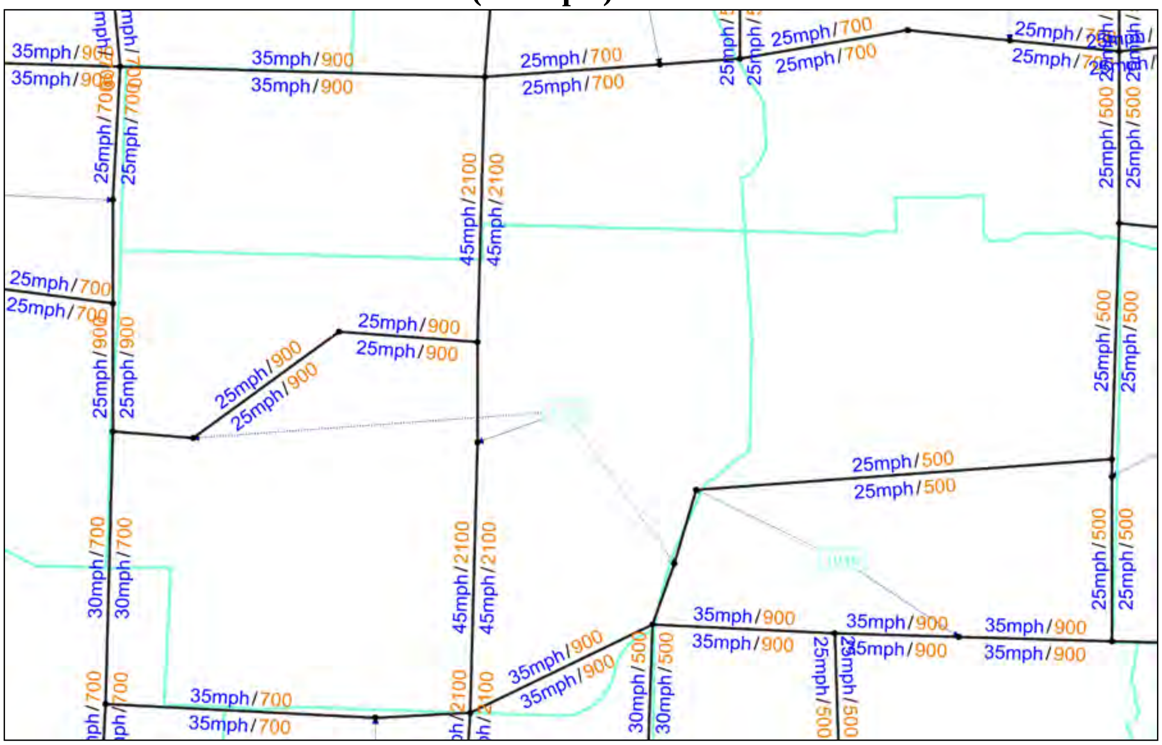
Ramp Meters

- Ramp meters can create bottlenecks that can significantly restrict the throughput based on timing parameters. To reflect this “hard” capacity restriction, a volume delay function that has significantly more delay when over capacity may be an option for emulating a ramp meter.
- Additional solutions for reflecting ramp meter impacts may be available depending on the presence of intersection control elements in the model.

8.3.4 Examples and Scaling

As listed in Exhibit 8-7, the focusing methodology can be applied in varying degrees and scaled to fit the needs of each project and analysis. The following examples demonstrate different scales of application. For reference, Exhibit 8-10 provides a sample of a typical regional base model. This model includes a sparse system of links that represent the major roads (typically collectors and arterials), a zone network, and limited connector loading.

Exhibit 8-10 Base Network Model (Example)



Examples for different scales of applying focusing are shown in Exhibit 8-11 and Exhibit 8-12.

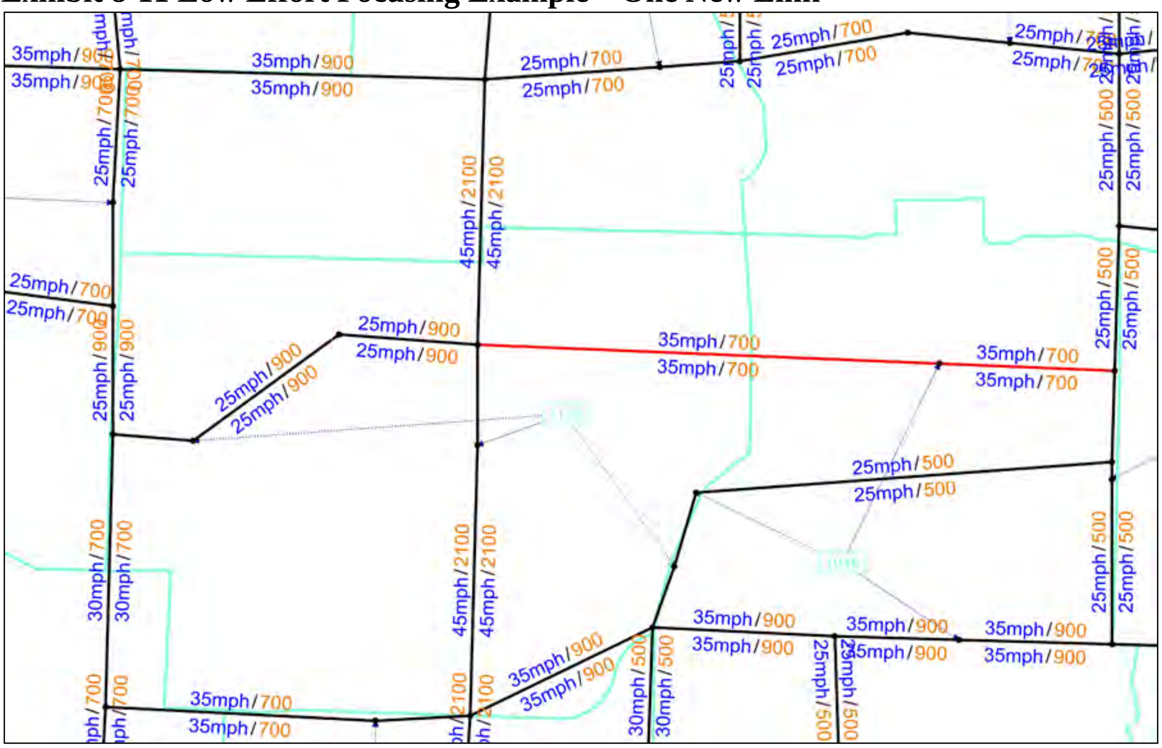
Focus Model Example 1: Low Effort

A low effort application may include making a minor refinement to the travel demand model, including any combination of the following:

- Add a new link or adjust the attributes of a link
- Add connector location or weighting
- Split a zone

Exhibit 8-11 shows an example of adding an additional link and centroid connector to refine the network. The new link (relative to the base network shown in Exhibit 8-10) is shown in red. These two refinements (new link and new centroid) may be all that are needed to focus the subarea model, and represent a minimal, low effort application. Low effort applications may also involve adjustments to the zone system (disaggregation), or minor additional link and centroid connector edits.

Exhibit 8-11 Low Effort Focusing Example – One New Link



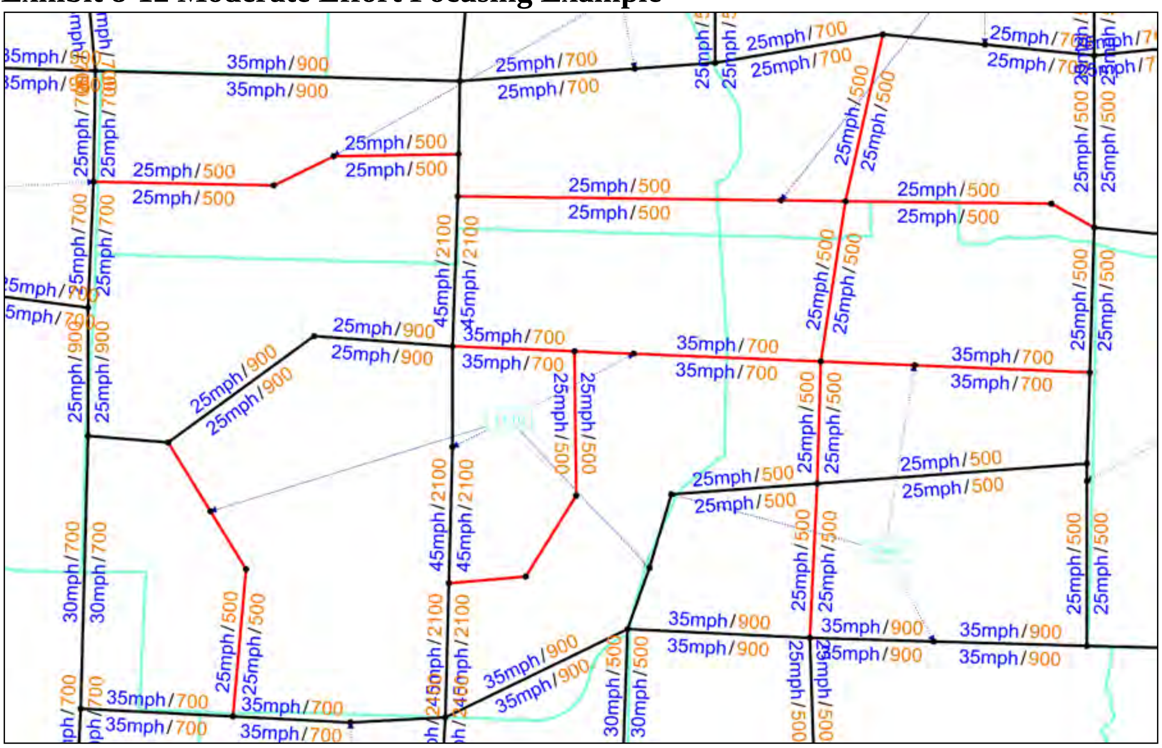
Focus Model Example 2: Moderate Effort

A moderate effort application may include making refinements to several elements in the travel demand model, including any combination of the following:

- Adding several new links or adjust the attributes
- Add connector location or weighting for several zones
- Split several zones

Exhibit 8-12 shows an example of adding several additional links and centroid connectors to refine the network. Similar to the previous example, the new links (relative to the base network shown in Exhibit 8-10) are shown in red. These additional refinements (beyond the level of detail shown in Exhibit 8-11) represent a moderate effort application and would further refine the assignment of traffic in this subarea. Moderate effort applications may also involve adjustments to the zone system (disaggregation).

Exhibit 8-12 Moderate Effort Focusing Example



8.4 Windowing

Windowing is primarily intended to be a motor vehicle reassignment exercise to improve traffic circulation routing and analysis within a subarea. It is likely that continuity of certain elements of the full regional model, such as the ability to perform full 4-step model runs and assigning transit, will be broken by windowing a subarea model. If these components are critical for the application, consider performing a focus model or other methods.

The following sections provide an overview of how to apply windowing:

- Process – What steps are involved?
- Potential Issues – What are issues that may occur?
- Calibration – How to know when the model is "ready"?
- Application – When is this tool needed?
- Examples – When has this tool been used?

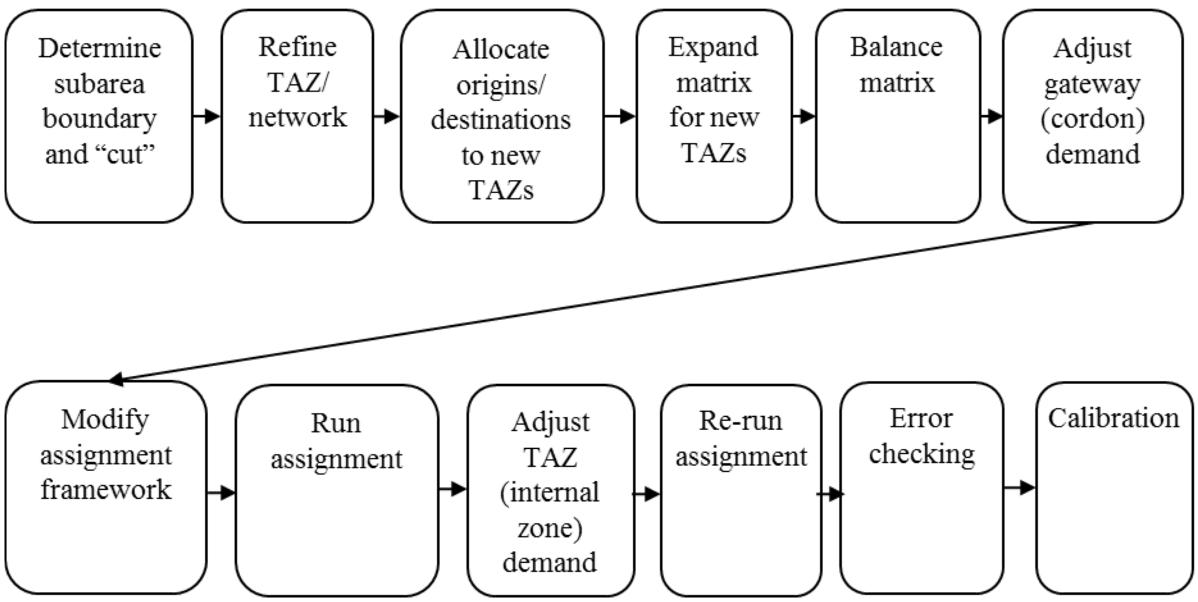
8.4.1 Process

The following steps, simplified in Exhibit 8-13, are used to apply windowing. Note that the windowing process contains more steps than the focusing process shown in Exhibit 8-8. Steps that are also included in the focusing process may have more details in Section 8.4. Additional information, including key considerations and checks, is provided in the following sections.

1. **Determine subarea boundary and "cut" –** Identify locations (links) that will become "cordon" zones with fixed demand from the regional model run. The selection of the boundary is a key decision since demand will be fixed.
2. **Refine TAZ/network** - The standard TAZ system and network are refined in the subarea. This may include adding elements that were not included in the base model (such as additional roads), or adjusting elements that were in the base regional model (such as modifying link capacity).
3. **Allocate origins/destinations to new TAZs** - The origins and destinations for the original or "parent" TAZs from the model run are allocated to the disaggregated TAZs using one of several possible weighting schemes.
4. **Expand matrix for new TAZs** - The assignment trip matrix from the model run is expanded to reflect the revised TAZ system.
5. **Balance matrix** - The expanded matrix is balanced using the disaggregated origins and destinations from Step 3.
6. **Adjust gateway (cordon) demand –** Adjust the demand at the windowed gateways as appropriate for analysis needs (such as adjustment to another analysis year or season).

7. **Modify assignment framework –** Adjust the assignment parameters[6] (such as assignment type and method for determining cost or travel delay), as appropriate, to meet the analysis needs.
8. **Run assignment** - A new trip assignment is run with the trip matrix from Step 5 and the refined network.
9. **Adjust TAZ (internal zone) demand –** Adjust the demand for internal zones as appropriate for analysis needs (such as scenario testing and scaling for additional growth or development). Verify that the resulting demand matches the target values.
10. **Rerun Assignment** - A new trip assignment is run with the updated demand parameters.
11. **Error checking –** Compare model output (such as link and/or turn assignments and general routing patterns between zones) to check for consistency with base model.
12. **Calibration** – Compare model output to data sources and make adjustments to calibrate model.

Exhibit 8-13 Windowing Process



8.4.2 Potential Issues

The following potential issues may arise with a subarea model that uses windowing:

- The subarea needs to be sufficiently large to capture variations in traffic circulation patterns among alternatives. Since the edge of the subarea becomes

[6] While windowing provides some flexibility to add additional detail and refinements that may support using different assignment techniques than the regional model (such as moving from a link-based volume-delay function to an intersection turn-based delay calculation), it is critical to coordinate with TPAU and ODOT regional staff to confirm these methods and assumptions are appropriate for the network detail.

fixed demand, traffic circulation that may be impacted outside the study area is not considered.

- The additional flexibility ("more knobs to turn") with windowed models can require more care and make them more difficult to calibrate. The iterative process that considers adjustments to the network, demand, assignment parameters, and other elements, often requires that those elements are revisited after subsequent adjustments have been made to other parameters. Conceptually-sound adjustments are critical.

8.4.3 Demand Adjustments

Demand adjustments (scaling or changing the demand for a zone or specific O-D pairs) allow a layer of flexibility not present in focus area models that can be used to account for subarea demand magnitudes and traffic patterns that may not be captured in a regional scale model. While subarea models should typically stay consistent with the regional demand model, there may be cases where demand adjustments are needed. Section 8.5.5 includes additional considerations for demand adjustments.

- Demand adjustments may be needed for a number of reasons (including, but not limited to, adjusting to count data for another analysis period/year, or capturing internal trips ends and driveway counts along a corridor) to represent demand well enough that the model has the ability to adapt to network modifications and alternatives.
- A sample application of a demand adjustment could include the following analysis scenario:
    - The purpose of the analysis is to test a variety of transportation improvements, which include operational improvements such as intersection control and lane configurations.
    - A windowed area model is created that includes a high level of detail for the traffic network in order to capture the operational behavior and impacts in the model. This model uses intersection-level turn delay based on HCM methodology.
    - Traffic circulation in the model depends on the delay calculated for turn movements in the system. In order to capture existing traffic flow patterns, a realistic approximation of the travel demand between zones is needed to produce accurate turn delays in the model.
    - A comparison of the subarea model and traffic count data indicates that some of the gateway (cordon) traffic volumes in the model do not match well with count data.
    - A demand adjustment is performed in order to scale the gateway traffic volume (or other isolated internal demand that is not related to routing errors) closer to actual count data. By having a model demand that reflects actual count data, the model will be able to better account for traffic circulation patterns that result from changes in the transportation system. By better estimating the traffic circulation, the model will also better estimate actual traffic volumes on the road system.

- Demand adjustments typically rely upon having traffic count data that demonstrate the basis for the adjustment. However, it is important to be mindful of daily traffic fluctuations, which may be 10% on a day-to-day basis.
- Demand adjustments (as noted in Section 8.5.4) should only be performed after exhausting other model checks and calibration items related to assignment routing in the model.
- Demand adjustments should not be performed indiscriminately without a conceptual basis for why the adjustments are being made.
- Demand adjustments require existing year count data or similar thresholds to act as constraints to determine the degree of adjustment. Demand adjustments made to future year models should be applied in such a way that the resulting model growth (difference between the original base and future models and differences between the adjusted base and adjusted future models) does not change. Demand adjustments are performed to better fit traffic data and routing behavior in the model, not to influence the growth projected by the model (unless analysis year is changed).

8.4.4 Calibration (Model Checking)

Analysis Procedure Manual Version 2 Chapter 6 includes additional information on model checks.

The calibration process is a critical step to ensure that the focused subarea model is compatible with the base model. If issues arise during the calibration process, it may be indicative that another method (such as focusing) may be more appropriate. The calibration process for windowing is similar to the process used for focusing (Section 8.4.3); however, some key differences exist due to additional allowances and flexibility that the modeler has through the windowing process. The calibration process for windowing should generally include the following considerations, at minimum:

(Before demand adjustments or network refinements)

- Confirm the window model is consistent with the base model
    - Are demands at cordon zones or gateways on the edge of the windowed model consistent with the assigned link volumes at these locations in the base regional model?
    - Do internal TAZ retain their original scale of demand (total trips in and out)?
- Confirm the window model is compatible with the base model
    - Does rerunning the assignment procedure for the windowed area give similar assignment results as the regional base model?

(After network refinements)

- Confirm the window model is providing realistic results

    - Do the sums of connector volumes for a zone match the total zone demand?
    - Does the routing for traffic into and out of new zones make sense? Common checks include a select zone analysis (Emme) or a zonal flow bundle analysis (Visum).
    - Do the routes, origins, and destinations for traffic using a new link make sense? Common checks include a select link analysis (Emme) or a link-based flow bundle analysis (Visum).
- Confirm the model reflects existing traffic data
    - Does the magnitude of resulting traffic demand match base year traffic count data?
    - If assigning multiple vehicle classes, does the traffic demand match classification data?
    - Does the distribution of traffic match O-D patterns anticipated from license plate, Bluetooth, cell-phone, or other data survey means?

(After demand adjustments)

- Confirm the demand adjustments had the intended effect
    - Are adjusted totals for in and out trips by zone consistent with the targets?
    - Does the overall matrix sum change by the intended amount (if applicable)?
    - Do demand levels at zones that weren't intended to change remain the same?
- Confirm the window model is providing realistic results (again)
    - Do the sums of connector volumes for a zone match the total zone demand?
    - Does the routing for traffic into and out of new zones make sense? Common checks include a select zone analysis (Emme) or a zonal flow bundle analysis (Visum).
    - Do the routes, origins, and destinations for traffic using a new link make sense? Common checks include a select link analysis (Emme) or a link-based flow bundle analysis (Visum).

Given the unique characteristics and applications of subarea models, an appropriate level of calibration (e.g., metrics such as R^2 values for observed versus modeled link or turn volumes) for one model may not be sufficient for another model. For that reason it can be difficult to determine when a model is calibrated "enough", since it ultimately depends on how the model is applied and the decisions that will be made with the analysis. However, at a minimum, the above process and qualitative checks should be followed to ensure that due-diligence is performed, though additional checks and actions may be required. In addition, the analyst/modeler should provide documentation that shows that convergence criteria (such as R^2) were tracked through the calibration process.

8.4.5 Key Considerations (Application and Issues)

The following considerations are intended to help guide decisions regarding application of window models. In general, the level of detail and refinement in any particular element should generally reflect the detail of other model elements.
These considerations are not absolute but are intended to function as a general guidance when developing an approach and methodology.

Study Area Size/Boundary
When selecting an area of the model to window, consider the following:

- The boundary of the windowed area creates gateway or cordon zones that have fixed demand based on the assigned link traffic in the regional base model.
- Refinements to the regional base model may be needed before applying the windowed cut in order to refine demand at the cordon zone locations.
- Since demand is fixed at cordon locations, ensure that the model is sufficiently large to test the set of alternatives that will be analyzed. Running a sensitivity test in the regional base model that includes the most extreme network alternative can be used to determine the area of impact and potentially the size of windowed area model that may be needed.
- Since demand is fixed at cordon locations, it is generally beneficial to minimize the number of cordons and select a boundary that has less network redundancy. Constraints to network connectivity (rail, water, extreme topography, etc.) can provide physical boundaries that serve windowed models well.

Network Element Refinements (Links, Zones, Nodes)
When adding refinement to the network elements in a windowed subarea model, many of the same considerations for focusing should also be applied and are provided in Section 8.2.4. Additional considerations that result from the added flexibility that windowing allows:

- Do other attributed need to be considered and coded in the windowed area that may not be included in the regional base model? Unique elements within the windowed area (downtown parking, pedestrian activity, etc.) may be considered and added to the network attributes. These elements may also need to be accounted for in the assignment process.

Demand Adjustments
When applying demand adjustments, consider the following (and items noted in 8.5.3):

- Applying network refinements before demand adjustments aids in error-checking these edits before introducing additional changes (demand).
- What is the demand adjustment trying to achieve and what is the appropriate level of detail to meet this need? Some adjustment scenarios may require simply applying a factor to a single zone, while other adjustment scenarios may include a more elaborate process that considers changes to distribution.
- What type of demand needs to be adjusted? Will all model trips be adjusted, or will adjustments be made to specific internal zones and/or cordons?

- Do the trip length frequencies of the adjusted demand matrices remain similar to the original demand matrices?

Future Year Scenarios
When preparing future year scenarios, consider the following:
- What is the most appropriate method for combining the windowed area network with the future year demand? The most efficient process often includes windowing the future demand matrix and inserting it into the windowed base year model network and then adding additional improvement projects (if any) to reflect the future model road network.
- Were demand adjustments made to the base year model that needs to also be applied to the future year model? Section 8.5.2 provides additional information about conducting demand adjustments and application for future year models.

8.4.6 Examples and Scaling

As listed in Exhibit 8-7, the windowing methodology can be applied in varying degrees and scaled to fit the needs of each project and analysis. The following examples demonstrate different scales of application.

Window Model Example: Astoria Downtown

The Astoria-Warrenton model was windowed to capture potential circulation impacts related to potential traffic control changes in the Astoria downtown area. The base regional model is shown in Exhibit 8-14 and includes Astoria, Warrenton and the surrounding rural areas.

Exhibit 8-14 Astoria Warrenton Regional Model

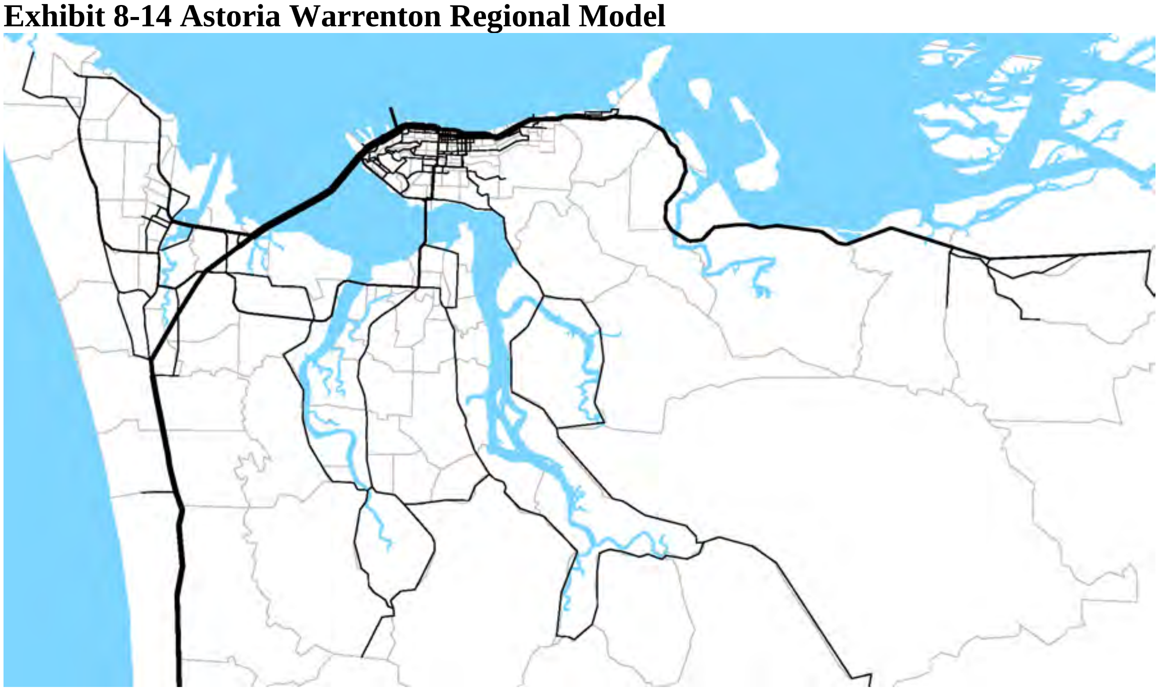
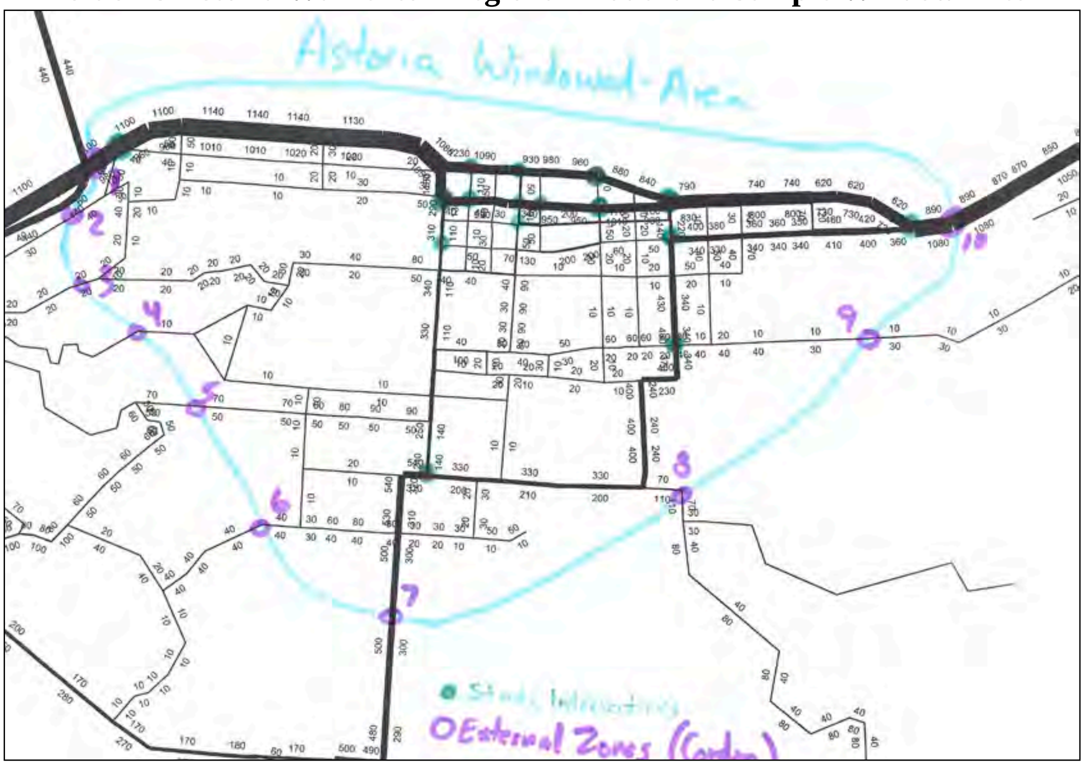


Exhibit 8-15 shows the Astoria downtown area in the regional model, as well as a sketch showing the proposed window boundary and cordon zones. This area was selected to minimize the amount of streets in the regional travel demand model that would be cut, while allowing for a large enough area to sufficiently capture potential traffic circulation changes due to future transportation network alternatives. Specifically, the window boundary was selected to account for and balance the following needs:

- Adequate size to include subset of study intersections and circulation impacts
- Minimize the number of cordon zones (cut base regional links)
- Zone boundaries were retained and not split (windowed matrix retained total trips for internal zones)

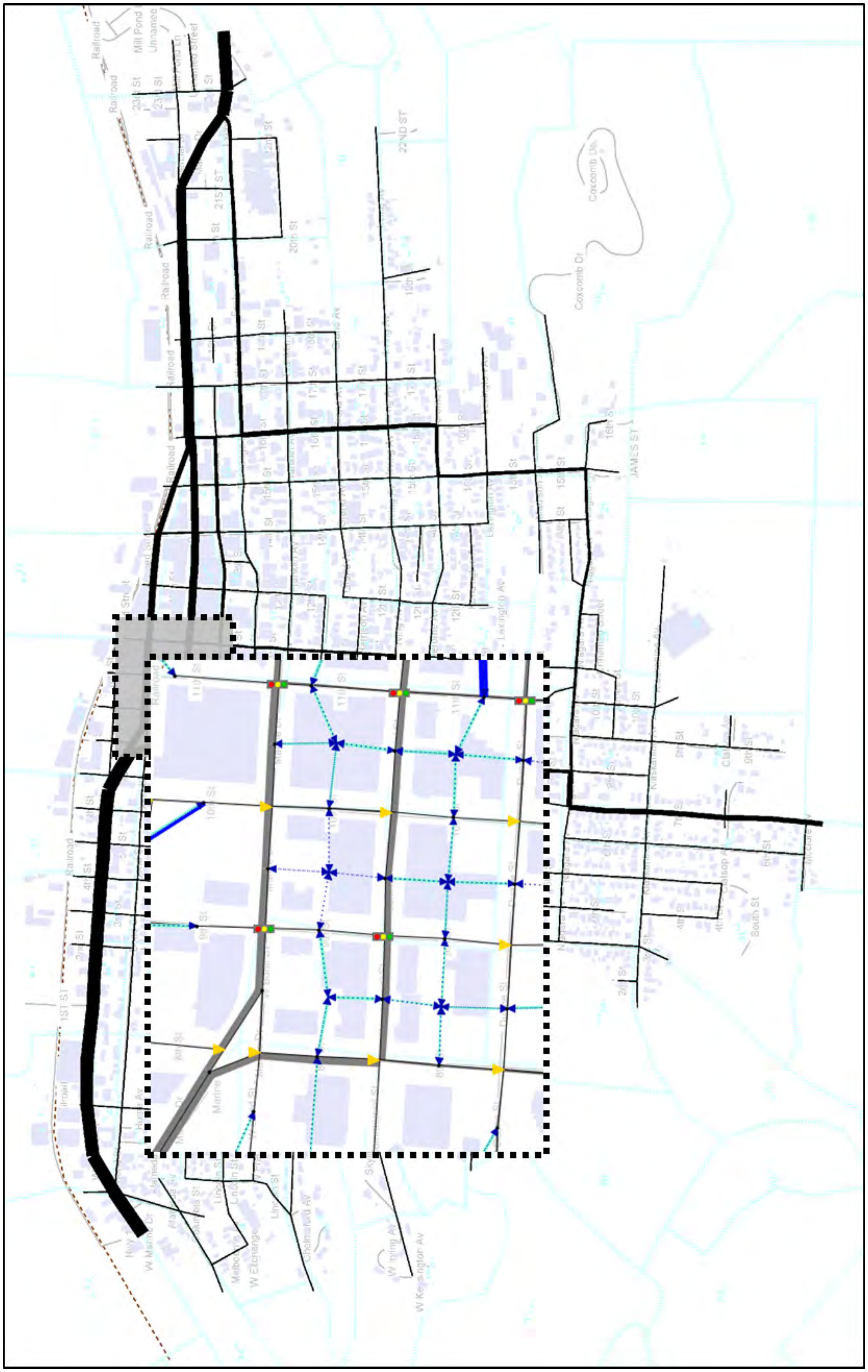
Exhibit 8-15 Astoria Warrenton Regional Model and Sample Window Area



The resulting windowed area model is shown in Exhibit 8-16. The windowed model included the following refinements to capture circulation impacts:

- Assignment – Incorporated HCM intersection delay for intersection turns.
- Link Network – Added all public roadways in windowed area
- Zone Network – Added centroid connectors to reflect major driveways and parking locations.

Exhibit 8-16 Astoria Downtown Windowed Model



8.5 Dynamic Traffic Assignment (DTA)

This section provides an overview of Dynamic Traffic Assignment (DTA) concepts and general considerations for application in Oregon. DTA tools and software packages are still relatively new, emerging tools that have the potential to improve forecasting and analysis and are becoming more widely used around the country. Due to the potential benefits of these tools, they may become more prevalent as they become better known and understood. While this section provides a general overview of DTA, additional information about DTA is provided by the Transportation Research Board[7].

8.5.1 DTA General Concepts

Traditionally, travel demand models have generally included a "static" assignment – one that provides a fixed path (or paths) for each origin-destination (O-D) pair during a given time interval. The time interval is generally the duration of the model period (such as a full day or a one-hour evening peak hour), or may be divided into shorter durations (such as one hour intervals within a day, or 15-minute periods within an hour). Generally, these static models have four common characteristics:

1) The O-D trips for each time interval are a function of input static time-of-day factors that are unrelated to the modeled assignment results. This means that the trips planned for each time interval are largely insensitive to how congestion varies through time.
2) All O-D trips are assumed to be completed during the time interval. That is, the travel time for each O-D pair is less than or equal to the time interval (generally one hour). In the event that a trip exceeds the time interval, demand is still forced through the system, leading to unreasonable demand-to-capacity (d/c) in highly congested networks.
3) The assigned path (or paths) for each O-D pair are not affected by the volumes in earlier time periods. Therefore, preexisting congestion on the roadway (from previous time periods) is ignored.
4) The representation of intersection Level of Service (LOS) is very simple (or non-existent) as the network usually lacks intersection geometry and signal timing. This significantly limits the static assignment model in its usefulness for traffic operations analysis.

Such static assignment models tend to work fairly well as long as recurring congestion is not present in a network with multiple alternatives routes (i.e. a redundant network). In addition, static assignment models are not able to fully capture additional details related to the complex effects of congestion that causes traffic to:

- Divert to another route (spatial spreading),
- Leave earlier or later (during another time period – peak spreading), or
- Travel through the congestion (with a significantly higher realized travel time).

[7] TRB Transportation Research E-Circular E-C153: Dynamic Traffic Assignment: A Primer, Transportation Research Board, Washington DC, June, 2011.

Beyond traditional aggregate trip-based demand models, activity-based demand models (ABM), are a better framework for integration with DTA. ABMs microsimulate a day's worth of travel for each individual in a region, operate at a finer temporal resolution (such as 30 minutes), and include improved models for scheduling trips (both departure and durations). Unlike aggregate trip-based models which output matrices by time period, ABMs output a list of individual trips with departure times. Integrating an ABM with a DTA therefore looks significantly different than integrating with a trip-based model. Currently, ODOT is in the process of developing ABMs.

In addition, static assignment models are not designed for dynamic changes in network capacity related to temporary closures, changes in traffic control, or non-recurrent diversion related to an incident. Other tools are needed for application in these scenarios. To better understand the potential impacts of the above cases, DTA can provide a more comprehensive assessment.

Unlike static models, DTA models allow travel routes to change through time. Traffic that is existing on the system (and its impacts on travel time) is considered, much like a seeding interval for a traffic simulation model. While there are many differences between different DTA platforms, the following general characteristics are common:

- Time-dependent paths; the path through the network is influenced by travel times that vary depending on when travelers arrive at a given network link, as opposed to assuming that travel times are constant throughout the period being simulated.
- Travel routes typically change at shorter intervals than static models (typically minutes instead of hours).
- Network congestion estimates, which are often based on traffic operations, are typically more detailed than a static model to account for travel time difference between routes.
- Vehicles queue on the network and are not forced through over-capacity conditions due to a timer interval constraint. Thus some demand may remain unserved during the analysis period.
- Individual vehicle "simulation" (whether visualized or not) is generally present to account for vehicle interaction and operational impacts.
- Like static assignment, multiclass assignment (including trucks) can be captured in DTA tools, which allow for different path sets and attributes among vehicle classes.
- Transit elements are included in varying degrees based on each DTA tool, but may include the ability to include service information including routes, stop locations, and schedules. Next generation DTA tools are starting to model individual transit persons as well.

DTA models assign demand in a much shorter time interval than static equilibrium traffic assignment models; often demand must be segmented into 15, 10 or even 5 minute time slices. This shorter interval duration is a feature that allows for better reflection of traffic flows, but it is also a requirement that necessitates better trip estimates than typically provided by demand models. Developing the initial set of OD trip tables by time period that are fed into the DTA is an important task that needs to be done with care, and is discussed later in this section.

In mesoscopic modeling, DTA models represent the application that comes closest to achieving the same details that microsimulation models achieve. While microsimulation models maintain more advanced driver behavior algorithms and settings, DTA models usually provide individual vehicle simulation, car-following logic, intersection delay and queuing components, and the ability to introduce vehicle and/or driver profiles, all while maintaining trips to load onto a routable network.

A key differentiator among DTA models is the overall fidelity and detail for which traffic flows are captured along a road. The two types of models are referred to as “link-based” and “lane-based:”

- **Link-Based Models –** Traffic flow along a roadway is analyzed macroscopically, where the total number of lanes is considered as an overall link capacity. Differences among individual lanes (including the amount of traffic demand for an individual lane), interactions among individual vehicles, and friction related to movement and lane changing are not directly modeled.
- **Lane-Based Models –** Traffic flow along a roadway is analyzed for each lane, using car-following algorithms that account for interactions among vehicles and flow differences in each lane. Storage of vehicles related to turn bay lengths and other differences between lanes along a given link may influence operations at the adjacent node/junctions/intersections.

As discussed in later sections, the fundamental differences between these model types should be considered when selecting a DTA tool for project application.

8.5.2 DTA Scoping

The following sections provide an overview of scoping DTA modeling efforts and an introduction to various tools and software packages.

“…DTA models are not the universal cure that can cost-effectively address all types of problems at hand. DTA models take more time and resources to construct and calibrate (as compared with static traffic assignment models) and represent traffic dynamics in a coarser granularity (as compared with microscopic traffic simulation models.) Practitioners are advised to match the choice of modeling approaches to the problem at hand. For long-term planning, for example, the available level of input data required for DTA may not be available, so that one may have to make many assumptions in order to construct such models. If the additional detail and precision in their output data compared to conventional network forecasting is not beneficial for the modeling question in mind, then it may not be worth the additional modeling effort. For a smaller bounded area in which a detailed representation of multiple modes (e.g., auto, transit, pedestrians) and facilities (roadways, parking, crosswalks, etc.) are required, microscopic models may be more appropriate and useful” [1]

[1] TRB Transportation Research E-Circular E-C153: Dynamic Traffic Assignment: A Primer, Transportation Research Board, Washington DC, June 2011.

Tool Selection

Unlike static demand models that typically can achieve similar analysis under different graphical user interface (GUI), DTA models generally have many more fundamental differences related to model structure, network detail, and assignment algorithms. Section 8.5.3 provides an overview of DTA tools and includes additional information comparing key differences among some of the DTA tools and software packages. It is important to consider these fundamental differences when selecting the correct tool (among DTA software or other tools) to address the transportation question.

Data

The general types of data needed to code and calibrate a DTA model are similar to a subarea model (Sections 8.2 through 8.4) and require the following general types of data:

- Network Data – Attributes of the transportation network such as traffic control, lane geometries, and signal timing information.
- Demand Data – Data such as initial O-D trip matrices by time period, land use inventories and/or traffic counts to estimate the level of overall traffic demand.
- Calibration/Validation Data – Data that can be used to compare traffic flows and operations from the model, such as tube or intersection turn counts, and travel time runs along a corridor.

However, the key differences are that DTA traffic datasets (demands, flows, and operations) typically need to address finer-grained resolutions of traffic volume and travel time data over sequential time periods. For instance, data (such as traffic volumes, speeds, etc.) may be needed for individual 15-minute intervals (3:00 to 3:15 p.m., 3:15 to

3:30 p.m., … 6:45 to 7:00 p.m.) rather than a single, aggregated 5:00 to 6:00 p.m. one-hour period. Key differences in the type and resolution of data needs include:

- O-D Trips (travel demand)
    - Finer time resolution trip matrices
    - Trips matrices that will not overload the DTA network (i.e. the overall magnitude of traffic demand is appropriate for the network)
- Traffic Volumes (intersection turn or roadway tube)
    - Flow profiles that cover the modeled time period (which may exceed a simple peak hour)
    - Flow profiles in time intervals of an appropriate resolution to be compared to the DTA model (such as 15-minute periods)
    - Flow profiles at key locations upstream, downstream and/or within bottleneck areas to understand differences in traffic demand and actual supply output
- Traffic Operations
    - Average speed profiles along a corridor (may come from Bluetooth and/or third party vendor speed data such as INRIX or TomTom)
    - Average delay for intersection movements (may be compared to an HCM intersection model)
    - Actual signal timing (such as observed average phase durations and cycle lengths) to develop the network model (traffic control details).
    - Queue lengths to understand the vehicle spacing and jam density

Given the duration and resolution of datasets identified above, it can be difficult (if not cost-prohibitive) to acquire complete data coverage of the model area. For that reason, it is important to strategically make use of existing datasets and focus new data collection efforts on key locations that capture real world behavior and attributes that need to be reflected in the model. The following types of answers should be addressed in model validation and calibration and may guide the collection of data:

- Is the overall magnitude of traffic demand appropriate? If not, then a common correction (for current and short term forecast years) is to adjust the O-D trip matrix to better match traffic counts (see Section 8.5.3) (Check volumes at network entry points and/or screenlines)[8]
- Is the correct routing being reflected in the model? (Check volumes on screenlines and turn movements at key junctions)
- Is the correct volume profile (and peaking) being realized on key network links? (Check volume profiles at key locations as well as upstream/downstream of bottlenecks)
- Is the correct travel time profile being realized on key network links? (Check traffic volumes and determine if delay estimates are being applied appropriately)

[8] There are some DTA model development projects that rely upon activity-based demand, provided in the form of trip lists (simulation output) rather than an ODO matrix.

Resource Needs
Section 8.3.3 reviewed the level of resources needed to apply windowing and subarea models. In many cases a DTA model may be applied as a windowed model. For reference, application of a DTA windowed model will generally fall under the “high” range of effort listed in Exhibit 8-7.

Due to the additional refinements and details available in a DTA model, the time needed to set up a DTA model (like a microsimulation model) is probably best summarized as weeks or months, rather than hours or days.

Since DTA models typically require additional effort to build and calibrate beyond a typical static macroscopic model, it can be helpful to approach the process from the perspective of scoping a microsimulation model. While DTA models may be used for a variety of purposes ranging from developing better demand volumes for microsimulation to using operational measures to screen alternatives, the details and data needed to build and calibrate the models can emulate a microsimulation model. ODOT has developed a process[9] for building Vissim microsimulation models that provides a general structure for how to approach and scope a microsimulation effort. Depending on the DTA application and similarity to a microsimulation model, some of the framework of this approach may be able to provide a generalized resource for a DTA modeling endeavor.

8.5.3 Overview of DTA Tools and Software Packages

There is a wide array of tools available to perform DTA at varying scales and resolution levels. These tools range from enhancements of macroscopic models to application within microsimulation – essentially covering the full spectrum of mesoscopic modeling in terms of network detail and level of effort. The various types of tools available and the range of these tools allows for broad application ranging from enhancing volume development for separate traffic analysis to performing the actual traffic analysis in place of microsimulation. The following section provides an overview of some of tools used in the Portland metropolitan area. This list is not meant to be a review of all the DTA tools on the market; instead it is meant to illustrate the spectrum of DTA tools. Exhibit 8-18 summarizes the DTA tools that have been applied in the Portland metropolitan area.

[9] *Protocol for Vissim Simulation*, Oregon Department of Transportation, June 2011.

The following tools are grouped along the spectrum of mesoscopic modeling, ranging from those closer to macroscopic models to those that are more similar to microscopic models. Each tool has numerous features and fundamental differences that provide opportunities and limitations that ultimately impact the analysis results. This list is grouped for demonstrative purposes only and analysts are encouraged to understand the full abilities and limitations of these tools before project application. Groups closer to macroscopic models would require less detail and resources to use, while those closer to microsimulation would be more data and labor intensive to apply.

DTA tools within **macroscopic framework** (individual vehicles are not modeled):

- Visum Dynamic User Equilibrium (DUE) – A model within PTV's macroscopic Visum model that constrains vehicle flow due to out-link intersection capacity and allows spreading of vehicle demand to adjacent routes. If other routes are at capacity (for a given time period), demand can be spread to originate during adjacent time periods. DUE is a link-based model and does not require coding intersection geometry or signal timing.

DTA tools with **link-based traffic flow** (individual vehicles modeled):

- DynusT – An open-source stand-alone DTA package that models individual vehicles but does not model individual travel lanes. Because of its simplifications in network representation, it can be applied to larger network sizes. See Exhibit 8-18 for additional details.

DTA tools with **lane-based traffic flow** (individual vehicles modeled):

- Dynameq – This stand-alone DTA package developed by INRO models individual vehicles and lanes, includes car-following logic that (due to the lane-based nature) models interactions in a more advanced manner than the link-based models but is more simplified than a microsimulation model. See Exhibit 8-18 for additional details.

DTA tools within **microscopic framework** (individual vehicles modeled)

- Vissim DTA – A model within PTV's Vissim microsimulation package that allows for dynamic routing of individual vehicles. This application makes use of the fine-grained car following logic within the microscopic model.

Exhibit 8-17 Continuum of DTA Tools and Typical Level of Detail

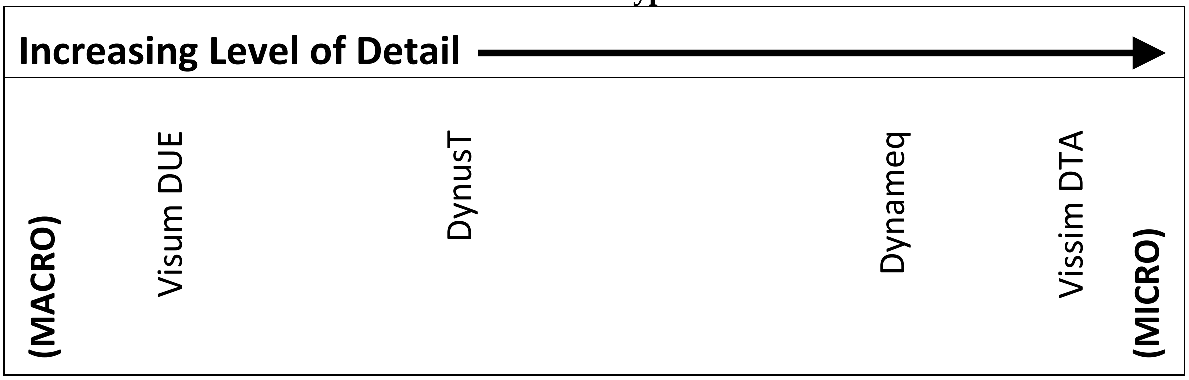


Exhibit 8-18 Comparison of DTA Tools Applied in Portland Metropolitan Area

Element	DynusT	Dynameq
Segment Geometry and Control	**Link**-based (overall capacity of the cross-section is considered)	**Lane**-based (individual capacity for each lane and interaction with adjacent lanes and vehicles are considered)
Intersection Geometry and Control	Intersection turn bays are based on single global distance parameter.	Allows specific lane storage via creation of new links at any change in cross section.
Traffic Signals	Includes general traffic signal settings as well as actuated timing on a cycle by cycle basis.	Includes general traffic signal settings as well as actuated timing on a cycle by cycle basis. (beta feature)
Typical Model Size/ Application	Regional application for major roads is common.	Typically more restrictive model size that may be smaller scale subarea due to increased detail (all streets).
Includes Vehicle Simulation	Vehicles simulated at specific time steps (every 3 seconds or other duration)	Vehicle interaction is simulated for events and calculation is performed when there is a change
Visualization	Synthetic visualization of individual vehicles based on link-level information.	Macro flow (density, speed, etc.) at link level - individual vehicles not shown (but are simulated).
Randomness	Stochastic – allows multiple runs to reflect system variability (similar to microsimulation)	Deterministic model (profiles are applied for characteristics)
Ability to Import and Export Data	Allows network import/export with travel demand models and microsimulation. Direct integration may be difficult since network may be modified based on geometries.	Allows network import/export with travel demand models and microsimulation. Direct integration may be difficult since network may be modified based on geometries.
Output	Data/Measures can be visualized in GUI or raw data exported for processing.	Data/Measures can be visualized in GUI or raw data exported for processing.
Scripting / Application Programming Interface (API)	Custom dynamic-link library (DLL) for ramp metering and signal control; full API planned	Python API
License	Open source without support; support for a fee	Purchase (includes software updates and support)

8.5.4 DTA Calibration

> *Analysis Procedure Manual Version 2 Chapter 6 includes additional information on model checks. Calibration information in section 8.5.4 may also be relevant, particularly when applying a DTA model as a windowed subarea model.*

Model Checking

An important aspect of DTA calibration is finding and fixing network coding problems and performing a model checking process. In the case of a DTA model, miscoding signal timing, approach geometry, intersection connectivity, turn prohibitions, etc. can have major impacts on the simulation. Very often the calibration process requires detailed network coding checks, and network defaults must be revised where they are found to be insufficient to reflect true operations in key locations. In addition, any O-D demand adjustment should be re-done if significant network errors are corrected.

Calibration

In general, the following broad observations are common to DTA model calibration:

- DTA models include more settings and "knobs" to turn (which may or may not be appropriate to adjust) than a traditional static assignment model. This makes the calibration process more complex, costly and time consuming.
- More data are needed than a typical static model (see Section 8.6.2)

Due to the varied level of detail and application of DTA tools, it is important to consider the resolution of the model and what is an appropriate level of calibration. In addition to information provided for calibration subarea models, some strategies for calibrating DTA models include:

- If specific targets are used for traffic volumes and/or travel times, consider thresholds that vary by project priority. Corridors that are the primary focus of the analysis should have the tightest calibration targets while other roads in the model that compose the supporting network may not be as critical. These targets should be developed through coordination with agency staff.
- Signal timing can be a critical element that affects calibration on non-freeway facilities. Spending the time to code signal timing at an appropriate level of detail will facilitate calibration.
- Flow models (relationship of vehicle speed, density and flow) are an important component of freeway segments. Be sure that the settings are reasonable and appropriate for the model area.
- Understand the basis, limitations, and location-dependency of data that were used to develop the model, specifically in regards to if the data represent a total traffic demand or an actual (potentially capacity-restricted) traffic volume that is realized in the network.

- Consider if bottlenecks and capacity constraints are being correctly reflected in resulting flow profiles. Gateway adjustments may be necessary to approximate bottleneck effects.
- Depending on model resolution and purpose, calibration may be sufficient at aggregate levels.
- Review high-level items (such as total volume magnitude of a screenline) and general routing patterns before focusing on network details.
- Check for unserved demand or “lost trips” on the travel network. These are vehicles that cannot actually get on the network or cannot make it from their origin to their destination according to their chosen route because of congestion. Many DTA packages report these trips and the analyst must address them by changing the demand profile or fixing the network.
- O-D demand adjustment may be needed to fit a demand matrix to the network conditions (as observed by traffic counts). However, such action should be approached with caution and should be re-done once the network is error free. . Related guidance for demand adjustments is provided in Section 8.5.3.

> *Some parameters (such as vehicle flow densities and signal timing) have the potential to greatly influence the model network capacity and the resulting traffic assignment and overall results. These parameters should be verified before focusing on and adjusting gateways or micro-level details for calibration.*

8.5.5 Measures of Effectiveness (MOE)

The fundamental nature of DTA models and core differences from traditional static models allows for an expanded set of model data and reportable measures of effectiveness. The increase of available measures reflects the key differences introduced by DTA models in level of detail (such as intersection operations) and changes over time that are typically not available in static models.

The amount of MOE that are available directly through the GUI of the DTA platform may vary by software version and type of DTA model (link-based versus lane-based, see Section 8.5.3). However, additional measures are typically available indirectly by processing raw data that are produced through the model runs. These possibilities leave the analyst with a wide selection of potential MOEs. It is important to select MOEs that are relevant for the decisions being made and are appropriate for the level of detail coded into the model.

Some potential types of MOEs include:
*(Note: additional detail may also be available for individual lanes in lane-based DTA models when denoted with *)*

- MOEs typically produced by a static model
    - Roadway segment (link) traffic volume during a time period *
    - Roadway segment (link) average travel time during a time period *

        - Roadway corridor average travel time during a time period *
        - System MOEs (vehicle hours of delay, etc.)
- MOEs typically produced by a DTA model
    - Available due to additional operation detail
        - Average delay by intersection movement
        - Vehicle queuing or link occupancy (available on a lane basis for lane-based DTA models) *
    - Available due to nature of model simulation (individual vehicles)
        - Reliability metrics related to range of travel time for all vehicles along a corridor during a time period (Exhibit 8-20) *
        - Individual travel time trajectories along a corridor *
    - Available due to differences between time periods
        - Map-based simulation of traffic volumes, occupancy, or other metric shown on a link by link basis (Exhibit 8-19) *
        - Segment or corridor travel time profile (for each hour of the day or model time interval) (Exhibit 8-20)
        - Space-time ("brain scan") heat maps showing changes in travel time, occupancy, speed, etc. along a corridor (Exhibit 8-21) *

Some MOEs or outputs that are commonly produced using static models (such as model difference plots) may not be as easy to reproduce in DTA packages. The summary of this output may require additional processing rather than being directly available/automated through the model interface.

The following exhibits demonstrate some of the MOEs that are possible with DTA models.

Exhibit 8-19 DTA MOE Sample 1 – Map-based Shockwave Animation (Regional or Subarea) Shown for Four Sequential Time Periods

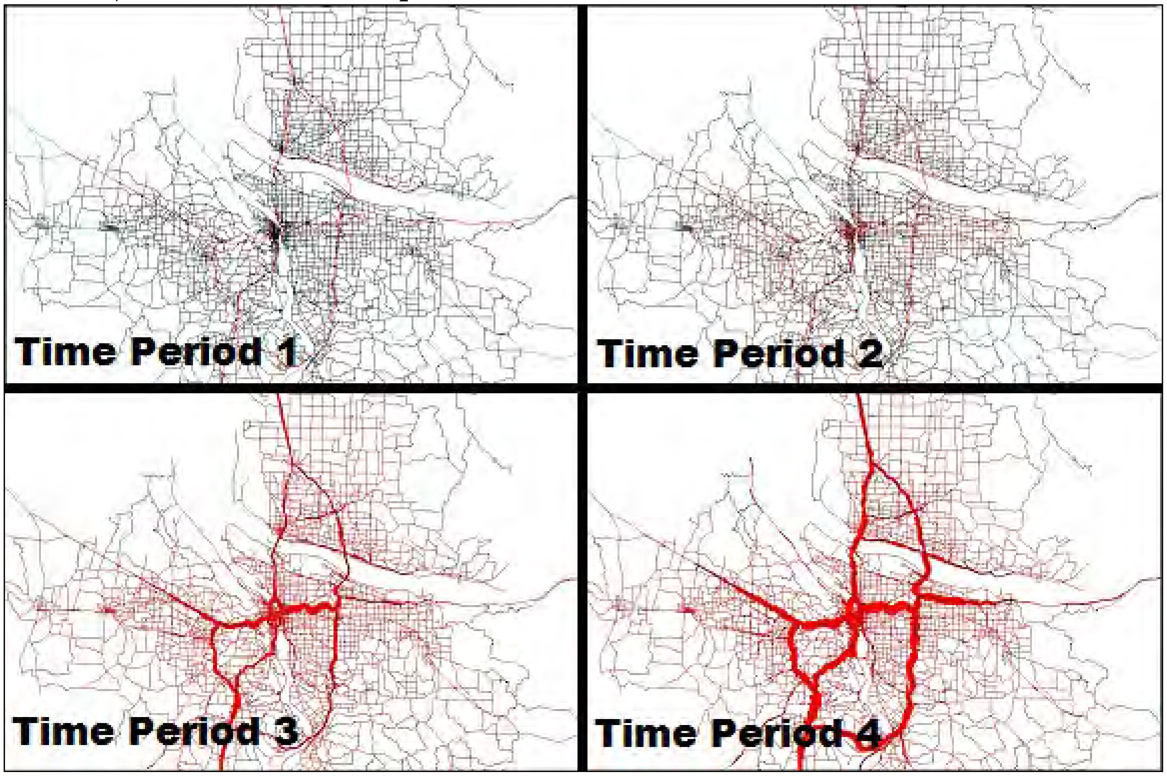


Exhibit 8-20 DTA MOE Sample 2 – Travel Time Profile along a Corridor

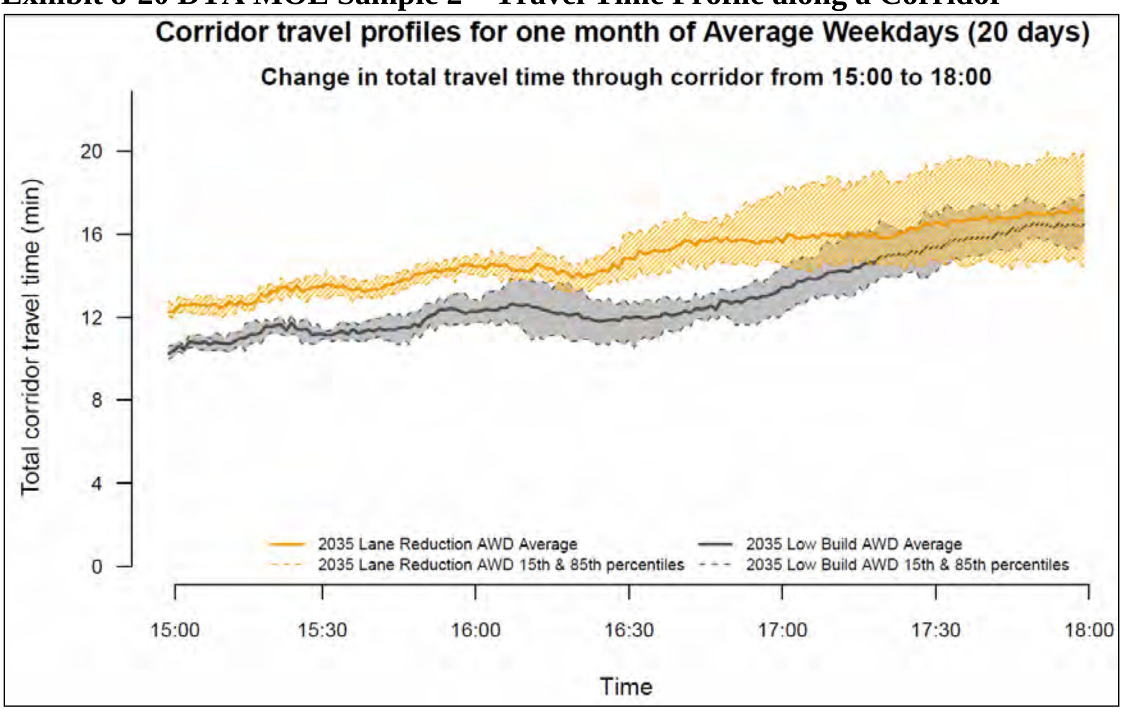
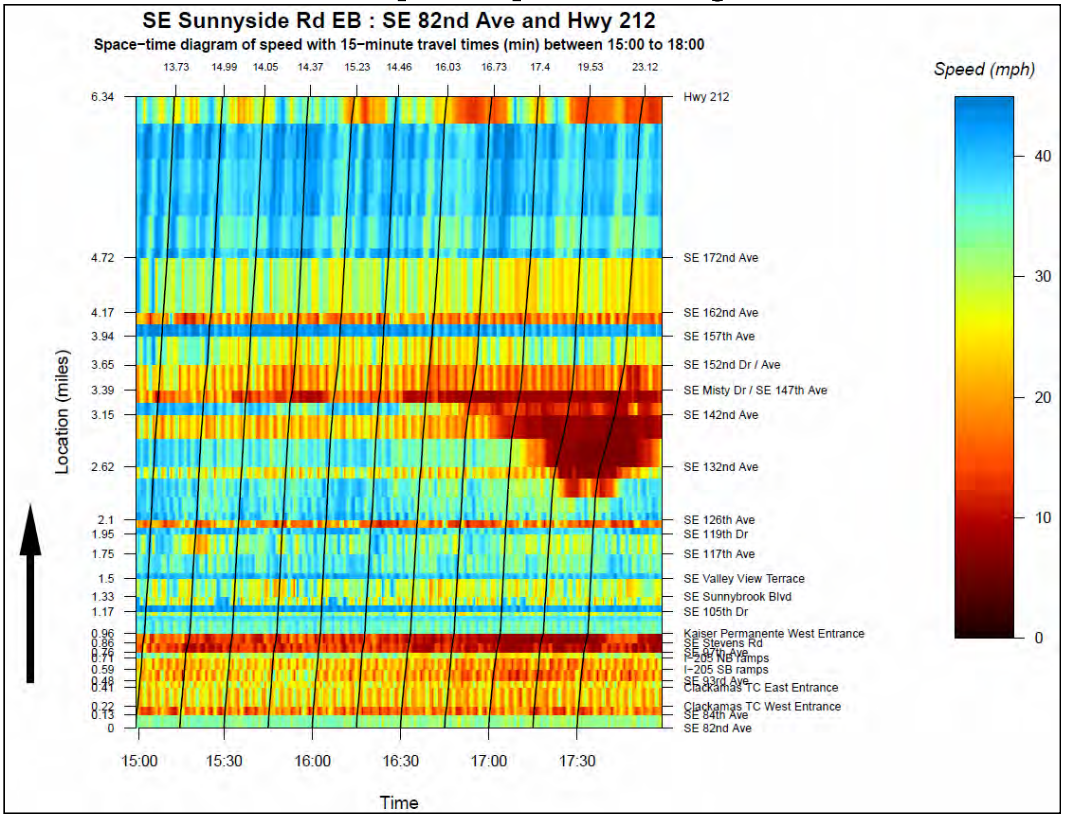


Exhibit 8-21 DTA MOE Sample 3 – Space-time Congestion Plot



8.5.6 Key Considerations (Application and Issues)

The following considerations are intended to help guide decisions regarding application of DTA models. In general, the level of detail and refinement in any particular element should generally reflect the detail of other model elements. Since application of a DTA model may be applied as a windowed subarea model to a regional static model, many of the same considerations for windowing would apply. However, due to the fundamental differences for DTA models (including increased resolution for traffic operations), additional considerations would apply. These considerations are not absolute but are intended to function as a general guidance when developing an approach and methodology.

General considerations related to windowing a DTA subarea from a regional static model (see Section 8.5.5 for additional detail)

- Study Area Size/Boundary
- Network Element Refinements (Links, Zones, Nodes)
- Demand Adjustments
- Future Year Scenarios

The following considerations reflect those that are introduced by DTA models that may not be primary issues when dealing with static models.

- Balancing model refinements with overall purpose - The ultimate purpose of the DTA model will application will vary from project to project and will ultimately guide the level of refinements needed in the model. Some examples of the range of typical applications include:
  - Volume development (may include higher level model)
    - Developing base year alternative volumes for operations analysis
    - Forecasting future year volumes
  - Operations analysis (may include finer resolution and detail)
    - Screening alternatives within the DTA model (relative comparisons)
    - System measures
    - Corridor/Intersection measures

- Understanding Traffic Flow Theory and Operations Concepts - DTA models introduce traffic flow details that are not generally present in static models. These elements may vary from those that are simplified in static models (such as traffic control types), to those that are non-existent in static models (flow model and jam density values). In order to correctly understand and use the models, the analyst needs to have an understanding of traffic flow theory that goes beyond what is generally needed for regional-scale travel demand modeling using static models. Some of these key concepts include:
  - Intersection control and movement priority
  - Traffic signal timing (and realistic settings)
  - Speed/Flow/Density relationships (including jam density[10])

Fundamental knowledge in these areas is needed to correctly code operational elements in the model and to consider the reasonableness of model results.

- While DTA software generally include the ability to import and export data, these abilities, and the differences in the underlying network data models, vary by platform and can make it difficult to transfer data between packages in an efficient manner. For this reason, the analyst should consider these differences when considering application of multi-resolution models and combinations of DTA platforms for project application.
- Applying DTA models for future conditions introduces several other considerations for the future year model:
  - Assumptions for traffic control, specifically future traffic signal timing parameters for existing or proposed traffic signals. In some cases (such as a fixed timed grid network or along a coordinated corridor) it may not be reasonable to assume that signal timing parameters would change in the future. However, depending on the intersection location, ownership, and potential for growth, it may be reasonable to anticipate that traffic signal timing would be updated in the future to reflect changing needs. For example, an intersection on the urban fringe that currently runs free and

[10] Jam densities vary by location and vehicle mix but generally range from a spacing of 25 to 30 feet per vehicle or 175 to 210 vehicles per lane per mile)

has low average splits and cycle lengths may have longer average splits and cycle lengths as demand increases in the future. However, for intersections that are part of a coordinated system, particularly when located along a highway route that maintains a green band, future signal timing modifications may not be as likely. These assumptions should be coordinated with TPAU and/or ODOT Region traffic group.

- Adjusting demand through O-D matrix demand adjustment is problematic for future year models because it assumes the relationships that exist between the seed demand matrices from a regional travel demand model and the traffic counts hold true in the future (see Sections 8.5.3 and 8.6.4)
- Method used for developing traffic analysis volumes. Many factors (such as the type of DTA model used, the amount of data available, the effectiveness of the calibration process, the amount of pre -processing of traffic demand, etc.) will influence the balance of traffic analysis that is performed within the DTA model versus the use of external tools (including HCM or microsimulation).

8.6 Peak Spreading

This section describes the concept of peak spreading and summarizes different analysis methods, needs, and considerations based on location and context (type of study and implications such as short term development impacts for a TIA or long-term regional planning needs). Peak spreading (as described in the following sections) occurs when there is travel demand for a sustained period that exceeds the available capacity of the transportation network. For this reason, peak spreading is unique to congested conditions, and is unlikely to exist for an extended period in a relatively uncongested network.

> *While peak spreading methods can be used to estimate how traffic routing, assignment, and/or travel time may be influenced by congestion during different periods of the day, the existence of peak spreading may indicate that other facets of the travel estimation (such as trip generation and trip distribution) may need to be reconsidered. As Oregon moves towards activity based models (ABM) and Dynamic Traffic Assignment (DTA) models the integrated abilities of these tools will limit the need for separate peak spreading processes described in this section.*

8.6.1 Peak Spreading General Concepts

The following sections provide an introduction and overview of general peak spreading concepts. For the purposes of the following material, it is important to understand the differences between two terms that are commonly used interchangeably but have very different meanings:

- **Travel demand –** the amount of unconstrained traffic that wants to travel on a road or make a turn movement during a specified period of time. An example of traffic demand would be the amount of traffic that a static travel demand model reports wants to use a road in the future. In reality, there may be constraints (such as capacity of a downstream intersection) that prevent this amount of travel demand from being achieved in the real world. However, this demand" can still provide insight for planning purposes. Travel demand is sometimes reported as a ratio to available capacity as demand/capacity (D/C), which can exceed a value of 1.0.
- **Traffic volume –** the amount of traffic that is actually served on a road during a specified period of time. An example of a traffic volume would be the number of cars that are observed travelling in a lane or making a turning movement during a vehicle count data collection. These volumes are the actual amount of vehicles that are able to use the system, given constraints that exist. The traffic volumes may be equal to the travel demand, but in congested conditions the traffic volumes will be less than the travel demand due to constraints (insufficient green

time at the signal, downstream bottlenecks, etc.). Traffic volumes are commonly reported as a ratio to available capacity as volume/capacity (V/C). By definition, these ratios cannot exceed a value of 1.0.

How Travel Demand Models Deal With Time-of-Day

Before defining peak spreading in detail, it is important to understand how travel demand models generate traffic demand forecasts since traffic demand is often revised during peak spreading procedures.

Aggregate trip-based models create time period specific travel demand trip matrices based on static user defined time period factors. These factors usually vary by trip purpose and direction of travel (production to attraction and attraction to production). By applying these time-of-day factors, the estimated daily travel demand is split into time period specific demand. As a result, travel demand models are largely insensitive to peak spreading since the user specifies how demand is spread across the hours of the day.

Activity-based models (ABM), such as those being developed for the Portland metropolitan area regional government (Metro) and ODOT, take a much different approach to modeling time-of-day. ABMs have explicit trip departure time and duration models that operate at the hourly or half-hour time period that are sensitive to a much more comprehensive set of information, including differences in network congestion by time-of-day. As a result, ABMs can produce more reasonable travel demand by time period and are much more able to model peak spreading.

What is Peak Spreading?

Traffic demands and traffic volumes constantly fluctuate based on many factors. While traffic is always changing, traffic analyses generally consider peak traffic conditions to assess the needs of the roadway system. Traffic analyses commonly account for traffic peaks or profiles in three general ways.

1) Annual traffic profile - Daily traffic volumes for a given location may be assessed to determine how they change over the year. These considerations are used to adjust for seasonal factoring, which account for changes in daily traffic demand throughout the year. (see APM Section 5.4 for additional information)
2) Daily traffic profile and “peak hour” occurrence - Traffic volumes by time of day (generally divided into an hour) are considered to determine at what time the traffic is generally highest. These periods are commonly referred to as peak periods and (based on land use and transportation factors[11]) generally occur during a “peak hour” in both the morning and evening periods that correspond to commute patterns. In order to account for this peaking, traffic counts may be conducted during both periods for a traffic impact analysis, such as the 7:00 a.m.

[11] Such as proximity to land uses that may have atypical trip generation peaks (schools, major employment centers with off-peak shift changes) or variable elements of the transportation system (time of day control or meters that influence other network options) that cause routing patterns to change.

to 9:00 a.m. and 4:00 p.m. to 6:00 p.m. periods to identify the peak hour.

3) Traffic peaking during the highest hour and peak 15-minutes[12] – Traffic analysis following HCM procedures further accounts for peak traffic demands during a peak 15-minute period. This is measured as the peak hour factor (PHF), which corresponds to the ratio of: (peak hour volume / (4 * peak 15-minute volume)). A PHF of 1.0 indicates a uniform traffic volume during each 15-minute segment of the peak hour, while lower values indicate a sharp peak (such as a shift change near a major employer or end of school day near a school).

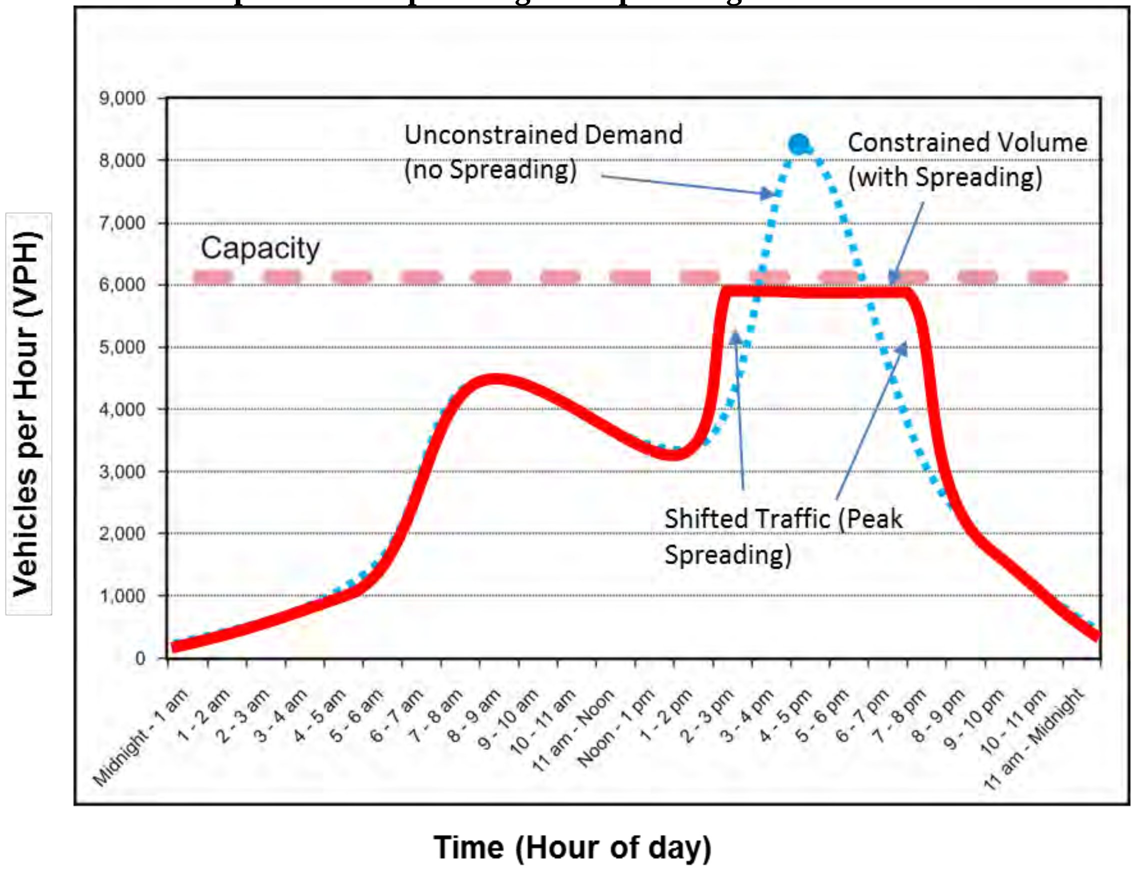
On a smaller scale, the example of a PHF at or near 1.0 is indicative of the general concept of peak spreading within the peak hour. **Peak spreading** is when traffic demand exceeds capacity and the resulting traffic volumes are served over a longer peak duration (temporal spreading) or may shift to other routes (spatial spreading).

Exhibit 8-22 demonstrates temporal peak spreading along a corridor. First, the x-axis represents the hour of the day and the y-axis represents the traffic on the freeway segment. The example includes four primary elements:

- The capacity of the segment is approximately 6,100 vehicles per hour (VPH), represented by a dashed red line.
- The unconstrained traffic demand (shown in dashed blue) is the amount of traffic that wants to travel on the corridor. This demand may be the result of applying a growth factor (resulting from historical trend or travel model forecast to an existing profile. The unconstrained demand is highest around 3 p.m. with a value of 8,200 vehicles. This unconstrained demand is well above the capacity of 6,100 vehicles per hour.
- The constrained traffic volume (shown in solid red) is the amount of traffic that can actually use the corridor given the capacity constraints. The constrained traffic volume does not exceed the capacity of 6,100 vehicles per hour. During early hours of the day and late hours of the night (when traffic demand is low), the traffic volume is the same as the traffic demand. However, when traffic demand exceeds the capacity, the traffic volume is limited at the capacity.
- The peak spreading (shown as the area above the unconstrained demand and below the constrained volume) is traffic volume that is using the system earlier or later than the unconstrained peak in order to be accommodated by the capacity. In this case, some of the traffic volume would occur later (approximately between 6 p.m. to 9 p.m.) due to congestion and longer travel times. In addition, some of this traffic may have desired to use the system between 2:00 p.m. and 5:00 p.m. but may have chosen to plan their route differently to leave later. In addition, the peak spreading shown to occur between 5:00 a.m. and 11:00 a.m. would be caused by traffic that would be leaving earlier to avoid congestion later in the day.

[12] Refer to APM Chapter 3 for additional information regarding traffic counts and congested conditions.

Exhibit 8-22 Temporal Peak Spreading Example along a Corridor

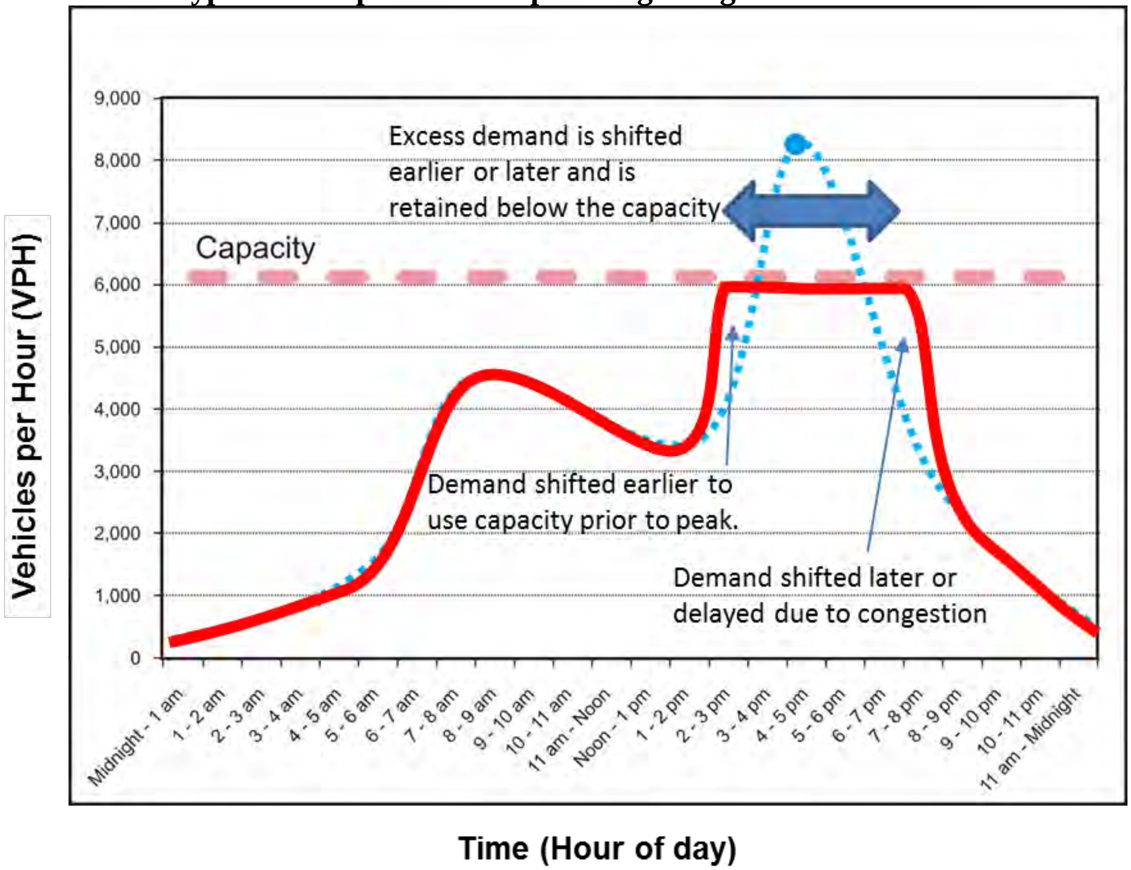


What Causes Peak Spreading?

As demonstrated in Exhibit 8-23, peak spreading is generally caused by one or both of the following conditions related to congested traffic conditions:

- Driver decision making - A driver changes their travel behavior to leave before/after the peak conditions to decrease the overall travel time and avoid waiting longer in congestion.
- Bottleneck capacity - A driver does not change their travel behavior and leaves at the desired time to start their trip. However, congestion prevents the driver from completing their trip in a timely manner and they do not reach a downstream location until later due to upstream bottlenecks.

Exhibit 8-23 Types of Temporal Peak Spreading along a Corridor



Why Does Peak Spreading Matter?
Peak spreading as an outcome of congested traffic conditions can influence the amount and distribution of actual traffic on the system. In some cases, the traffic volumes may be less than the travel demand that was projected (during the peak period), but in other cases the traffic volumes may be greater than the projected travel demand due to the effects of peak spreading. In cases where actual traffic volumes are required for an analysis (which is the majority of all traffic analysis), and there is potential for congestion and peak spreading, considerations for peak spreading are important to ensure that findings are reasonable.

In particular, peak spreading considerations are critical for the following use cases:

- Realistic operational analysis (including microsimulation analysis) – Realistic traffic demands are necessary to produce realistic traffic operations results. In order to derive more realistic operations (such as v/c or LOS), appropriate consideration needs to be taken for other network elements and behavior that may influence the traffic demand. Microsimulation models generally function poorly (and may reach a gridlock state) when estimates of O-D trips are not reasonable to account for upstream metering or capacity constraints. In addition, by not correctly accounting for peak spreading, the analysis may lead to unrealistic conditions, misidentification of traffic impacts and queues, and poor design decisions.

- Understanding duration of congestion (and implications of potential improvements) – As new performance measures and policies are explored, it is critical to have a better understanding of what congested conditions will result. By portraying the actual conditions that people will experience, analysts and decision-makers will have a better context for choices related to transportation improvements and policy needs. This consideration will also provide a means for assessing the implications for potential improvements outside the traditional design hour case.

8.6.2 Peak Spreading Scoping

The following sections provide an overview of scoping peak spreading considerations.

Peak spreading analysis and approach depends highly on the preexisting tools (such as the type of travel demand and assignment model) that exist for a location as well as the level of detail needed. Some sketch level analysis may be completed in a matter of hours while more in-depth questions may require additional travel surveys and/or (further) development of a travel demand model.

Identifying Analysis Needs

When analyzing congested conditions, peak spreading may need to be accounted for in the following conditions:

- Developing travel demand volumes for microsimulation
- Determining peak hour factors for future intersection capacity analysis
- Evaluating compliance with OHP mobility targets and possibly identifying alternate mobility targets

Tool Selection

Ultimately, the tools and methods available to assess peak spreading will vary greatly for each unique application based on a variety of factors, including:

- Purpose of the analysis
  - What questions are being asked and what level of detail and precision is needed? Having a sketch level estimate of congestion duration at one location is much different than producing peak period travel volumes along a corridor for microsimulation analysis.
- Location and context
  - Single road location versus network level demand
  - Commuter travel base that may adjust travel patterns and driver behavior versus recreational trips that may not have information (the knowledge of regular occurrence) to change travel behavior.
- Availability of existing data sources (see Data section)

- Availability of existing modeling tools (travel demand models or other tools)

Exhibit 8-24 provides an overview of peak spreading tools and methods that are covered in additional detail in the following section.

Exhibit 8-24 Overview of Peak Spreading Tools and Methods

Element	Cursory Method (No Model)	Travel Demand Model (OSUM[1]/ JEMnR[2])	Metro Hours of Congestion (HOC)	Metro Peak Spreading	DTA
Area	Areas without models	Urban areas with models (outside Portland)	Portland region	Portland region and other OSUM/ JEMnR models	Portland region
Location Scope	Manually applied on link by link basis	Systematic for model area	Systematic for model area	Systematic for model area	Systematic for model area
Data Needed	Existing traffic counts and profile	Depends on application and post-processing	Depends on application and post-processing	Depends on application and post-processing	Depends on application and post-processing. See Section 8.6.2
Output	Scaled volume profile for a location	With additional processing - regional matrix and systematic link level travel demand by hour	Systematic link level travel demand by hour	Regional hourly demand matrix from 2-7 p.m. that can be assigned	Link by link or O-D pair travel time for time intervals.

[1] Oregon Small Urban Models (OSUM) - Population under 50,000
[2] Joint Estimated Model in R (JEMnR) Population over 50,000 (MPOs)

Data

The following types of data may be useful or necessary to conduct peak spreading analysis:

- Existing travel profiles – existing profiles can provide some insight into how peak spreading currently occurs or could develop given additional traffic growth or transportation network changes.
- Bottleneck data – data that indicate the maximum capacity for bottlenecks such as intersections. Saturation flow studies may be needed to determine actual capacities.
- Travel survey – Driver decision making has the potential to influence peak spreading characteristics. The decisions people make related to how much congestion they are willing to endure, the flexibility of their schedule, and the likelihood to leave earlier or later to avoid congestion can all affect the degree of peak spreading.

- Travel time data – The collection of existing travel time data may be used to calculate a travel time index (TTI) or other measure to better understand existing congestions levels (the degree to which travel time is impacted by existing congestion) and consider when estimating driver sensitivity and the future potential for peak spreading.
- Seasonal count data – In absence of specific data about the share of system users that are aware of recurrent system conditions (such as commuters) and those that are not (such as recreational travelers), a comparison of seasonal count data may provide insight into the portion of travelers that are seasonal users. These data can be used to inform assumptions about whether trips would occur at other periods in the presence of congestion. (See Section 8.7.3).

8.6.3 Peak Spreading Application and Procedures

The following sections provide an overview of tools and methods available to analyze peak spreading. Each tool is location-based and therefore options and methods may be limited. In all cases, it is appropriate to coordinate with TPAU or Region 1 Traffic staff to ensure that the method and application is appropriate based on the analysis requirements. In some cases existing tools and data may limit the degree and certainty of peak spreading analysis.

Areas without Travel Demand Models (Cursory Method)

The following section provides an overview of a cursory method to estimate peak spreading in the absence of a travel demand model. While this method can provide useful estimation on peak spreading potential, locations that are projected to be congested to the degree that trip generation, distribution, or route choice are affected may require a travel demand model to better assess this impacts.

At a cursory level, peak spreading can be assessed for locations that do not have a travel demand model. The application may vary based on the specific location, context or purpose of the analysis (such as short-term analysis or long-range planning needs), and availability of data. However, the general process would be the same. While additional processing and adjustments may be needed and applied, the following method is intended to be applied at a cursory level as an informative approximation of peak spreading.

To manually assess peak spreading, three elements are needed to first develop the un-spread profile, including:

- **Reference traffic volume at location to be analyzed –** This may be an existing traffic count or a projected future traffic volume, typically for a peak hour. This value is needed to determine how to appropriately scale the traffic profile.
- **Traffic profile at a generally representative location –** This may be data from an ATR or data collected from a multi-hour (typically 16-hour or daily) traffic count. The location would optimally be at the same location that is studied for the

effects of peak spreading. However, it may be determined that other locations may be representative (or may be all that is available) and may be sufficient for analysis. Other representative locations would typically be along the same corridor either upstream or downstream of the analysis location. Depending on the use of the cursory analysis, other locations may be considered.

- **Relative comparison (or scaling factor) to adjust ("fit") the traffic volume to the representative location on the profile –** A factor to convert the traffic profile to the profile for the analysis condition is needed. This factor is derived from the following method and will generally vary by location:
  - **Reference volume / Profile volume during period = Profile Factor** For a like time period (such as 5-6 p.m.) (Exhibit 8-25):
    - **For same location -** This factor may be equal or near 1.0 depending on when the profile data and reference data were collected. However other values may result from impacts such as seasonal factors or annual growth.
    - **For different location -** This factor may be more or less than 1.0 depending on where the data were collected and the relative difference between the two locations (e.g., the profile data may be located upstream of the study location in an area that generally has higher traffic volumes). Like data collected from the same location, variability also may result from impacts such as seasonal factors or annual growth.

Exhibit 8-25 Profile Factor Calculation

$$\textbf{Profile Factor} = \textbf{Volume}_{\textbf{(Reference)}} \,/\, \textbf{Volume}_{\textbf{(Profile)}}$$

Profile Factor: Scalar factor that will be applied to each hour of the existing profile to achieve the demand profile, which is representative of another location or another analysis period (or year)
Volume $_{\textbf{(Reference)}}$: Volume that represents the analysis volume (peak count or post-processed) for a given time reference time period. This may be the 30HV traffic volume
Volume $_{\textbf{(Profile)}}$: Volume that is indicated in the daily profile during the same reference time period as the Volume $_{\text{(Reference)}}$.

In addition, a general assumption about spreading behavior is needed to determine how the excess demand will be spread:

- **Capacity or maximum traffic flow that can be served –** At the location being analyzed, determine the maximum traffic flow. This may be based on an HCM intersection or other types of capacity analysis.
- **Seasonal data/user system familiarity, or assumptions related to the propensity for drivers to shift earlier, later, or not at all –** Roadway context,

location, and fluctuation in seasonal demands have an influence on peak spreading. The decisions made by a traveler (destination, route choice, and time of departure) have the potential to greatly influence the observed peak spreading trends. The outcomes of these decisions can vary greatly based on the characteristics of the travelers, particularly with respect to how familiar the traveler is with the system condition and their level of flexibility. Commuters or other drivers that know the system conditions may choose to leave earlier or later, or may select a different route, based on their expectation of the system condition. This could include unfamiliar drivers if they have access to traveler information on congested conditions. Thus, they may tend to make peaking spread both earlier and later. However, trips made by users that are not familiar with the system (such as regional/recreational trips) may not have the ability to leave at a different time (being committed to a longer regional or interstate trip) or may not be aware of the local conditions before committing to a route. These users would tend to cause peak spreading to occur with a bias towards later. Therefore, the types of drivers on the system and the mix of seasonal and daily users can have an influence on how peak spreading occurs. Related discussion about data is provided in Section 8.7.2. In the absence of other data, the following spreading trends could be applied based on the traffic user:

- Commuters – Assume that these users that are familiar with the reoccurring system conditions would be evenly likely to change their daily behavior to leave before and after the peak period - half of the travelers would leave before the peak and the remaining half would leave after the peak.
- Recreational Users (Portion derived from seasonal data) – Assume that these users, which are not familiar with the reoccurring system conditions, would not spread to other periods (i.e., these users would not leave before or after the peak period in order to reduce their overall travel time).

To manually apply the cursory method, the following two steps are needed:

1) Determine the projected (un-spread) traffic profile (each hour) of the profile
2) Determine the method used to develop the adjusted (spread) traffic profile by the following method::
    a. No shifting - Trips leave at original times, but take longer to complete due to congestion. (Trips over capacity will be shifted to later period of arrival)

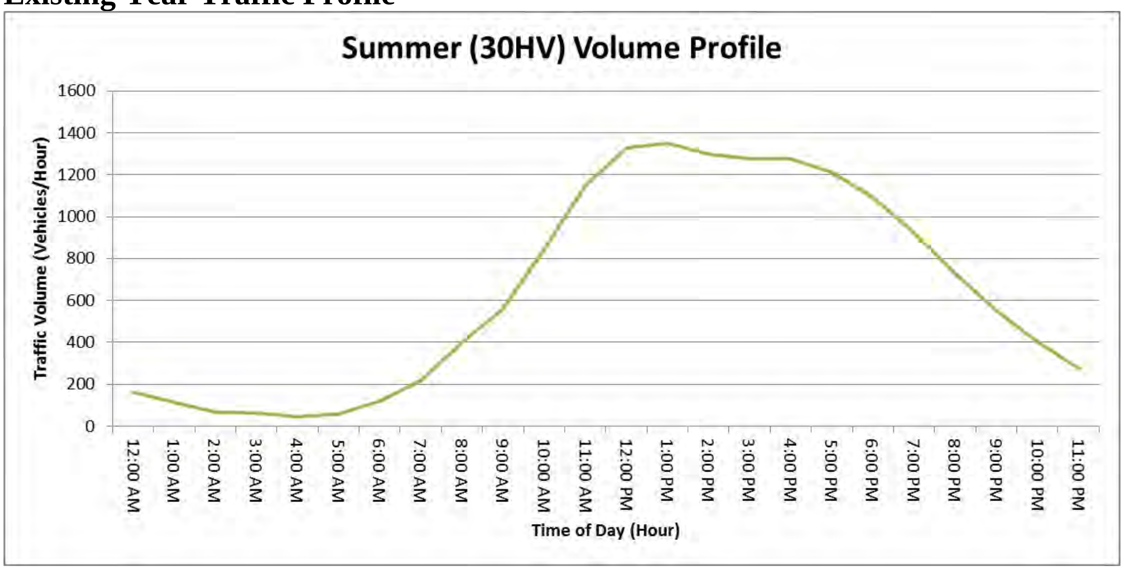
b. Shifting – A share of commuter or other users may shift their trip-making to an earlier and/or later period. (Determine portion to shift)
    i. Identify the portion of trips that will be shifted earlier or later based on the familiarity with the system.
    ii. Based on the split of excess demand that will be shifted earlier and later than the peak, identify when this period occurs. This will serve as the "divide" where excess demand is shifted before or after the peak.

Example 8-1 Cursory Method for Estimating Peak Spreading (To Determine Duration of Congestion)

In this example, the Cursory Method is applied to determine the duration of "congestion" at the US 101/ D River intersection in Lincoln City. This example assesses the southbound direction in the peak summer month (August). The duration of congestion, accounting for peak spreading, is estimated using the following steps:

- **Step 1: Determine the base year volume profile.** An Automatic Traffic Recorder (ATR) exists near the study intersection (D River Wayside ATR) and used to determine the base year volume profile. To determine the summer volume profile, each hourly volume is averaged for the entire month of August. The resulting profile is shown below.

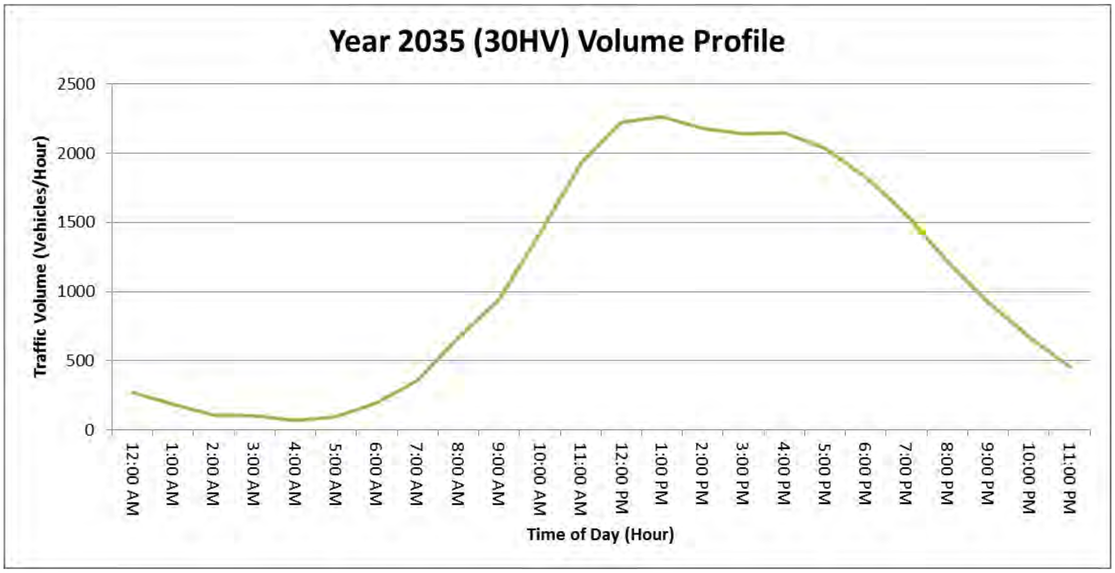
Existing Year Traffic Profile



- **Step 2: Factor the volume profile to the year 2035 p.m. peak hour volumes.** Through the forecasting process, the resulting year 2035 p.m. peak hour volume for the southbound approach is 2,145 vehicles/hour; the peak hour is 4 p.m. The corresponding base year volume at 4 p.m. (determined in Step 1) is 1,279 vehicles/hour. The resulting factor is 1.68 (i.e., 2145/1279). To determine the

2035 volume profile, each hour of the base year volume profile is multiplied by 1.68. The resulting year 2035 volume profile is shown below.

Factored Future Year Unconstrained Demand Profile



- **Step 3: Determine capacity constraint.** Through HCM analysis (e.g., Synchro HCM report), the southbound mainline approach capacity is 2,091 vehicles/hour. The same value of capacity is assumed for each hour of the 24-hour volume profile.

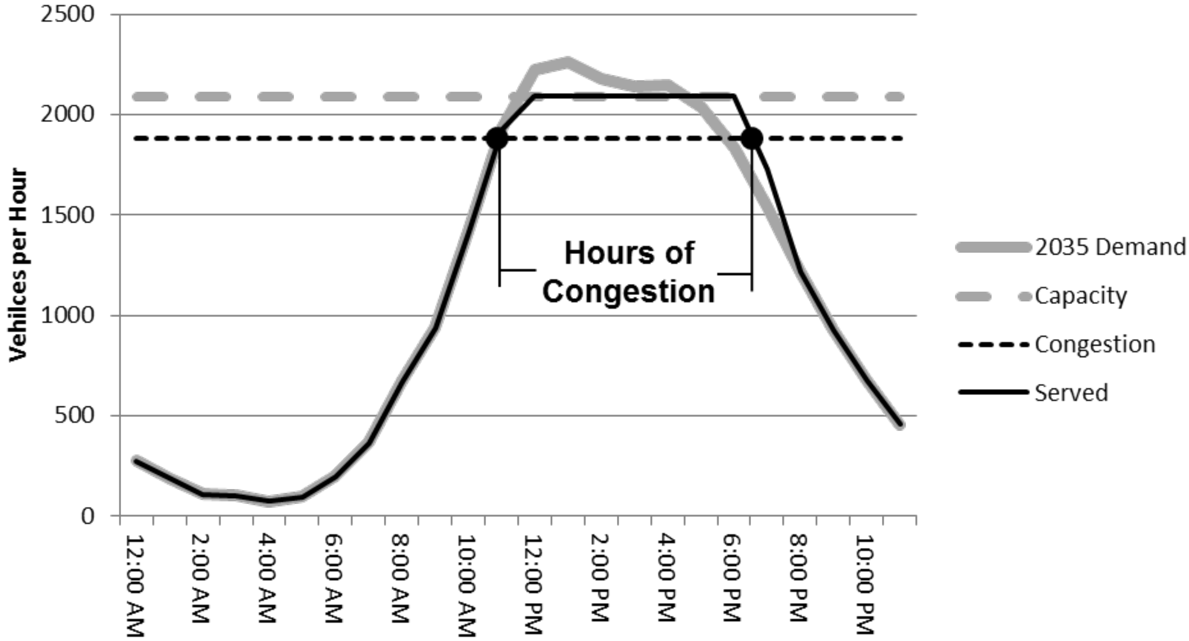
- **Step 4: Apply Peak Spreading by Shifting Demand to Determine the Volume Served.** First, for this analysis, it was assumed that due to the high amount of recreational traffic during the seasonal peak that no trips would shift before the peak. Therefore, all peak spreading would occur by shifting to a later period. This exercise included comparing each projected (unconstrained) hourly demand to the capacity. When an hourly volume exceeds capacity, the difference is added to the demand of the next hour. After shifting demand, demand is capped at capacity (2,091 vehicles/hour in this example). The following table shows the original demand projected for each hour as well as the resulting shifted capacity. Time periods where the ultimate volume is different than the original demand are shaded.

Shifted Volume Constrained by Capacity

Time	Demand	Capacity	Volume
12:00 AM	275	2091	275
1:00 AM	189	2091	189
2:00 AM	110	2091	110
3:00 AM	103	2091	103
4:00 AM	73	2091	73
5:00 AM	95	2091	95
6:00 AM	197	2091	197
7:00 AM	365	2091	365
8:00 AM	671	2091	671
9:00 AM	940	2091	940
10:00 AM	1416	2091	1416
11:00 AM	1927	2091	1927
12:00 PM	2226	2091	2091
1:00 PM	2263	2091	2091
2:00 PM	2178	2091	2091
3:00 PM	2140	2091	2091
4:00 PM	2145	2091	2091
5:00 PM	2036	2091	2091
6:00 PM	1831	2091	2091
7:00 PM	1549	2091	1731
8:00 PM	1219	2091	1219
9:00 PM	925	2091	925
10:00 PM	676	2091	676
11:00 PM	457	2091	457

- **Step 5: Determine "congestion."** Congestion can be defined many ways, but in this example, congestion is assumed as volume/capacity (v/c) of 0.90 or higher. To determine the equivalent volume at which the approach becomes congested, the v/c ratio is multiplied by capacity: 0.90*2,091 = 1,882 vehicles/hour.
- **Step 6: Determine hours of congestion.** For 8 hours of the day (11 a.m. to 7 p.m.) the volume of the southbound approach exceeds 1,882 vehicles/hour. This information is summarized in the figure below.

Cursory Method for Estimating Peak Spreading



Areas with Travel Demand Models (excluding the Portland Metropolitan Area)

Travel demand models have been developed for a number of areas around the state.[13] These models generally fit into one of two categories, each with distinctions that may provide ability to estimate peak spreading.

- Oregon Small Urban Models (OSUM) - Population under 50,000
- Joint Estimated Model in R (JEMnR) Population over 50,000 (MPOs)[14]

The following sections provide an overview of peak spreading available with each type of model. These methods could be applied systematically over the entire model area. Additional post-processing and refinements that incorporate more real-world data (such as spot location use of actual traffic profile data) and/or use of subarea modeling methods could provide information for peak spreading and traffic profiles on other network locations.

In addition, there is the potential that Metro's methods for pre-processing the trip table (shifting the demand of O-D pairs to other adjacent time periods based on a TTI ratio), described later in this section, could be applied to these other models with additional effort and coordination. In these urban areas, the modeler would need to identify whether there is systematic peak spreading, or if spreading is limited to a specific corridor through investigation of the variability of the TTI. This endeavor (and travel time data collection) would require further discussion and scoping with TPAU and Region traffic to determine the extent of the analysis based on the unique characteristics of the urban area. Application of this method would require development of a TTI ratio based on a perceived accepted maximum level of congestion (and travel delay) that currently occurs.

The following sections provide a general overview of the model tools and abilities. Coordination with TPAU modeling staff is critical to ensure appropriate use of the individual modeling tools based on issues related to model age, data sources, and calibration.

<u>OSUM</u>

The OSUM models can produce hourly trip matrices based on static input factors that can be assigned for each hour of the day. These trip matrices provide the ability to determine raw, model level demand profiles for links in the transportation model network. Additional post processing of these demands using real-world traffic data (depending on analysis need) has the potential to readily provide information about peak spreading. A key limitation for this method is that reliability would be a question for time periods that haven't been validated (e.g., 10 p.m. to 11 p.m.). Metro's method of pre-processing the

[13] For more information: https://www.oregon.gov/ODOT/Planning/Pages/Technical-Tools.aspx

[14] The Eugene-Springfield travel demand model is not a JEMnR model, but it shares many of the same features as it relates to modeling time-of-day and (the lack of) sensitivity to peak spreading.

trip table (shifting individual demand of OD pairs to adjacent time periods) may be an option, as noted in the previous section.

JEMnR (MPO)

The JEMnR model by default does not produce hourly matrices. Instead, the model is typically set up for the daily, a.m., and p.m. peak periods. It may however be set up for different time periods if desired. To develop matrices for other time periods, static input factors would be needed to split the daily person trip matrices by time-of-day and direction. Separate factors are used for each trip purpose. Once individual matrices were developed, the tool could be applied in a similar method as the OSUM models – with assignment of individual time periods and additional post-processing (using real-world traffic data), as needed, based on specific application needs. Metro's method of pre-processing the trip table (shifting individual demand of O-D pairs to adjacent time periods) may be an option, as noted in the previous section.

Portland Metropolitan Region

There are two recent tools that have been developed to analyze peak spreading in the Portland metropolitan region. A summary of each of these tools is presented in the following section, but additional documentation for each tool may be available[15].

Hours of Congestion (HOC) Tool [Interim Method]

ODOT Region 1 developed this tool as an interim method to assess the duration of congestion in the Portland region. The following excerpts summarize the purpose and methods used to develop the tool. Additional documentation is available from ODOT.

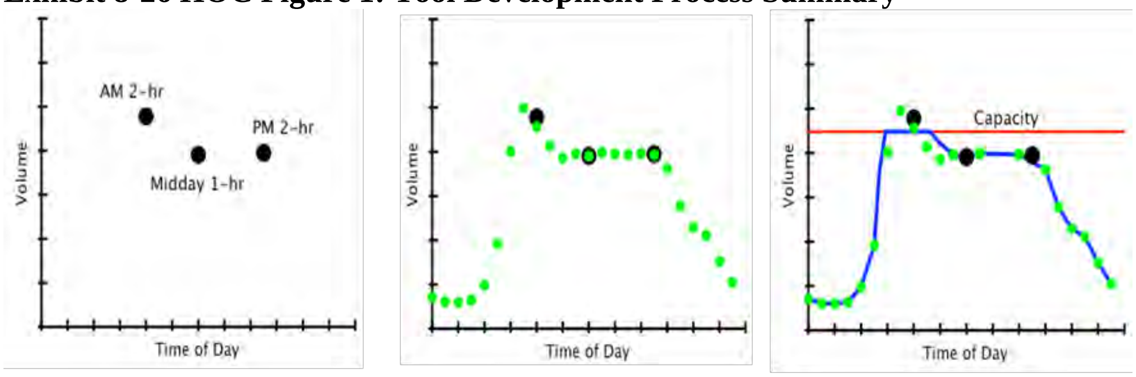
The tool is in the form of a spreadsheet that imports and processes data from the travel demand model to calculate hourly demand for every link in the model. The following excerpts from the documentation provide an overview of the procedures used by the tool:

> "The purpose of this study was to develop a method to address peak hour spreading given the limitations of a traditional four-step travel demand model. This was accomplished by using a link-based approach that post-processes travel demand model outputs to generate peak-spreading measures at the link level. The link based approach is viewed as an interim method for addressing hours of congestion where a more robust method would adjust the travel demand model trip tables. However, the link based approach is viewed as a reasonable estimate that will allow the congested hours on freeways and arterial roadways in the Portland metropolitan area to be approximated for purposes of policy discussions and project prioritization.
>
> Hours of Congestion Application

[15] For additional information, contact ODOT Region 1 Traffic Unit for the Hours of Congestion study or Metro's Transportation Research and Modeling Services for information related to regional model peak spreading.

To capture peak spreading on a corridor with a link based approach, traffic volumes produced from a travel demand model need to be post-processed to reveal an hourly volume profile. For this study, an application was developed that accomplishes this process in three basic steps, as shown in Figure 1 [Exhibit 8-26]. The first step involves gathering Metro's model data for the AM, midday, and PM peak periods. Next, the model data are used to estimate Average Daily Traffic (ADT) and hourly traffic volumes for the entire 24-hour period. Finally, the hourly volumes are compared to the link capacity and, where volumes exceed capacity, peak spreading is applied to spread the volume into shoulder hours.

Exhibit 8-26 HOC Figure 1: Tool Development Process Summary



Step 1: Metro Model Data

Step 2: Hourly Estimation

Step 3: Apply Peak Spreading

Methods

To determine the congested hours on roadways in the Portland metropolitan area using the link based approach described above, a data mining effort was undertaken to build the Hours of Congestion application based on observed traffic characteristics in the Portland area. The following sections describe the four major components of the data mining.

Data Collection

Data for ODOT facilities in the Portland area were collected for the most recent four years from PORTAL, ODOT Automated Traffic Recorders (ATRs), available 24-hour tube counts, and TriMet GPS bus travel time records. Reality checks and data quality screenings were performed on the PORTAL, ATR and tube-count data to remove data from outside the area of interest, incomplete or suspect data, and/or data that did not meet data quality diagnostics. For an example, of the original 665 PORTAL detector locations, 455 remained in the database after the data screening and quality checks were performed.

Volume Profile Analysis

Metro's regional travel demand model provides forecasts for a 2-hour AM period, a 1-hour midday period, and a 2-hour PM period. Daily traffic volumes are not directly forecasted. To estimate a 24-hour vehicle volume profile, regression analysis was conducted to first develop estimation factors for the total daily volume (ADT). With the predicted daily volumes and the peak period volume forecasts, a 24-hour volume curve was estimated to represent an "unconstrained"[16] forecast for each link.

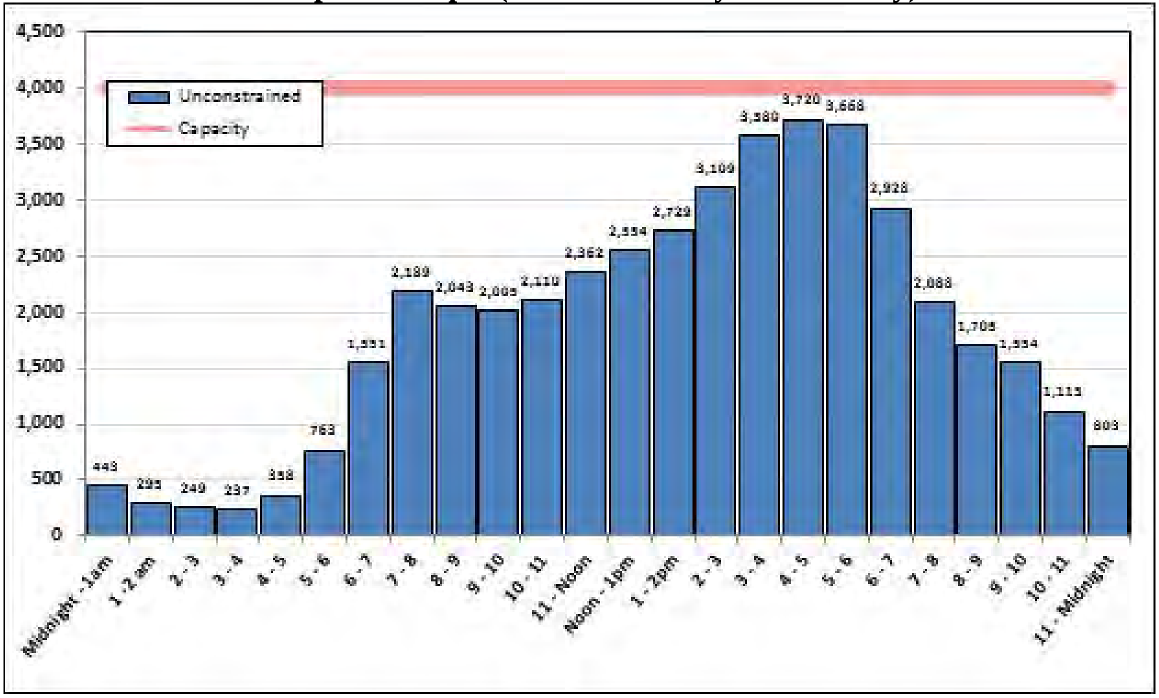
Peak Spreading Analysis
To develop an application that can spread excess volumes in peak periods, existing peak spreading in the Portland area was investigated in the PORTAL traffic volume database. Traffic volume data were examined to essentially identify congested vs. uncongested traffic flow days at locations where peak spreading was found to occur. At these locations, AM and PM peak periods were examined to determine how spreading occurs (i.e., which direction traffic shifts relative to the peak hour). These volume shifting factors were then used in the Hours of Congestion application to adjust 24-hour profiles when demand was forecasted to be above capacity. This shifting of traffic volume is an important distinction, as total daily traffic volume is conserved whereas some methods used by other agencies for similar efforts "trim" peak period volumes and do not maintain ADT.

Congestion Threshold Analysis
Review of Hours of Congestion analysis compared to speed data in the PORTAL system found that the congestion threshold (where vehicle speeds are significantly reduced from free-flow speed) may be well below the ODOT mobility standard, which is essentially where volume reaches capacity. An evaluation of speed congestion information compared to link v/c estimates was conducted, which found that a v/c ratio of 0.80 (instead of 1.00) would be a reasonable threshold for congestion. Therefore, the Hours of Congestion application was built to track both how many hours exceed a v/c ratio of 0.80 and how many hours reach a v/c ratio of 1.0."

The output of the Hours of Congestion tool is link-level traffic volumes for each hour of the day as shown in Exhibit 8-27. This example demonstrates a case where the link demand during the evening peak approaches the link capacity (4,000 vph) shown by the red line but does not reach the capacity. If the demand had exceeded the capacity, the tool would process the demand to the shoulder periods.

[16] It should be noted that while Metro's model does not provide temporal spreading of the traffic volumes in the peak periods when congestion occurs, it does spatially spread volumes to parallel corridors when possible to balance travel times in the system.

Exhibit 8-27 HOC Output Example (Link Volume by Hour of Day)



Metro Travel Demand Model with Peak Spreading

Metro has developed a methodology for pre-processing raw demand volumes from the static travel demand model to feed into dynamic traffic assignment (DTA). The output of the tool is an adjusted origin-destination matrix for each hour between 2:00 p.m. to 7:00 p.m. The following text is an excerpt for Metro's tool documentation, which is distributed with other travel demand model assignment tools[17]:

> "Metro's peak spreading algorithm is a method for measuring congestion through a travel time index (TTI), which is simply *travel time / free flow travel time*. For example, if the free flow travel time between an O-D pair is 10 minutes and the peak period travel time is 15 minutes, then the O-D pair has a TTI of 1.5 (15 minutes / 10 minutes).
>
> A proxy 'threshold' for congestion is based on the highest TTI corridor in our region within Existing Year (2010) regional model--I-5 NB from 5pm to 6pm. It's a corridor that currently experiences a good deal of peak spreading, which is represented in the relatively long congested peak (3pm – 6:30pm or later on most work nights). The TTI for this corridor is 1.6, and establishes a threshold for comparing all other congestion against. It is assumed that travelers are willing to accept congestion up to a TTI of 1.6 before they begin peak spreading.[18]
>
> The TTI threshold is used to adjust future year demand for each hour in the PM peak period. Since congestion is widely prevalent in the future, the peak period is

[17] Contact Metro's Transportation Research and Modeling Services for additional information.
[18] Application of this methodology to areas outside of the Portland metropolitan region would require assumptions about an appropriate TTI to use.

measured from 3pm-6pm. Hour-long shoulder periods are also produced, resulting in a full 5 hours of PM peak period trip tables (2pm-7pm).

To begin the peak spreading algorithm, a 5pm-6pm (peak of the peak) hourly static assignment is run, and the TTI for each O-D pair is then calculated. For each O-D pair in which the TTI exceeds the TTI threshold (1.6), trips are removed based on how much the TTI exceeds the threshold. For example, if an O-D pair has a 2.0 TTI, and the TTI threshold is 1.6, the difference is 0.4. This number is 25% above the TTI threshold (0.4 / 1.6), therefore 25% of the trips from this O-D pair are removed from the 5pm-6pm trip tables (SOV and HOV only) [Note that latent demand for other O-D pairs may potentially be reassigned to links belonging to paths and O-D pairs that were adjusted][19].

The trips that are removed from the 5pm-6pm hour are added to the 4pm-5pm and 6pm-7pm trip tables. For each of these hours, the above process is repeated: Produce hourly static assignments with the new trip tables, calculate the O-D TTIs, and remove excess trips based on the TTI-to-threshold ratio. The excess trips get added to the next shoulder hour (e.g., 3pm-4pm for the 4pm-5pm hour). The process is continued on through every hour of the 5-hour PM peak period.

There are a few additional rules that are observed during the process, such as never removing more than 50% of the trips in an O-D pair for the 5pm-6pm hour and never reducing the O-D trips in subsequent shoulder hours to values less than the final 5pm-6pm peak spread tables.

The final result is a set of hourly trip tables encompassing the 2pm-7pm time period, with O-D pairs reflecting different degrees of peak spreading depending on the initial amount of 'congestion' (i.e., comparison against the TTI threshold) measured during each hour."

Additional frequently asked questions for Metro's peak spreading methodology are available in Appendix 8A.

8.6.4 Key Considerations for Peak Spreading Application

Policy and Performance Measures

While accounting for peak spreading generally improves the reasonableness of traffic volumes, it may not be compatible with some performance measures and policies that are based on those performance measures. For example, mobility targets in the Portland region are based on V/C ratios, which for some locations (such as a Town Center) exceed

[19] The spreading is applied to the O-D matrix itself and not the assigned link volume. Therefore, after the O-D matrix has been adjusted it will need to be reassigned. The reassignment may result in different paths than the original assignment and (while the demand for the original O-D pairs using a link will be reduced) other latent demand from other O-D pairs may shift to the link that was originally over capacity. Therefore, reducing all O-D pairs assigned to a link by 25% may not ultimately result in a 25% demand reduction.

1.0. These policies are based on traditional tools that do not fully account for limitations related to peak spreading and capacity constraints within congested systems. Peak spreading methods have the potential to greatly improve the reasonableness of reported measures; however, it is important to understand the limitations and differences between policy measures and the tools that are used to analyze them.

Existing and Future Year Methods

Projecting peak spreading for future year conditions adds another aspect of uncertainty. Given limited resources and data, one approach may be to scale an existing demand profile to a forecasted future magnitude. However, assuming that existing year demand profiles (shape, not magnitude) are similar in the future may be affected by the following:

- Location of future growth areas and travel patterns on the facility
- Traveler decision making based on future changes to society and technology
- Future conditions on other areas of the travel network, including upstream/downstream bottlenecks and adjacent routes

[**Appendix 8A – Peak Spreading Procedure**]

[**Appendix 8B – PTV Vision Software Network Setup Guide**]

9 TRANSPORTATION ANALYSIS PERFORMANCE MEASURES

9.1 Introduction

Transportation analysis performance measures, sometimes referred to as measures of effectiveness (MOEs), are quantitative estimates on the performance of a transportation facility, service, program, system, scenario or project with respect to policies, goals and objectives. Some common performance measures used in traffic engineering include v/c ratio, level of service (LOS), crashes, vehicle delay, travel time, mode share and capacity. Performance measures in this chapter focus on the objectives of mobility **and** safety.

Performance measures can be based on empirical observations/data measurements of existing conditions or may be outputs of models that estimate or predict the performance of potential future scenarios, programs or alternative strategies.

Performance measures typically have some type of established threshold or target value or rating which defines the acceptable conditions for a facility. Any case where conditions do not meet that level is defined as a deficiency or need that should be reviewed. The term 'need' as used by transportation professionals has generally been defined as any case where the current or planned facility conditions fall below an established threshold."

The greater the deviation of the measured value from the performance threshold, the greater the need. Thresholds provide a critical element of the decision-making framework for assessing deficiencies and improvement alternatives since they are developed to maximize overall system performance while limiting liability to the agency responsible for construction, operations and maintenance. Thresholds may be known as goals, targets, or benchmarks. Thresholds may be adopted by a jurisdiction as part of a plan or policy.

Most road authorities (state, county or city) maintain adopted performance standards for operational efficiency that identify specific performance thresholds. It is important to identify all applicable performance standards and corresponding performance measures and thresholds for study roadways to provide a basis for evaluating the results of transportation analysis and to determine if project goals and objectives are being achieved. Methods of calculation or tools may also be prescribed.

9.1.1 Selecting Performance Measures

Performance measures to be used in a system plan, corridor study, development review, or project alternatives analysis are driven by the goals and objectives of the project and are identified during scoping and methodology development. Appendix 9A is provided as a general guide to aid in the consideration of potential transportation analysis performance measures by plan or project type, including RTPs, TSPs, MMAs, Facility Plans, Development Review, and NEPA/project development. For each type of study, potentially applicable performance measures are noted and identified as best practice/recommended, supplemental, or for screening purposes.

Performance measures may require varying levels of efforts depending on factors such as project type and tools used. Refer to Chapter 2 for level of effort information.

Some analysis performance measures are required. For example, state highway project v/c ratios are needed in order to compare the performance of alternatives with ODOT Highway Design Manual (HDM) mobility thresholds. Other analysis performance measures are often necessary, depending on the needs of the project. These selected performance measures become project evaluation criteria by defining them specifically to the project. In addition, project-specific thresholds and desired confidence or significance levels may be defined.

Performance measures should be SMART: Specific, Measurable, Agreed upon, Realistic, and Time-bound. See Chapter 10 for guidance on the process of developing project evaluation criteria and performance measures. Performance measures need to be sensitive enough to differentiate between analysis years and alternatives, scenarios or options. For example, a highly congested area with v/c's in excess of 1.0 and LOS F will not be sensitive to an increase in volume. In this case, different/additional measures would be needed such as travel time, safety, and reliability.

There is no one-size-fits-all performance measure that can address all the policies or objectives of a plan or project. Many performance measures address only one dimension of a problem while ignoring other important considerations. For example, a ratio or percentage based performance measure such as v/c ratio by itself does not indicate the number of users affected. Two roadways, one with a high volume and one with a low volume, may both have the same v/c ratio, but the high volume roadway affects more users than the low volume roadway. Multiple performance measures are typically needed.

The applicability or priority of performance measures depends on the purpose, need, goals and objectives of the project or plan, as well as on the facility and area type. In some cases the same performance measure can address multiple objectives. For example travel time can be used to assess emergency vehicle trips, or freight, or other modes. The number of performance measures chosen for any particular aspect for a project should be minimized. Too many performance measures for a given area may create conflicts, confusion, unnecessary work, and may result in measures not being used for that decision process. Some measures may not be clearly understandable to the desired audience or practically creatable based on data and tools available.

A matrix of typical analysis performance measures including definitions, purpose, modes, level of resolution, data and tool requirements is available in SPR Report 716[1].

[1] Development and Sensitivity Testing of Alternative Mobility Metrics, SPR 716, ODOT, John P. Gliebe and James G. Strathman, March, 2012, Table 3.1.

9.1.2 Purpose of this Chapter

Performance measures in this chapter are grouped into categories. Transportation can be measured in terms of its primary functions such as safety, accessibility and mobility. It can also be measured in terms of its impact or consequence, such as on the environment, and socioeconomics. It should be noted that while the performance measures identified below are assigned to a single primary category, some measures relate to multiple objectives and categories.

This chapter is limited to performance measures commonly reported out using APM methods and tools. These measures are used in plans and projects to identify needs, compare scenarios and alternatives, and identify benefits and impacts. The chapter focuses on facility level performance measures. System level performance measures generated by APM tools are discussed at a higher level. The performance measures covered are:

- Mobility
- Reliability
- Level of Service (LOS)
- Accessibility
- Safety
- Other Multimodal Performance Measures
- Infrastructure

The performances measures contained in this chapter are not an exhaustive list, but focus on those that are the most widely used and practical. Measures which have a good potential for application for a range of studies are also discussed. TPAU can provide assistance in selecting appropriate analytical performance measures for a specific project. A given project will use only a small subset of all possible measures. This chapter provides measure definitions, calculations, strengths and weaknesses, but leaves the application of performance measures to other referenced chapters. The use of performance measures to evaluate alternatives is discussed further in Chapter 10. For a broader discussion on mobility performance measures, see FHWA Traffic Analysis Tools Volume VI [(1)].

Not addressed in this chapter

- Factors that contribute to or are components of performance but are not typically reported out as stand-alone performance measures. Although not performance measures per se, in many instances these can provide additional useful information on the causes behind performance, which helps to understand or interpret the performance measure result. This includes analysis outputs used as inputs into performance measure calculations performed by other methods. For example, forecasted traffic volumes and speeds which are used to report out air quality and noise performance, or predicted crashes or delays which are used in economic analysis to report out reduced travel time or crash reduction cost performance.
- Preliminary screening criteria or flags are more intermediate in nature, such as those that are used as inputs into following steps and are not typically reported out as performance

measures, for example preliminary signal warrants or turn lane criteria. Refer to Chapter 10.
- Performance measures or evaluation criteria produced using methods or tools outside of the APM. For example, measures produced for managing the ODOT TSMO program such as average time to clear an incident. This also includes right of way, construction cost, funding, economics, design criteria, and environmental impacts.
- Performance measures relating to broad high level policy areas that are not specifically transportation-related, such as economic vitality; land use; environmental stewardship; quality of life, livability and health; equity; and funding and finance.
- Agency key performance measures (KPMs)/benchmarks/goals/targets for agency-wide or policy/strategic planning/investment strategies or monitoring purposes such as performance measure reporting required by the FAST Act; for example, ODOT Key Performance Measures. Refer to Chapter 18 for more information on the performance management planning and programming.

The broader KPMs are not comparable to analysis performance measures because they are different in purpose and resolution, and are likely to be based on different measurements, tools, level of aggregation, networks, assumptions, definitions, variables, data sources, formulas, and/or time periods. In contrast, analysis project alternatives are typically smaller in scale with greater resolution, focusing on study area roadway sections and intersections. Large scale performance measures would not be as useful on smaller projects and plans such as small city TSPs because they would not be likely to show a significantly measurable change in order to make comparisons useful.

9.2 Mobility

Mobility refers to the movement of both people and goods regardless of mode. Mobility performance relates to both supply and demand, as affected by land use and other policies. Supply could include the road network, transit routes, bicycle lanes, or any other modal infrastructure. Demand is the rate of flow which could include total persons, motorized vehicles, transit vehicles, etc., desiring to be able to traverse a point or section over a period of time such as an hour or a day. For additional detailed information on multimodal mobility related performance measures refer to HCM 6.

9.2.1 Volume to Capacity Ratio

The principal performance measure ODOT uses when evaluating motor vehicle operating characteristics on the state highway system is the volume to capacity (v/c) ratio, which is a measure of how close to capacity a roadway is operating. It reflects the ability of a facility to serve motorized vehicle traffic volume over a given time period under ideal conditions such as good weather, no incidents, no heavy vehicles, no geometric deficiencies. The volume to capacity ratio is the degree of utilization of the capacity of a segment, intersection or approach. The v/c ratio is not defined over 1.0. Under those conditions it is considered to be a demand to capacity ratio. A lower ratio indicates smooth operations and minimal delays. As the ratio approaches 1.0, congestion increases and performance is reduced. At 1.0 the capacity is fully utilized.

For example, when v/c equals 0.85, 85 percent of a highway's capacity is being used ; 15 percent of the capacity is still theoretically available. However, as the v/c ratio approaches 1.0, flow becomes unstable, speeds decrease, and bottlenecks can easily occur.

<u>Performance measures</u>

- Critical Intersection v/c ratio X_c (signalized intersections)
- Intersection Approach v/c ratio (unsignalized intersections)
- Segment v/c ratio (freeways, uninterrupted flow multilane highways and two-lane highways)
- Weave, merge, and diverge v/c ratio (freeways and uninterrupted flow multilane highways)

<u>Example evaluation criteria</u>

- Number of locations exceeding applicable v/c ratio standards
- Percent of intersections operating at V/C > 1.0

The generic formula for v/c ratio is as follows.

$$\frac{v}{c} = volume/capacity$$

There are other variations on calculating the v/c ratio. For example, for signalized intersections, Xc is used for total intersection v/c ratio as shown below. Refer to the HCM for fully detailed procedures.

Critical Intersection Volume to Capacity Ratio (for signalized intersections)

$$X_C = \left(\frac{C}{C-L}\right) \sum_{i \in ci} y_{c,i}$$

With

$$L = \sum_{i \in ci} l_{t,i}$$

Where

X_C = critical intersection volume to capacity ratio
C = cycle length (sec)
$y_{c,i}$ = critical flow ratio for phase $i = \frac{v_i}{(Ns_i)}$
$L_{t,i}$ = phase i lost time $= l_{1,i} + l_{2,i}$ (sec)
ci = set of critical phases on the critical path
L = cycle lost time (sec)

v_i = lane group flow rate for phase i
N = number of lanes for lane group i
s_i = lane group saturation flow rate for phase i

The v/c ratio can account for changes in either volume or supply (capacity). Volume is a measure of the rate of flow of traffic expressed as the number of vehicles passing a given point on a roadway over a specified time period, such as vehicles per hour or day. Volume is most commonly reduced at a location or facility by adding alternative routes or connections which may shift traffic to other routes. Other means of shifting volume include TDM or TSMO measures such as ramp metering, traveler information, tolling or congestion pricing. Procedures for developing traffic volumes are found in APM Chapter 5 for Existing and Chapter 6 for Future Year.

Capacity is the supply side measure of the ability of a facility to carry traffic. It is the maximum number of motorized vehicles per hour that can travel on a particular stretch of roadway under relatively ideal conditions such as proper lane widths, no parking, no bus blockages, etc. Capacity is a function of a number of variables including number of lanes, lane width, shoulder width, presence and type of control devices, free flow speed, and other features. Capacity may be calculated by HCM methods or measured in the field in locations and conditions where demand exceeds capacity. Procedures for calculating capacity and v/c ratios are found in the APM chapters on segments and intersections and are primarily based on methodologies in the Highway Capacity Manual and implemented by various software tools.

ODOT uses v/c-based measures for reasons of application consistency and flexibility, manageable data requirements, forecasting accuracy, and the ability to aggregate into area-wide targets that are fairly easy to understand and specify. In addition, since v/c is responsive to changes in volume as well as in capacity, it reflects the results of demand management, land use and multimodal policies. Other advantages of v/c ratio include:

- Standardized calculation methodologies and tools
- Easily applied and forecasted
- Planning level methods are available to estimate segment v/c ratios. Volumes are estimated using AADTs along with K_{30} factors and directional factors. Capacity estimates can include the use of default values in estimating v/c ratios with the results reported out as below, near, or at capacity, as example, HERS-ST performs this level of v/c ratio analysis (refer to Chapter 7). For urban signalized arterials, segment capacity can be estimated using approximate green time to cycle time (g/c) ratio assumptions.
- Can be calculated for segments, intersections, approaches, and turn movements
- Travel demand models calculate a link-based demand to capacity ratio (d/c). Refer to Section 9.2.5 on model based demand to capacity ratios.

Requirements/Limitations

- Does not directly apply to or address safety, non-motorized vehicle modes, operational improvements, and other policy objectives often under consideration because these

aspects of the transportation system cannot be directly measured in terms of vehicle demand and vehicle capacity.
- Identifies when capacity is exceeded, but does not address the extent or duration of congestion or queue spill-back effects. By definition, the volume of traffic using a roadway cannot exceed the roadway's capacity. When demand exceeds capacity, a demand-to-capacity (d/c) ratio may be used (see section on Demand to Capacity Ratio). A d/c ratio that exceeds 1.00 indicates that more vehicles would use a roadway in a given time period if capacity constraints were not present.
- Is focused on a recurring peak period and does not address non-recurring congestion such as from incidents, weather, or special events.

9.2.2 Oregon Highway Plan Mobility Targets

ODOT has adopted specific v/c ratio thresholds for identifying current and future needs in the Oregon Highway Plan (OHP) which are used for identifying needs in planning. These are different from the performance thresholds for project design in the Highway Design Manual (HDM), which are accepted by FHWA for design and need to be lower than the planning need threshold in order to allow for the project to have a design life.

Volume to capacity ratio was selected as the performance standard for motor vehicle mobility on state highways in the Oregon Highway Plan (OHP) after an extensive analysis of candidate highway performance measures. The review included the effectiveness of the measure to achieve other policies (particularly OHP Policy 1B, Land Use and Transportation), implications for growth patterns, how specifically ODOT should integrate transportation policy with land use, flexibility for modifying targets, and the effects of Portland metro area targets on the major state highways in the region.

Targets for state highway motorized vehicle mobility needs are established in the current OHP Policy 1F. Tables 6 and 7 within Policy 1F contain the v/c ratio targets for various combinations of highway classifications and surrounding land uses, with Table 7 applying to the Portland metropolitan area and Table 6 applying to the remainder of the state.

The targets vary the priority for mobility according to facility, area and designation type; mobility is a high priority on freeways, expressways and freight routes, but is a lower priority on District highways or local interest roads in Special Transportation Areas (STA) and Metropolitan Planning Organizations (MPO). It should be noted that the text within Policy 1F contains exceptions to the targets listed in these tables and, therefore, must be consulted as well. Furthermore, the OHP Registry of Amendments webpage should be checked for amendments to the OHP mobility policy where alternative mobility targets have been adopted; for an example refer to the report US 101 Seaside Alternate Mobility Standards.

The analyst should refer to OHP Policy 1F for appropriate application of the OHP mobility targets in specific contexts. For plan amendment applications also refer to TPR 0060.

9.2.3 Oregon Highway Plan Alternative Mobility Targets

The v/c ratio targets were generally designed to provide continued operation in an under capacity condition. Increasingly in urban areas, there are roadways that are projected to be over-capacity, or that are currently operating in an over-capacity mode. Circumstances exist where v/c targets cannot reasonably be met due to financial, environmental or land use constraints. In these circumstances, where it is not feasible or desirable to make infrastructure investments to fully accommodate the existing and projected vehicular demand, it is possible to explore alternative mobility targets.

If meeting OHP v/c ratio targets is not practical or feasible due to financial, environmental or land use constraints or impacts, OHP Action 1F.3 contains provisions for creating alternative mobility measures and targets through a planning process and adoption by the Oregon Transportation Commission (OTC). Adjustments to the OHP targets may include changing the v/c ratio target (increase or decrease), changing the analysis methodology (e.g., from 30th highest hour to average annual traffic volumes or adjusting peak hour factors), and/or acknowledging that a facility will likely operate at capacity for more than just a single peak hour. Alternative (non v/c-based) performance measures may involve other analysis methods that address safety performance, travel time reliability and delay.

The process for consideration of alternative mobility targets is detailed in the Planning Business Leadership Team PBLT Operational Notice PB-02. This process involves the participation, commitment and mutual agreement of local and regional jurisdictions and includes exploring a variety of transportation-related solutions, including a number of system and demand management activities to maximize the efficiency of transportation movements and to identify solutions that are realistic to implement and have the potential to be effective. Under most circumstances, local jurisdictions must adopt appropriate local policies, codes and ordinances that are necessary to help support and implement the alternative mobility target and achieve other policy and performance objectives.

In some cases such as a rural interchange area management plan, more restrictive alternative v/c ratio targets may be adopted as part of OHP Action 1F.4. More restrictive targets may help to maintain mobility in an identified area. This can be an effective tool where it is desirable to further preserve a significant investment, such as in the vicinity of an interchange.

Developing OHP Alternative Performance Targets or Measures

The following v/c-based methodology is recommended as a first option when developing alternative mobility targets for state highways outside the Portland Metro area. OHP policy and current analysis practices use a v/c-based methodology as the initial measure to standardize and simplify implementation through a quantifiable, consistent and reproducible measure. Where v/c-based approaches may not meet all needs and objectives, developing alternative mobility targets using non v/c-based measures may also be pursued. Any alternative mobility target per the OHP, including new methodology, will not be final until adopted by the OTC.

1. In cases where v/c is forecasted to be greater than the OHP mobility target but less than capacity (v/c = 1.0) during the design hour using standard analysis procedures, establish the proposed alternative target consistent with the v/c values used in the OHP (0.75, 0.80, 0.85, 0.90, etc.).

2. In cases where v/c is forecasted to be greater than or equal to capacity during the design hour using the standard analysis procedures evaluate the actual peak hour traffic volume for future year design hour projections rather than expanding the peak 15 minutes to be the design hour traffic volume (e.g. peak hour factor) for projection purposes. If v/c is less than 1.0, establish the proposed alternative target.

3. In cases where v/c is forecasted to be greater than or equal to capacity during the design hour using the actual peak hour projection of traffic and in areas where design hours are affected by high seasonal traffic volumes, evaluate the Annual Average Weekday PM Peak as the future year design hour rather than the 30th highest hour. If v/c is less than 1.0, establish the proposed alternative target.

4. In cases where v/c is forecasted to be ≥1.0 using the Annual Average Weekday PM Peak as the future design hour, determine the duration of the period during which the future Annual Average Weekday PM Peak hour will have a v/c ≥1.0. Establish the proposed alternative target by increasing the number of hours that v/c can be ≥1.0 (i.e., v/c ≥1.0 for not more than 1 hour, or not more than 2 hours, etc.).

If a v/c-based mobility measure does not by itself meet the needs of the jurisdiction, the state or the particular facility under consideration, then it is reasonable to explore non v/c-based measures for defining mobility on the state highway system. At a minimum, all non v/c-based measures must:

1. Be consistent with OHP Policy 1F, with particular attention to Actions 1F.1 and 1F.3;

2. Follow the PBLT Operational Notice PB-02 Attachment A Checklist; and

3. Develop a measurable and defensible target value, with defined geographic limits and a defined analysis methodology that can be compared between alternatives, recognizes data needs, availability and quality, and considers requirements for implementation including the availability of analysis tools, staff responsibilities and associated costs.

Recognize that, even when exploring non v/c-based measures, there may still be advantages to keeping v/c measures as well. The v/c ratio along with other measures provides a complete picture of operations.

9.2.4 Highway Design Manual Mobility Guidelines

Motor vehicle mobility thresholds for design of modernization projects are identified in Exhibit 10-1 of ODOT's HDM. These v/c ratios (the functional equivalents of the LOS standards in the American Association of State Highway and Transportation Officials [AASHTO] Green Book) represent the level of operation for which state facilities are expected to be designed and are intended to be applied to an analysis year occurring 20 years beyond the year of completion. These thresholds are applicable to future build alternatives on state highways associated with all project types except Traffic Impact Studies associated with development, unless an interchange or interstate freeway is involved. It should be noted that for ramp terminals, the HDM mainline maximum v/c ratio is the standard that applies. There is no equivalent ramp terminal v/c ratio in the OHP as there is in the HDM.

Exhibit 9-1 illustrates the appropriate sources of adopted mobility performance measure standards for different project types.

Exhibit 9-1 Sources of Adopted Mobility Targets/Standards for State Highways by Study Type

	TIS/TIA	Projects	TSPs	Corridor and Refinement Plans
Existing Conditions	OHP	OHP	OHP	OHP
Future No-Build	OHP	OHP	OHP	OHP
Future Modernization Build(s)	OHP	HDM	HDM (OHP in Portland Metro Area[1])	HDM

[1] In the Portland metropolitan area, future modernization build alternatives on state highways are scoped and analyzed in corridor plans, refinement plans or projects rather than as part of TSPs.

HDM mobility thresholds are generally more restrictive than the OHP mobility targets; however, there is a design exception process that allows variation from the HDM when appropriate. Transportation System Plans (TSPs) generally identify needs and the function, mode, location, and parameters (e.g. number of lanes) of solutions. The precise location, alignment, and preliminary design of solutions is typically deferred to refinement studies or project development.

HDM FHWA-ODOT MOU

In order to be used as baseline standards for future project design, alternative mobility targets being considered as an amendment to the OHP must be established in coordination with FHWA. This process is described in the Memorandum of Understanding (MOU) between ODOT and FHWA, provided as an attachment to PB-02. Through this process, the alternative mobility target may be adopted as an amendment to the HDM.

9.2.5 Supplemental Vehicle Mobility Measures

Many of the mobility analysis procedures summarized in the APM have direct (or equivalent) v/c ratio results for performance assessment. The compliance with the appropriate target (maximum v/c ratio thresholds defined in the OHP) is the first tier of the evaluation. The other category of performance measures focuses on travel time/speed, including progression analysis, arterial analysis and selected outputs of many simulation models. The vehicle speed outcomes can be compared to target or design speeds to assess relative benefit, but there is no direct comparison with v/c ratio in these analyses. It is recommended that these types of measures be used in conjunction with either intersection or segment analyses that do have v/c ratio related outcomes to compare to mobility targets.

Typical travel demand model-based performance measures are calculated using model generated outputs that yield general system performance of the scenario. Scenarios would be considered relatively the same if there is no significant difference in the performance measure (less than 10%) because of the model's limited accuracy. Performance measures can be system-wide or segregated into select facilities, corridors, areas, or zones.

Quantity of Travel

Quantity of travel represents the amount of use of a facility or service. It is both a performance measure and an input into the calculation of other performance measures. Quantity of travel is usually expressed as the number of motorized vehicles, persons, pedestrians, bicyclists, or transit vehicles per unit of time. Methods to estimate the quantity of travel range from simple historical trends to cumulative analysis to complex urban, regional or statewide travel demand models.

For more information refer to APM Chapters 6 and 7 and the ODOT Planning Section Technical Tools webpage.

Percent Change in Volume

Percent change in motor vehicle volume is used to compare different scenarios or alternatives such as No Build versus Build, different land use scenarios, or multiple Build alternatives. Build alternatives that increase capacity and/or reduce travel time often result in network volume changes, for example due to demand shifting between competing routes. Volume change is commonly obtained by comparing similar segments across screenlines or difference plots between two scenarios from a travel demand model, or can be estimated using deterministic methods. Volume change can be used as a high level analysis or preliminary screening measure. Changes of more than 10% are generally considered the minimal level of significance. Changes of more than 20% have a large impact, depending on the absolute volume level.

Performance measures

- Design hour volume on segment or screenline

Example evaluation criteria

- Percent change in volume crossing screenline
- Percent change in volume on link
- Diversions or neighborhood cut-through traffic due to temporary/permanent lane closures including road diets, work zones, congestion, incidents.

Vehicle-Miles of Travel (VMT)

VMT is the amount of vehicle travel on a system in terms of both vehicle volume and distance. VMT is the relationship of the total vehicle volume on the specified links multiplied by the total link lengths. VMT is typically a system performance measure reported for large-scale, regionally significant changes or regional/MPO areas and should generally only be used in high-level planning analyses. Although VMT can be calculated on any facility, it is not typically reported out. VMT is also calculated as part of energy analyses in Environmental Impact Statements.

Performance measure

- Vehicle-miles of travel on segment or facility

Example evaluation criteria

- Change in area/region VMT
- Change in facility/corridor VMT
- Change in segment VMT

$$VMT = AADT \times length$$

VMT is typically reported as a daily value but may be specified as an average annual value based on 365 days a year. The analyst should be aware that VMT can be calculated based on different data sources, tools or methodologies. For example, gasoline sales based VMT (when combined with average vehicle MPG), official VMT from HPMS used in HERS (link-level, statewide, state-owned facilities only), RSPM (all days average, household-based, all roads), or from a travel demand model (average weekday, mostly state system, within model area only).

VMT is typically reported as an annual average daily value per segment:

$$Daily\ VMT = AADT \times Length$$

VMT is also commonly reported for an entire facility, system or subset of roads by summing individual segment VMTs:

$$Facility\ or\ System\ VMT = \sum Segment\ VMTs$$

For trucks,

$$Truck\ Miles\ Traveled = AADT \times length \times \%\ trucks$$

Oregon historical VMT data at a state facility level or broad regional level, reported as part of the Highway Performance Monitoring System (HPMS), may be obtained by contacting ODOT Road Inventory & Classification Services.

Regional VMT within an urban area is a common travel demand model measure. Reporting can be for the entire model area or for roads within a sub-area (e.g., UGB, MPO boundary), for all trips or a portion of the trips (e.g., internal-internal (I-I) trips only, truck-only). Model produced VMT may be reported by mode and by trip purpose. Even where total demand is the same, VMT can increase due to changes in trip lengths, such as a scenario where trips lengthen when land use growth is mainly on the fringe of the urban area, or increasing road congestion may result in either shorter trips or forces trips to take alternate routes which may be longer.

VMT is closely related to both the demand and the supply side of the urban setting. Levels are lower in communities that are more walkable and compact and in communities that have a strong public transport system. Increasing population density can lower VMT as well, although increased density may increase the VMT in the local area but may reduce the overall system VMT. VMT can also drop due to economic downturns, when unemployment is high and people have a smaller shopping budget. Vehicle operating costs including fuel costs, per mile fees and vehicle MPG, can also significantly impact VMT. Population shifts or new population estimates can change VMT trends significantly. Many of these factors are outside the agency's control.

VMT results can be subject to misinterpretation as many factors can contribute to a particular increase or decrease in the value. For example, a VMT increase could be due to more people driving, but it also could be due to new growth on the fringe of an area with a subset of the population having to drive longer distances.

The TPR requirement of VMT per capita is limited to internal trips only, even though models can produce VMT per capita for all trips. The TPR measure can be skewed based on the relative size of the model area, the proportion of external trips, or other individual characteristics of the urban area such as demographics (i.e., high retiree population). The measure can also be skewed when population forecasts change.

Person-Miles Traveled (PMT)

PMT is similar to VMT except that person travel is measured rather than vehicle travel. Person travel includes motorized vehicle drivers, passengers, transit riders, rail passengers, pedestrians or bicyclists. For motorized vehicle travel, person-miles traveled is typically calculated as follows:

$$PMT = AADT \times length \times vehicle\ occupancy$$

Performance measure

- Person-miles of travel

Example evaluation criteria

- Change in area/region PMT
- Change in facility/corridor PMT
- Portion of drive-alone mode (SOV) trips

Total PMT on a facility would need to add non-motorized vehicle person trips. The typical method of calculation involves use of travel demand model VMT divided by mode share.

The amount of person travel a corridor or system serves, PMT is directly related to VMT as it is VMT multiplied by a vehicle or transit occupancy factor. PMT should only be used for high-level planning processes because of the high level of estimation required. PMT can also be calculated for modes on a regional basis if the mode split is known like from a MPO travel demand model. Bicycle and pedestrian counts could also be used to determine PMT if trip lengths are known on a facility basis. PMT has the same limitations as VMT. Calculating PMT may be difficult as occupancy factors may not be available or not enough bicycle/pedestrian counts may be available. OSUM models assume a static value for auto occupancy by trip purpose. In JEMnR and SWIM models, auto occupancy reacts to land use and transportation policies and projects and can be reported. The analyst should coordinate with the modeler as to the applicability of its use.

A commonly reported mode share performance measure is the portion of travel by drive alone mode, or single occupant vehicle (SOV). This can be reported for a region or corridor.

$$Portion\ of\ Drive\ Alone\ Mode = \frac{SOV\ VMT}{Total\ VMT}$$

Portion of SOV trips can be used to evaluate alternatives that encourage non-drive alone trips, such as park and ride lots.

Throughput

Throughput is the hourly volume of traffic that a facility serves or discharges.

Performance measure

- Throughput on segment or intersection

Example evaluation criteria

- Change in facility/corridor throughput
- Segments and time periods where demand is metered due to upstream bottlenecks
- Intersections and time periods or number of cycle lengths where green time is starved due to upstream bottlenecks

Vehicle throughput is also a calibration measure used in microsimulation. Refer to APM version 1 Chapter 8. Vehicle throughput as reported in SimTraffic is known as “Vehicles Exited”.

Throughput is sometimes confused with capacity. Even where peak hour demand equals or exceeds capacity, vehicle throughput is often less than capacity for several reasons, including:

- Metered volume – volume at an approach to a segment or intersection may be metered or constrained due to an upstream bottleneck or similar condition, so fewer vehicles can be served than otherwise could be.
- Congested bottleneck – once traffic flow has broken down on a free-flow facility, the queue discharge flow rate from the bottleneck is less than the capacity of the freeway.
- Other temporary conditions such as inclement weather, incidents, and work zones.

Degree of Utilization/Congestion

Degree of utilization is the percent of a facility’s capacity that is being used by the traffic volume, typically for a peak hour. The most commonly used measure is the v/c ratio. As the degree of utilization increases, mobility (freedom of movement) and speed decrease and density increases. Eventually, as volumes increase beyond a certain level, vehicles become impeded enough that traffic flow breaks down, and speeds drop to near zero and the facility is considered congested.

Degree of utilization is sometimes reported for other modes such as pedestrians, bikes, and transit. In Oregon, with a few exceptions, pedestrian and bicyclist degree of utilization is not typically reported because most pedestrian and bicycle volumes do not typically approach the physical capacity of the facility.

Duration of Congestion

The measures discussed in this section evaluate recurring congestion. See Travel Time Reliability section for measures that evaluate non-recurring congestion. Duration of congestion reflects the temporal extent of congestion. Hours of congestion has been used as an alternative mobility performance measure per OHP 1F.3. It is the period of time, that a segment, facility or area is congested. A facility or area may experience multiple recurring periods of congestion, such as an AM period and a PM period. Refer to APM Chapter 8 for procedures. Duration of congestion may be visualized with exhibits such as contour diagrams or heat maps, see example in Exhibit 9-2.

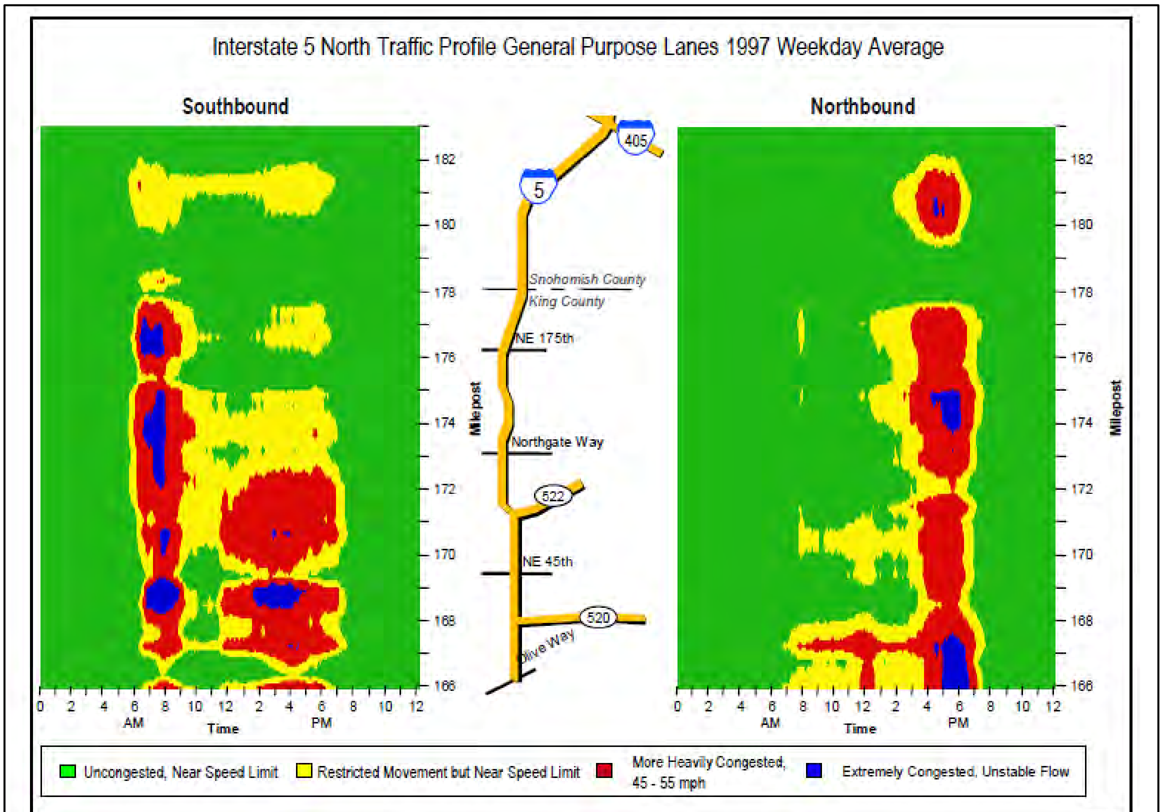
Performance measure

- d/c ratio above 1.0
- Speed below an agreed-upon threshold
- Excess/unserved demand
- Queue on uninterrupted flow facility
- Average Daily Traffic to Capacity Ratio (ADT/C)

Example evaluation criteria

- Number of hours facility exceeds capacity, v/c ratio > 1.0
- Number of hours the facility is rated at LOS F
- Number of hours that speed is below a designated threshold

Exhibit 9-2: Sample Heat Map



Source: Traffic Congestion and Reliability: Trends and Advanced Strategies for Congestion Mitigation, FHWA, 2005

For analysis purposes non-recurring congestion is typically assumed to start when the demand in the analysis time period exceeds capacity. The congested period ends when there is no longer excess or unserved demand in the analysis time period. Other threshold definitions of congestion are sometimes used for other purposes such as for performance monitoring or investment decisions, such as using a speed threshold and speeds obtained from a travel demand model.

Travel speeds from a travel demand model are approximate and should only be used on a relative basis to compare alternatives/scenarios.

Average Daily Traffic to Capacity Ratio (ADT/C)

The ADT to C ratio (ADT/C) is the average daily traffic divided by the peak hour capacity. ADT/C has been used by ODOT as a rough indicator of the level of congestion. The ADT/C methodology was developed as part of studies prepared for FHWA [2] and has been used by ODOT as part of the statewide congestion management system. It is a way to estimate peak spreading on a road system. It is a higher planning level rating of the level of congestion as compared to duration of congestion or queueing. ADT/C can be used for segments or intersection approach analysis for planning estimates of congestion. For segments ADT/C should be reported out by direction. For intersections sum the sum the lane group capacities for each approach. The highest approach ADT/C is reported for the intersection.

Performance measure

- ADT/C

Example evaluation criteria

- Model links with ADT/C ratio exceeding threshold
- Alternative change in ADT/C ratio

Model capacities can be coded as a segment capacity or as a lane capacity depending on the model platform. The per lane capacity must be multiplied by the number of lanes before calculating the ADT/C ratio.

ADT/C thresholds are offered (from) as shown in Exhibit 9-3 below:

Exhibit 9-3 ADT/C Congestion Level Thresholds

Level	Condition	Description	Lower ADT/C	Upper ADT/C
1	Uncongested	No decrease in speeds during the peak hour.	0.00	6.75
2	Uncongested to Moderately		6.75	8.25
3	Moderately Congested	Speeds decrease slightly during portions of the peak hour.	8.25	9.25
4	Moderately to Congested		9.25	9.75
5	Congested	Speeds decrease significantly during portions of the peak hour.	9.75	10.75
6	Congested to Very		10.75	12.25
7	Very Congested	Speeds decrease substantially for substantial portions of the peak hour.	12.25	13.75
8	Very to Extremely		13.75	15.25
9	Extremely Congested	Speeds decrease substantially for more than the peak hour.	15.25	24.00

Queue Length

Motor vehicle queue length is typically a peak period performance measure. Queues occur in both under and over-saturated conditions. Undersaturated queues occur on interrupted flow facilities at traffic control locations. Both segments and intersections will experience oversaturated queuing when demand exceeds capacity. Oversaturated queue lengths measure the spatial extent of congestion in length (typically feet). Normally free-flow segments can experience queues when oversaturated.

Performance measures

- 95th percentile queue length

Example evaluation criteria

- 95th percentile queue length by approach (refer to APM v1 Chapter 7 and v2 Chapter 8)
- Queue blocking of turn or through lane
- Intersection queue blocking percentage exceeding 5% of peak hour
    - Queue spillback to railroad crossing
    - Queue spillback to functional area of intersection
    - Queue on exit ramp extending to deceleration area or mainline as calculated using design speed
    - Queue occurring in area with insufficient sight distance

Undersaturated queueing at a signalized intersection approach tends to build and dissipate with every green phase, with a maximum value reached during the peak period. Undersaturated queueing at a stop controlled intersection approach tends to gradually build to a maximum value during the peak period and then dissipate.

A queue blockage or spillback condition should be reported when the duration exceeds five percent of the peak hour. See Chapters 12 and 13 of the APM. Queue spillbacks need to be evaluated with other contextual information to determine the extent and nature of the problem. Spillback queues can reduce both safety and capacity. Spillback occurs when a queue at one intersection extends into a second signalized intersection. This is typically reported as the total length of oversaturated queue beginning from bottleneck where the queue started.

Queuing is usually reported as the number of vehicles or length of vehicles in queue at the 95th percentile. 95th percentile queues are typically used to identify the extent of queuing problems and to evaluate alternatives that reduce queue lengths.

95th percentile queues are calculated using deterministic tools following HCM methods, or by microsimulation. Depending on the solutions being evaluated, microsimulation is typically needed for final design in congested conditions. Methods of calculation vary by facility type and level of analysis detail. Refer to APM Chapters 12 and 13 for deterministic queue calculation procedures. Queuing is provided by microsimulation models where v/c ratios are high or conditions are congested (refer to APM Chapter 15).

Demand to Capacity Ratio

When the estimated v/c ratio exceeds 1.0, it is referred to as a demand to capacity (d/c) ratio. Travel demand models generate demand which can be used to calculate d/c ratios.

> *Typically a travel demand model run would be a constrained run. An unconstrained (infinite capacity) run can be requested that will show the full desired demand on a facility.*

This means that for a given time period, there are more vehicles desiring to use a facility than it can accommodate. This is also known as oversaturation. The actual volume will never exceed the capacity of the facility. Instead, the excess demand (unserved trips) may do one or more of the following: divert to other routes; change the time of the trip; distribute to other destinations; change the travel mode; or queue up to be served in following time periods (incurring additional delay).

Performance measure

- d/c ratio

Example evaluation criteria

- Travel demand model links with directional link peak hour d/c ratio exceeding 1.0
- Number of locations on state highways with a d/c ratio of 0.90 or higher
- Number of urban area lane-miles over 1.0 d/c ratio

Travel demand model d/c ratios are link-based and can only be relatively compared on a large-scale basis such as below, at, or over capacity. They cannot be compared with the Oregon Highway Plan or Highway Design Manual volume-to-capacity ratios as these require that volumes are based on the 30th highest hour from actual ground counts, while raw (not post-processed) model volumes typically only represent an average weekday condition and have been calibrated to the facility level. Also, model capacities are generically estimated based on functional class and speed rather than using HCM methods. Model d/c ratios represent a full 60-minute period rather than the peak 15-minute period. Model d/c ratios provide a planning level indication of the extent of demand on segments, including the level of potential congestion, without pinpointing specific intersection bottlenecks. For preliminary screening purposes model d/c ratios may be reported as below, near, or over capacity rather than reporting specific values.

- Over capacity: d/c > 1.10
- At capacity: d/c between .90 and 1.10
- Near capacity: d/c between .80 and .89
- Below capacity: d/c < 0.80

The d/c ratio can be used to evaluate and rank or prioritize oversaturated links, and to evaluate alternatives that reduce demand or increase capacity.

Travel Time

Travel time is a measure of the length of time a segment, facility or route can be traversed in a given time period. It is most often reported for a given direction during the peak period and expressed as the average travel time of all vehicles. Influences include design speed (encompassing facility geometrics), free flow speed, control delay, traffic volume, and travel distance.

Performance measures

- Average travel time during peak period
- Freight travel time
- Emergency services response time

Example evaluation criteria

Average peak period travel time is the sum of travel times for all trips on the segment over a given time period, divided by the traffic volume, typically expressed as minutes or seconds.

$$Travel\ Time = \frac{Distance}{Speed}$$

Speed is based on segment running speed or field or archived speed data between representative locations such as intermodal facilities, employment centers, CBDs, medical centers, park and ride lots, or transit centers.

> *As an input to noise analysis, speed is calculated as the speed at LOS C, except is reported as the posted speed if volumes are less than volumes at LOS C.*

Field or archived speed data such as from private sector probe data may be used as a measure of existing travel times as well as for reasonability checking of modeled speeds. Differences in methodologies need to be taken into account when comparing existing speeds from different sources and modeled speeds.

Total vehicle travel time is the average travel time per vehicle multiplied by the vehicle volume over the analysis period. At a very basic level:

$$Average\ Travel\ Time = \frac{Segment\ Length}{Average\ Travel\ Speed} \times 60$$

Where

Travel Time = Average travel time of all vehicles traversing segment (min)
Travel Speed = Average travel speed of vehicles (mi/hr)
Segment Length = Length of section (mi)

Travel time may be developed from travel demand models, based on zone to zone travel for an origin-destination (O-D) matrix or by summing link travel times between major intersections. MPO model travel times may be produced for a variety of modes such as SOV, HOV, freight, and transit. Travel times for all modes are used as inputs into measures of accessibility. Model based travel time travel time information can also be classified by trip type (i.e., work-based trips). Travel times from a travel demand model are approximate and should only be used on a relative basis to compare alternatives/scenarios.

For corridor or facility level analysis travel time may be developed from operational models such as HERS and HCM methods.

Travel time can be used on a relative basis to evaluate emergency services by making assumptions about faster speeds for an emergency vehicle to travel a given O-D path under emergency conditions. Contributing factors include the ability of the emergency vehicle to move through congestion and traffic control devices and the provision of emergency pre-emption.

Average Delay

Performance measures

- Average delay per vehicle (sec/veh)

Example evaluation criteria

- Average delay for intersection
- Average delay for lane group

Delay is the additional vehicle travel time beyond the free-flow travel time for a given facility. Free-flow travel time is defined differently depending on the tools used. For reliability it could be based on empirically determined speeds, or posted speeds can be used in some HCM deterministic procedures. The analysis period is typically the peak 15-minute period of the design hour. In some instances free-flow conditions may be replaced by a designated acceptable target travel time or speed. Delay is typically calculated using HCM procedures, which also include Level of Service thresholds based on delay for many facility types.

Delay is also calculated by travel demand models and microsimulation methods. These delay outputs must be post-processed in order to compare with HCM delay values.

$$Delay = [Actual\ Travel\ Time - Threshold\ Travel\ Time] \times \frac{1\ hour}{60\ min}$$

Where

Delay = Delay for all vehicles the segment over the study period (hours)

Actual Travel time = (minutes)

For uninterrupted flow facilities, threshold travel time may be defined in different ways; free-flow travel time, travel time at posted speed limit, or a policy definition of congested speed (minutes). For interrupted flow, delay is computed by HCM methodologies and is the sum of segment delay and control delay. Control delay is delay occurring due to traffic control devices such as signalized intersections, roundabouts, or stop signs. Delay per vehicle does not account for the total number of vehicles being delayed which can result in underestimation of the impact, as compared to using vehicle hours of delay, which is generally a better performance measure.

Total Delay (Vehicle Hours of Delay)

Vehicle hours of delay is the delay per vehicle for a given segment, multiplied by the total number of vehicles in the study period, typically daily or annual. The delay per vehicle is based on the travel time minus the free-flow or threshold travel time. Unlike delay per vehicle, vehicle-hours of delay evaluates the total number of vehicles that are delayed.

Performance measures

- Daily vehicle hours of delay (veh-hr/day)
- Annual vehicle hours of delay (veh-hr/yr)

Example evaluation criteria

- Peak period vehicle hours of delay for segment
- Annual hours of delay per 1000 VMT

$$Vehicle\ Hours\ of\ Delay = \frac{[Average\ travel\ time - threshold\ travel\ time] \times traffic\ volume}{\frac{60min}{hour}}$$

Where

Delay = vehicle-hours of delay per given time period for the study segment

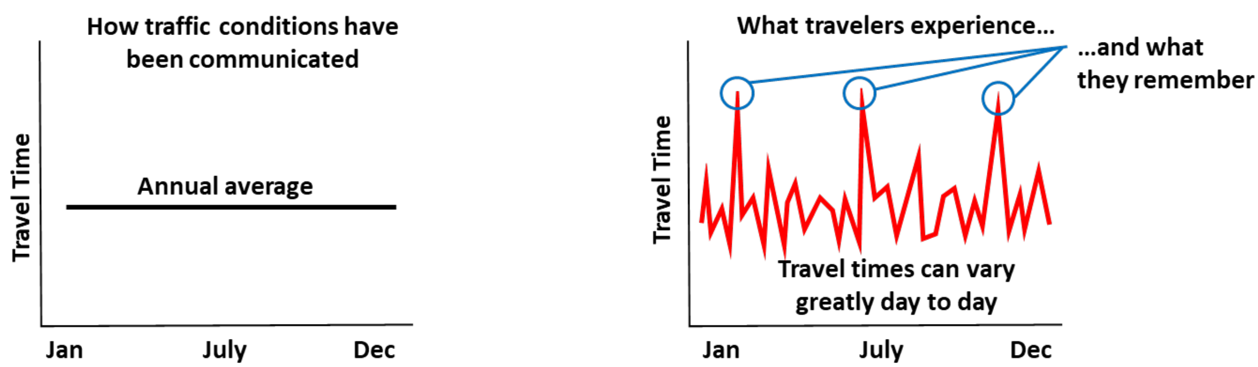
The threshold travel time is typically based on a threshold or target speed which could be free-flow speed, posted speed, or some other policy speed determined to be the minimum desirable operating speed. Several methods are available for estimating vehicle hours of delay, including the HCM, PPEAG, HERS-ST, and microsimulation. Facility VHD is obtained as the sum of the VHD of the individual segments. VHD is useful for evaluating an entire study area across multiple segments and is useful in sketch-level cost estimation.

In-vehicle person-hours of delay is VHD multiplied by the average vehicle occupancy.

9.3 Travel Time Reliability

Travel time reliability considers (1) the range of potential travel times roadway users may experience, (2) the consistency of travel times, and (3) the ability of a roadway to provide a desired travel time. Traditional measures of roadway operations, such as volume-to-capacity ratios or average travel speeds, reflect conditions during a design or analysis hour, such as the 30th-highest volume hour of the year. However, demand variation is just one of a number of factors that affect roadway operations. The effects of severe weather, incidents (e.g., stalls, debris), crashes, construction and maintenance activities, and special events (e.g., festivals, college football games) can all contribute to roadway operations that are different (and generally worse) than the average condition, as illustrated in Exhibit 9-4.

Exhibit 9-4 Difference Between Traditionally Reported and Actual Roadway Operations



Source: Derived from FHWA Travel Time Reliability website,
https://ops.fhwa.dot.gov/publications/tt_reliability/brochure/index.htm

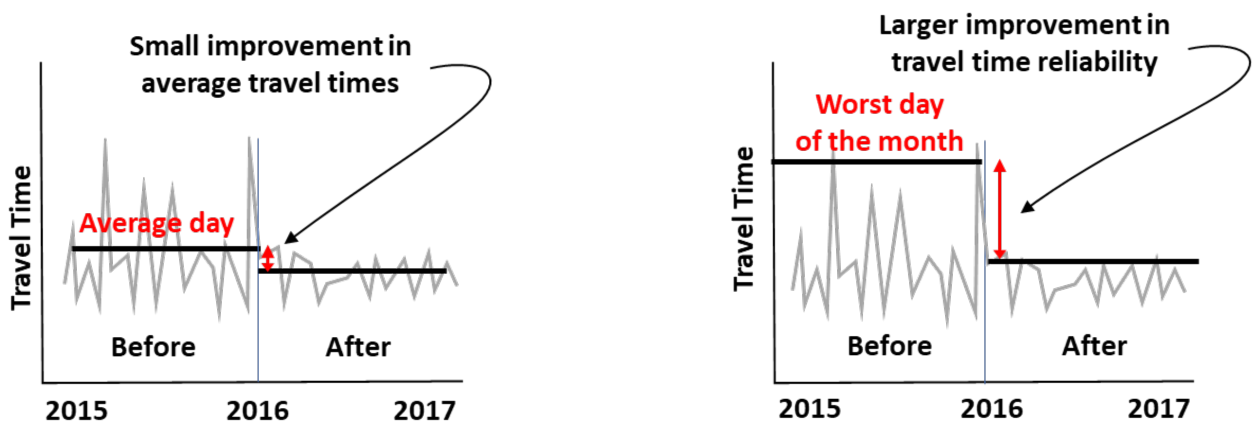
Measures of travel time reliability incorporate all of the factors influencing roadway capacity and free-flow speed to describe the variability of travel times along a particular roadway section or facility, or for a given trip. This variability affects roadway users in important ways, including:

- **Commuters,** who must plan extra time into their commute trip to avoid arriving late at work, even though they may not need that extra time most days;
- **Freight shippers,** who incur extra costs when shipments take longer to reach their destination, as well as their customers, whose supply chains may be disrupted by late deliveries; and
- **Transit operators,** who may need to add buses and drivers (at a significant added cost) to ensure frequent, reliable bus schedules that attract and retain customers.

Evaluating travel time reliability can also help roadway agencies better evaluate the effects of traffic operations strategies, such as ramp metering, dynamic part time shoulder use, and freeway service patrols. As illustrated in Exhibit 9-5, these strategies may produce relatively small effects on roadway capacity and average travel speed under normal conditions, but can have much greater effects on travel time reliability. For example, ramp metering may delay the onset of freeway breakdowns, or even reduce the number of days when freeway operations break down.

Traffic management centers and service patrols can help roadway agencies more quickly identify, respond to, and clear incidents, minimizing the effects of the incident both spatially and temporally.

Exhibit 9-5 Difference Between Traditionally Reported and Actual Roadway Operations



Source: Derived from FHWA Travel Time Reliability website, https://ops.fhwa.dot.gov/publications/tt_reliability/brochure/index.htm

The remainder of this section uses the term *reliability* as shorthand for travel time reliability. This section introduces methods of evaluating reliability, describes potential applications for a reliability analysis, and presents data sources and analysis tools currently available for evaluating and forecasting reliability. Methods for evaluating reliability on freeways and other uninterrupted-flow facilities are presented in APM Section 11.5.

9.3.1 Applications

Travel time reliability analysis has a number of potential applications, including:

- Performance reporting,
- Project planning, and
- Traffic management planning.

Performance Reporting

The federal MAP-21 and FAST Act transportation funding legislation requires states and metropolitan planning organizations (MPOs) to measure roadway performance. The FHWA's final rule implementing this legislation defines four reliability-related system performance measures as part of the set of National Performance Management Measures (*Federal Register,* Vol. 82, No. 11, January 18, 2017, 23 CFR Part 490). These performance measures can be evaluated using travel time data contained in the National Performance Management Research Data Set (NPMRDS) maintained by FHWA and made available to states and DOTs, or can be evaluated using an equivalent travel time dataset acceptable to FHWA.

Although performance management is outside the scope of this chapter, one potential analysis application is to forecast the contribution of project alternatives toward meeting roadway system performance targets. Furthermore, because the most difficult part of conducting a travel time reliability analysis is assembling a travel time dataset, the work that ODOT and MPOs invest in calculating FHWA's required performance measures can readily be extended to other applications, such as those described next.

Project Planning

Typical planning applications include problem identification, project evaluation, and project prioritization. The first of these requires (desirably) actual travel time data, while the latter two require both a reliability analysis model and actual travel time data for use in calibrating the model. Sources of travel time data are discussed in APM Section 9.3.5, while descriptions of currently available analysis models are provided in APM Section 9.3.6.

<u>Problem Identification</u>

In a typical problem identification application, reliability performance measures are evaluated for a defined roadway network (e.g., all freeways in a metropolitan area, all Interstate highways in Oregon). Roadway sections where the performance measure exceeds a threshold value, or alternatively, the worst *X*% of all roadway sections, are then flagged for further analysis to identify the cause(s) of the unreliability and, subsequently, potential projects or operational strategies to improve reliability. Any of the four primary travel time data sources available to ODOT (described later in Section 9.3.5) can be used to assemble a travel time dataset. Once this dataset has been created, the full range of reliability performance measures can be calculated from it. In addition, as described later in Section 9.3.6, planning-level estimates of some reliability performance measures can be developed without having a travel time dataset available. These planning methods require estimates of a roadway section's free-flow speed, average travel speed, and volume-to-capacity ratio.

> *ODOT has not yet set any targets or thresholds for reliability performance measures; doing so will require additional investigation and experience using these measures. The FHWA's National Performance Management Measures and the HCM's reliability rating (described below) are examples of measures with built-in threshold values for unreliable travel.*

<u>Project Evaluation</u>

Individual projects can be evaluated by comparing the values of one or more reliability performance measures with and without the project, following this general process:

1. Evaluate reliability performance for existing conditions using actual travel time data.
2. Calibrate a reliability-capable analysis tool, such as those described in Section 9.3.7, to replicate existing conditions.
3. Adjust the model parameters to reflect the project aspects that influence reliability.
4. Re-run the model to forecast future reliability performance with the project.

The additional effort required for the application is the effort to code the roadway facility in the analysis tool and to then calibrate the analysis output to reasonably match existing conditions. Once the existing facility is coded, it is relatively quick to forecast the reliability performance of a potential project in software. Exhibit 9-6 lists a variety of potential roadway capacity, modernization, and operations projects, along the factors influencing reliability that these projects affect.

Exhibit 9-6 Reliability Factors Influenced by Roadway Infrastructure Projects and Operations Strategies

Project Type	Reliability Factors Influenced
Add general-purpose (GP) lane	Capacity, the timing and amount of demand
Modernization	Free-flow speed, capacity
Ramp metering	Capacity, demand
Traffic management center	Incident detection and response times
Road patrols	Incident response and clearance times
Speed harmonization	Free-flow speed
Managed lanes	Capacity, demand in GP and managed lanes; benefits reflected in lowered person delay
Bus-on-shoulder	No bus volume in GP lanes; benefits reflected in lowered person delay
Part-time shoulder use	Capacity, possibly incident clearance times
Traveler information	Timing and location of demand
Traffic demand management	Timing and amount of demand

The effects of many operational strategies have not yet been well-quantified; therefore, it may be desirable to test a range of values for how a given strategy may affect reliability, to determine the sensitivity of the result to the assumptions used. Appendix B of the *IDAS User's Manual* (Cambridge Systematics and ITT Industries 2000), no longer supported but available in the Technical Reference Library section of HCM Volume 4 (http://hcmvolume4.org), provides default values for the effects of a number of operational strategies, although the information is somewhat dated at this point.

Project Prioritization

Once the effects of a given project or strategy have been forecasted, this information can be incorporated into a prioritization process, for example by considering both the magnitude of the reliability improvement and the number of vehicles or people that would benefit.

Traffic Management Planning

Reliability can also be incorporated into the development of various types of traffic management plans, such as:

- **Incident management planning—**forecasting the relative benefits of different strategies under consideration
- **Work zone planning—**identifying suitable work zone start and end times and number of lanes closed

- **Special event planning—**evaluating different event schedules, evaluating special traffic operations strategies for freeway off-ramps (e.g., temporary lane controls, signal timing adjustments, traffic control officers)

Considerations for Performing a Reliability Analysis

Evaluating reliability is most useful when a roadway facility operates, or is forecast to operate, over capacity on a regular basis, leading to highly variable travel times. In these cases, even if it is not financially or physically feasible to provide extra capacity through road widening, the effects of incremental improvements can still be evaluated in terms of reducing worst-case travel times, providing more consistent travel times, and/or reducing overall person delay.

For future-year forecasting, the additional effort required to conduct a reliability analysis using default values is minimal, once the facility has been coded and calibrated in an analysis tool that implements the HCM freeway facilities method. In other words, if a project would require a facility analysis using the core freeway facility methodology anyway, there is little reason not to go ahead and generate a set of reliability performance measures at the same time.

When forecasting the effects of project alternatives on a roadway's reliability, it is desirable to incorporate local reliability-related input values to the extent that the alternatives affect those inputs. For example, if an intersection improvement would be expected to affect the intersection's crash rate, using a local existing-conditions crash rate in lieu of a national default value is desirable. Similarly, when comparing and prioritizing potential projects on different roadways, it is desirable to account for differences in local traffic demand patterns. If the projects are located in different parts of the state with different climates, then using local weather data would also be desirable. Developing local input data for reliability methods is discussed in APM Chapter 11, Appendix 11F.

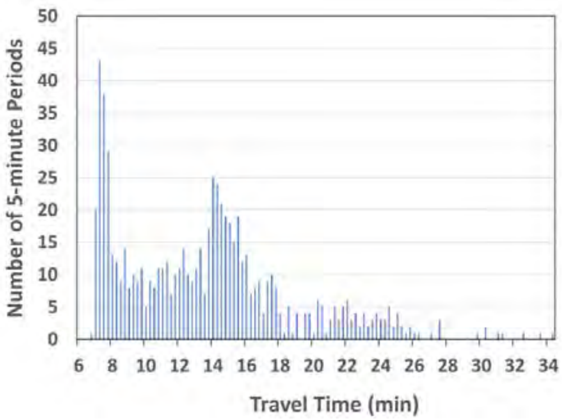
9.3.2 The Travel Time Distribution

A travel time distribution is a collection of travel time observations or forecasts for a defined roadway section (e.g., segment, facility) over a relatively long period of time (e.g., all nighttime time periods over the course of a year; all weekday time periods between 6:00 and 10:00 a.m.). Each observation represents the average travel time to traverse the road section during a defined time period, typically 5 or 15 minutes.

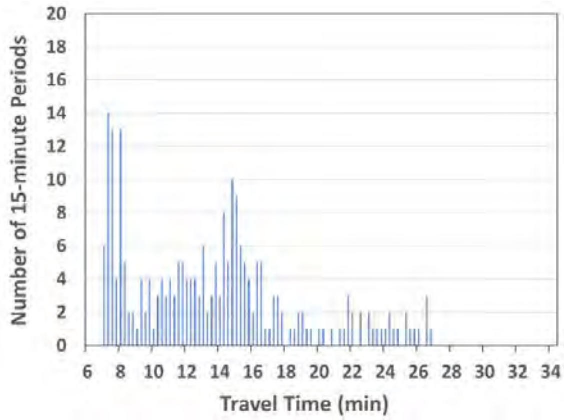
Once a travel time distribution has been created, nearly any reliability performance measure can be directly developed from it, except for certain measures where the measure's travel time reliability component is weighted by another variable (e.g., traffic volume, truck volume, person trips, regional population). The travel time distribution can be developed through direct observation of travel times (see Section 9.3.5) or by forecasting travel times using analysis tools (see Section 9.3.6).

Exhibit 9-7 illustrates travel time distributions developed for northbound I-5 in the Portland area between the Highway 217 and I-405 (south) interchanges, for all weekday a.m. peak (6:00 to 9:00 a.m.) time periods during February 2017. One distribution was developed using 5-minute data, while the other was developed using 15-minute data. The data were originally acquired for NCHRP Project 07-22 (*Planning and Preliminary Engineering Applications Guide to the Highway Capacity Manual*) from the PORTAL database maintained by Portland State University. Both of these graphs are frequency distributions: the *x*-axis represents travel times in 15-second bins, while the *y*-axis represents the number of travel time observations associated with each bin (i.e., the number of 5- or 15-minute time periods experiencing an average travel time within the 15-second range represented by the bin).

Exhibit 9-7 Examples of Travel Time Distributions Developed from 5- and 15-Minute Data



(a) 5-minute Data



(b) 15-minute Data

Both distributions show a peak on the left side, corresponding to free-flow or nearly free-flow conditions. Both also show a secondary peak in the left-center area of the distribution, corresponding to typical peak-period traffic congestion. Finally, both distributions have a long tail to the right, corresponding to conditions during severe weather (e.g., freezing rain) and/or when incidents occur (e.g., crashes, stalls, water on the roadway). The 15-minute distribution is more compact than the 5-minute distribution, as the extremely low speeds reported during individual 5-minute periods occur less often over a longer 15-minute period.

Fifteen-minute data are generally adequate for performing reliability analyses and have the following advantages over 5-minute data:

- One-third the amount of data must be manipulated
- Reduced quality-control effort, due to fewer time periods with missing data or outlier travel times
- Compatible with FHWA requirements for National Performance Management System reporting
- Compatible with HCM analysis output

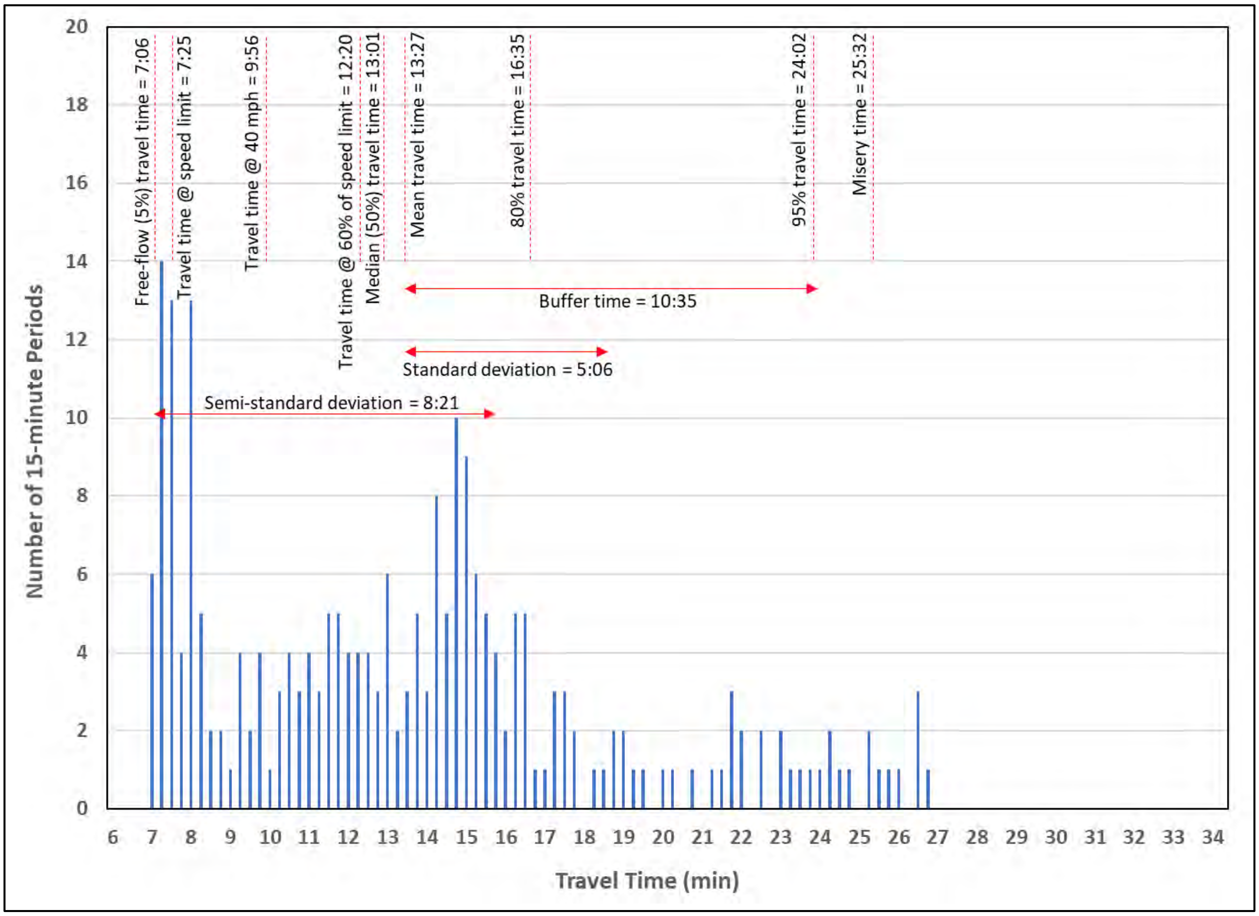
The greater detail provided by 5-minute data can be useful for diagnosing the causes of unreliability along a roadway. Diagnosing reliability problems is beyond the scope of this chapter, but is addressed in the HCM Planning Guide workshop material on performance management, available on HCM Volume 4, www.hcmvolume4.org.

9.3.3 Reliability Performance Measures

Identifying Key Travel Times from a Travel Time Distribution

The starting point for measuring reliability is identifying the travel times required to traverse a roadway section under specified conditions. These travel times can represent a fixed value (e.g., the travel time required to traverse the section at the posted speed limit), a percentile value (e.g., the 95th percentile highest travel time, a difference between two other travel times (e.g., the difference between the 50th and 95th percentile travel times), or statistical descriptors of the distribution such as the standard deviation. Exhibit 9-8 depicts common types of travel time values that can be obtained from the travel time distribution shown in Exhibit 9-7(b). These travel time values are described in the subsections that follow.

Exhibit 9-8 Examples of Travel Time Values Obtained from a Travel Time Distribution



Free-flow Travel Time

Free-flow travel time is the time required to travel a roadway section under low-volume conditions. It is preferably calculated as the average vehicle speed during low-volume periods (i.e., 500 pc/h/lane or less), with good weather and no work activity or incidents. Alternatively, when the study roadway is a freeway, multilane highway, or two-lane highway (i.e., uninterrupted flow without traffic signals), and the distribution clearly contains congestion-free periods, free-flow travel time can also be estimated as the 5th-percentile travel time, as shown in

Exhibit 9-8. Typically, free-flow travel time is not reported by itself, but is used instead to calculate other reliability measures, such as the *travel time index,* discussed later. *Highway Capacity Manual* (HCM) methods also calculate delay based on the difference between the actual travel time and the free-flow travel time.

Travel Time at the Speed Limit

The time required to travel a roadway section at the speed limit can be used as an alternative starting point for calculating delay, and as an input to reliability measures based on the percentage of time the roadway operates at or above a target percentage of the posted speed. This value can also be used as a check that the free-flow travel time estimate is accurate; the free-flow travel time will normally be slightly less (i.e., faster) than the travel time at the speed limit.

Target (Policy) Travel Time

This is the time required to travel a roadway section at a designated speed (e.g., free-flow speed, posted speed, speed producing maximum vehicle throughput, speed considered “congested”, speed at which greenhouse gas or particulate emissions significantly increase). It is typically not reported by itself, but is used in calculating other reliability measures, such as the policy travel time index and the percent of time or percent of travelers experiencing conditions where the target travel time is achieved.

ODOT uses the travel time at the posted speed limit rather than at the free-flow speed as the basis for calculating reliability performance measures for ODOT purposes. Different travel time targets may be required for FHWA reporting purposes.

Average (Mean) Travel Time

This is the average time to travel a roadway section during a given time period. HCM segment and facility methods predict average 15-minute travel times for a particular set of conditions.

Percentile Travel Time

A percentile travel time is the travel time over a roadway section achievable a given percentage of the time. Percentile travel times may be reported by themselves, but are also often used in calculating other reliability measures. The most common percentile travel times are:

- **50th-percentile (median) travel time—**this time typically will be slightly lower than the mean travel time, due to the influence of exceptionally long (outlier) travel times on the mean travel time;
- **80th-percentile travel time**—the travel time achievable 80% of the time; research has shown that the 80th-percentile time is more sensitive to roadway operational changes than the 95th-percentile time, making it useful for evaluating project effects on reliability; and
- **95th-percentile (planning) time—**for a segment or facility, the travel time achievable 95% of the time; for a trip, the travel time one would need to budget to ensure an on-time arrival 95% of the time (e.g., late to work approximately once a month when commuting).

Percentile Truck Travel Time

This measure, not depicted in Exhibit 9-8, is similar to *percentile travel time*, but is calculated from a distribution of truck travel times. These times may be different from overall vehicle travel times due to lower truck speed limits, severe roadway geometry (e.g., steep grades), presence of truck weigh stations, etc. FHWA's Freight Reliability performance reporting measure incorporates 50th- and 95th-percentile truck travel times.

Buffer Time

Buffer time is calculated as the 95th-percentile travel time minus the average travel time. It represents the extra amount of time a traveler would need to budget for a trip to ensure an on-time arrival 95% of the time.

Misery Time

Misery time is the average of the highest 5% of travel time observations in the distribution, approximating a reasonable worst-case condition.

Standard Deviation of Travel Times

This is a statistical measure of how much travel times may vary from the average travel time. The larger the standard deviation, the greater the variability of travel times from day to day along the roadway.

Semi-Standard Deviation of Travel Times

This is a statistical measure of how much travel times may vary from the free-flow travel time. It is calculated from the set of travel time observations slower than the free-flow speed as follows:

$$SSD = \sqrt{\frac{1}{n} \times \sum_{i=1}^{n} (FFTT - TT_i)^2}$$

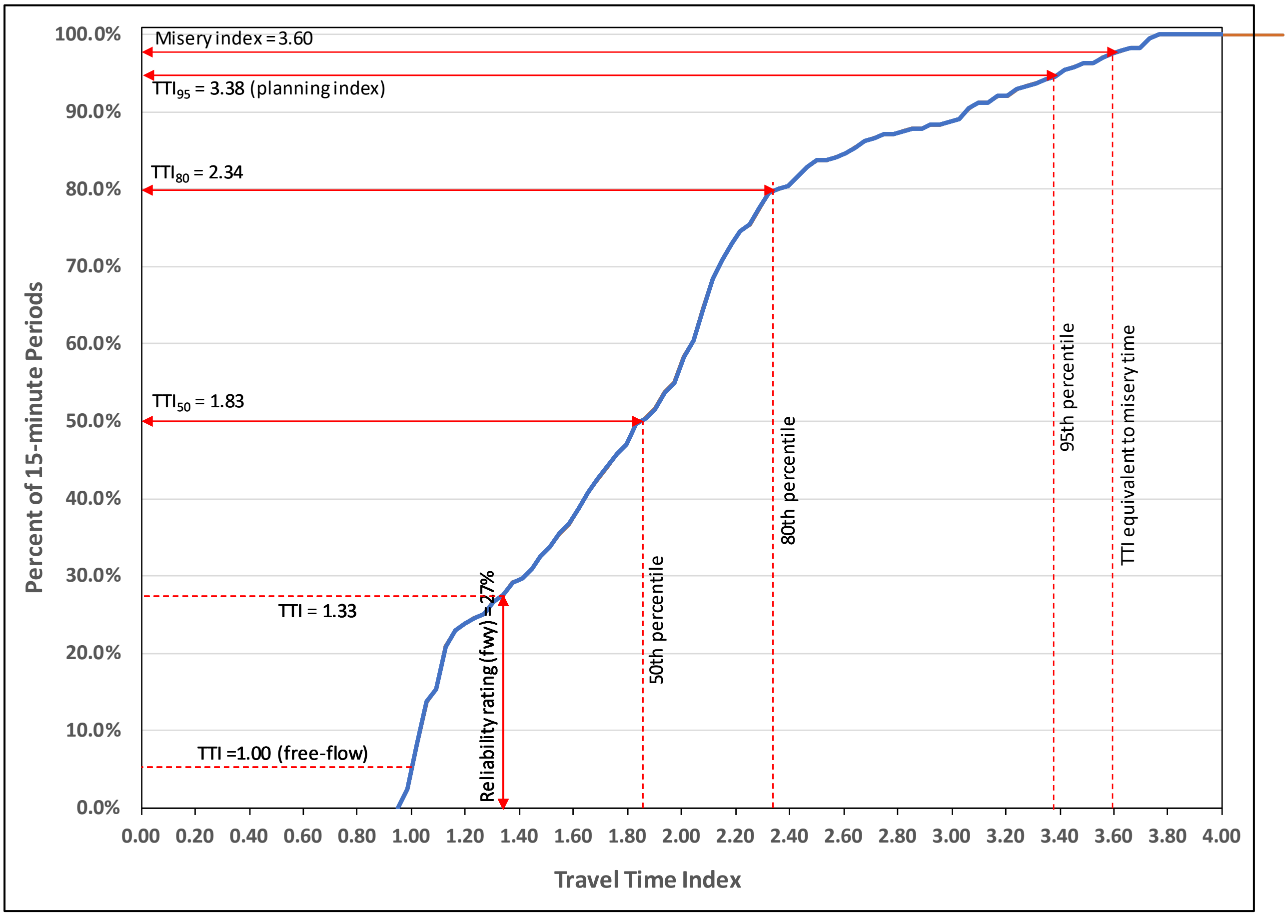
where

SSD = semi-standard deviation,
n = number of travel time observations slower than the free-flow speed,
$FFTT$ = free-flow travel time (s); and
TT_i = travel time observation i (s).

Ratio-Based Reliability Performance Measures

Travel time values are frequently used to create other measures of reliability. For example, one travel time value can be divided by another to create a ratio. When an observed travel time is divided by the free-flow travel time, the resulting ratio is known as a travel time index (TTI). The TTI indicates how much longer the observed travel time was, relative to the free-flow travel time. Exhibit 9-9 provides examples of ratio-based performance measures derived from the travel time distribution shown in Exhibit 9-7(b). The travel time distribution in Exhibit 9-9 is depicted as a cumulative distribution, with the x-axis containing TTI values and the y-axis showing the percentage of travel time observations occurring at or below a given TTI.

Exhibit 9-9 Examples of Ratio-Based Reliability Measures



Travel Time Index (TTI)

A TTI is calculated as a travel time divided by the free-flow travel time. A TTI value of 1.00 indicates travel at the free-flow speed, while a TTI value of 2.00 indicates travel that is twice as long, compared to free-flow conditions. Commonly reported TTIs include the 50th-percentile TTI (TTI_{50}, the 50th-percentile travel time divided by the free-flow travel time), the 80th-percentile TTI (TTI_{80}), the 95th-percentile TTI (TTI_{95}, also known as the planning index), and the mean (or average) TTI (TTI_{mean}, not pictured in Exhibit 9-9).

Policy Travel Time Index (TTI_P)

ODOT's policy TTI is calculated as a travel time divided by the travel time at the posted speed limit. A TTI_P value of 1.00 indicates travel at the posted speed, while a TTI_P value of 2.00 indicates travel that is twice as long as travel at the posted speed limit. Similar to the TTI, a variety of percentile values can be reported, including TTI_{P50} (the 50th-percentile travel time divided by the travel time at the posted speed limit), TTI_{P80}, and TTI_{P95}.

ODOT uses TTI_P instead of TTI for ODOT reporting purposes. Analysts should be aware that software packages may report TTI by default.

Level of Travel Time Reliability (LOTTR)

The LOTTR (not shown in Exhibit 9-9) is defined as the 80th-percentile travel time divided by the 50th-percentile travel time. The greater the LOTTR value, the longer travel times are on relatively poor (but not uncommon) travel days, compared to travel times on typical days. The FHWA incorporates LOTTR into its Interstate Travel Time Reliability measure, described below. The FHWA considers an LOTTR value less than 1.50 as indicating "reliable" conditions for reporting purposes.

Truck Travel Time Reliability (TTTR) Index

The TTTR Index (not shown in Exhibit 9-9) is defined as the 95th-percentile truck travel time divided by the 50th-percentile truck travel time. The FHWA uses this measure as the basis for the Freight Reliability component of its National Performance Management measures. For each roadway section, a TTTR Index is calculated for each of the following five reliability reporting periods:

1. All weekday a.m. peak periods (6 a.m. to 10 a.m.) during a calendar year
2. All weekday midday periods (10 a.m. to 4 p.m.) during a calendar year
3. All weekday p.m. peak periods (4 p.m. to 8 p.m.) during a calendar year
4. All weekend daytime periods (Saturday and Sunday, 6 a.m. to 8 p.m.) during a calendar year
5. All nighttime periods (Sunday through Saturday, 8 p.m. to 6 a.m.) during a calendar year

FHWA's Freight Reliability measure is calculated as the length-weighted average of the maximum of the five TTTR Index values for each Interstate roadway segment in the state.

Misery Index

The misery index indicates how much longer a reasonable-worst-case travel time is, relative to the free-flow travel time. It is computed as the misery time divided by the free-flow travel time.

Buffer Index

The buffer index (not shown in Exhibit 9-9) is the 95th-percentile travel time divided by the average travel time. Although this measure appears in the reliability literature, the HCM 6th Edition recommends against using it for tracking travel time trends "because it is linked to two factors that can change: average and 95th percentile travel times. If one factor changes more in relation to the other, counterintuitive results can appear." This same issue applies to other ratio-based measures incorporating the 50th-percentile or mean travel time, such as LOTTR and the TTTR Index.

Percentage-Based Reliability Measures

Percentage-based measures can indicate the percentage of time that a roadway operates at or better than a specified travel time or TTI. Percentage-based measures can also indicate the percent of people experiencing a specified condition (e.g., the percentage of people that were able to travel a roadway at 45 mph or faster).

On-time Percentage

The on-time percentage is the percentage of time periods when travel can occur at or above a specified target speed. *Failure rate* is a similar, but opposite, measure of the percentage of time periods when the target speed is not achieved. The target speed could be the posted speed, a percentage of the posted speed, the speed at capacity (i.e., the speed that maximizes throughput), or any other speed that makes sense for a particular analysis need. The *duration of congestion* (with “congestion” defined as travel slower than the target speed) is the study period length multiplied by the failure rate.

Reliability Rating

The HCM defines the reliability rating as the percentage of time periods where the TTI is no greater than a threshold value of 1.33 for freeways and 2.50 for urban streets. The threshold value represents the point at which facility operations typically break down; thus, the reliability rating approximates the percentage of time that a roadway operates below capacity.

Interstate Travel Time Reliability

The FHWA uses this measure, along with a companion National Highway System (NHS) Travel Time Reliability measure, as the reliability components of its System Performance measures for performance reporting. For each directional Interstate roadway section, a LOTTR is calculated for each of the following four reliability reporting periods:

1. All weekday a.m. peak periods (6 a.m. to 10 a.m.) during a calendar year
2. All weekday midday periods (10 a.m. to 4 p.m.) during a calendar year
3. All weekday p.m. peak periods (4 p.m. to 8 p.m.) during a calendar year
4. All weekend daytime periods (Saturday and Sunday, 6 a.m. to 8 p.m.) during a calendar year

Interstate Travel Time Reliability is then calculated as the length- and person trip–weighted percentage of Interstate roadway sections that have LOTTRs less than 1.50 during all four periods.

Delay-Based Reliability Measures

All measures of delay require defining a threshold where delay starts. Possible thresholds include:

- **Free-flow travel time—**the HCM uses free-flow travel time as the starting point for delay (i.e., any travel time slower than the free-flow travel time is considered to be delayed). This approach allows an apples-to-apples comparison of delay between roadways with different speed limits. However, this approach also allows delay to include travel times faster than the travel time at the posted speed, but less than the free-flow travel time, which may be inconsistent with both agency and traveler expectations.
- **Travel time at the posted speed—**any travel time slower than the travel time at the posted speed is considered to be delayed. This approach is probably the most consistent with traveler expectations for freeways and rural highways, but may not be particularly helpful in identifying or prioritizing problem areas, as higher-volume roadways with relatively high speeds will produce as much or more delay as lower-volume roadways

with lower speeds. ODOT uses the posted speed as the starting threshold for calculating delay.

- **Travel time at a target speed—**a minimum speed is specified as the target for satisfactory operations, and the portion of any travel time longer than the travel time at the target speed is considered to be delayed. For national performance reporting purposes, the FHWA defines a speed of 60% of the posted speed as the point where "excessive delay" begins.

Vehicle Hours of Delay

Delay is defined as the larger of (1) the actual travel time during a given time period minus the threshold travel time, or (2) zero. Vehicle hours of delay (VHD) is then:

$$VHD = \sum_i \frac{d_i \times V_i}{3{,}600}$$

where

VHD = vehicle hours of delay (veh-h),
d_i = delay during time period i (s),
V_i = volume during time period i (veh), and
3,600 = number of seconds in one hour (s/h).

Person Hours of Delay

Person hours of delay (PHD) is calculated similarly to VHD, but accounts for the vehicle occupancy of each mode using the roadway. Roadway operations strategies that reduce delay for higher-capacity modes (e.g., carpools, vanpools, transit) will show a greater percentage improvement in PHD than in VHD.

For roadways where all travel modes experience identical delays, PHD is calculated as:

$$PHD = \sum_i \frac{d_i \times V_i \times OF}{3{,}600}$$

where

PHD = person hours of delay (person-h),
OF = average vehicle occupancy (persons/veh), and
all other variables are as defined previously.

For roadways where different travel modes experience different delays (e.g., facilities with managed lanes or bus-on-shoulder operations), PHD is calculated as:

$$PHD = \sum_i \frac{\sum_m d_{i,m} \times V_{i,m} \times OF_m}{3{,}600}$$

where

$d_{i,m}$ = delay of mode m during time period i,
$V_{i,m}$ = volume of mode m during time period i,
OF_m = average vehicle occupancy of mode m (persons/veh), and
all other variables are as defined previously.

The Oregon default for average vehicle occupancy of private vehicles is 1.4 persons per vehicle, based on the 2009–2011 Oregon Household Activity Survey.

Recommended Reliability Performance Measures

The following performance measures provide a good starting point for evaluating reliability:

- **80th-percentile TTI_P—**this measure reports the upper limit of commonly occurring (e.g., once a week) travel conditions. This measure is more sensitive to roadway operations strategies such as ramp metering and road patrols than is the 95th-percentile TTI_P. This is because the longest travel times in the travel time distribution tend to be associated with major crashes and/or severe weather, both of which are less affected by operations strategies.
- **95th-percentile TTI_P—**this measure reports uncommonly poor, but not worst-case, conditions that roadway users would account for as part of their trip planning (e.g., a once-a-month occurrence on a commute trip). The planning time associated with this measure can be valued in terms of commuter time that could have been spent at home, extra freight shipment time that must be planned for, and longer transit trips that must be scheduled (possibly requiring additional vehicles and drivers). However, the use of an index rather than a pure travel time allows facilities with different lengths and different free-flow speeds to be compared on an apples-to-apples basis.

Additional reliability measures, such as TTI_{P50}, person delay, and reliability rating, can also be evaluated, depending on the specific needs of the analysis. For example, the FHWA national performance management measures would be forecasted if the purpose of the analysis was to investigate the potential contribution of different project alternatives toward meeting state or metropolitan system performance targets.

9.3.4 Reliability Reporting Periods

Reliability quantifies the uncertainty in travel times that a traveler might experience from day to day, across different times of day, over a period of time from a few months up to a year. Key reliability time periods are defined below.

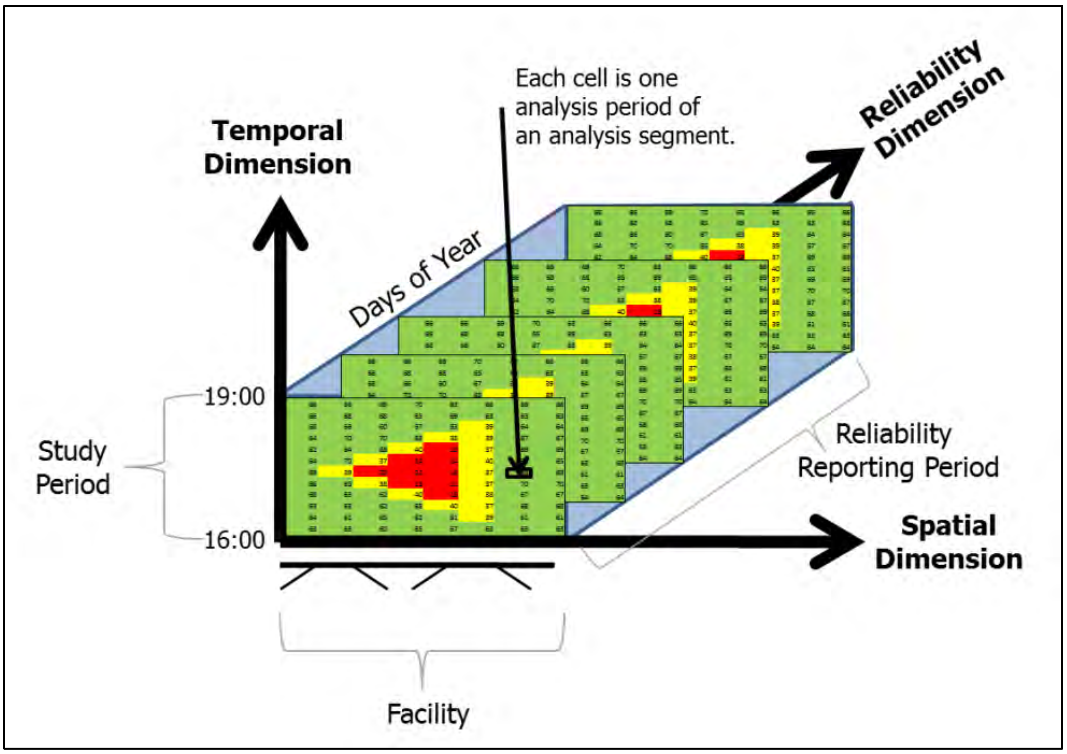
1. The *reliability analysis period* is the smallest time unit for which the analysis procedure is applied. In the case of freeway and urban street facility analysis, the analysis period is

typically 15 min, although it can be of greater or lesser duration, at the discretion of the analyst. Alternative tools may define different analysis period lengths.

2. The *study period* is the sum of the consecutive analysis periods for which the facility analysis procedure is applied (e.g., an a.m., midday, or p.m. peak period). The study period is defined by the analyst for each specific application. A study period of multiple hours is preferred, as a single congested peak hour could be very reliable but with poor travel times, while the shoulder hours could be much less reliable but with better travel times.
3. The *reliability reporting period* is the period over which reliability is to be estimated (e.g., the 250 non-holiday weekdays in a year). In essence, the reliability reporting period specifies the days within the year for which the reliability analysis is to be performed.

Exhibit 9-10 depicts these different time periods. The *y*-axis in the figure represents the time dimension on a given day, with each vertical cell representing one analysis period and the combination of all the individual analysis periods representing the length of the study period. The *x*-axis in the figure represents the facility's spatial dimension, with each horizontal cell representing one roadway section or HCM segment and the combination of all the individual sections or segments forming the length of the study facility.

Exhibit 9-10 Reliability Analysis Time Space Domain



Source: HCM 6th Edition, Exhibit 11-1

The combination of the study period length and the facility length should be large enough to contain all analysis periods and segments where demand exceeds capacity (for example, the red and yellow areas shown in Exhibit 9-10). If congestion spills out of the study facility length and/or the study period length, the analysis results will understate the facility's operational performance and reliability.

The study period length should be carefully scoped when the analysis is not intended to cover a full day. Including numerous time periods that rarely, if ever, experience congestion will tend to lower reliability performance measure values, which in turn may mask problems on the facility or fail to show much benefit from operational treatments.

The *z*-axis in Exhibit 9-10 introduces the reliability dimension. The facility analysis is repeated for each day of the year represented in the reliability reporting period, each of which experiences, to a greater or lesser degree, a different set of conditions. The travel times required to travel the facility during each analysis period of each day in the reliability reporting period are then aggregated into a travel time distribution.

The days to include in the reliability reporting period will depend on the type of facility being analyzed and the purpose of the analysis. For example, a study of the reliability of a major commute route within a metropolitan area might define a reliability reporting period of all non-holiday weekdays during the year. In contrast, a study of the reliability of a highway leading from the Willamette Valley to the Oregon Coast might define a reliability reporting period consisting of Saturdays, Sundays, and holidays during the summer.

9.3.5 Travel Time Data Sources

Travel time data are most commonly obtained from online databases of probe-vehicle speed data. The data are generated from commercial vehicle fleets and users of cell phone–based navigation systems, with the probe devices recording speeds that are reported to a central database. Through post-processing, speeds are attributed to a reporting segment called a TMC (Traffic Message Channel). On freeways, a TMC is typically defined from ramp gore to ramp gore. The recorded speed data are then converted to travel times across the TMC and stored in online archival databases.

ODOT has access to four primary sources of travel time data:

- Iteris Performance Measurement System (iPeMS),
- HERE Traffic Analytics,
- National Performance Management Research Data Set (NPMRDS), and
- Portland, Oregon Regional Transportation Archive Listing (PORTAL).

ODOT is subscribed to the Iteris Performance Measurement System (iPeMS), which is a web-based database for speed, travel time, and other data for Oregon roadways. The iPeMS system collects, filters, processes, aggregates and visualizes speed and travel time data derived from the probe data collected by HERE. Access to the Oregon iPeMS database is available at https://odot.iteris-pems.com; TPAU approval is required for access by non-ODOT staff. ODOT also has access to raw HERE travel time data, which can be downloaded for automobiles only, trucks only, or both automobiles and trucks. APM Section 18.2.3 provides more information about iPeMS and HERE data.

The NPMRDS dataset is provided by FHWA to MPOs and state DOTs without charge. The NPMRDS dataset is accessed through the RITIS (Regional Integrated Transportation Information System) online data portal (www.ritis.org) and can be downloaded for a region or corridor of interest using the *Massive Data Downloader.* The download is in the form of a CSV (comma separated value) file containing 5-minute sample summary data for the selected TMCs over the time period of interest. The NPMRDS further contains separate records for passenger cars and freight traffic. Additional information on the NPRMDS is available in APM Section 18.2.3 and through FHWA at http://www.ops.fhwa.dot.gov/freight/freight_analysis/perform_meas/vpds/npmrdsfaqs.htm. The NPMRDS dataset only covers the National Highway System.

PORTAL is the data archive for the Portland metropolitan region, which has been a collaborative effort between ODOT and Portland State University's Intelligent Transportation Systems (ITS) Laboratory, with additional data supplied by WSDOT, PBOT, TriMet, and Clark County, among others. PORTAL archives speed and count data from approximately 500 inductive loop detectors in the Portland metropolitan region dating back to July 2004. PORTAL has a web-based interface that provides performance metrics designed to assist practitioners and researchers. More information on the PORTAL system can be found on the Portland State University website at http://portal.its.pdx.edu and in APM Section 18.2.4.

In addition to these online databases, travel time reliability data can be obtained from other devices that allow longitudinal measurements of speeds and travel times. For example, Bluetooth or Wi-Fi readers can be used to monitor individual vehicle travel times over extended periods of time. These raw travel time data can be aggregated to derive a travel time reliability distribution.

9.3.6 Methods for Forecasting Reliability

Differences Between Reporting Existing Reliability and Forecasting Future Reliability

Existing travel time reliability is normally determined from actual travel time data from one of the sources described in the previous section. Reliability performance measures derived from these travel time distributions thus *report* the actual conditions that occurred during the period of time covered by the data.

In the absence of travel time data, it is also possible to *forecast* various measures of travel time reliability using one of the analytical methods described in this section. Analytical methods will tend to predict somewhat worse reliability performance than would typically occur in any given time period (e.g., 1 year). This is because analytical methods account for very rare events (e.g., unusually severe weather) that have very large travel time impacts. These events may not occur in any given reporting year, and therefore are not necessarily used in planning decisions, but nevertheless are the events that "travelers remember," as was highlighted in Exhibit 9-4.

When reporting travel time reliability, the majority of the effort involves manipulating the travel time data and (potentially) matching the data to information from other databases, such as traffic volumes. Some travel time data sources provide an analysis tool that performs this data manipulation and analysis, while other sources provide only the raw travel time data, which analysts must manipulate themselves.

When performing a detailed forecast of travel time reliability, the majority of the effort involves coding and calibrating the facility in the analysis tool. The analysis tool then takes care of creating various reliability scenarios, generating the travel time database, and reporting reliability performance.

Categories of Reliability Forecasting Methods

Reliability forecasting methods can be divided into three main groups: (1) sketch-planning methods developed through the SHRP 2 program, (2) the detailed HCM freeway and urban streets reliability methods, and (3) Oregon's implementation of HERS-ST, which incorporates elements of the other two methods.

Although in theory microsimulation can also be used to estimate reliability, it is not currently practical to do so in a way that addresses the multitude of potential scenarios the way the HCM or HERS-ST can, because of the time required to develop, code, run, and analyze the many different reliability scenarios that would be required to accurately estimate reliability. For example, the HCM method allows random variation in the location, severity, and time of day of incidents; severity and start time of severe weather events; and so on. HCM-implementing software can evaluate hundreds of scenarios for a facility covering up to 24 hours a day for an entire year in a matter of seconds. In contrast, FHWA's pilot tests of evaluating reliability using simulation used only 8 or 9 scenarios (combinations of demand and incidents) in two cities to represent relatively common peak-period conditions, and without consideration of weather effects. Such an approach may be sufficient to demonstrate some benefit from traffic management strategies, but not to forecast future reliability.

SHRP 2 Sketch-Planning Methods

The SHRP 2 program developed planning-level methods for estimating selected travel time reliability measures. Unlike reporting methods and the detailed HCM method, these methods do not assemble a travel time distribution. Instead, they use equations to estimate what a roadway's reliability performance would be, using a minimum number of inputs: free-flow speed, volume-to-capacity ratio, and number of lanes. These equations were developed from research into the reliability performance of a variety of roadways in different parts of the U.S.

SHRP 2 Project C11 Method

This method estimates delay due to recurring and nonrecurring congestion using just two inputs: volume-to-capacity ratio and facility type (freeway, arterial, collector, ramp, local road). Facility type is used as a proxy for free-flow speed. Predictive equations are then used to estimate common reliability performance measures. The method is capable of forecasting reliability impacts and costs for individual projects, and can be applied to any roadway type.

Roadway segments are the basic unit of analysis. Segments can be of any length, but it is recommended that they not be so long that their characteristics change dramatically along their length. Reasonable segment lengths would be:

- Freeways: between interchanges;
- Signalized highways: between signals; and
- Rural highways (non-freeways): 2–5 miles.

The method first estimates the mean TTI. The mean TTI then becomes an input to other predictive equations for estimating:

- Recurring delay (hours)
- Incident delay (hours)
- Total delay (hours)
- 95th-percentile TTI
- 80th-percentile TTI
- 50th-percentile TTI
- Percent of trips < 45 mph
- Percent of trips < 30 mph
- Cost of recurring delay
- Cost of unreliability
- Total congestion cost

The reported reliability values apply to a single weekday analysis hour (the hour used in calculating the volume-to-capacity ratio supplied to the method) over the course of a year. The results from multiple calculations can be combined and weighted to produce reliability values for longer weekday study periods.

HCM Planning Guide Method

The *Planning and Preliminary Engineering Applications Guide (PPEAG) to the HCM* presents a method for estimating freeway reliability. It is based on the SHRP 2 C11 method, but allows specific roadway characteristics to be used to estimate the free-flow speed, and it simplifies the calculation of incident-related delay. Because the HCM does not currently provide reliability methods for multilane and two-lane highways, the PPEAG limits itself to forecasting freeway reliability. However, because the underlying SHRP 2 C11 equations can be applied to any facility type, the PPEAG method can also be applied to any facility type.

Required inputs to the method are:

- **Free-flow speed:** Estimated using Appendix 11A of APM Chapter 11.
- **Analysis-hour speed:** Estimated using the screening method in APM Section 11.3.5 (for freeways and multilane highways) or from the appropriate PPEAG method for other roadway types.
- **Number of directional lanes:** 2 to 4 (if less than 2 lanes, use 2; if more than 4 lanes, use 4)
- **Volume-to-capacity ratio:** Estimated using APM Chapter 11 screening-level methods.

The method predicts the same performance measures described above for the SHRP 2 C11 method. The reliability reporting period is also the same: one or more weekday analysis hours over an entire year.

Oregon HERS-ST Method

The HERS-ST software does not directly calculate reliability performance measures. However, ODOT has used HERS-ST to generate the inputs required for the SHRP 2 C-11 mean TTI equation, namely: free-flow speed, recurring delay rate, and incident delay rate. Once the mean TTI has been determined, all of the other performance measures described above for the SHRP 2 C11 method can also be predicted.

ODOT has also demonstrated the application of HERS-ST for developing reliability scenarios combining a variety of severe weather, incident, and work zone events. Appropriate demand and capacity, and free-flow speed adjustments for a given scenario are made in HERS-ST before re-running the model. The individual scenario results are then weighted by their probability of occurrence when calculating an overall performance measure result. Because HERS-ST results apply to individual roadway sections, they may not fully reflect the delay associated with queue spillback from one section into other upstream sections.

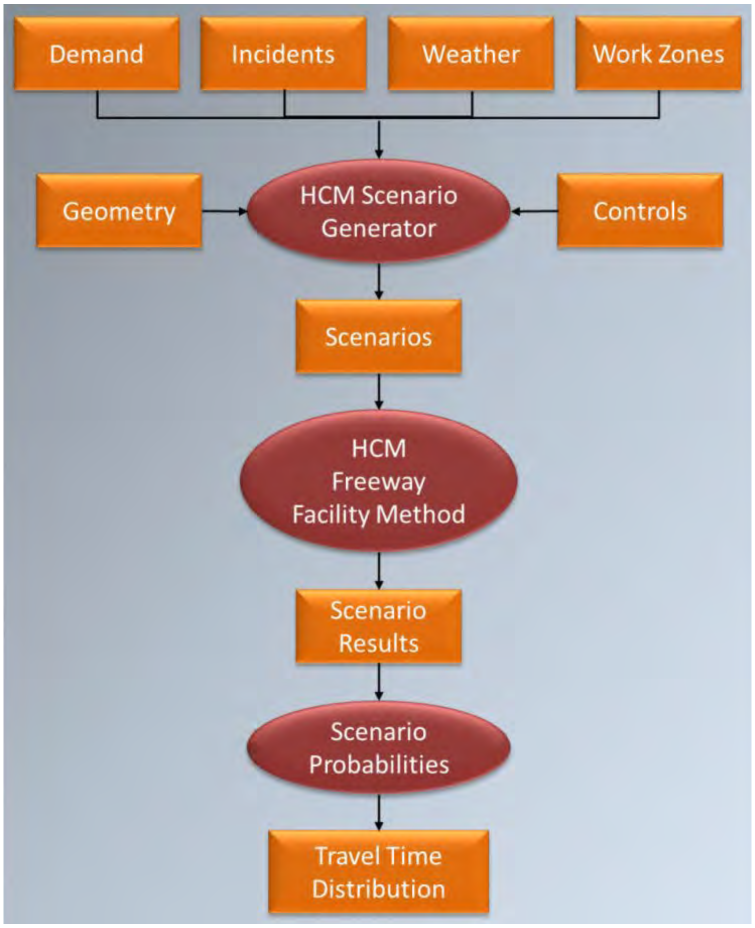
The HERS-ST method can be applied to any roadway type, for a reliability reporting period consisting of the weekday peak hour over an entire year.

HCM Freeway Reliability Method

The HCM freeway reliability analysis methods are described in Chapters 11 and 25 of the HCM 6th Edition. A reliability analysis starts by coding a base scenario for the facility, consisting of all the data normally entered for an HCM operations analysis using the HCM's core freeway facility methodology (described in APM Chapter 11). The HCM reliability method then creates a series of scenarios representing various combinations of demand, severe weather, incidents, work zones, and special events, along with a probability of occurrence for each scenario. Each reliability scenario adjusts the base scenario's demand, capacity, and/or free-flow speed in some way, resulting in a different set of performance results (e.g., travel times) for each scenario. Finally, a travel-time distribution is generated based on the weighted probability of each scenario occurring.

Exhibit 9-11 illustrates the method's flow from scenario generation through outputting a distribution of travel times.

Exhibit 9-11. Flowchart of the HCM Freeway Reliability Analysis Method



The HCM provides national default values for incident probabilities and durations by incident severity, and demand variations by day of week and month of year. It also provides probabilities of 10 categories severe weather by month for the 101 largest metropolitan areas around the U.S. (Portland is the only Oregon metropolitan area represented in the HCM's default weather data) The analyst can choose to replace any or all of the default values with local values, and can also optionally provide data regarding long-term work zones and special events that significantly alter traffic demand and/or traffic operations strategies.

The analyst must supply the following: the day of year represented by the base scenario's traffic volume (so that each scenario's demand adjustment can be applied relative to that day), the study period length coded in the base scenario (e.g., 6–10 a.m.), and the days to include in the reliability reporting period.

The HCM does not provide much guidance on time periods to include in a reliability analysis, other than to state that reliability reporting periods spanning one year are most common and that the study period length should be long enough to allow queues to dissipate by the end of the study period. The choice of days to include in the reliability reporting period will depend in part on the use of the facility—a commuter route might analyze weekdays, while a recreational route might analyze weekends. For national reporting purposes, the FHWA defines four study periods for the Interstate Travel Time Reliability measure (weekday a.m. peak, weekday midday, weekday p.m. peak, weekend daytime) and five for the Truck Travel Time Reliability Index (the four listed above, plus nighttime periods).

Method Comparison

The reliability forecasting methods discussed above vary in the following respects:

- Input data requirements
- Ability to be adapted to local conditions
- Number of scenarios used to model travel time variability
- Facility types covered
- Types of events modeled that influence reliability

All of the methods have tools available to assist in applying the method. Exhibit 9-12 compares the capabilities of the different methods.

Exhibit 9-12. Comparison of Travel Time Reliability Analysis Methods

	SHRP 2 C11	PPEAG	Oregon HERS-ST	Simulation	HCM
Scenarios used	1	1	1/100s*	≤10	100s to 1,000s
Scenario generation process	NA	NA	NA/Manual*	Manual	Automated
Facility types covered	All	Freeways (extendable to all)	All	All	Freeways, urban streets
Required inputs	FFS, v/c, # lanes	FFS, v/c, # lanes, average speed	Obtained from HPMS	All required by simulation tool	All required for freeway facility analysis
Local adjustment capability	No	Values used to generate input data	Scenario generation	Inputs, scenario generation	Inputs, scenario generation
Reliability measures output	Most common	Most common	Most common/ any*	Any	Any
Creates travel time distribution	No	No	No/Yes*	Creates sub-distributions for each scenario	Yes
Reliability reporting period	Single analysis hour for all weekdays in one year**	1–24 analysis hours for all weekdays in one year	Weekday peak hour for one year	Typically, 1+ analysis hours for all weekdays in one year	Any, up to one year
Models weather impacts	No	No	No/Yes*	No	Yes
Models incident impacts	Indirectly	Indirectly	Indirectly/Yes*	If included as scenarios	Yes
Models work zone impacts	No	No	No/Yes*	If included as scenarios	Yes

Notes: NA = not applicable, FFS = free-flow speed, v/c = volume-to-capacity ratio.
*In a batch-processing application using multiple scenarios.
**Calculations can be repeated for additional weekday analysis hours if desired.

The number of scenarios used by a method affects (1) the variety of conditions analyzed that can impact roadway operations and (2) the ability to incorporate local conditions into the analysis.

Sketch-planning methods produce a single estimate of reliability, based on regression equations developed from nationally representative travel time datasets. These methods do not account for the effects of local weather conditions, differences in incident frequencies or detection and clearance times, or other local factors. The base SHRP 2 C11 method is also insensitive to differences in roadway characteristics that would affect the roadway's FFS or capacity and thus the reliability result. In contrast, the PPEAG method can account for these differences.

As the number of reliability scenarios increases, the greater the analyst's ability to account for the various factors affecting reliability, but also the greater the effort required—either up front or for each analysis—to develop the scenarios. Even the method using the greatest number of scenarios, the HCM, places constraints on the types of situations considered in order to reduce analysis complexity. (For example, the HCM limits consideration of weather to weather events that decrease capacity by at least 4%.) Similarly, FHWA's pilot tests of simulation used just 8 or 9 scenarios, because of the effort required to develop individual simulation models for each scenario, along with determining the probability of each scenario. Although multiple simulation runs can be performed for each scenario and the results compiled into a travel time distribution, what the analyst ends up with is a single mean travel time for each scenario, each with its own distribution around the scenario mean. This collection of sub-distributions does not match the full spectrum of travel time observations that would be measured in the field.

HERS-ST offers the option of producing a single estimate of travel time reliability, using the SHRP 2 C11 equations, or using its batch-processing feature to generate a true travel time distribution from a series of reliability scenarios. For example, an analysis of a section of US 97 between Sunriver and LaPine incorporated 8 demand levels and 89 capacity-reducing events (combinations of severe weather, incidents, and/or work zones) were included, for a total of 712 reliability scenarios. Capacity reductions for each event were derived from the default values given in the HCM 6th Edition. The probabilities of each demand level and capacity-reducing event occurring were also determined and used to weight the scenario's resulting travel time.

The scenario-generation approach taken by the HCM is different than that used by simulation or HERS-ST. Rather than rely on the analyst to define scenarios and decide which ones to include or exclude, the analyst provides information on demand variability by day of week and month of year, the probabilities of various types of severe weather by month, and probabilities of various types of incidents. This information can come from the HCM's national defaults, from a one-time effort to create local or regional defaults, or from location-specific data. The analyst also specifies the number of replications of each day–month demand combination; the HCM suggests 4 for a reliability reporting period spanning one year, corresponding to each day being modeled approximately four times in a given month. If a shorter reliability reporting period is used, the HCM recommends increasing the number of replications so the total number of scenarios (replications × months × days) generated is at least 240. The HCM method then randomly assigns weather and incident events (or non-events) to each scenario, along with random start times for each event and (for incidents) random locations. This process recognizes, for example, that heavy rain that occurs in the middle of the night will have a different impact on roadway operations than a downpour in the middle of rush hour. The process also allows rarer events to be considered as part of the overall analysis, without needing to arbitrarily decide which events to include or exclude—it may not snow in Portland every winter, but ODOT prepares for the possibility of snow anyway because of its severe impacts on roadway operations.

9.3.7 Tools for Forecasting Reliability

This section introduces software tools available to predict travel time reliability for freeways and uninterrupted flow facilities. Three of the tools implement the HCM 6th Edition method, while the other three implement versions of the SHRP 2 C11 planning-level equations.

FREEVAL

FREEVAL is the official computational engine of the HCM 6th Edition freeway facilities and freeway reliability chapters. It can be downloaded for free on the HCM Volume 4 website (http://www.HCMVolume4.org). A FREEVAL reliability analysis builds on a calibrated and completed freeway facilities analysis (described in APM Chapter 11), and then adds the reliability dimension. FREEVAL applies user input or national defaults for incident distributions and day-of-week and month-of-year demand variability, along with historical weather data and user-specified work zone inputs. FREEVAL further integrates the HCM 6th Edition method on Active Travel and Demand Management (ATDM) with methods for evaluating impacts of traffic system management and operations strategies such as ramp metering, part-time shoulder use, and managed lanes.

Highway Capacity Software (HCS)

HCS is commercial software for the Windows operating system that is developed, distributed, and supported by the McTrans Center at the University of Florida. Similar to the process used by FREEVAL, HCS builds from a calibrated freeway facilities analysis by adding the HCM's reliability method and (optionally) the HCM's ATDM method. Users can apply the national default values for demand variability, weather patterns, and incidents, or supply their own local values. Users supply facility-specific work zone information.

TTR/ATDM

TTR/ATDM is a reliability analysis tool based on the HCM 6th Edition developed by SwashWare and the University of Florida Research Foundation. The tool is an extension of the HCM Calc tool for freeway facility analysis (described in APM Section 11.2.5). TTR/ATDM implements the HCM reliability and ATDM methodologies, similar to what was described for FREEVAL above. The tool can be downloaded for free through the Microsoft store.

PPEAG

The PPEAG's freeway computational engine, available on HCM Volume 4, can be used to estimate a freeway segment or facility's volume-to-capacity ratio and average speed, given a user-provided free-flow speed and number of directional lanes. These results can then be used with the PPEAG reliability equations (either manually or by setting up a simple spreadsheet) to estimate any of the reliability performance measures supported by the SHRP 2 C11 method.

HERS-ST

HERS-ST can estimate any roadway section's free-flow speed directly, while HERS-ST output can be used to develop the section's recurring delay rate and incident delay rate. These results can then be used to estimate (either manually or by setting up a simple spreadsheet) any of the reliability performance measures supported by the SHRP 2 C11 method. ODOT has demonstrated the ability to model weather and work zone scenarios with HERS-ST to develop estimates of travel time reliability for non-freeway roadways and corridors containing a mix of facility types. See APM Section 7.3 for more information about HERS-ST.

SHRP 2 C11 Spreadsheet Tool

The C11 reliability spreadsheet tool is an Excel spreadsheet that calculates all of the reliability measures supported by the C11 method. The spreadsheet also calculates the value of reliability improvements based on the following assumptions, which can be changed within the spreadsheet:

1. For passenger travel, it assumes a $19.86/hour average value of time multiplied by a 0.8 reliability ratio (i.e., hours of delay are multiplied by 0.8 when calculating the value of changes in reliability),
2. For commercial travel, it assumes a $36.05/hour average value of time multiplied by a 1.1 reliability ratio.

For ODOT projects, values of travel time should be consistent with the most recent version of "The Value of Travel Time: Estimates of the Hourly Value of Time for Vehicles in Oregon," available at https://www.oregon.gov/ODOT/Data/Pages/Economic-Reports.aspx. APM Section 10.6.8 provides more information about economic analysis.

Tool Comparison

Exhibit 9-13 summarizes key features of software tools that implement the HCM's reliability method. Exhibit 9-14 provides similar information for tools that are based on the planning-level SHRP 2 C11 reliability equations.

Exhibit 9-13 HCM-Implementing Tool Comparison

Overview	HCS	FREEVAL	TTR/ATDM
	Tool Overview		
Source	McTrans	hcmvolume4.org	University of Florida
Cost	License Fee	Free	Free
Operating system	Windows	Windows/Mac	Windows 10
Installation required	Yes	No (need Java)	Yes
Widespread use	High	Medium	Low
	Staff and Support Needs		
Learning curve	Medium	Medium	Medium
Complexity	Medium	Medium	Medium
Training available	●	◐	○
User guide	●	●	●
Instructional videos	○	●	○
Technical support	●	◐	◐
	User Experience		
Copy/paste	○	◐	◐
Load/save	●	●	●
Import/export	●	●	○
Auto-fill	◐	●	○
	Specialized Features		
Charts/visualizations (reliability)	●	●	●
Charts/visualizations (ATDM)	○	●	○
Automated report generation	●	○	◐
Built-in scenario comparison	○	●	○
Calibration (adjustment factors)	●	●	●
Built-in weather adjustments	●	●	◐
Incident scenario analysis	●	◐	◐
Work zone scenario analysis	●	◐	◐
ATDM method	○	●	○
Ramp metering	○	◐	○

Notes: ● = fully supported, ◐ = partially supported, ○ = not supported.

Exhibit 9-14 SHRP 2 C11 Implementing Tool Comparison

Overview	SHRP 2 C11 Reliability Tool	PPEAG Tool	ODOT HERS-ST
Tool Overview			
Source	tpics.us/tools	hcmvolume4.org	ODOT
Cost	Free	Free	Free
Operating system	Windows/Mac	Windows/Mac	Windows
Installation required	No (need Excel)	No (need Excel)	Yes
Widespread use	Low	Low	Low
Data source for reliability inputs	Defaults or another tool	Calculated	Imported from HPMS
Reliability calculations	Automated	Manual or separate spreadsheet	Manual or separate spreadsheet
Staff and Support Needs			
Learning curve	Low	Low	Medium
Complexity	Low	Medium	Medium
Training available	○	○	
User guide	●	◐	
Instructional videos	○	○	○
Technical support	○	○	
Specialized Features			
Congestion cost estimates	●	○	○

Notes: ● = fully supported, ◐ = partially supported, ○ = not supported.

9.4 Level of Service (LOS)

Level of Service (LOS) and quality of service (QOS) are indicators that cannot be measured directly in the field and are a letter grade based on an underlying performance measure value.

9.4.1 Motorized Vehicle Level of Service

Motorized vehicle Level of Service is a commonly used performance measure computed following Highway Capacity Manual methodologies. It is a rating of the level of mobility (typically as a function of delay or density) of a facility, segment, intersection or approach on a scale of A-F. LOS A, B, and C indicate conditions where traffic moves without significant delays over periods of peak hour travel demand. LOS D and E are progressively worse operating conditions. LOS F represents conditions where average vehicle delay has become excessive and demand has exceeded capacity. This condition is typically evident in long queues and delays.

Performance measure

- Level of Service letter grade A-F

Example evaluation criteria

- Unsignalized intersection approach LOS compared to local jurisdiction LOS standard
- Freeway segment LOS

Motorized vehicle LOS is determined for the following facility types using the following quantitative measures (all specified in the HCM):

- Freeway segments, facilities, merge, diverge and weaving segments
  - Density – specifically, average number of vehicles per lane mile (pc/mi/ln). LOS F where demand exceeds the capacity of the segment.
- Two-lane highway segments
  - Density – specifically, follower density (veh/mi/ln) of directional segment (refer to APM v2 Addendum 11B).
- Intersections – signalized, unsignalized, by approach, lane group or intersection as a whole
  - Delay – specifically, average delay (sec/veh) (by approach, lane group or intersection as a whole).
- Urban Streets – segment or facility
  - Speed (mi/hr)

Many local jurisdictions have adopted LOS as a performance measure for facilities under their jurisdiction and have adopted LOS thresholds as standards. The analyst needs to evaluate LOS and compare to the adopted local standards when analyzing those facilities. Some jurisdictions have dual performance thresholds for both v/c ratio and LOS in general or by facility type. Reporting LOS for state highways is optional, although reporting LOS on state highways is best practice to obtain a complete picture of operations versus reporting v/c ratio alone. Facilities with low v/c ratio could still have high delays. Refer to the HCM 6th Edition for specific calculations and LOS thresholds for each facility type.

- Basic freeway segments – Chapter 12
- Two lane highways – Chapter 15
- Signalized intersections – Chapter 19
- Unsignalized intersections – Chapters 20-22

9.4.2 Multimodal Level of Service (MMLOS)

MMLOS is a Quality of Service (QOS) measure. QOS measures the perceived level of comfort by the user, which could be a pedestrian, a bicyclist, or a transit rider. While vehicular LOS includes factors for the effects of pedestrians on vehicular mobility, pedestrian/bicycle/transit LOS reflects the point of view of the pedestrian, bicyclist or transit rider. The methodology creates a score which is equated to a Level of Service rating. Refer to APM Chapter 14 for procedures.

A qualitative multimodal methodology is also available as an alternative to the full HCM MMLOS method. It uses the same data categories as the HCM method, but is a qualitative assessment which can be used where HCM methods do not apply or where data are not available.

Pedestrian and Bicycle Level of Service

The APM methodologies to calculate Pedestrian and Bicycle LOS are simplified versions of the HCM Pedestrian and Bicycle LOS. Refer to APM Chapter 14 for detailed procedures.

The APM Pedestrian and Bicycle LOS are based on user perception scores of the level of comfort a user would experience on a given facility. Performance ratings for pedestrians are provided for roadways with and without sidewalks and multi-use paths. PLOS evaluates sidewalk width, posted speed, number of through traffic lanes and vehicle traffic volume. Additional performance measure methods are under development for midblock crossings, signalized intersections and unsignalized intersections.

Performance measure

- Level of Service letter grade A-F
- Qualitative MMLOS (good/fair/poor)

Example evaluation criteria

- Pedestrian facility LOS
- Bicycle facility LOS
- Multi-use facility LOS
- Signalized intersection ped or bike LOS (TBD)
- Unsignalized intersection LOS (TBD)

Performance ratings for bicyclists are provided for roadways with and without bike facilities, separated paths, and intersections. Bike facilities evaluated include shared lanes, bike lanes, buffered bike lanes, protected bikeways, and bike signals. Bicycle LOS evaluates the number of through travel lanes, presence of bike lane or paved shoulder, posted speed and number of unsignalized intersections and driveways.

Pedestrian and bicycle LOS can be used to evaluate walk and bike networks such as for a TSP to identify needs, as well as to evaluate alternatives affecting sidewalk width, bike facility type, volumes, lanes, posted speeds, and driveways.

Capacity-based HCM pedestrian performance measures evaluate the utilization of available space. These measures are generally not used in Oregon due to the lack of pedestrian density.

Transit Level of Service

> *The APM methodology to calculate Transit LOS is a streamlined version of the HCM Transit LOS. Refer to APM Chapter 14 for detailed procedures.*

The APM Transit LOS is based on user perception scores of transit service on a segment. Transit LOS relates to passengers' perception of walking to a transit stop on the street, waiting for the transit vehicle, and riding on the transit vehicle. The method applies to buses, street cars, and other types of transit vehicles operating with mixed traffic on the roadway. The measure does not apply to transit operating in separated right-of-way. Transit LOS is a function of transit schedule speed, transit frequency and pedestrian LOS. Transit LOS can be used to evaluate alternatives that affect route speed, frequency, and pedestrian LOS.

Performance measures

- Segment Transit LOS letter grade A-F

Example evaluation criteria

- Transit LOS letter grades by segments along a transit route

Transit LOS is not an indicator of ridership, which may involve several contributing factors such as land use density, transit frequency, reliability, wait time, walk time, transfers, fares, bus stop amenities, and parking availability and cost.

9.4.3 Truck Level of Service Index

Truck Level of Service is a recently developed measure of the quality of service provided by a facility for truck hauling of freight, as perceived by shippers and carriers. Truck LOS was developed as part of NCFRP Report 31 [(3)]. It is a composite index based on the percentage of ideal truck operating conditions achieved by a facility. Ideal conditions are defined as a facility usable by trucks with legal size and weight loads, with no at-grade railroad crossings, that provide reliable truck travel at truck free-flow speeds, at low costs (i.e., no tolls). Truck Level of Service (TLOS) Index is the ratio of the actual utility to the utility for ideal conditions (free-flow speed and no tolls). Methodology details are found in the HCM Planning & Preliminary Engineering Applications Guide (PPEAG) to the HCM 6th Edition.

Performance measure

- Truck LOS on highway facility

Example evaluation criteria

- Relative difference in facility Truck LOS letter grade

Truck Level of Service Index

$$\%TLOS = \frac{1}{(1 + 0.10e^{-200U(x)})}$$

Where

%TLOS = truck LOS index as a percentage of ideal conditions (decimal),

U(x) = truck utility function, and

e = exponential function.

Truck Utility Function

$$U(x) = A \times (POTA-1) + B \times (TTTI-1) + C \times (Toll\ mi) + D \times (TFI-1)$$

Where

U(x) = utility of facility for truck shipments,

A = weighting parameter for reliability, sensitive to shipping distance = 5 / ASL, for Oregon = 0.025

B = weighting parameter for shipment time, sensitive to free-flow speed = -0.32 / FFS, FFS = free-flow speed,

C = weighting parameter for shipment cost = -0.01,

D = weighting parameter for the facility's truck friendliness = 0.03,

POTA = probability of on-time arrival = 1 if the mixed flow (autos and trucks) travel time index is ≤1.33 (freeways and highways) or ≤3.33 (urban streets),

TTTI = truck travel time index for the study period, the ratio of truck free-flow speed to actual truck speed,

Toll/mi = truck toll rate (dollars per mile), a truck volume–weighted average for all truck types, and

TFI = truck friendliness index, where 1.00 = no constraints or obstacles to legal truck load and vehicle usage of facility and 0.00 = no trucks can use the facility.

The truck utility function is based on several parameters including the probability of on-time arrival for the truck shipment, the travel time index for trucks, tolls paid by trucks, and the truck friendliness index.

The Truck LOS index is focused on the heavier long-haul trucks that travel intercity (externals in travel demand models), rather than commercial vehicles that are typically lighter (can be pickups and panel vans) and are used to distribute goods within an urban area. The response of these two groups can be significantly different. For example, long haul trucks have more potential to be shifted to rail. Commercial vehicles are often fleets of vehicles whose owner can be influenced by local policies, e.g., availability of CNG fueling stations or EV charging infrastructure, or simply levels of local congestion.

Level of Service thresholds are based on %TLOS and class of freight facility. Three classes of freight facility are defined. Truck LOS can be used to evaluate facilities of a uniform freight class, and alternatives that affect travel time reliability, weight or dimensional restrictions, or tolls.

> *For oversize/overweight vehicles, the ORS 366.215 approval process must be followed if considering alternatives that reduce roadway widths on certain freight routes. For more information refer to [ORS 366.215 Implementation Guidance]*

(4)

9.5 Accessibility

Accessibility is a point, zonal, district or area-wide measure of the availability of a range of opportunities such as employment, schools, shopping, medical, recreation, etc, by mode. Accessibility measures typically include travel distance, travel time, population and employment data. Other factors such as level of congestion, parking availability/cost, tolls, and safety may be included directly or indirectly. A destination may be physically close by but if obstructed by a freeway or river or other restrictions, it may not be very accessible. No single accessibility measure captures all possible factors. Accessibility is typically an area or point measure and requires a network. However, the Place Types land use methodology (see Chapter 7) produces a rough accessibility measure of jobs within five miles on a straight line basis. As such, it does not require a network, and is a rough measure of centrality of the location.

Accessibility is important for the bike, walk and transit modes as a travel option and equity measure. The automobile mode generally has good accessibility in most areas. Bike, walk and transit often do not have good accessibility due to incomplete networks or services. Maximizing travel options is more likely to focus on those modes.

9.5.1 Accessibility for Motorized Vehicles, Pedestrians and Bicyclists

JEMnR (MPO-level) travel demand models include accessibility utilities. Accessibility can be analyzed by mode, trip purpose or time of day. OSUM (non-MPO) models can produce accessibility information by trip purpose only. Zone to zone demand and travel time is available from trip matrices, which can also be mapped. Accessibility from travel demand models identifies the potential level of interaction between zones. Applications include evaluating a zone’s potential for development. Thematic or heat maps may be produced identifying the most likely locations for development or transit for example. Accessibility can also be used for equity analysis, based on household income, race, limited English proficiency and other socio-economic factors.

Performance measure

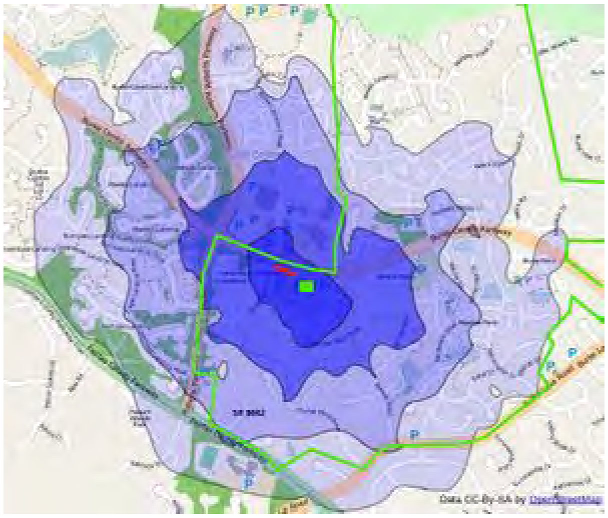
- Area that can be accessed within a given travel time, by mode of travel

Example evaluation criteria

- Percent change in regional accessibility (to jobs or shopping) in each TAZ stratified by walk, transit and auto (JEMnR models)
- Number of essential destinations within a buffer travel time such as 20-minutes by any mode. Essential destinations are defined as hospitals, grocery stores, parks, schools, major retail, etc.
- Number of essential destinations or daily needs accessible within a certain travel time by all modes to low-income, people of color, or limited English proficiency households
- Number of households within a market area such as a travel time radius of a commercial or shopping zone, by mode. Percent of jobs or households within walking or bicycling distance or travel time.
- Number of jobs within certain travel times for all modes accessible to low income, people of color, and limited English proficiency households
- Percent of households/population located with ¼ mile of a bikeway or transit stop.
- Percent of population with X minutes between work and home
- Percent of population located within a mixed use or transit oriented development
- An index of the ratio of direct travel distances to actual travel distances. Well connected streets result in a high index. Less connected streets with large blocks result in a lower index.

The area or distance that can be accessed within a given travel time can be shown using isochrone lines or by shading of TAZs to reflect numbers or percent changes in accessibility variables. Isochrones are contour lines which show the spatial extent of the area or network that can be accessed from a given location given different travel time thresholds, such as 5 minutes, 10 minutes or 20 minutes. Travel demand models and GIS tools are typically used to produce isochrones lines for motorized vehicle travel, whereas GIS tools are typically used to measure bicycle and pedestrian accessibility. It is critical to measure actual network travel time and distance, not “as the crow flies”, especially for pedestrian and bicycle accessibility. Pedestrian network analysis which codes each side of a street separately and which includes improved crossings is superior to street centerline-based analysis, since lack of improved street crossings can be a major barrier to safe pedestrian travel. See Exhibit 9-15 below for an example of an isochrone map.

Exhibit 9-15 Isochrones



Source: Wikipedia

Accessibility may be used to evaluate land use scenarios/changes, such as those that increase density and diversity, for example Transit Oriented Developments (TODs). Travel demand models can evaluate these changes as well as RSPM for use in scenario planning (refer to Chapter 7).

When evaluating pedestrian or bicycle accessibility, any unofficial routes such as trails or unofficial crossing locations should be noted as they indicate actual usage and shortcuts which have potential to be improved as official walkways, bikeways or crossings. For example, [Strava's heat map](#) can be used to find non-traditional bicycle pathways.

Accessibility for pedestrians and bikes may be used to compare alternatives or scenarios that affect densification of land use or walk or bike connections, such as completing paths, adding new paths, or improving crossings. Accessibility does not typically address facility adequacy. Accessibility is one of many contributing factors that affect the amount of walking or biking trips. Other contributing factors include level of comfort, completeness, and safety on the facilities being traveled. Pedestrian and bike travel typically works best for travel distances under one mile for pedestrians or five miles for bicyclists. Refer to Chapter 14 for multimodal considerations.

Bike, Walk and Transit Scores

Bike score, walk score, and transit score are types of accessibility ratings, by mode of travel, of locations based on the number and variety of nearby activities/amenities and the travel time to access them. For example, for a given point (origin), what is the area that can be accessed within a walking distance of ¼ mile, and what amenities are available within that area. A highly accessible location for a given mode of travel would be one with a variety of amenities available within a reasonable travel time for that mode. A less accessible area would be one that has limited or no amenities located within a reasonable travel time. These types of scores typically do not take into account the comfort, safety or quality of the travel facilities or services being used. The scoring method can be expressed in terms of accessibility to important destinations such as schools, shopping areas, parks, medical facilities and transit facilities.

Such scoring methods may be identified using GIS, or using commercial products such as Walk Score, an application used in evaluating the accessibility of candidate residential properties. Commercial tools may create a combined measure or index that accounts for factors other than just travel time. For example, a more complete bicycle score might include consideration of topography. Other non-travel time based measures or indices may be included as well, such as crime statistics, or hilliness of an area. Scoring methods may be aggregated into a rating for an entire city, region or neighborhood, or can be localized to individual properties by address. Heat maps may be created to visualize variations in accessibility throughout an area.

Accessibility scores may be useful in a sketch planning level analysis, but may be limited to existing conditions only. Network-based accessibility measures can show improvements in accessibility when certain links or crossings are added to the network. Commercial software is generally used, as GIS analysis effort can be high, although there may be a cost associated with obtaining the data. Commercial software may not share the complete algorithm or data sources the score is based upon. ODOT has no proposed way to calculate these scores.

9.5.2 Accessibility for Transit Riders

Accessibility for transit riders measures the proximity of transit service available. It is used to evaluate areas with or considering transit service. It may be used to evaluate or prioritize alternatives that affect land use proximity via transit, such as land use densification, adding new routes, or increasing frequency or span of service.

<u>Performance measures</u>

- Proximity of households/population to transit stops
- Proximity of households/population to destinations via transit

<u>Example evaluation criteria</u>

- Percent of population living within "X" miles or "Y" minutes that can access fixed route transit.
- Percent of jobs or households within ¼ mile walking distance of transit stops.
- Percent of households in environmental justice (EJ) communities within half mile of high capacity transit or quarter mile of frequent bus service. As used in equity analysis, such as for environmental projects or TSPs, for identification of affected populations such as minorities, income level, age, etc.

Some general rules of thumb for transit corridors to be potentially viable are those with the following characteristics

- Walk distance to/from transit stops less than or equal to ¼ mile, or ½ mile to high capacity transit stations
- Residential density greater than 4-5 dwelling units/acre for local bus service (1 bus per hour)
- No more than one transfer required

MPO travel demand models include transit lines, fares, and transit stops, and assign transit trips to routes. These models can calculate accessibility to potential transit stops. Small urban area models do not model transit. Metro's model is more sophisticated with the ability to estimate transit loadings at stops. Activity-based models promise a more dis-aggregate treatment of transit, which is likely to be significantly more detailed and accurate, leading to more flexibility in terms of transit performance measures.

Other transit accessibility measures include the use of isochrones to visualize how far a transit rider can get from a given starting point within 15, 30, 45, and 60 minutes of travel using only transit and walking. These illustrate the extent of activities that can be reached from points on transit at different times. For information on other transit tools see the ODOT Public Transit tools webpage.

Transit accessibility is just one of many contributing factors that may affect potential transit ridership. Other factors include land use density, transit coverage, span, frequency, total travel time, pedestrian level of stress/comfort, transit stop amenities, safety, transit fare, transfers required, accommodation of bicycles, and bus occupancy.

Accessibility to frequent transit service may address equity by measuring the ease of access to transit by specific groups such as lower income households. It may be part of the environmental analysis process or may also be performed in some planning studies. GIS databases are able to provide distance information. Factors include travel distance, level of comfort, safety, traveler demographics, and frequency of service.

9.6 Safety

Safety performance measures evaluate historical or are predictors of future potential of crashes on networks and facilities, including crash type and severity. Crashes or crash rates can be displayed using GIS or other mapping tools to identify hot spots for network screening. Detailed procedures are provided in APM Chapter 4.

9.6.1 Crash Rate

Crash rates are a commonly used safety performance measure for a wide range of planning and project analysis studies as part of identifying safety improvement needs. Crash rates are easy to calculate and require little data. Crashes should be based on the official data published by

ODOT's Crash Analysis and Reporting (CAR) Unit. AADTs are required for segments and Total Entering Volume (TEV) AADTs are required for intersections. See APM Chapters 3 and 4 for detailed information.

<u>Performance measure</u>

- Intersection crashes per million entering vehicles (MEV)
- Segment crashes per million vehicle-miles traveled (MVMT)
- Fatal and Severe Injury crashes
- Fatal and Severe Injury crash rates per 100 million vehicle miles traveled

<u>Example evaluation criteria</u>

- Segment crash rate exceeding average for similar facility type from Oregon State Highway Crash Rate Tables
- Intersection crash rate exceeding published 90th percentile intersection crash rate for similar intersection type (APM Exhibit 4-1)
- Crash rate exceeding site critical crash rate based on reference population of similar sites
- Pedestrian and/or bicycle involved fatal and severe injury crashes

The critical crash rate is a Highway Safety Manual screening method of the likelihood that a site crash rate is high as compared to a reference population of similar site types. Critical crash rate is used to flag and prioritize high crash rate locations for further study. See APM Section 4.3.4.

Requirements/Limitations

- Segment crash rates can be heavily influenced by the length of the segment.
- Lack of crashes inhibits usefulness of the measure for evaluating pedestrian and bicycle safety
- Does not account for regression to the mean (RTM) (See Chapter 4 for definition)
- Critical rate requires sufficient number of sites in reference population

Crash Severity – indicator of need and priority based on the level of injury of crashes, the highest priority being the reduction of fatal and severe injury crashes.

- Does not account for regression to the mean
- Requires AADT volumes
- Does not address future safety performance or alternatives

9.6.2 Safety Priority Index System (SPIS)

SPIS is a screening method developed by ODOT that computes a safety index based on crash and volume history on segments. The SPIS index is a function of crash frequency, rate and severity. The Traffic-Roadway Section (TRS) calculates SPIS numbers annually for the entire public road system in Oregon. SPIS sites exceeding threshold scores based on the top 5% or 10% percentile are identified and flagged for further safety investigation. SPIS site maps are available including on TransGIS. The annual SPIS index is calculated based on the last 3 years of reported crash history. Refer to APM Chapter 4 for more detailed information.

Example evaluation criteria

- Top five and ten percent SPIS locations

9.6.3 Change in Crash Frequency Using Crash Modification Factors (CMFs) or Crash Reduction Factors (CRFs)

CMFs and CRFs are typically used to evaluate candidate countermeasures for safety solutions. The initial source for countermeasures should be the ODOT-approved set of proven countermeasures and associated CRFs that are used for the All Roads Transportation Safety (ARTS) Program. See Chapter 4 for detailed procedures.

Example performance measures

- Reduction or percent change in average annual crash frequency, type and/or severity by application of a countermeasure, as calculated using CRFs or CMFs

9.6.4 Excess Proportions of Specific Crash Types

Excess proportion of specific crash types is an HSM screening measure of the extent that a crash type (for example, fatal and serious injury, or pedestrian or bicycle crashes) at a site is overrepresented. Crash sites can be intersections or segments. This is based on a comparison to a reference population of similar sites. Excess proportion of crash types is an indicator of the likelihood that a site will benefit from a countermeasure targeted at the collision type under consideration.

Example evaluation criteria

- Target crash type or severity exceeding threshold

Excess proportion is most frequently used in large area studies such as TSPs. Refer to APM Section 4.3.5 for procedures.

The method does not account for regression to the mean. It does not require traffic volumes. It does not address future safety performance or alternatives. It requires a sufficient number of sites of a similar type in the reference population.

9.6.5 Expected or Predicted Crash Frequency

Expected or predicted crash frequency is an HSM predictive measure of long term crash frequency. This is based on Safety Performance Functions (SPFs) which factor in geometrics, traffic control, volumes, and operations. The Empirical Bayes adjustment methodology factors in crash history. These methods account for RTM error, the natural fluctuation of crashes that occurs over the long-term independent of the contributing factors the analysis is trying to review. Predictive crash analysis is used most often for detailed analysis of alternatives. Expected or predicted fatal and serious injury crash frequency should always be reported as a sub-category of total crashes. Crash frequency can also be reported out by crash type such as bicycle or pedestrian crashes if sufficient data exist. The method predicts reported crashes. There are no established thresholds but the measure may be used for ranking/prioritizing and for comparing alternatives. Refer to APM Section 4.4 Predictive Methods for details.

Example performance measures

- Excess Expected Crash Frequency using Empirical Bayes (EB) Adjustments
- Net Change in Expected or Predicted Crashes

Excess Expected Crash Frequency using Empirical Bayes (EB) Adjustments is used to evaluate the extent that a site's long term average crash frequency differs from that of similar sites.

Excess Expected Average Crash Frequency = Expected Crashes – Predicted Crashes

The EB method requires a calibrated prediction model (with overdispersion factor) and substantial similarity between the analysis period for which crash data exist and the analysis period being used for the predictive method.

Net Change in Expected or Predicted Crashes is used to compare alternatives. Expected crashes can be determined for alternatives if the only changes are in AADT. Otherwise, net change in predicted crashes is used.

Net Change in Expected/Predicted Average Crash Frequency = Expected/Predicted No Build Crash Frequency - Expected/Predicted No Build Crash Frequency

9.6.6 Conflicts

Conflicts are a measure of the number and type of locations where paths cross, merge or diverge at an intersection or junction. Bicycle, pedestrian and transit vehicle conflicts can also be reported as multimodal safety performance measures. Conflict points are potential crash locations, although the number of conflict points does not indicate the probability of occurrence of a crash, which would depend on additional factors such as traffic volumes. Paths that cross are considered major conflicts while those that merge or diverge are considered minor conflicts. Refer to APM Section 4.8.3 for procedures.

Example performance measures

- Number of conflict points at an intersection, total or by type of conflict

Conflicts are typically reported when analyzing alternative intersection types, alignments or lane configurations.

9.6.7 Access Spacing

Access spacing is a measure of the distance between driveways and public street intersections along a roadway segment, or between interchanges along a freeway or expressway. ODOT access spacing standards are provided in Appendix C of the OHP. Local jurisdictions may have their own access spacing standards. A related measure is driveway density which is a factor in bicycle Level of Service. Refer to OAR 734-051 and APM Chapter 4 for procedures. Substandard access spacing can lead to safety and operational problems. Access density is a factor in bicycle LOS.

<u>Example performance measures</u>

- Number of accesses not in compliance with spacing standards
- Percent of roadway in compliance with spacing standards
- Percent deviation from spacing standard
- Number of deviations required

Access spacing is commonly evaluated in corridor plans or refinement plans such as IAMPs or AMPs, in approach permitting, and in projects considering new or modified accesses or roadway connections.

Functional Area

The functional area of an intersection is a measure of the adequacy of spacing between intersections and/or access points to accommodate vehicle paths. Functional area inputs include speed, perception-reaction time, deceleration, lane changing, and queueing/storage lengths. Refer to APM Chapter 4 for detailed procedures.

<u>Performance measures</u>

- Access or junction within functional area of an intersection

<u>Example evaluation criteria</u>

- Functional area of a new connection to the roadway
- Extent of overlapping functional areas

9.7 Other Multimodal Performance Measures

9.7.1 Mode Share

Mode share, typically an area measure, is a function of many contributing factors. Factors include trip purpose, travel time, level of stress/comfort, Level of Service, directness of route, route completeness/connectivity, safety, accessibility, land use, travel costs, and household characteristics. Typical automobile cost factors include auto ownership, maintenance, fuel, parking, and tolls, and is highly influenced by the vehicle's fuel efficiency (e.g., electric vehicles can cost a fraction to fuel relative to internal combustion vehicles, with hybrids somewhere in-between, depending upon the share of miles driven with electricity). Typical transit cost factors include bus fares and subsidies. Bike mode share is also affected by topography, and increasingly bike-share programs (e.g., Portland and Rogue Valley) and their cost schedule.

Performance measure

- Mode share

Example performance measures

- SOV mode share
- Change in mode share
- Percent share of total trips by mode – pedestrian, bicycle, transit, auto
- Percent share of total VMT by drive alone mode (SOV)

Mode share would typically be evaluated for transportation system plan performance or scenarios or alternatives that may significantly change mode share. Examples include transit route changes, transit subsidies, or parking availability/cost.

Mode share is typically obtained from a travel demand model as an estimate that may not represent observed data and is not calibrated. In Oregon there are two levels of travel demand models. In small urban models mode share is assumed from the household survey used to build it (observed travel behavior). It is static and does not react to land use and transportation policies / projects. MPO models have a mode choice model that does react to policies and projects and is an important measure to be aware of and should be requested for all MPO-level model runs.

9.7.2 Transit Service Miles per Capita

Transit Service Miles per Capita is a measure of transit service coverage. It is calculated as fixed route transit revenue service miles divided by area population. Data sources include the National Transit Database (NTD), local transit agency plans and the General Transit Feed Specification (GTFS). RSPM uses this measure and it is also an Oregon Statewide Transportation Strategy (STS) monitoring measure. It can be used as a screening or supplemental for RTPs and TSPs.

Performance measure

- Transit service miles per capita

For the base year, transit service is provided in units of bus-equivalent fixed route transit revenue miles (not counting miles for transit vehicles when not in service). The number of miles traveled while in service for each fixed route transit vehicle is summed over all transit vehicles, for a period of one year. The measure is reported in units of annual service miles per capita.

This measure allows comparisons between alternatives that involve changes in transit service in terms of routes or frequencies, including either expansions or reductions in service. The measure does not reflect ridership.

Transit revenue miles can typically be obtained directly or calculated from miles on various routes combined with hours of operation and headways from the local Transit Agency. This should only include fixed route service. The National Transit Database (NTD) also reports annual service miles by transit provider. Future transit service inputs are provided in units of growth of the region's bus-equivalent revenue miles per capita. It is also important to note that revenue miles are reported in bus-equivalent units.

9.7.3 Multimodal Mixed-Use Area (MMA)

A multimodal Mixed-Use Area (MMA) is an Oregon land use designation that may be adopted by a local government pursuant to the Transportation Planning Rule (TPR – OAR 660-012-0060-10)) to promote mixed-use, pedestrian-friendly, transit oriented, compact land use and transportation activity centers. In order to encourage these types of centers, an MMA designation allows plan or land use regulation amendments to be approved without applying performance standards related to motorized vehicle congestion levels, including volume to capacity ratio, delay or travel time.

Performance measures for evaluating proposed MMA designations within interchange areas are primarily safety related

- TPR requirements
  - Crash rates compared to statewide average for similar facilities
  - Top ten percent SPIS locations
  - 95th percentile queue lengths on freeway exit ramps
- Suggested supplementary measures
  - Critical crash rate
  - Excess proportion of specific crash types
  - Excess expected average crash frequency

For more information including definitions and maps of mixed use areas refer to the DLCD Place Types webpage.

9.8 <u>Infrastructure</u>

Infrastructure performance measures evaluate the supply of transportation networks or services.

9.8.1 Network Connectivity and System Completeness

Network Connectivity and System Completeness are measures of network completeness, redundancy, and availability of alternative routes, which could include streets, intersections, sidewalks and bicycle facilities. Connectivity and completeness is typically evaluated in system planning as well as when considering the potential for re-routing of trips such as for TSMO purposes. Inventories of network elements are prepared and displayed on maps, identifying gaps. Data sources include the FACS-STIP tool for state highways, Google maps and Google Earth, city or regional GIS databases, and Active Transportation Needs Inventories (ATNI) in Regions 1, 4 and 5.

<u>Performance measure</u>

- Network connectivity – extent that the network is inter-connected
- System completeness – percent of planned facility elements such as sidewalks, bike lanes, or improved pedestrian crossings that currently exist

<u>Example evaluation criteria</u>

- Percent local I-I versus regional external –internal or internal-external (E-I/I-E) versus external-external through trips (E-E) on highway – using select links relative to study area
- Percent completeness of bike and walk facilities within ¼ mile of transit stops or ½ mile of schools
- Percent of planned network with sidewalks and/or bicycle facilities
- Percent of network restricted to heavy vehicles
- Capacity available on parallel local facilities
- Ratio of shortest network path distance (driving, walking, or biking) to shortest straight-line distance (as shown in Exhibit 9-16). This is a theoretical minimum distance. Ratios closer to 1 are preferred.
- Number of roadway links divided by the number of roadway nodes or intersections (as shown in Exhibit 9-17).[2]

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[2] A higher index indicates that travelers have increased route choice, allowing more direct connections for access between any two locations. Links are the segments between intersections, nodes the intersections themselves. Cul-de-sac heads count the same as any other link end point. A higher index means that travelers have increased route choice, allowing more direct connections for access between any two locations. According to this index, a simple box is scored a 1.0. A four-square grid scores a 1.33 while a nine-square scores a 1.5. Dead-end and cul-de-sac streets reduce the index value. This sort of connectivity is particularly important for nonmotorized vehicle accessibility. A score of 1.4 is an example threshold for a ‘walkable’ community.

Exhibit 9-16 Shortest Network Path versus Straight-Line Distance

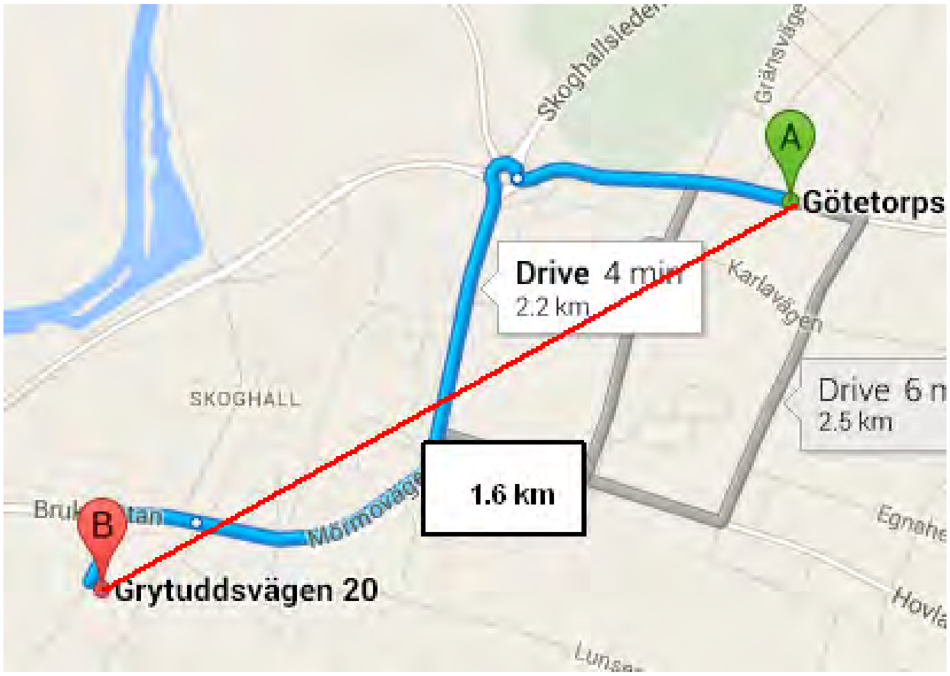
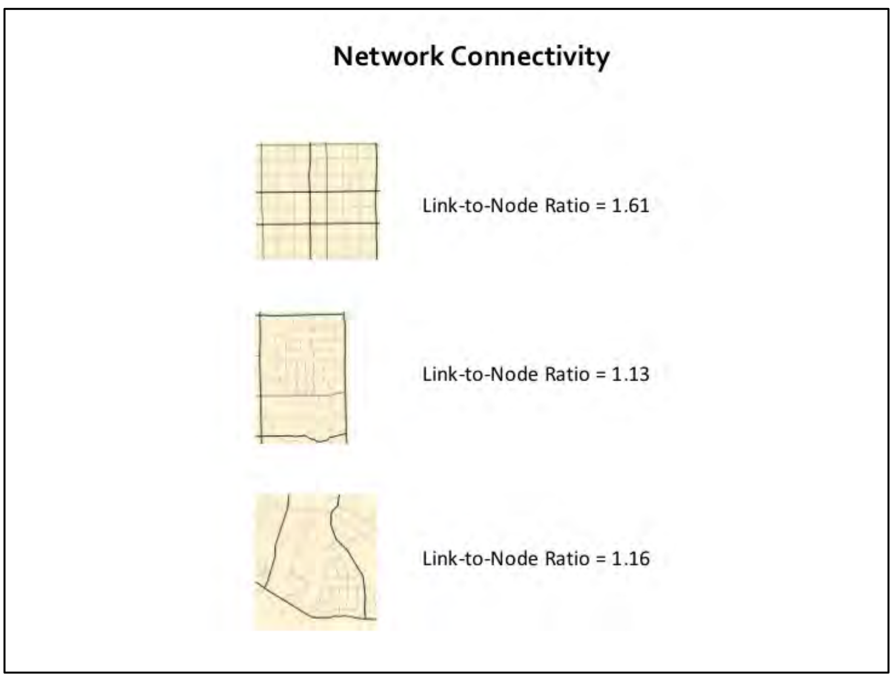


Exhibit 9-17 Links/Nodes Ratio



Out-of-direction Travel

<u>Example performance measures</u>

- Distance out-of-direction (mi or feet), by mode
- Additional VMT

This is the amount of additional travel time and/or distance for a trip or movement due to out-of-direction travel, as compared to a base case. In other words, this is a measure of circuitousness of a route as compared to a direct path. An example would be the out-of-direction travel for an indirect J-turn or at-grade jug handle alternative as compared to a direct left turn. Excess out-of-direction travel for motorized vehicles adds to travel time and VMT and may result in driver frustration which could lead to violations or safety problems. Excess out-of-direction travel for pedestrians (greater than approximately 0.10 mile) may deter use or lead to improper roadway crossings. Excess out-of-direction travel for bicyclists (greater than approximately 0.33 mile) is likely to deter use.

Intersection Density

Intersection density or multi-modal street density are not a common performance measure but are occasionally used as a potential indicator of urban form, i.e., network redundancy, connectivity, or pedestrian friendly paths in an area. Intersection density would be high value for a grid system and low for an area with cul-de-sacs or public street access control is used in JEMnR travel demand models used in many regions of the state. The similar street density is used in Place Types, utilizing block group level data.

<u>Example performance measures</u>

- Number of intersections per square mile within a region or area
- Density of pedestrian-oriented/local streets and/or multi-modal streets miles per square mile within a region or area.

9.8.2 Bicycle or Pedestrian Level of Traffic Stress (LTS)

Bicycle or pedestrian LTS is an ODOT APM Chapter 14 methodology that rates the level of comfort of bicyclists or pedestrians traveling along or crossing a roadway. Scores range from 1 to 4, with 1 being the most comfortable and 4 being the least comfortable. Factors for pedestrian LTS include sidewalk width, condition, and ADA ramps. Target scores are generally either 1 or 2, depending on nearby land uses and demographics such as schools, transit stops, downtown cores, medical facilities, etc. It is useful to display LTS on maps to identify connectivity islands and high stress locations such as major road crossings. Such locations create discontinuities which if fixed could improve the LTS of an entire route. Refer to APM Chapter 14 for procedures.

LTS is not by itself an indicator of the potential use of a walk or bike facility, which would need to take into account additional factors such as the proximity and size of land use origins and destinations, topography, and competition with other modes.

Example performance measures

- Pedestrian or bicycle LTS score on a roadway segment, intersection, approach, or crossing
- Project study area network locations not meeting LTS threshold

Appendix 9A – Applicability of Analysis Performance Measures by Plan or Project Type

Appendix 9B – Alternative Mobility Targets (Planning Business Line Team Operational Notice)

References

(1) Dowling, Richard. *Traffic Analysis Toolbox Volume VI: Definition, Interpretation, and Calculation of Traffic Analysis Tools Measures of Effectiveness.* No. FHWA-HOP-08-054. 2007.

(2) Estimating the Impacts of Urban Transportation Alternatives, Participant's Notebook, FHWA/NHI December, 1995.

(3) Dowling, R., G. List, B. Yang, E. Witzke, and A. Flannery. NCFRP Report 31: Incorporating Truck Analysis into the Highway Capacity Manual. Transportation Research Board of the National Academies, Washington, D.C., 2014.

(4) Guidance for Implementation of ORS 366.215 (No Reduction of Vehicle-Carrying Capacity), ODOT Transportation Development Division, April 17, 2015

10 ANALYZING ALTERNATIVES

10.1 Purpose

This chapter provides guidance on facility level alternative transportation analysis for corridor plans, refinement plans, and project development with or without National Environmental Policy Act (NEPA) involvement. If NEPA is involved or intended to be involved in the future such as in the creation of an Environmental Impact Statement (EIS) or an Environmental Assessment (EA) then the NEPA guidance for alternative analysis must be followed. Most projects are deemed "CE" for Categorical Exclusion (CE) as there are little to no adverse impacts and typically do not go through the NEPA process. An exception is if the CE project goes through federal lands such as Forest Service or Bureau of Land Management. The NEPA process is usually done by the federal agency, but ODOT can still prepare the environmental document.

The traffic analysis portion of the alternative development is only a small part. A NEPA process requires a much broader analysis of the alternative encompassing many diverse areas. Other areas such as right-of-way, air quality, socioeconomics, or noise may have an equal or greater influence on the decisions made. The purpose of alternatives analysis is to analyze Existing or Future No Build needs or deficiencies to develop and evaluate solutions. This process is similar for both planning and project development, the main difference being level of detail of the analysis. The guidance covered in this chapter is based on an adaptation of the EIS process, as it is the most comprehensive and most closely follows the planning process. Smaller CE projects may not go through all the steps described. More and more the planning process is linked to the NEPA process to minimize rework and to speed up the ordinary long timelines. This Planning and Environmental Linkage (PEL) creates NEPA-compatible planning documents with a proper purpose and need, goals and objectives, preliminary screening of alternatives and documentation. The preferred alternative out of a PEL-compatible plan could be rolled straight into a NEPA EA process to be compared with the no-build without any new analysis work or rework for example.

Analysis of alternatives includes definition of the project evaluation criteria, creation of screening processes and documentation for multiple types of solutions. Needs and deficiencies can relate to mobility, safety, multimodal, geometric design, water quality, land use, utilities, etc. Solutions can be either interim or long-term, may involve capacity improvements as well as operational elements or strategies. It is necessary to evaluate trade-offs between solutions as part of the decision-making process. This includes practical design considerations. Contact the Geo-Environmental Section or the regional environmental coordinator for additional guidance, review, or questions on the overall alternatives analysis process especially if NEPA is involved.

10.2 Project Coordination

The development of potential improvement alternatives should be done in cooperation with any groups within ODOT or other agencies that will be involved in the design, implementation, construction, maintenance or operations of the facilities. The district and regional units within ODOT that may be contacted during this process are listed in Chapter 1.

10.2.1 Traffic Analyst

The traffic analyst may be solely a resource to the project team, providing technical analysis results. In some cases the analyst may be a voting member of the project team, being a part of the decision-making process. The technical results that the analyst provides are objective conclusions from the traffic analysis. These need to be vetted and considered along with many other objectives by the larger project team. The traffic analysis results are only part of the overall picture and may not be a primary objective or a deciding factor in the alternative selection process.

10.2.2 ODOT Staff

Environmental or Planning Region staff lead the NEPA process in project development or PEL-compatible plans. Environmental project managers/coordinators govern over the typical contractor that is developing the environmental document and are responsible for ensuring that the NEPA process is followed. Project teams may also have regional Environmental Program Coordinators as team members. Environmental section headquarters staff are subject-matter experts on NEPA guidance, noise, air & water quality, etc. and may also be involved on project teams directly or as resources.

Typically, the highway design and traffic operations engineers within ODOT have a key role in assisting the review and confirmation of the selected alternatives. This includes both headquarters staff as well as at the regional technical centers. For example, the Traffic Engineering Section, specifically the State Traffic Engineer, must approve certain traffic control devices. Design exceptions are also approved at the headquarters level. It is a good idea to have headquarters staff perform a preliminary review of project alternatives as they may find issues that may be an impediment to approval. This coordination should occur early in the alternative’s evaluation process. Planning staff should also be coordinated with to ensure the project does not potentially conflict with past or current planning efforts. The regional technical center staff that would be responsible for the design and implementation of the selected alternative should be included in the concept development, performance assessment and suggested for further refinement.

The Crossing Safety Team under the Rail Safety Section, which is part of the Commerce & Compliance Division based in Salem, has jurisdiction over railroad crossings and traffic control devices used within crossing areas. They also have exclusive legal authority over public grade crossings and provide coordination with the railroads for affected private rail crossings. The Crossing Safety Team should be contacted any time a project will have an impact directly to or within 500 feet of a railroad or rail crossing.

10.2.3 Other Federal, State and Local Agencies

Other agencies such as FHWA, the Oregon Aviation Department, State Marine Board, Department of Forestry/US Forest Service, and Bureau of Land Management may need to be coordinated with depending on the context of the project. Projects on the interstates or NHS system require FHWA coordination. FHWA approves interchange modification requests and oversees the NEPA process for Federal Aid projects. The local authorities for affected roadways, other than the state, should be included in the selection and review of alternatives. Typically this includes local cities, counties, regional metropolitan planning organizations, transit agencies, etc. Tribal governments need to be coordinated with as applicable.

10.2.4 Project Teams/Committees

The project team(s) control the overall flow of the project. The actual teams and composition of them is dependent on the specific planning or project development effort at hand. For more information see ODOT's Project Delivery Guide. This group may also be known as a Technical Advisory Team (TAC) on a planning project. Typical attendees are ODOT /consultant staff representing different technical areas (i.e. traffic, roadway, environmental, right-of-way, mode experts, etc.) and local jurisdiction staff (i.e. planners, public works, etc.). Some other state or federal agencies (i.e. FHWA) may be represented. The Project Development Team (PDT) reviews the information provided by the analyst, consultants, other staff, other committees, and provides direction, comments, and decisions/recommendations on next steps. The PDT may have voting to screen down alternatives or may have encompassing discussions on an alternative evaluation matrix to help decide what alternatives move into the next step of the alternative development process.

The Citizen's Advisory Committee (CAC) (or similarly named) consists of local partners such as business owners, city council members, bike/pedestrian advocacy groups, freight companies, police and fire departments, transit agencies, legislators, tribal representatives, and private citizens. The CAC reviews and provides comments on information (reports, presentations, etc.) and gives comments. Preferences and priorities may be given on alternatives but the decisions here are not binding and only in an advisory capacity. CAC's may be more common on planning efforts or in EIS's and not every project will have one. There also may be a "local partners" team instead that has similar make-up. Sometimes the PDT-CAC are combined and there may be cross-over in the attendees between both groups.

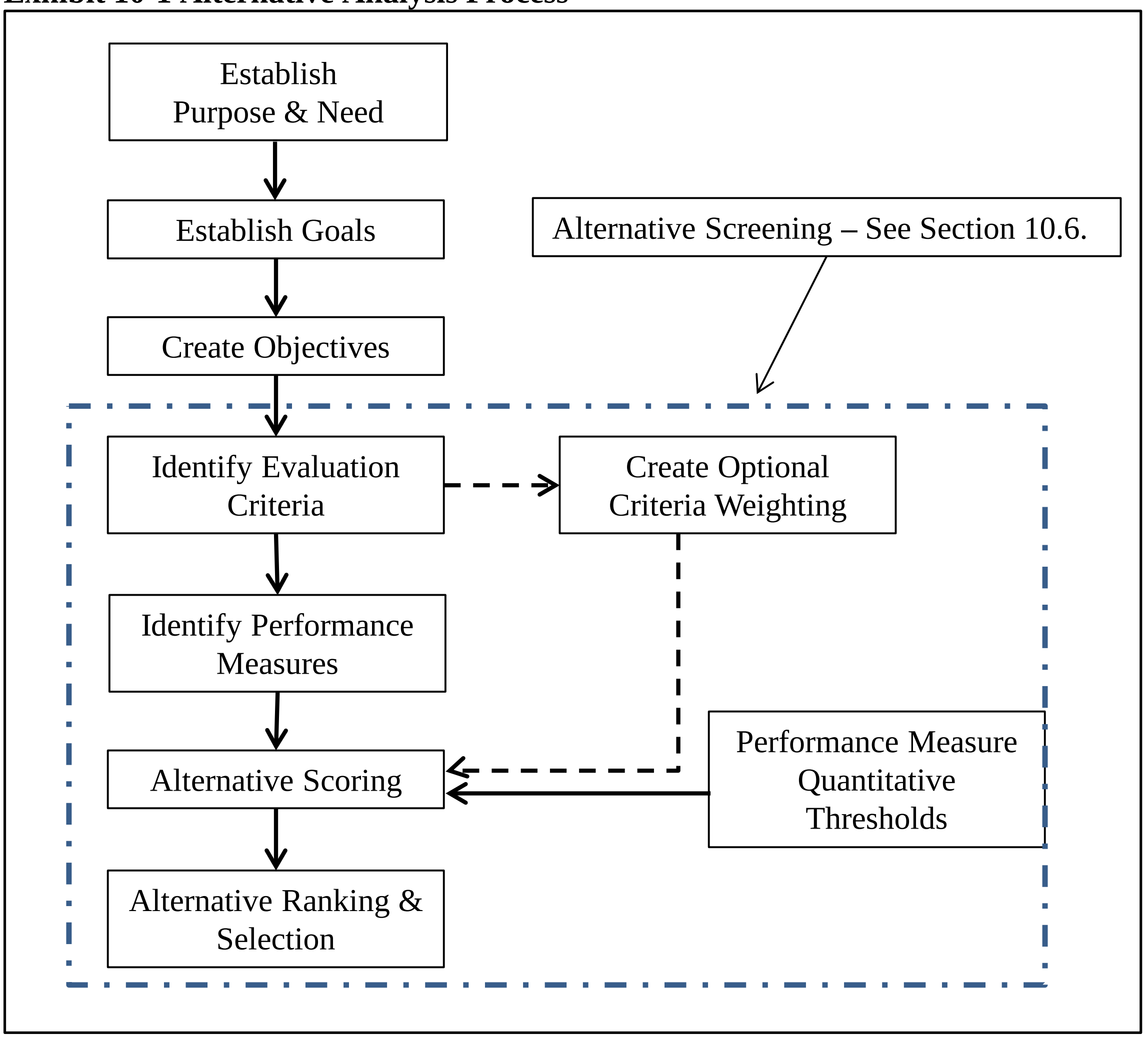
Some regions may also have a Steering Team (ST) or Steering Committee which controls overall direction of the project. These teams come in many forms, but their purpose is to ensure that the project moves forward. The ST is typically made up of high-level officials such as an ODOT Region Planning Manager, mayor or city manager, and county commissioners. When these exist, recommendations from the PDT are given to the ST and the ST makes the final decision on a proposal or alternative that is not delegated to an individual or agency. If an alternative includes a traffic control device for example, the State Traffic Engineer has the approval authority over that element and not the ST.

10.3 Overall Alternative Evaluation Process

Every project or plan must have a purpose (what is trying to be done) and a need (what is the project justification) which is developed by the project teams. Alternatives are developed to meet the purpose and to address the needs. An objective alternative analysis process is necessary to compare the alternatives without bias so the alternatives can be reduced to a final set or to a single preferred alternative. However, it may be necessary to use professional judgment and some subjectivity when deciding on alternatives as the process can be complex. Exhibit 10-1 illustrates the overall evaluation process.

The purpose and need are developed into goals and objectives which are quantified with evaluation criteria and related performance measures. Plans may also develop policies in concurrence with the goals and objectives. The alternatives analysis results using the performance measures are converted into scores which are then totaled and ranked and the top alternative(s) selected. The screening process can repeat several times with increasing detail and specificity for the evaluation criteria and performance measures.

Exhibit 10-1 Alternative Analysis Process



Goals are overarching principles that are sorted based on different considerations such as safety, mobility, multimodal, environmental impacts, livability, socio-economic impacts, accessibility, economic development, etc. Typically safety, mobility, and multimodal goals are required based on current statewide policies and others may be added based on discussions with the project team(s) and stakeholders. Some planning projects will focus on policies in addition to goals. Some example goals are:

- Provide transportation improvements that will accommodate future travel demands safely and efficiently
- Improve transportation system connectivity
- Improve bicycle and pedestrian safety and connectivity
- Provide local and regional access
- Develop a cost effective and environmentally sustainable project that can be funded within the planning horizon
- Consider economic development opportunities
- Minimize impacts to the natural, or built, environment
- Compatibility with local and statewide plans
- Improve freight mobility

Each goal has multiple objectives to provide direction in how to meet the goal. Objectives are specific to one topic so they can be quantified. For example, objectives for mobility and safety goals or improving transportation system connectivity goals could be stated as:

Goal - Improve mobility and safety:

Objective:

- Provide improvements that safely accommodate demand for 20 years
- Provide improvements that are consistent with the classification of the highway per the Oregon Highway Plan

Goal - Improve transportation system connectivity:

Objective:

- Identify local street impacts
- Maintain or improve function of state highway route
- Maintain or improve emergency service response times

Goals may conflict with each other as trying to achieve one may cause significant impact to another. The overall alternative analysis process is a matter of compromise, cooperation, and collaboration with all the involved team members and stakeholders.

From the objectives, evaluation criteria are created to guide the alternative analysis. Evaluation criteria can be weighted, but weighting is optional and more often done in plans rather than projects. It is important that weights be determined before performing any alternative analysis to avoid accidentally creating a biased process. Many methods for weighting exist, but generally they may be performed to develop quantitative index values which total into a single value for each alternative. Weights are typically assigned based on the relative importance of each goal via a consensus or vote of the project team(s). If weighting is used, then it needs to be clearly defined to state why a certain criteria was selected to be weighted versus other criteria. The use of weighting needs to be specifically documented as much as possible to avoid creating the appearance of subjectivity. Weighting can be used to help address tradeoffs between disparate goals such as the need for mobility versus minimizing environmental impacts. However, turning this process into a mathematical exercise is not always possible, especially if there are multiple sensitive issues, and can create more issues if done improperly.

The evaluation criteria should be readily explainable, quantifiable, and data driven. In addition, evaluation criteria should be tracked through the development of performance measures. The evaluation criteria will change based on the level of detail that the alternative process is in, such as purpose and need based screening (fatal flaw), goals and objectives-based screening (shown in this section), and operational based screening (full detailed, micro-simulation, etc.). From the sample objectives above, the corresponding evaluation criteria could be stated as:

- Objective - Provide improvements that safely accommodate demand for 20 years
  Evaluation Criteria:
    - Conflict points
    - Relative degree that interchange spacing standards are met on the corridor
    - Relative degree that interchange crossroad spacing standards are met
    - Relative degree that weave/merge-diverge spacing are met

- Objective - Provide improvements that are consistent with the classification of the highway per the OHP classification
  Evaluation Criteria:
    - Volume-to-capacity (v/c) ratio

- Objective - Identify local street impacts
  Evaluation Criteria:
    - Demand-to-capacity (d/c) ratio
    - Are there parallel local facilities that can capture trips currently on the state highway?
    - Relative extent that local streets are severed by alternative

- Objective - Maintain or improve the function of state highway route
  Evaluation Criteria:
    - Compatibility of highway with OHP spacing standards
    - Volume-to-capacity (v/c) ratio

- Objective - Maintain or improve emergency service response times
  Evaluation Criteria:
    - Travel distance
    - Clearance width for emergency responders

Performance measures/indicators and analysis methods (i.e. how to measure performance) are assigned to each evaluation criteria to measure the impacts of the alternatives. Established or potential performance measures are in Chapter 9. These need to be achieved within the project schedule, consistent with the level of detail of the alternatives, and understandable to project team members. From the sample evaluation criteria listed above, possible performance measures could be:

- Conflict points = Number of conflict points
- Relative spacing met = Number of exceptions when standard could not be met
- Demand-to-capacity ratio = Number of locations on state highways with a d/c ratio of 0.90 or higher
- Demand-to-capacity ratio = Number of locations on local streets with a d/c ratio of 1.0 or higher
- Parallel network = Number of local facilities available
- Relative extent of severed facilities = Number of local facilities that are severed by alternative
- Compatibility to OHP spacing standards = Number of deviations required
- Travel distance = Average distance between fixed provider origin and neighborhood destination pairs
- Emergency vehicle clearance width = Number of locations with substandard widths

Each of the performance measures may have additional thresholds applied to classify the results into groups such as Met, Partially Met, Not Met or Good, Fair, Poor. For example, for spacing standards, no exceptions would be Good, one exception would be Fair and two or more would be Poor. These thresholds can be quantified by assigning values to each (i.e. 2 pts for Good, 1 point for Fair and 0 points for Poor). Additional weighting of the evaluation criteria would further modify the values for each alternative, eventually resulting in a single alternative score.

To screen alternatives, an evaluation matrix should be developed and applied to all alternatives, and those alternatives that do not meet the basic criteria should be removed from further consideration. Exhibit 10-2 shows a sample evaluation matrix based on the sample goals, objectives, and evaluation criteria used in this section. Thresholds, weights and scores were added for each alternative following the indicated performance measure. For Example, Alternative 1 had four total locations where the d/c ratio exceeded 1.0, had no parallel local facilities, and one severed facility. The project team determined threshold rankings for each performance measure, along with criteria weighting, giving parallel facilities three times the impact and severed local roadways twice the impact of an over capacity segment. These are applied to the alternative screening results to convert the screening results into a score for the transportation system connectivity goal. This would be repeated for each goal and a total score computed for each alternative.

Alternatives that passed the initial screening should be advanced to the broader assessment of operational performance analysis, project refinement, and preliminary cost estimates, as appropriate. The alternative evaluation process needs to be done in a team environment using the project team, TAC, CAC, stakeholders, decision makers, etc.

Exhibit 10-2 Sample Alternative Evaluations Screening Matrix

Goal: Improve transportation system connectivity						
Objective	**Evaluation Criteria**	**Performance Measure**	**Threshold**	**Criteria Weight**	**Alt 1**	**Alt 2**
Identify local street impacts	d/c ratio	Number of local street locations with d/c higher than 1.0	2 pts = none 1 pt <5 0 pts> 5	x1	4	2
	Parallel local facilities	Number of parallel local facilities available	1 pt = Yes 0 pts = No	x3	No	Yes
	Relative extent of severed facilities	Number of severed facilities	2 pts = none 1 pt <5 0 pts >5	x2	1	6
Total Score					3	4

All solutions should take into consideration the context of the study area and address the project purpose.

10.4 Practical Design

Practical design is an integral part of the project and alternative development process. Practical design is about creating the appropriate scope for a project based on a system context but developed within existing resources to deliver specific tangible results. Projects need to be evaluated on a system basis. For example, a district highway route that is duplicated by other faster high-capacity routes would likely remain in its current form as it would not make much sense to widen a section of it when most of it will never be improved in terms of the long-term vision for the highway. Tangible results can include safety, mobility, condition, multimodal, livability, economic growth, and the environment. Practical design is handled through the project design process by collaborative multi-discipline project teams that rely on good project descriptions, purpose and need statements, and a clear long-term system vision of the corridor. Detailed information on the practical design approach for ODOT is available in the Project Delivery Guidebook.

Project alternatives must address the overall purpose and need of the project. These alternatives need to make the system better as a whole, address changing needs, and maintain current functionality by meeting (but not exceeding) the project's purpose, need, and related goals and objectives. Potential alternatives need to have key issues (i.e. advantages/disadvantages) identified, be screened using an evaluation process, and result in a single choice or range of recommended alternatives that can gather the buyoffs of the major project sponsors (i.e. ODOT HQ/Region, local communities, etc.). The five key Practical Design values (SCOPE) below will help project teams and members meet these basic goals.

- Safety: Overall system safety will not be compromised, but will be made as safe as practical by maintaining or improving the facility safety level.
- Corridor Context: A corridor approach should be used in determining design and other criteria and applied consistently along it. Facilities need to match the overall context and character of the area (see Section 10.4).
- Optimize the System: Developing specific strategies that optimize life-cycle investment in a particular asset (i.e. interchange, bridge, sidewalks, etc.).
- Public Support: Must work together with local communities in creating solutions and considering needs on a multimodal basis.
- Efficient Cost: Making the best decisions that benefit the entire system by prioritizing the most critical elements. Elements of solutions can be incremental improvements if the specific project purpose and need is met.

Project team members will work together to determine the project's purpose and need, identify goals, objectives and related criteria to evaluate the proposed alternatives, and document the decisions made regarding the alternatives. The traffic engineer/analyst as part of the project team must participate in the project and alternative development processes by sharing how their individual discipline contributes to the total project. The project teams must be aware of when potential shifts occur (i.e. an access change that would require future volumes and analysis to be revised) that change the original assumptions or parameters of the project, and they must inform the project team of how those changes may affect different elements of the project, the project schedule, or the project cost.

10.5 Context Sensitive Solutions

The facility design concepts are initially generated based on their potential ability to meet the needs of the project, but each concept must further balance its features against the physical, social, and environmental constraints found at that location. A planning study should provide sufficient preliminary information about a range of constraints that could complicate or preclude a particular solution. Environmental criteria should be established as part of the project's evaluation and selection process. Environmental impacts may be allowed only if there are no other feasible alternatives. The analyst should coordinate with the Project Leader and Environmental Program Manager on these issues.

The typical environmental and physical issues to be considered include the following:

- **Exclusive Farm Use (EFU) Lands**: State regulations are very restrictive about the nature of highway improvements that are allowed within these lands. Without exceptions, no facility improvements are allowed that add capacity to serve nearby urban areas. Limited safety improvements are acceptable.
- **Environmentally Sensitive Zones**: Proximity of fish bearing streams, open space, riparian zone, etc., requires substantial setbacks from any improvements. In federal environmental parlance these are known as "4(f)" zones and may include wildlife refuges, riparian zones in designated recreational area or parks, historic sites, parks, schools and cemeteries. Impacts should be minimized to these areas as mitigation will generally involve more analysis to see if impact can be avoided. Parks and other

recreational properties purchased with all, or partial federal funds are referred to as a "6(f)" zone and generally cannot be converted into a roadway use without an extensive approval process. Generally solutions that avoid these kinds of environmental impacts are required to be selected over those that do not with everything else being equal.

- **Environmental Justice (EJ):** Disproportionate impacts to a sector of the community, such as low-income or minority populations. Impacts to EJ-affected properties are not ideal and could include displacement, relocations, or increased traffic through a neighborhood. EJ impacts may require substantial mitigation measures.
- **Built Environment**: Impacting existing buildings and structures generally should be avoided. It is usually very difficult as part of a typical project not to disturb the built environment to some degree. This requires consideration of historic buildings, schools, hospitals, parks, large developments, low-income areas, utilities, land use, visual impacts, noise, economic impacts, and environmental justice issues.
- **Right-of-Way (ROW)**: In general, improvements should be limited to minimize right of way impacts. Acquisition of additional right-of-way adds costs and may not be feasible in some locations.
- **Multimodal**: There will be the need to service pedestrians and bicycles in solutions as applicable. Where applicable, transit and freight will also need to be considered as well as impacts on air, rail, marine, and pipeline facilities. Multimodal aspects must be considered through all stages of the alternative development process from initial concepts to preferred alternative selection. Concepts should be developed that serve all allowed modes and should address adverse conditions for non-auto travel.
- **Physical Limitations**: Topography and other geographical features may physically create challenges of full implementation of an alternative. Examples include slope stability, rivers, wetlands, other roadways, railroads, utilities, power lines, etc., which may make it impractical to reroute or widen to the fullest extent required. Other full or interim solutions may need to be considered. For example, an interchange that has one ramp terminal hemmed in by a nearby river and the other ramp terminal by a railroad would make it impractical to try to increase the terminal spacing to allow for longer turn lanes. In this case the structure would need to be widened to accommodate side-by-side turn lanes.
- **Access Management**: Alternatives may impact property access points or access rights which may not be able to be resolved or mitigated. Resolving access issues may be challenging and are governed by statutes and Oregon Administrative Rules (OAR).
- **Funding Feasibility**: Current funding limitations may preclude many alternatives. It is important to document the reasons why a particular alternative cannot meet funding restrictions. It is important to be realistic and not create a whole set of alternatives that are too expensive to build or have phases that cannot be broken down further into manageable pieces. Phases (or sub-phases) need to have some sort of independent utility that will incrementally work toward the final solution. Alternatives need to be able to be broken into phases either with interim short to medium range solutions or a series of phases for long term implementation. A project may start with larger more expensive alternatives then screen them down to a set that is more manageable. The project leader and Region planning generally take the lead on identifying funding availability. The determination of funding availability should be made as early as possible to avoid analysis of alternatives that may not be feasible.

- **Fiscally Constrained**: Alternatives are evaluated and a preferred is selected and becomes a project after being adopted into a Regional Transportation Plan (RTP) or Transportation System Plan (TSP). Projects within RTPs and state highway projects within TSPs must be fiscally constrained. Generally, projects within TSPs should be fiscally constrained[1]. Fiscally constrained means at least the first phase of the project is likely constructible within the funds available in the plan horizon. Other projects may be identified in an illustrative list which is a list of projects that cannot be relied on in reviewing land use changes. For RTPs, Tier 1 is a common nomenclature for the financially constrained list while Tier 2 is the illustrative list. Within the Tier 1 financially constrained list, projects are sorted by short/medium/long term based on yearly funding projections.

10.6 Considerations for Evaluating Build Alternatives

A Build Alternative refers to any combination of proposed or potential facility improvements to the current transportation system within the study area. Alternatives that are substantially similar except for some distinct areas are usually called “Options” instead. Build Alternatives are compared to each other as well as to the No-Build scenario to assess relative performance benefits of the various alternatives and options using the selected evaluation criteria. Comparisons are usually made on a quantitative basis, but some resources may require use of qualitative data.

> *The No-Build is a viable alternative. The No-Build includes committed (funded in a City CIP or ODOT’s STIP) projects other than the subject project being analyzed. The No-Build alternative needs to be analyzed in same way as all Build alternatives for consistency.*

The alternatives selected for evaluation should be reviewed to determine if new model forecasts (or new manual traffic forecasts) are required to reasonably represent the traffic flow conditions with the proposed improvements. For larger study areas, typically a travel demand model is the best tool for evaluating changes in travel patterns associated with potential system improvements and access management plans. However, in smaller studies these changes can be reasonably represented by making manual re-assignments of travel demand, assuming sufficient background volume and travel pattern data are available. For more information see Section 6.12.2 discussion on induced and latent demand.

Typically, the horizon year travel demand forecast used for the No-Build scenario should be applied for each build scenario unless it is determined that the Build scenario would alter the future forecasts for that alternative. For example, where the No-Build scenario is heavily capacity constrained, it is likely that diverted traffic will return in the build scenario. If a model is available, both scenarios would be modeled separately. There are two major aspects to consider in making the new travel forecasts: the effects on travel demand and any reasonable changes to the network or operating parameters.

[1] PBLT Operational Notice on Financial Feasibility in System Planning, PB-03, 09/04/2014

10.6.1 Travel Demand Issues

One outcome of the new travel forecasts may be higher overall volumes on a facility compared to the no-build scenario. This is a common result in a highly congested corridor where a share of existing trips use parallel routes and when sufficient capacity is provided nearby, the trips will be re-assigned to the new facility. Typically travel demand model assignments consider the total travel times between the beginning and end of a trip. When new routes are added with shorter travel times, the model compensates by assigning more trips to the improved facility. For a smaller study area, the total travel demand within the system remains constant, but the locally assigned traffic volumes may be re-distributed. This is a common outcome for most projects.

In a larger regional system, the latent demand for travel that was constrained by corridors with severe delays during commute hours can experience changes in both travel mode and time-of-day when new facilities are introduced. The net result is a higher total travel demand compared to no-build. For example, if a new interstate bridge were constructed across the Columbia River between Portland and Vancouver, several changes to the no-build demand forecast would occur. First, the number of commute bus trips would likely decrease as more travelers opted to drive to take advantage of faster travel times. Second, because the peak travel times would be shorter, more commuters would leave their home closer to the start of their work shift. The combination of these factors would dampen the effectiveness of the new bridge facility because of higher total vehicle trips and more vehicle trips during the peak hour.

10.6.2 Network and Operational Issues

Care should be taken to consider network or access changes that would substantially change the no-build forecasted volumes on the build network. For example, if the build alternative includes a parallel street extension, major access closure, traffic control change, or other action that could re-route traffic flows from one facility to another or one access point to another within the study area, these adjustments should be made before re-evaluating performance. These types of changes indicate that the no-build forecast should not be used for the build analysis. If a travel model is being used, then the analyst should review the build assignments to ensure that they reasonably reflect the proposed improvements, including comparing them to the no-build assignments. If these forecasts are done by manual methods, a controlling factor in making these adjustments is to maintain the total trip origins and destinations for each land use generator within the study area.

For example, if the build alternative consolidates access to a shopping center, the sum of vehicle trips in and out of the shopping center should be the same before and after the project. The volumes that used the driveways that would be closed by the project must be re-assigned to other driveways that are accessible from the shopping center. This is an example of maintaining the same trip totals around a periphery of an activity center.

Another example would be where a street extension is proposed to offload local trips from the highway. In this example, the study area includes a one-mile section of a north-south highway that connects to east-west arterials at either end. Before the project there is only one route for all north-south trips. After the project a new parallel north-south collector road is proposed that connects to both east-west arterials. The reasonable check in this case would use a screenline across where the north-south routes connect to the east-west arterials. The total two-way north-south volume should be approximately the same, except for shifts in travel that may have occurred due to the project, for all facilities connecting to the arterials before and after the street extension. For more information see Section 6.12.4 guidance on screenlines.

10.6.3 Traffic Signal Optimization or Coordination

The background traffic signal timing parameters should be modified to be consistent with the proposed improvement. Caution should be applied when changing the background signal cycle assumptions for the purposes of future analysis. Signal timing is continually re-adjusted over time, so future signal timings should be optimized within the typical cycle maximum. The analyst should coordinate with the agency responsible for operating the signals to identify how the signals would likely operate in the field. Typically the cycle length for the analysis should not exceed 60 seconds for a two-phase traffic signal, 90 seconds for a three-phase traffic signal (e.g., protected highway left turns, and permissive side streets left turns) or 120 seconds for a four- or more phased traffic signal. In larger or more complex intersections or systems, the cycle length may be longer than 120 seconds. Demand-responsive or adaptive traffic control systems continually vary the cycle length, so the use of optimized timings for base and future conditions is necessary. Coordinate with the Traffic Engineering Section if analysis indicates that cycle lengths in excess of 120 seconds are likely. For more information on signalized intersection analysis see Chapter 13.

10.6.4 Intersection Approach Lane Changes or Additions

Any proposed additions or revisions to an intersection approach should be reflected in the capacity analysis and signal phasing, as appropriate. A typical example is adding left-turn lanes to serve higher demand during peak hours. New turn lanes may require changes to the background signal phasing to operate safely, and the phasing changes should also be reflected in the analysis. In addition, the geometry of the intersection should be reviewed to determine if the added approach lane can be served on the exit leg. For example, a second left turn lane on one approach requires a second exit lane on the receiving leg of that intersection for a minimum distance to operate effectively.

10.6.5 Storage Length Changes

Another change would be the modification of storage lengths as indicated by the capacity and/or micro-simulation analysis. Phasing or cycle length changes will also likely cause the storage needs to change. Caution should be exercised if storage lanes exceed 300 feet and especially if a bike lane is located between a long left and right turn lane as this will cause a “sandwich” effect on the bicyclist having to travel between two lines of vehicles without any additional buffering. It is recommended that the analyst coordinate with Region Traffic and/or the Traffic Engineering Section in these cases.

10.6.6 Multimodal and Safety Tradeoffs

Both benefits and disbenefits of all solutions need to be identified and evaluated with the project team. Modal staff from Region, Bike/Ped, Motor Carrier, Rail, and the Traffic & Roadway Engineering Sections, should be involved in these considerations.

Some potential solutions to improve flow and safety for one mode may have adverse impacts on other modes. For example, building a long right turn lane may create an effect of sandwiching bicycle riders between the through lane and right turn lane, creating a deterrent for bicyclist use of that facility segment. All modes need to be considered from the beginning as each concept or alternative is created, instead of evaluating impacts on other modes as an afterthought. Some solutions may be deemed unworkable and dismissed due to multimodal or safety impacts.

For more information on multimodal analysis see Chapter 14. Other examples:

- Increasing a turn radius to mitigate rear-end vehicular crashes will result in an increased crosswalk distance thus increasing pedestrian exposure and risk.
- Adding turn lanes or auxiliary through lanes to improve flow will increase crosswalk distance and likely will increase speeds through the intersection
- Adding sidewalk bulb-outs, landscaped medians, or reducing the number of lanes or lane widths to reduce pedestrian exposure may impact the ability to move oversize vehicles. Reducing widths on certain freight routes is covered by the ORS 366.215 approval process. For more information see the ORS 366.215 Implementation Guidance.
- Traffic signal timing changes could increase queuing at a railroad crossing, creating a safety concern.
- Certain designs such as an overpass could interfere with an airport runway protection zone.

10.6.7 Evaluating Severely Congested Facilities

The performance analysis of severely congested roadways and intersections should recognize that many of the conventional (or default) assumptions used in computer software tools are not necessarily appropriate in these cases. For this discussion, severe congestion occurs when the observed demand exceeds facility capacity (v/c is over 1.0). The HCM analysis methods for roadways and intersections are not appropriate in cases where the volume substantially exceeds facility carrying capacity.

When the facility is presently heavily congested, the analyst should verify through field studies, additional surveys or other measurements that the observed conditions are reasonably like the computer software results. For example, if an intersection analysis indicates v/c ratio near 1.0, it should be noted that intersection evaluations are based on the number of vehicles entering the intersection during the assessment period and may not be the same as the total demand at that location. A field observation may show that heavy vehicle queuing occurs during the peak hour, and a substantial share of the actual demand is queued and not served at the intersection during the peak analysis period (refer to Chapter 3 section on counting congested conditions). In this case, the demand is greater than the actual amount of traffic that enters the intersection during the analysis period.

When facilities approach capacity levels during the peak hour, one result is for commuters to shift their travel times outside of the busiest hour to reduce their overall travel times. This phenomenon is referred to as peak hour spreading. Refer to Section 8.6 on peak spreading analysis methodologies.

For future analysis, a v/c ratio calculation may result in a value higher than 1.0 for an isolated intersection. This condition may result from existing latent demand or excessive future demand of vehicles at an intersection. This should be considered as a d/c rather than an actual v/c ratio and would indicate conditions where mitigation could be considered to improve intersection operations.

Severe forecasted congestion at one location may influence and impact conditions at other intersections within the local transportation system. For example, spillback from one intersection may block traffic from proceeding through a nearby intersection, even when the traffic signal indication permits it. In addition to the intersection v/c ratio analysis, the analyst should review average and 95th percentile vehicle queues within a congested local system to identify potential cases of secondary congestion impacts, which could reduce the performance otherwise indicated by an isolated intersection analysis for that location. In these types of situations, it is not sufficient to only conduct isolated intersection methods. A more reasonable tool would be either micro or mesoscopic simulation, which accounts for interaction between locations, queue spillbacks, blocked intersections and serving excessive demand between signal cycles. See Chapter 8 on mesoscopic analysis and Chapter 15 on micro-simulation.

Large numbers of alternatives need to be reduced first with an established screening process (See Section 10.7.2), such as with a transportation demand model. It will take too long (which will also have a large budgetary impact, especially if contractors are used) to analyze alternatives at the full micro-simulation detail at a month apiece versus a couple weeks for a dozen or more at the screening level. No more than three to five alternatives should be fully analyzed in detail to keep the workload, schedule, and budget reasonable.

10.6.8 Benefit-Cost Analysis

Overview

Benefit-cost analysis (BCA) is often used to compare the cost of projects relative to the benefits to evaluate whether investments make good business sense. Theoretically, all benefits and all costs associated with a project are monetized to produce a ratio of benefits to cost. A ratio greater than one indicates the benefits are greater than the costs, indicating a positive outcome for the investment. When BCA is required, the analysis is prepared by economists, either consultants or the economists in the ODOT Program Implementation and Analysis Unit (PIAU). Most BCA tends to require customized applications as the methods used are very specific to an individual project's goals and objectives, issues and questions being asked. An increasing proportion of ODOT projects require BCA, making it important for traffic analysis to generate the information needed for economists to prepare this metric.

Some BCA may be required as part of grant or other funding programs such as Better Utilizing Investment to Leverage Development (BUILD) grants or Highway Safety Improvement Program (HSIP). Such programs may have specific methodological requirements such as the use of national travel time values instead of local values.

Much of the time project level BCA is performed as part of programming or final design. BCA of environmental studies, such as EAs and EISs, tend to be larger efforts involving an assessment of a wider range of impacts and are usually done by the contractor responsible for the environmental document. If a Planning and Environmental Linkages (PEL) study, Environmental Impact Statement (EIS), or Environmental Assessment (EA) are conducted, there will likely be an a Socio-economic technical report prepared to analyze the alternatives' economic impacts and benefits, which are disclosed in the NEPA document.

Sketch level BCA can be prepared by non-economists for planning or project analysis to obtain general order of magnitude estimates of certain types of benefits and costs. Simplifications are made, and as such, it is supporting information which should not be used as a sole factor in deciding between alternatives. The limitations of sketch level BCA and level of uncertainty should be clearly stated and documented. The more common project parameters estimated in sketch level BCA analysis include the change in travel time, miles traveled, crashes, emissions, and vehicle operating costs. The largest components are typically travel time, crashes and miles traveled. Change in distance traveled due to a project is occasionally a large component as well.

Estimates of delay are obtained from models or other tools including simulation. Value of travel time differs by automobile versus trucks versus bus so truck and bus percentages are needed.

A quick way of estimating the economic impact of delay is to use a Queue and Delay Cost worksheet. This spreadsheet uses an hourly volume profile over a day and a measure of the directional hourly roadway capacity. Capacity can be varied during the day to show the impacts of a short or long-term workzone or other reduction such as an incident. Queues and delays are created when the hourly arrival rates exceed the capacity of the segment and will continue until the demand drops enough that the segment can discharge all of the extra demand. The spreadsheet will show estimates of queue length, queue duration, delay and delay cost. Example 10-1 shows part of this spreadsheet with the important values highlighted.

Example 10-1 Queue and Delay Estimates

An analyst is trying to determine the impacts of a short-term workzone during the day on a section of urban freeway. The freeway has a nominal hourly capacity of 5700 vph with a reduced workzone capacity of 5000 vph. The workzone will be in place from 5 AM to 1 PM. The hourly volume profile from a nearby count (or ATR) is entered into the worksheet along with the hourly capacities and the current estimates of the value of travel time. The results come back with queuing starting at 7 AM and continuing until about 2 PM with a maximum queue of 2100 vehicles at around 8-9 AM (which is approximately four miles). This creates a total of 11,100 vehicles –hours of delay at a cost of over $106,000 daily.

Queue and Delay Cost Worksheet Example

TIME	Hour	*Demand	Cumulative Arrival	Capacity	Departure	Cumulative Departure	Queue	Cumulative Delay
12-1 AM	1	600	600	5700	600	600	0	0
1-2 AM	2	300	900	5700	300	900	0	0
2-3 AM	3	350	1250	5700	350	1250	0	0
3-4 AM	4	300	1550	5700	300	1550	0	0
4-5 AM	5	700	2250	5700	700	2250	0	0
5-6 AM	6	2000	4250	5000	2000	4250	0	0
6-7 AM	7	5200	9450	5000	5000	9250	200	200
7-8 AM	8	6400	15850	5000	5000	14250	1600	1800
8-9 AM	9	5500	21350	5000	5000	19250	2100	3900
9-10 AM	10	5000	26350	5000	5000	24250	2100	6000
10-11 AM	11	4500	30850	5000	5000	29250	1600	7600
11-12 PM	12	4700	35550	5000	5000	34250	1300	8900
12-1 PM	13	5200	40750	5000	5000	39250	1500	10400
1-2 PM	14	4900	45650	5700	5700	44950	700	11100
2-3 PM	15	4800	50450	5700	5500	50450	0	11100
3-4 PM	16	5100	55550	5700	5100	55550	0	11100

Duration of Queue

Capacity reduction

Total delay (veh-hrs)

Workzone duration

Maximum queue

Results

Max.Queue		**Total Delay**		**ᵇTotal Cost**
2100	vehicles	11100	veh-hrs	$106,560
4.2	miles			

**ᵇ Unit User Costs: $8/hr -Cars;$24/hr -Trucks*

Traffic Data for Benefit Cost Analysis (BCA)

To prepare project benefit-cost analysis, the economist requires project-specific traffic data from the project traffic analyst. The traffic data is typically requested for both the no-build and build alternative. The data is generally needed for both the base year (existing conditions) and future year (typically 20 years from opening). The build year (year of opening) data will be automatically interpolated by the BCA spreadsheets used to enter the data. The data typically includes design hour or peak period traffic volumes, section travel time/delay, and crash reduction factors for the build alternative. Other data may be needed as well such as percentage of heavy vehicles and average daily traffic. The data are used by the economist to monetize project benefits such as reduced travel time and crash cost savings.

Usually, the road sections for which traffic data are needed are defined by the beginning and ending mileposts of the project as identified by the Region. If the traffic impact area extends beyond these mileposts, the analyst should coordinate with the economist to define the road sections for which data are needed. The road sections needed may also depend on the type of facility, the context of the project, the stage the project is in, the type of BCA being performed, and other factors. It is important that all requested traffic data values are clearly identified, and data sources documented.

The traffic data typically requested are listed below, for both no-build and build alternatives and for both base year or year of opening and future year. The aggregation of the data by roadway sections will be defined by the economist.

- Traffic Volume
  - AADT
  - Percent trucks and percent buses
  - Percent annual growth rate
- Demand-to-Capacity Ratio
  - Design hour or peak period d/c ratio, by direction of travel
- Travel Time and Delay
  - Design hour or peak period hours of travel, by direction of travel
  - Design hour or peak period vehicle-hours of delay, by direction of travel
- Safety
  - Crash reduction factors applicable to the build alternative

Projects Requiring a Benefit-Cost Analysis due to ORS 184.659 (HB 2017)

> *Scoping-level traffic data are needed for those projects that require a benefit cost analysis under ORS 184.659 (HB 2017) for OTC consideration of project adoption into the STIP. The traffic data need to be prepared as part of project scoping activities.*

New modernization projects (that were not earmarked in HB 2017) having a cost estimate near or over $15 million are required to have a benefit-cost analysis prepared by an economist prior to being adopted into the STIP. Project traffic data are needed to support this analysis. This traffic data must be prepared as a project scoping activity.

Traffic data for BCA will be needed prior to preparation of the project traffic analysis. Timelines will typically not allow for detailed data collection and volume development, and the project will not be fully defined at this stage, so the traffic data usually will be developed at a scoping level. Volumes will be based on AADTs, K factors and D factors. The analyst needs to coordinate with the economist early on regarding traffic data needs and assumptions.

Further details on the criteria and process for BCA is contained in [ODOT Scope & Select Leadership Team Operational Notice SS-03]() on Benefit/Cost Analysis for Large Projects

Grant Application BCA (Post-Project Traffic Analysis)

At the stage of a grant application, the project-level analysis should have been completed. Traffic data needed for grant application benefit-cost analysis should be obtained from the project analysis. The project level analysis is likely to have been prepared at a greater level of detail using different tools than those used in scoping. For example, if the analysis created a microsimulation model, the traffic data for BCA would be obtained from the microsimulation model results. It should be noted that other federal grant requirements may apply.

In many cases it will be necessary to supplement the project traffic analysis with additional computations, such as calculating peak hour VHD or VHT, or reporting different segments as requested by the economist. The procedures presented in the section on Scoping Level Traffic Data can be followed for such supplemental calculations. For HSM predicted crash frequency, historical crash data can be used by the economist to estimate other values such as the number of persons injured or number of vehicles involved in the PDO crashes. If historical crash data were reported as part of the project analysis, it should be provided to the economist to check for and reconcile any significant differences from the economist's historical crash data.

If travel time reliability analysis was performed in the project study, it may be used for the traffic data for BCA if a more refined estimate of the variation in travel time and delay due to non-recurring events (such as incidents and weather) is desired. If non-recurring delay values are provided, they should be clearly identified and provided separately from recurring delay values, since the value of travel time for non-recurring delay is different than that for recurring delay.

For projects that are not fully funded, traffic data may need to be provided separately for each component of the project that has independent utility.

The project-level traffic data are entered into the economists' spreadsheet like the scoping level traffic data illustrated in Example 10-2.

Scoping-Level BCA (Pre-Traffic Analysis)

In the scoping stage, the design of the build alternative is in general terms, not details. For example, it may only be known that the project will expand a roadway from two travel lanes to four. Individual elements of the project such as connections, intersection treatments and auxiliary lanes may be unknown. In addition, available data may be limited and/or impractical to collect in detail at this early scoping stage. For these reasons, a sketch planning-level methodology is recommended to develop scoping level traffic data. Default values will generally be used except where project-specific information is readily available. For the BCA, the scope of the build alternative will be provided by the Region. Where there are gaps in project scope decisions, the economists may be required to make assumptions about the project, which should be clearly documented, to complete the BCA.

The overall steps for the scoping level process are as follows:

1. Working with the project manager and economist, define the roadway sections needed for both the no-build and build alternatives. Section boundaries may vary by facility type and direction of travel. Sections may be segmented at major junctions or intersections where highway volumes change significantly, at lane adds or drops, or at changes in terrain type. Establish the years for which data is needed. Define the roadway configurations for both the no-build and build alternatives.
2. Establish the analysis hour or period.
3. Gather data for each roadway section and direction of travel.
4. Calculate the performance measures identified below, for each roadway section in each direction of travel, for the no-build existing, no-build future, build existing, and build future alternatives.
    - Free-flow speed
    - Capacity
    - Demand
    - Demand-to-capacity ratio
    - Section length
    - Average speed
    - Travel time
    - Delay
    - Crash reduction factors (Build Alternative)

Typical methodologies, data sources and assumptions to develop scoping level traffic data are described below. These are generally based on the Highway Capacity Manual (HCM) and the Planning & Preliminary Engineering Applications Guide (PPEAG). Use of default values is likely needed to minimize data collection. The calculations are typically performed manually or with spreadsheets.

The methodology for demand estimation uses AADTs, K factors, D factors and other readily available traffic data. Results are provided for the design hour or peak period, typically based on the 30th highest hour. The economist may estimate values for other time periods if needed. The analyst should clearly identify the analysis period the data represent, i.e., peak hour, peak two hours, etc.

Free-Flow Speed

At a scoping level, the base free-flow speed of existing roadway sections can be assumed equal to the posted speed plus either 5 mph or 10 mph depending on the facility type, as discussed further below and in the PPEAG.

Where a speed differential exists for autos versus trucks, a weighted average base free-flow speed is calculated. The base truck FFS is estimated as the auto FFS minus the difference in the posted auto and truck speed limits. Then the weighted average base free-flow speed is calculated using the following formula, based on the proportion of trucks in the traffic stream P_T (refer to Appendix A of Chapter 11).

$$BFFS = (1-P_T)S_{auto}FS_{auto} + (P_T)FFS_{truck}$$

Freeways

The base FFS for freeways is estimated using the default value (Appendix C of Chapter 11) of posted speed + 5 mph. The adjusted FFS is calculated using HCM Equation 12-2:

$$FFS = \text{BFFS} - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}$$

where

f_{LW} = lane width adjustment:
- 0.0 mph (12-ft or wider lanes) (default)
- 1.9 mph (11-ft lanes)
- 6.6 mph (10-ft lanes)

f_{RLC} = right-side lateral clearance adjustment factors. Refer to HCM Exhibit 12-21

TRD = total ramp density, the total number of on- and off-ramps in one direction for 3 miles upstream and 3 miles downstream, divided by 6 miles

Multi-lane Highways

The free-flow speed on multi-lane highways is calculated using HCM Equation 12-3:

$$FFS_{adj} = BFFS - f_{LW} - f_{TLC} - f_M - f_A$$

where

BFFS = base free-flow speed. Use section design speed or estimate using a default value of posted speed plus 5 mph.

f_{LW} = lane width adjustment. Same as for freeways

f_{TLC} = total lateral clearance adjustment (left and right). Refer to HCM Exhibit 12-22.

f_M = median type adjustment. Refer to HCM Exhibit 12-23.

f_A = access point density adjustment (right side). Refer to HCM Exhibit 12-24.

Rural Two-Lane Roadways

The free-flow speed on rural two-lane highways can be calculated using HCM Equation 15-2:

$$FFS = BFFS - a(HV\%) - f_{LS} - f_A$$

where

BFFS = base free-flow speed. Use section design speed or estimate using a default value of posted speed plus 10 mph.

a = Refer to HCM Equation 15-4.

HV% = Percentage of heavy vehicles in the analysis direction (%).

f_{LS} = lane and shoulder width adjustment. Refer to HCM Equation 15-5.

f_A = access point density adjustment (right side). Refer to HCM Equation 15-6.

Urban Streets (Arterials and Collectors)

For scoping level analysis, the simplest way to estimate free-flow speed on urban streets is to use a default value of posted speed plus 5 mph. If more detailed information is readily available, the following methodology from the HCM can be used.

Calculate the Base FFS per HCM Equation 18-3. Adjustment factors are found in HCM Exhibit 18-11.

$$S_{fo} = S_{calib} + S_0 + f_{cs} + f_A + f_{pk}$$

Where

S_{fo} = base free-flow speed (mi/h)

S_{calib} = base free-flow speed calibration factor (mi/h) – can be assumed to be zero for scoping level analysis

S_0 = speed constant (mi/h)

f_{CS} = adjustment for cross section (mi/h)

f_A = adjustment for access points (mi/h)

f_{pk} = adjustment of on-street parking (mi/h)

Calculate the adjustment for signal spacing per HCM Equation 18-4.

$$f_L = 1.02 - 4.7 \times \frac{Sfo - 19.5}{\max\,(Ls, 400)} \leq 1.0$$

Where

f_L = signal spacing adjustment factor

S_{fo} = base free-flow speed (mi/h)

L_s = distance between adjacent signalized intersections (ft)

The adjusted FFS is calculated using HCM Equation 18-5.

$$S_f = S_{fo} \times f_L \quad \geq S_{pl}$$

Where

S_f is the free-flow speed (mi/h)

S_{pl} is the posted speed limit

Capacity

The methodology for computing section capacities varies by facility type. Computation procedures are provided below.

Freeways

For scoping-level analysis, generalized capacities for freeways can be estimated by applying Exhibit 129 of the PPEAG.

If more detailed information is available, freeway section capacities given as flow rates in pcph/ln under base conditions can be determined from HCM Equation 12-6.

c (base freeway segment capacity) = 2,200 + 10 × ($FFSadj$ − 50)

The base freeway capacity is adjusted for driver population using HCM Equation 12-8:

$$c_{adj} = c \times CAF$$

Default values for CAFpop are provided in Appendix C of Chapter 11.

If greater detail is desired, methodologies in Sections H.5 or H.6 of the PPEAG may be used.

<u>Multi-lane Highways</u>

Multi-lane highways are uninterrupted flow roadways where traffic signal spacing is greater than two miles. Use the urban street method for multi-lane highway sections preceding a traffic signal.

For scoping-level analysis, generalized capacities for rural multi-lane highways can be estimated by applying Exhibit 129 of the PPEAG.

If more detailed data is available, the capacity of a multi-lane highway section can be estimated using HCM Equation 12-7:

$$c = 1900 + 20 \times (FFS_{adj} - 45)$$

<u>Rural Two-Lane Roadways</u>

For scoping level analysis, generalized capacities for rural two-lane highways can be estimated by applying Exhibit 129 of the PPEAG.

If more detailed data is available, PPEAG Equation 198 can be used

$$c = PCCap \times f_{hv} \times f_g \times PHF$$

where

c = capacity (veh/h)

PCCap = HCM passenger car capacity = 1,600 for a single direction (pc/h/ln)

fhv = heavy vehicle adjustment factor for average travel speed (unitless)

$$f_{HV} = \frac{1}{1 + P_{HV} \times (E_{HV} - 1)}$$

P_{HV} = proportion of heavy vehicles. Heavy vehicle percentages on state highways can be obtained from TransGIS

E_{HV} = heavy-vehicle equivalency (PPEAG Exhibit 37) based on terrain type (level, rolling, mountainous). Specific grades (where a grade is at least ±3% and at least 0.6 miles long) can also be considered but are not required

fg = grade adjustment factor for average travel speed (unitless), refer to HCM Exhibit 15-9 and 15-10. Default value = 1.00

PHF = peak hour factor, default = 0.88

Urban Streets (Arterials and Collectors)

For scoping-level analysis, generalized arterial and collector capacities can be estimated by applying Exhibit 129 of the PPEAG. The values in the table for downtown, urban and suburban arterial and collectors outside of large MPOs (Portland, Salem and Eugene) need to be reduced by 8%.

If more detailed information is available for signalized roadways, Equation 199 of the PPEAG can be used. Alternatively, the PPEAG urban street segment planning tool may be used.

Peak Period Demand

Existing Year Volumes

Section directional demand (veh/h) is obtained by converting AADT into design hour volume by applying K factors and D factors.

1. Calculate the two-way design hour volume by multiplying the AADT by the average K-30 factor. Both AADT and the K-30 factor for state highways can be obtained from TransGIS. Calculate the average K-30 factor using a representative ATR and following procedures in Chapter 5.
2. Calculate the directional design hour volume (DDHV) using the D-30 factor, obtained from the OTMS Ranked Hour report (see Oregon Traffic Monitoring System (OTMS) Count Report Guide).

DDHV = AADT × K × D
where

DDHV = directional design-hour volume (veh/h)

AADT = annual average daily traffic (veh/day)

K = proportion of AADT occurring in the design hour (decimal)

D = proportion of design hour traffic in the peak direction (decimal)

Where K and D values are not available, typical K and D values from PPEAG Exhibit 7 and 8 may be used.

fhv = heavy vehicle adjustment factor for average travel speed (unitless)

$$f_{HV} = \frac{1}{1 + P_{HV} \times (E_{HV} - 1)}$$

P_{HV} = proportion of heavy vehicles. Heavy vehicle percentages on state highways can be obtained from TransGIS, for default values refer to Appendix 11C

E_{HV} = heavy-vehicle equivalency (PPEAG Exhibit 20)

$$c = PCCap \times N \times f_{hv} \times PHF \times CAF$$

where

c = capacity (veh/h)

PCCap = HCM passenger car capacity from PPEAG Exhibit 127 (pc/h/ln)

N = number of through lanes, ignoring auxiliary lanes

PHF = peak hour factor, default value = 0.88(rural), 0.95 (suburban)

CAF = capacity adjustment factor (locally developed and applied to match field measurements of capacity, when available), default value = 1.00

fhv = heavy vehicle adjustment factor for average travel speed (unitless)

$$f_{HV} = \frac{1}{1 + P_{HV} \times (E_{HV} - 1)}$$

P_{HV} = proportion of heavy vehicles. Heavy vehicle percentages on state highways can be obtained from TransGIS, default values = 5% (urban), 12% (rural)

E_{HV} = heavy-vehicle equivalency (from PPEAG Exhibit 20)

<u>Existing Year Build Alternative</u>

The build alternative with existing volumes is not normally fully developed but is needed to perform the interpolation calculations for the build year in the BCA spreadsheets. The build volume can be estimated by shifting the previously created existing volumes based on the following potential methods.

Potential methods to estimate redistribution of trips

- The rerouting of traffic due to simple network modifications, such as basic connection changes or interchange ramp reconfigurations, may be apparent and can be estimated manually.
- Volume difference plot from urban travel demand model.
- Volume difference plot from statewide integrated model (SWIM).
- Manual screenline method. Shift in demand can be ignored if anticipated to be less than 10%. Refer to APM Chapter 6.

For many projects the build alternative may be expected to result in a shift in traffic volumes from existing conditions which is known as latent demand. Examples include new roadways or connections or added lanes. If the project is located within a travel demand model area, a model run may be undertaken to compare the no build and build alternative volumes. For example, if a new roadway or crossing is proposed, the travel demand model could be used to estimate relative changes in demand on sections across a screenline. At a scoping level the model results should be applied as relative percent changes in no-build volumes rather than as actual volumes. Refer to Chapter 6 for additional guidance. If located outside a travel demand model area, use of the statewide integrated model (SWIM) may be considered for major roadways if the project is of substantial scope to cause a significant change in demand (regional impact). Contact TPAU for further information.

Future Year No-Build Volume

Scoping-level future year no-build volumes are typically developed by applying growth factors from the Future Volume Tables to the existing no-build volumes. If available, a travel demand model may also be used to develop growth factors. Refer to Chapter 5 for methodology.

Future Year Build Volume

Scoping-level future year build volumes are typically developed by applying growth factors from the Future Volume Tables to the existing build volumes. If available, a travel demand model may also be used to develop growth factors. Refer to Chapter 5 for methodology.

Latent Demand

The economist may request the percentage of additional demand or VMT of the Build alternative over the no-build alternative in the peak period. This may occur as latent demand, where an increase in travel within the project area may result from build alternative network improvements which provide more attractive travel paths than are available under the no-build network. This is typically estimated from a travel demand model run. Build demand is assumed equal to no-build demand unless a travel demand model run shows a change in demand of greater than 10%. If latent demand is modeled, the additional build demand is represented in the traffic data provided, i.e., there should be no further factoring up of the traffic data provided. Latent demand can be expressed as the percentage change in peak period demand or VMT over the no-build alternative within the project area. The causes of shifts in demand should be noted.

Peak Period Demand-to-Capacity Ratio

The d/c ratio is calculated for the analysis peak period for each scenario by dividing the DDHV by the directional capacity. Both demand and capacity must be in the same units, such as pcph/ln.

$$\text{Demand-to-Capacity ratio} = v_p \,/\, \text{capacity}$$

Section Length

No-build and build section lengths are needed in order for link changes in vehicle operating costs (through link VMT) to be calculated (by the economist).

Average Speed

Average speed for the peak period is estimated using a version of the Bureau of Public Roads (BPR) curve that has been fitted to approximate HCM results for different combinations of facility type and free-flow speeds (Equation 203 of PPEAG). This formula calculates speed as a function of v/c ratio. As the v/c ratio approaches 1.0, speed drops due to the effects of increasing traffic volumes.

$$S = FFS/(1 + Ax^B)$$

where

- S = average peak hour speed (mph)
- FFS = Free-flow speed (mph)
- A = speed-at-capacity ratio = (FFS / SC) – 1, values provided in PPEAG Exhibit 129
- SC = speed at capacity
  = (capacity [pc/h] / density at capacity [pc/ln/mi])
- x = demand-to-capacity ratio. Demand-to-capacity ratios are calculated by dividing the DDHV by the capacity of the section. For sections that end with stop or roundabout control, the higher v/c ratio of the section or the intersection is used in the formula.
- B = calibration parameter used to match HCM results when demand greatly exceeds capacity (d/c = 1.9), values provided in PPEAG Exhibit 129

Travel Time

The average travel time in the peak period can be calculated by dividing the length of the section in miles by the previously calculated average speed (mph).

Average travel time (sec)

$$T = \frac{Length\ (mi)}{Speed\ (mi/h)} \times 3600\ sec/h$$

Alternatively, travel time can be calculated using a rearrangement of the BPR average speed curve:

$$T = T_0(1 + Ax^B)$$

where

T = section travel time (h),

T_0 = section travel time at low near-zero volumes (h),

Peak period travel time can also be converted to vehicle-hours of travel (VHT) by multiplying the average travel time per vehicle by the peak hour volume.
Projects may reduce travel time in two ways, by reducing delay due to congestion (v/c ratio),

and/or by constructing new links or connections that result in shorter and/or faster paths. The total savings in travel time is represented by the difference in VHT between the Build and No Build alternatives, summed across all sections.

Delay

The portion of the travel time greater than the travel time at the speed limit is considered to be delay. The average delay on a section is calculated by subtracting the average section travel time from the travel time at the posted speed (section length divided by posted speed).

The total delay for the design hour in vehicle-hours (VHD) is calculated by multiplying the average delay per vehicle by the peak period volume:

$$\text{Vehicle-hours of delay in peak period} = \text{Average peak period delay} \times \text{volume} / 3600$$

If there are multiple peak hours in a peak period, the VHD in the peak period is the VHD in the peak hour multiplied by the number of hours in the peak period. Clearly state the number of peak hours included in the calculation.

Crash Data

The economist is responsible for obtaining historical crash data as needed. The analyst furnishes crash reduction factors (CRF) for the Build Alternative. The All-Roads Transportation Safety (ARTS) Crash Reduction Factor Appendix is the first source of crash reduction factors that should be investigated. If a CRF from the ARTS Appendix/List is not available/applicable, a CRF derived from the Crash Modification Factors (CMF) in the HSM Part D and/or the FHWA CMF Clearinghouse may be used if applicable. The ODOT CMF standard is to only use CMF's with a quality rating of 3 stars or better.

If an HSM predictive analysis was performed, the analyst furnishes the predicted crash frequency and severity from the project analysis.

An example of developing scoping level BCA traffic data is provided in Example 10-2.

Example 10-2 Scoping-Level BCA Traffic Data

Scoping-level traffic data are needed for a benefit cost analysis of a proposed freeway improvement project. The purpose of the project is to reduce congestion on an 18-mile section of I5 northbound between Albany and Salem. The Build alternative would add one travel lane in each direction.

In the northbound direction of travel, the project is broken into four sections as follows:

Section A: milepoint 234.00 to 234.99, 2 lanes, urban, level, 1 mile
Section B: milepoint 235.00 to 245.99, 2 lanes, rural, level, 11 miles
Section C: milepoint 246.00 to 248.99, 3 lanes, rural, rolling, 3 miles,
Section D: milepoint 249.00 to 251.99, 2 lanes, urban, rolling, 3 miles

---

The following sample calculations are provided for Section B in the northbound direction of travel only. A spreadsheet showing computations for this example is provided for the purpose of illustration at https://www.oregon.gov/ODOT/Planning/Documents/BCA_ExmplCalc.xlsx.

Gather Input Data

Section B is located in a rural area with level terrain. The following values were obtained from TransGIS. The posted speed is 65 mph for autos and 60 mph for trucks. There are two travel lanes in each direction. The two-way AADT is 68,100 with K = 9.0 and D = 52, with 18.4 percent heavy vehicles.

Free Flow Speed

The base auto FFS is estimated using the default value (Appendix C of Chapter 11) of posted speed + 5 mph:

$$BFFS_{aut}BFFS_{au} = 65 + 5 = 70 \text{ mph}$$

In Section B a speed differential exists for autos versus trucks. A weighted average base free-flow speed is calculated. The base truck FFS is estimated as the auto FFS minus the difference in the posted auto and truck speed limits:

$$BFFS_{truck} = 70 – (65 – 60) = 70 – 5 = 65 \text{ mph}$$

The weighted average base free-flow speed is calculated using the following formula, based on the proportion of trucks in the traffic stream PT (refer to Appendix A of Chapter 11).

$$BFFS = (1−P_T)S_{auto}FS_{auto} + (P_T)FFS_{truck}$$

$$= (1−0.184) \times 70 + (0.184) \times 65 = 69.1 \text{ mph}$$

The base free flow speed is adjusted for lane width, lateral clearance, and ramp density using HCM Equation 12-2.

$$FFS = BFFS - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}$$

Lane width – for Section B, the lane width is 12 feet. From Exhibit 12-20, f_{LW} = 0.0

Lateral clearance – for Section B, the right side lateral clearance is 6 feet or greater. From Exhibit 12-21, f_{RLC} = 0.0

Total ramp density TRD – for Section B there are 1.4 ramps per mile as measured starting from 3 miles upstream of the section and ending 3 miles downstream of the section.

Therefore the Section B adjusted free flow speed is

$$FFS = 69.1 – 0 – 0 – 3.22 \times 1.4^{0.84}$$
$$= 64.8 \text{ mph}$$

Capacity

Freeway section capacities given as flow rates under base conditions are determined from HCM Equation 12-6.

For Section B, the weighted average adjusted FFS was calculated as 64.8 mph.

The base capacity from HCM Equation 12-6 for Section B is

$$c \text{ (basic freeway segment)} = 2{,}200 + 10 \times (FFSadj - 50)$$
$$= 2200 + 10 \times (64.8\text{-}50)$$
$$= 2348 \text{ pc/h/ln}$$

Adjusted capacity is calculated using HCM Equation 12-8

$$c_{adj} = c \times CAF$$

The default adjustment factor CAFpop for Section B is 0.939 for a rural area (Appendix C of Chapter 11). The adjusted capacity for Section B is

$$C_{adj} = 2348 \times 0.939 = 2205 \text{ pc/h/ln}$$

Demand

The DDHV in mixed vehicles per hour (30th highest hour volume) is calculated as follows, per APM Chapter 5.

DDHV = AADT × K × D
For Section B, No Build

Existing Year 2017 = 68,100 vpd
Future Year 2042 = 96,672 vpd

Heavy Vehicle Adjustment

The heavy vehicle adjustment factor for Section B is calculated as follows.

fhv = heavy vehicle adjustment factor for average travel speed (unitless)

$$= \frac{1}{1 + P_{HV} \times (E_{HV} - 1)}$$

Section B has 18.4% trucks. From Exhibit 20 of the PPEAG, for freeways in level terrain, E_{HV} = 2.0. Therefore

$$f_{HV} = \frac{1}{1 + .184 \times (2 - 1)}$$

= 0.845

PHF Adjustment

The local value for PHF is the default value obtained from Appendix C of Chapter 11. For Section B, in a rural area, the default PHF is 0.88.

Demand Adjustment

The DDHV is converted to an equivalent flow rate in pcph.

Equation 12-9 of HCM:

$v_p = V/(PHF \times N \times fHV)$

$$v_p = \frac{DDHV}{PHF \times N \times f_{HV}}$$

Example calculation for Section B, Year 2017, No-Build, flow rate in pcph

Flow rate per lane
v_p = 3044/(0.88×2×0.845)
= 2321 pcphpl

For this project, latent demand is not anticipated to be significant.

Demand-to-Capacity Ratio

v/c ratio = v_p / capacity

where

v_p = demand flow rate in pc/h/ln

Example Calculation for Section B, No-Build, Year 2017:

No-Build Existing v/c ratio = 2144 / 2205 = 0.97

Average Speed

Section B average speed

$$S = FFS/(1 + Ax^B)$$

where x = v/c ratio

Values for speed-flow equation parameters A and B are found from Exhibit 129 of PPEAG for a rural freeway; A = 0.31; B = 7

S = FFS / (1+Ax^B)

Example calculation for Section B, No-Build, Year 2017:

No-Build Existing peak hour average speed
= 64.8 / (1 + 0.31 × 0.97^^{7}) = 51.6 mph

Travel Time

The average peak hour travel time for Section B is the section length divided by the peak hour average speed. Example calculation for Section B, No-Build, Year 2017:

No-Build Existing travel time = (11 mi / 51.6 mi/hr) × 3600 = 767 sec

Travel time can also be expressed in terms of vehicle hours of travel (VHT) by multiplying the average travel time per vehicle by the peak hour volume. Example calculation for Section B, No-Build, Year 2017:

No-Build peak hour Existing VHT
= 767 sec/veh x 3,187 veh / 3600 sec/h = 679 veh-hrs

Delay

Section B travel time at the weighted average posted speed
Existing = (11 mi / 69.1) × 3600 = 573 sec

Delay per vehicle is the average per vehicle peak hour delay. Example calculation for Section B, No-Build, Year 2017:

> No Build peak hour Existing average delay
> = 767 – 573 = 194 sec/veh

VHD – The peak hour vehicle hours of delay is the average delay per vehicle multiplied by the peak hour volume. Example calculation for Section B, No-Build, Year 2017:

> No-Build Existing peak hour delay VHD
> = 194 × 3187 / 3600 = 171 veh-hours delay

Crash Data

For scoping level analysis, crash reduction factors are provided for the Build alternative. In this example, no applicable CRFs were found from the ARTS Crash Reduction Factor Appendix. A study identified in the FHWA CMF Clearinghouse indicated that crashes were reduced by 25% due to a freeway lane addition (CRF = 0.25) for K, A, B, and C type crashes. PDO crashes did not change. (Source: Operational and Safety Trade-offs: Reducing Freeway Lane and Shoulder Width to Permit an Additional Lane).

Summary of Traffic Data

Once calculations are complete, the traffic data results are input into the economists' spreadsheet, as shown in the example screen capture below (inputs in yellow-colored cells). Note that in this example, only I5 northbound sections are shown. Actual traffic data would include sections in the southbound direction as well.

Example Traffic Data Input - Economist Worksheet

					Peak Period Analysis						
Modeled Base Year					NO BUILD			BUILD			
2017 peak hour	Old Length	New Length	Change	Average Daily Traffic	VHD				VHD		
Sub-Project:					VHD by Sub-Project:	V:C Ratio	V:C Ratio > 1? (enter Yes or No)		VHD By Sub-Project:	V:C Ratio	V:C Ratio > 1? (enter Yes or No)
A - I5 Northbound N. Santiam Exit 234 to Albany UGB MP 235	1.00	1.00	0.00	67,700	6.2	0.86	No		2.6	0.57	No
B - I5 Northbound Albany UGB MP 235 to Terrain Change MP 246	11.00	11.00	0.00	68,100	171.3	0.97	No		41.5	0.65	No
C - I5 Northbound Terrain Change MP 246 to Salem UGB MP 249	3.00	3.00	0.00	68,100	12.5	0.75	No		7.5	0.56	No
D - I5 Northbound Salem UGB MP 249 to Kuebler Exit 252	3.00	3.00	0.00	66,100	41.9	0.98	No		12.9	0.66	No
Name #5			0.00								

				Peak Period Analysis							
Modeled Future Year				NO BUILD				BUILD			
2042 peak hour	Old Length	New Length	Change	Assumed Annual Traffic Growth [1+% Change] Between A3&A16	VHD			Assumed Annual Traffic Growth [1+% Change] Between A3&A16	VHD		
Sub-Project:					VHD By Sub-Project:	V:C Ratio	V:C Ratio > 1? (enter Yes or No)		VHD By Sub-Project:	V:C Ratio	V:C Ratio > 1? (enter Yes or No)
A - I5 Northbound N. Santiam Exit 234 to Albany UGB MP 235	1.00	1.00	0.00	1.060	37	1.12	Yes	1.060	5.1	0.75	No
B - I5 Northbound Albany UGB MP 235 to Terrain Change MP 246	11.00	11.00	0.00	1.080	2321.1	1.38	Yes	1.080	180.6	0.92	No
C - I5 Northbound Terrain Change MP 246 to Salem UGB MP 249	3.00	3.00	0.00	1.080	104.4	1.06	Yes	1.080	22.3	0.79	No
D - I5 Northbound Salem UGB MP 249 to Kuebler Exit 252	3.00	3.00	0.00	1.080	619.1	1.42	Yes	1.080	51.4	0.95	No
Name #5	0.00	0.00	0.00					0.000			

Assumptions:				
Peak Hour (K) Factor for Delay Calculations	9.0	**Emission Type:**	**$ / ton ($2017)**	
Benefit per unit of measure (light vehicle)	$27.05	**Carbon dioxide (CO2)**	$1	metric
		Volatile Organic Compounds (VOCs)	$2,000	short
Benefit per unit of measure (truck driver)	$28.60	**Nitrogen oxides (NOx**	$8,300	short
Assume percentage trucks	18%	**Particulate matter (PI**	$377,800	short
Benefit per unit of measure (Bus)	$657.90	**Sulfur oxides (SOx)**	$48,900	short
Assume percentage of Buses	0.2%			
Benefit per unit of measure	$28.59	**Induced VMT in API**	**0%**	
		Maintenance costs per state hwy lane-mile in 2017 $s:	**$7,305**	

Sub-Project:	CRF A	CRF B	Source
A - I5 Northbound N. Santiam Exit 234 to Albany UGB MP 235	0.25		Study Title: C
B - I5 Northbound Albany UGB MP 235 to Terrain Change MP 246	0.25		Study Title: C
C - I5 Northbound Terrain Change MP 246 to Salem UGB MP 249	0.25		Study Title: C
D - I5 Northbound Salem UGB MP 249 to Kuebler Exit 252	0.25		Study Title: C
Name #5			

10.7 Screening Alternatives Overview

The alternatives analysis for potential improvement projects should be consistent with the established evaluation criteria. Alternatives for facilities should be developed, assessed and evaluated relative to the matrix of performance measures selected for the study. Depending on the scope and complexity of the study, it may be appropriate to have a tiered screening process. This process would begin with a brainstorming –type screening process that allows for a large range of potential alternatives (the "universe" of alternatives) to be defined (typically through a workshop or open house process). This enables many stakeholders to express any outstanding concerns and potential solutions at a sketch or concept level format.

These initial alternatives are then filtered down to a reduced set of alternatives through the first screening process. How many alternatives are filtered out at this point depends on the screening criteria. Initial alternatives are usually filtered using a "fatal flaw" analysis which involves comparing alternatives against the purpose and need or minimum design (i.e. AASHTO) standards. The remaining alternatives would then be advanced to the next level to select the best candidates for the purposes of alternative performance evaluations based on the goals and objectives. Alternatives are typically refined, combined or new ones created through the development process. Alternatives that are screened out should be documented as to why and tracked in the project files. This helps document the entire project selection process as well as reference to answer questions about alternative development. As the project advances through alternative development to project design, the process that was applied to develop alternatives should be documented to carry forward into an environmental review document. It is important to describe any initial alternatives that were developed and set aside from further consideration (based on the evaluation criteria) for this purpose. Any alternatives in an EA or EIS need to follow the appropriate NEPA requirements. These discarded alternatives should be included in the Alternatives Considered but Dismissed appendix in the traffic narrative report as frequently details on the "why" something was dismissed are overlooked. An EIS will also normally have a "Alternatives Considered" chapter that describes the overall alternative development process and timeline, and the traffic narrative appendix can be used to help re-construct this.

> *For many projects, alternative naming conventions can change, often more than once as the project progresses especially when alternatives are combined in the screening process. It is important to track the history of the name changes along with the alternatives, so that the appropriate variations can be included in the documentation. Many times early versions are referred to as "Concepts" or "Scenario" and only when it has proved to be reasonably viable in the screening process it is referred to as an "Alternative".*

The end result may be a preferred alternative or a set of final alternatives depending on the type of project or plan. TSPs will end up with multiple projects defined by short (0-5 yr), medium (5-10 yr), or long term (10 yr +) periods. Interchange Area Management Plans (IAMP) or refinement plans can also identify multiple projects. For projects to be considered officially they would need to be adopted into the TSP as part of the implementation of the corridor or refinement plan. Projects may also have multiple final alternatives to be analyzed further in a project development process. For more details on TSPs refer to the TSP Guidelines.

> *Solutions identified in TSPs should not be too specific and they should allow some flexibility as the improvement may be years off. An example of this overly specific language would be to specify "traffic signal" instead of a more general "intersection improvement" for a particular location in a TSP.*

Once a TSP is adopted by the local jurisdiction, amending the TSP will require new hearings, probably new analysis, and can take months or years for approval. TSPs in MPO areas may also require amending the RTP, RTSP, and re-working any air quality conformity especially if the "new" project is elevated into the fiscally constrained list. Sometimes if there are many issues it is best to specify a follow-on refinement plan to look at these issues in greater detail and to provide a more detailed solution.

10.8 Screening the Alternatives

At many points in the alternative screening process, there will be a need to apply different transportation analysis tools and methodologies to address the traffic-related criteria. Typically there are three levels or tiers of screening, Fatal Flaw (Purpose & Need), Goals & Objectives (Modeling), and Operations. Overall, in the broader NEPA analysis, the overall screening levels and approach are responsibilities of the Environmental Project Manager so not all of the following sections will apply or to the same level of detail on every project.

10.8.1 Fatal Flaw/Purpose & Need Screening

At the beginning of the process, many designs will be drafted up and the basic viability usually compared against "fatal flaw" criteria. These criteria are more than likely to be based on AASHTO or ODOT Highway Design Manual (HDM) or local design standards as applicable. This will eliminate transportation concepts that will not work on a geometric, policy, or general nominal safety basis. If a project has a purpose and need (P&N), then concepts are evaluated against it to see if they meet or generally meet the P&N with modifications. Concepts that do not meet the P&N are dropped. The concepts that pass the fatal-flaw and the general P&N screening also need to be evaluated on a transportation operation basis. For simple projects, the transportation screening can be a volume-to-capacity or other performance measure comparison at key locations. Urban areas typically have larger, more complex interdependencies that make a simple isolated point-by-point comparison insufficient. Travel demand models are one tool that allows for many concepts to be evaluated quickly and to arrive at a set of reasonable recommendations for forwarding onto the next step.

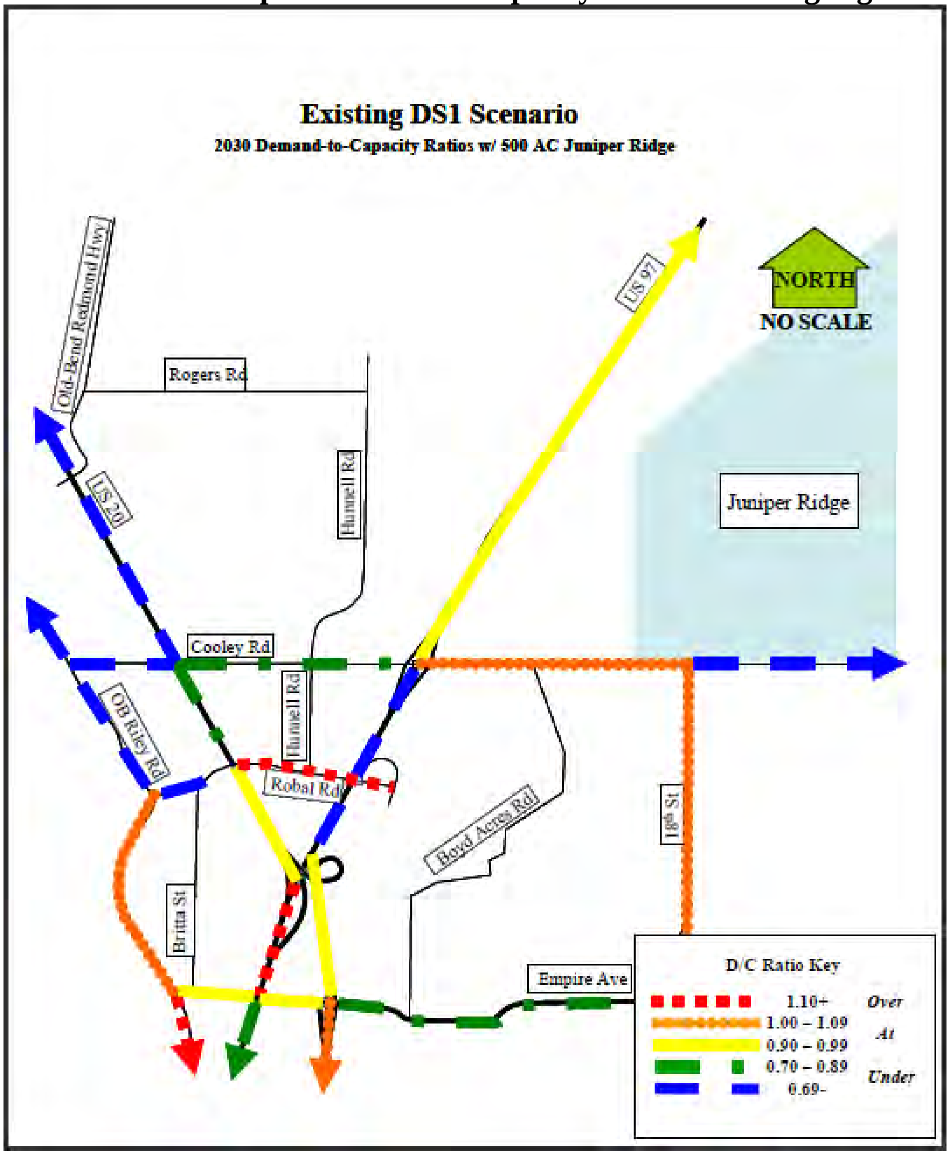
10.8.2 Goals and Objectives Screening

Goals and objective –based screening is more detailed and will focus on many different transportation elements such as mobility, safety, and operations and many non-transportation ones such as water quality, displacements, and historic resources. Typically this level is guided by evaluation criteria arranged in matrices. This is more objective and quantitative than the preliminary level.

Models can provide system level performance measures which can be useful in screening preliminary alternatives. Use of model outputs for preliminary screening can reduce the time and cost of full analysis of all alternatives and any variations. These initial assessments typically focus on more general performance indicators, such as d/c ratios on arterials and highways, d/c ratios across screenlines or approach volumes at major intersections and junctions. These findings can be useful for quickly assessing the general feasibility of a preliminary improvement concept and provide a basis for eliminating or further refining an initial concept. Tables and figures are preferred to summarize the issues rather than detailed text descriptions.

For example, a model scenario can be constructed for an individual design concept and a demand/capacity ratio plot could be requested to compare links on a relative basis to each other. The d/c ratios cannot be directly compared to the published OHP/HDM targets. Instead, they can be categorized as below (less than 0.70), near (0.70 – to 0.90), at (0.90 to 1.10) or over (greater than 1.10) capacity. Model links that are shown to be over capacity in a d/c plot have proven to be a good predictor of bottlenecks that are difficult to mitigate. Links that are at capacity generally can be addressed with mitigation, while links that are below likely will not have problems in the detailed analysis. Exhibit 10-3 shows an example from the US97 North Corridor Solutions project. It is preferred to show the d/c on a base map that reflects the actual roadway network with major street names shown. Model networks by themselves are simplified and may be difficult to tell locations apart. A simple graphical figure (note the use of colors and patterns so it can be discernable in black and white) such as this can quickly show the overcapacity areas that may or may not be addressable, which might be grounds for dropping an alternative.

Exhibit 10-3 Example Demand-to-Capacity Ratio Screening Figure



Use of screenlines to cut across multiple roadways at multiple points in different alternatives/scenarios can be used to compare the relative changes in volumes on those roadways or the effect of a specific issue/change on the overall travel patterns. For example, adding another river crossing would remove at least 25% of volumes from other roadways. Volumes are compared where study-area roadways cross the different screenlines to keep locations consistent between scenarios (See Exhibit 10-4 and 10-5). Use of shading in the result tables can quickly show the reader positives and negatives of the scenarios. Significant positive or negative changes (greater than +/-10%) can be used as justification to drop or forward a particular scenario.

Exhibit 10-4 Sample Screenline Locations

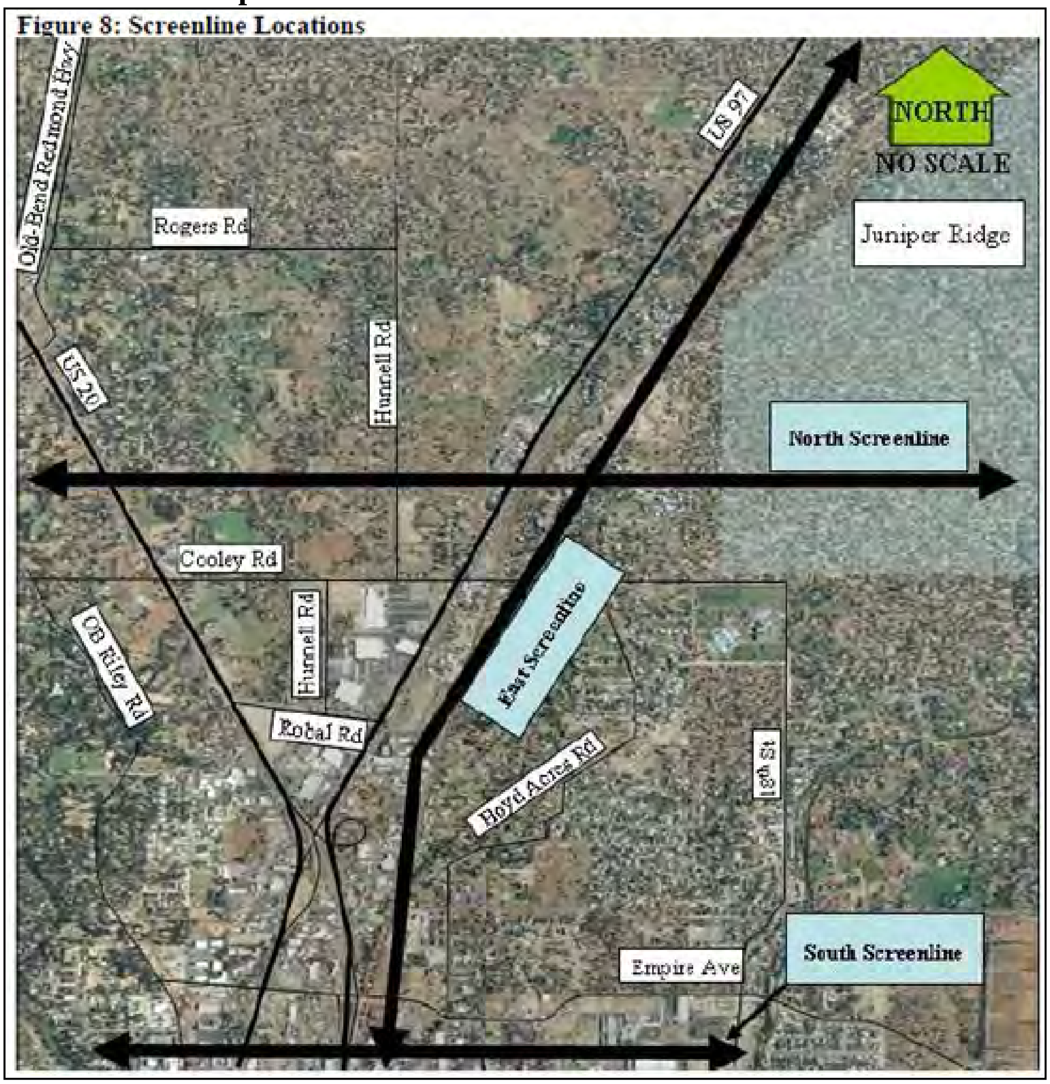


Exhibit 10-5 Volume Difference Screening between Scenarios

Table 1 North Screenline Percent Difference from Committed Scenario[1]

	US97		US20		Local Roads[2]	
Scenario	**w/o JR**	**w/JR**	**w/o JR**	**w/JR**	**w/o JR**	**w/JR**
Committed	0	**46**	0	4	0	**59**
MM-1	**-26**	**23**	-1	6	**-16**	**42**
RRA-1B	**-22**	**20**	**-21**	**-10**	4	**63**
RRA-2-2	**-29**	**-18**	4	**-13**	-2	**45**
RWA-1-2	**-35**	0	-7	0	2	**58**
RWA-3B-2	**-31**	**-35**	**-16**	**-19**	1	**60**
RWA-4	**-35**	**12**	**-16**	**-17**	-3	**34**

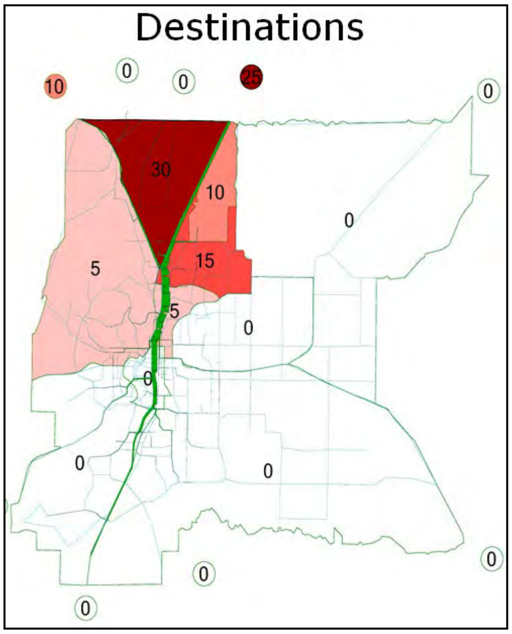
[1] Black shaded cells indicate a significant negative impact at the screenline because of increased volumes. Gray shaded cells indicate a significant positive impact at the screenline because of decreased volumes.
[2] Local roads include O.B. Riley and Deschutes Market Roads.

Summing volumes across a single screenline can determine if a scenario is simply shifting traffic between different roadways if differences are not significant (less than 10% additional volume) or maybe attracting traffic into an area. Significant traffic increases (greater than 10% additional) might indicate latent demand issues as volumes shift to study area roadways from other congested facilities nearby. These kinds of shifts may not be desirable (but might be expected) as they may require “improvements” to be larger than originally intended.

Summing volumes at study area boundaries can be used to determine whether a specific alternative creates significantly more vehicle-miles-traveled or greenhouse gases than others. Changes in other modes can use this method if the model is sufficiently detailed to represent these modes and is sensitive to them.

For larger regional areas, the model area TAZs can be aggregated into districts that represent areas such as a CBD or general sectors (i.e. West Salem or South Beach, Newport) or travel sheds (east county) or individual cities within an MPO model (i.e. Gresham from the METRO model). The general travel patterns between districts can then be determined. Using a select-link plot of the district-to-district flows, the analyst can answer traffic flow questions, such as percentage of through trips in the study area or distribution of trips to/from a specific location/district. Exhibit 10-6 shows the destinations aggregated by districts and external areas (circled values) from a location (where the volumes bars are the thickest – this is the location of the specific chosen link for the select-link analysis) on US97 north of downtown Bend.

Exhibit 10-6 District Plot of Destinations



Scenarios can also involve proposed or future land use changes along with network changes. This could be at the regional level all the way down to a specific development proposal. Models can be used to evaluate policy type questions such as land-use scenario planning with UGB expansions, nodal development, significant Comprehensive Plan changes and multiple growth scenarios. These changes between a base and a proposal should be modeled as referred to in the ODOT Modeling Procedures Manual for Land Use Changes. The results from these changes can be reflected in d/c plots, screenlines, or districts as shown above.

If the model is detailed enough, other measures can be screened. Mode split can be evaluated if the model is at a regional level. Models can also be used to evaluate policies other than land-use related to where parameters are included in the model such as restricting the overall capacity of arterials or changes to standards. Policies related to monetary issues such as parking, tolling, or VMT taxation require models with economic components. There are other models that are not travel demand-based such as land use, greenhouse gas/emission, and economic-based models that can be used in preliminary screening of related concepts.

Adding model travel times on the specific links that comprise a specified route can be used on a relative basis to determine the effectiveness of certain scenarios. Routes are typically determined on a shortest path by time method from a specific origin to a specific destination. Multiple routes can be averaged together to judge performance of a scenario as shown in Exhibit 10-7. The model travel times can be used to estimate emergency response times, freight trip times, school route trips, etc. on a relative basis keeping in mind that many models do not account for intersection congestion. These travel times can also be used as a surrogate for micro-simulation travel times when the origin or destination is outside of the project/study area.

Exhibit 10-7 Relative Average Model Scenario Travel Times

Table 7 Average Travel Time Percent Differences with the Committed Scenario[1]

Scenario	US20	US97	Overall Study Area
Committed	0	0	0
Juniper Ridge	8	**24**	**13**
MM-1	6	**-17**	0
MM-1 w/ Juniper Ridge	6	**-12**	1
RRA-1B	**22**	**-15**	**11**
RRA-1B w/ Juniper Ridge	**27**	-9	**18**
RRA-2-2	-1	**-18**	-5
RRA-2-2 w/ Juniper Ridge	**23**	**-17**	**12**
RWA-1-2	6	-8	2
RWA-1-2 w/ Juniper Ridge	9	1	7
RWA-3B-2	4	**-14**	-2
RWA-3B-2 w/ Juniper Ridge	6	-9	8
RWA-4	**10**	-9	3
RWA-4 w/ Juniper Ridge	**10**	-5	4

[1]Gray shaded cells indicate that travel time is significantly less than the committed scenario and black shaded cells indicate travel times that are significantly greater.

10.8.3 Operational Level Screening

Screening using operational-level measures is typically applied after fatal flaw or first cut screening such as using models. It generally is the third and final level of screening and involves a detailed evaluation of goals and objectives and is applied to a lesser number of alternatives. See Chapter 9 for details on performance measures.

- **Volume-to-Capacity Ratio**: This could apply to individual turning movements, average intersection conditions for all movements, roadway or highway segments, weaving movements and highway merge/diverge operations. This is the primary performance evaluation criterion for ODOT facilities.
- **Level of Service**: Many local jurisdictions use Level of Service ratings in their development code as performance criteria. Most facility evaluation methods provide both a v/c ratio result and a Level of Service result.
- **95% Queue Length**: Safety and operational impacts associated with the likelihood of a vehicle queue frequently blocking circulation or access. Use the 95^{th} percentile queue and compare it to the storage length.
- **Queue Blocking Percentage**: Generally applied to mainline travel lanes, this is the portion of the study period (percent of time) where standing queues block the advance of vehicles from the adjoining upstream intersections or block the entrance to turn lanes.
- **Other Operational Indicators**: Travel time (by corridor or by segment), travel time reliability, total delay and total number of vehicle stops.
- **Safety:** Screening for safety includes Highway Safety Manual (HSM) Part B methods such as critical crash rate and excess proportion of crash types which are detailed in Chapter 4. Other safety methodologies such as Crash Modification Factors (CMF), functional area, and spacing standards are also included in Chapter 4.
- **Multimodal:** Level of Traffic Stress methodologies can be used for screening pedestrian and bicycle systems. Multimodal Level of Service (MMLOS) methodologies may be

used to identify impacts to transit and may be used for pedestrian or bicycle modes. See Chapter 14 for further information.

- **Other Screening Measures:** Other typical screening measures that may be identified in evaluation matrices that may have traffic or design components include right-of-way, environmental (acres of impervious surface, air quality, noise, etc.), socio-economic (displacements, disproportionate impacts), emergency vehicle access, freight travel times, and access points.

10.9 Documentation of Screening Process - Alternatives No Longer Considered

As the screening process proceeds from the fatal flaw through the operational level, it is important to actively document the concepts and alternatives as they are eliminated. Frequently, concepts and alternatives change names, are combined with others, or completely dropped from consideration. It is very difficult to reconstruct this history after the fact due to the sheer number of concepts and alternatives that are typically developed in the project process. It is not uncommon that concepts/alternative naming conventions can change multiple times thus further obscuring their origins. Concepts can come from multiple sources – staff, consultants, project team(s), and the public. For each concept or alternative through each level of screening, the name/title, a detailed description including figures if available, and the disposition and reason (name change, combination, drop etc.) should be documented chronologically. Documentation is important, as alternatives that were previously dismissed may be re-introduced without realizing they were already dropped, potentially causing re-work or delay. Exhibit 10-8 shows an excerpt of the alternatives no longer considered in the US199 Expressway Plan traffic analysis report. Note that one alternative was dropped because it did not meet purpose and need, one was dropped as it was not unique, and one was dropped as it had the largest right-of-way and displacement impacts. This documentation is a critical appendix in traffic analysis reports and in any project that falls under an EA or EIS under the NEPA. While tracking of alternatives is the responsibility of the ODOT Environmental Project Manager, the traffic analyst should also document the alternative development.

Exhibit 10-8 Example Alternatives No Longer Considered Documentation

West 2 – This concept consists of a continuous two-way left turn from Midway Avenue to Dowell Road and included removal of the Willow Lane restriction. The CAC and PDT determined that this alternative did nothing to improve safety which was part of the purpose and need, therefore it was dropped.

West 3 – This concept has full median from Midway Avenue to Dowell Road except for three items: 1) Hubbard Lane improved south to Demeray Drive; 2) US 199 at Hubbard Lane is signalized; and 3) Dawn Drive is connected north to Redwood Avenue. All the components of this alternative were included in other ones, so this was not forwarded.

West 4 – The concept is referred to as a mini-couplet with a wide median (50'+) between Willow Lane and Midway Avenue with indirect left turns centered around Willow Lane and Midway Avenue. Hubbard Lane would be right-in/right-out only and a left-in only to Rogue Community College would be allowed. Dawn Drive would have a frontage road that would connect across from Arbor Ridge Drive with full access having stop-controlled medians. This alternative was dropped because it had the greatest right of way acquisition and relocations.

10.10 Final Alternative Selection

The project team will select a single alternative from the final group of alternatives, or a hybrid of alternatives, which could necessitate additional analysis. If the selected alternative is significantly different from the alternatives described in the Draft EIS then a Supplemental EIS will also be required so the analysis of all alternatives is consistent. For EIS projects, the Preferred Alternative may, or may not, be identified in the Draft EIS, however, a Preferred Alternative should be identified in the Final EIS. The Final EIS and Record of Decision (ROD) identify the "selected" alternative. For EAs, frequently only a build alternative and no-build alternative are evaluated, in which case the build alternative is typically considered the Preferred Alternative.

10.11 NEPA Projects – Post Draft EA/EIS

Following the publication of the draft EA or EIS, there is a required comment period, in which the local, state and federal agencies, stakeholders, and the general public may comment on the preferred alternative. Depending on the scope or level of controversy, additional time and analysis may be required to address the comments. Once all comments have been addressed, the final EA or EIS is published. After the comment period and the resolution of comments, FHWA will either issue a Finding of No Significant Impact (FONSI) for an EA, or require an EIS if there is a significant impact, or a ROD for an EIS. Once a FONSI or ROD is secured, the project is eligible for obtaining federal funds and may proceed into final design and right-of-way purchase. TSP amendments and IAMPs are considered land use actions and need to be completed before issuance of a FONSI or ROD. For more information visit ODOT's NEPA Coordination webpage.

10.12 <u>Potential Solutions</u>

10.12.1 Purpose

This section is intended as a general summary of a representative range of practical solutions for ODOT plans and projects. This guidance is not intended to duplicate or conflict with ODOT design guidance such as in the Highway Design Manual or Traffic Manual. The analyst will frequently need to refer to these manuals for more detailed guidance and needs to coordinate early on and closely with Region Tech Center and headquarter staff. Many solutions such as those involving new or modified traffic control devices are subject to review and approval of the State Traffic Engineer or Region Traffic Engineer/Manager as discussed the Traffic Manual.

The solutions identified in this section are not an exhaustive list but a reasonable starting point. Solutions can be for the HDM standard 20-year design life or can be shorter interim or incremental improvements if a design exception or concurrence is approved.

All alternative solutions for plans and projects need to be reviewed by Region Roadway/Traffic or headquarters Traffic Engineering/ Roadway Engineering Section staff for reasonableness, need for design exceptions, other preferred options, or other potential concerns. This will be especially needed if representatives are not already part of the technical advisory committee or project development team. Alternatives should follow ODOT design standards or the project team can seek a design exception. Alternatives not meeting design standards need to have a design exception approved for projects within five years of construction. For projects between 5 and 10 years from construction, an indication or concurrence is needed from Roadway Engineering in Technical Services that a design exception would be approved.

Potential solutions to address existing or future deficiencies can range within the following categories:

- Transportation System Management & Operations
- Potential Land Use or Regulatory Changes
- Access Control and Local Circulation Improvements
- Multimodal & Intermodal Improvements
- Safety Solutions
- System Improvements
- Segment Improvements
- Intersection Improvements
- Interchanges

In general, the analyst should first consider the least impact to existing development, natural systems and cost, then progress towards improvements that have potentially larger investments and associated impacts until the identified need is resolved. The impacts of long-term maintenance and other life-cycle costs should be considered when choosing between solutions.

This includes cost for power for signals and illumination, software upgrades for dynamic message signs, and even extra emissions from standing vehicle queues.

Many of the solutions in the following sections can either be stand-alone or interim projects. Interim solutions can be used to delay or phase in the implementation of more complex projects. Solutions should be phasable and limit throwaway (improvements needing to be replaced or reconstructed in the near future, inconsistent with the long-term design) for most efficient use of funds. Solutions need to strike a balance between safety, operations, and multimodal as it is unlikely that full standards for all areas will be achievable. For example, pure mobility-based solutions may adversely affect safety as speeds and crossing widths increase. Many solutions need to be evaluated as part of a larger system or corridor to capture potential effects. For example, downstream intersection spacing may cause backups into a signalized intersection or roundabout. Study areas should generally cover a larger area than the solution itself, at least to the next intersection or interchange and in some cases further. Bottleneck improvements should be evaluated to ensure the bottleneck is not just moving to another location.

Many solutions may overlap one or more of the categories discussed below. For example ramp metering could be considered a TSM, TDM, and an operational strategy.

10.12.2 Transportation System Management & Operations (TSMO)

TSMO strategies are covered in more detail in APM Chapter 18. The following summarizes the basic types of TSMO strategies.

Travel Demand Management (Transportation Options)

The future analysis may also include elements that modify the initial travel demand that are expected in the future no-build forecasts. There are many techniques and programs that effectively manage future traffic demands, both on a temporal and modal basis, to work towards reducing the overall travel demand within the project area. The initial assessment of the project area should consider solutions that do not require physical improvements to the transportation system. Travel demand management generally includes the following types of programs and services that can marginally reduce the estimated travel demand where these types of programs are not in place. In general, these types of programs are most suitable for urban areas where commute traffic represents a significant component of the study period flows. Common demand management techniques could include:

- Increase or enhanced transit services.
- Carpooling/ridesharing
- Transit fare subsidies
- Flextime/compressed work week
- Bike parking/on-site lockers and showers
- Telecommuting
- Parking management can range from time-based measures to increase turnover to cost-based strategies to manage long-term monthly parking demand.
- Comprehensive Travel Demand Management (TDM) programs applied to larger employment centers that increase auto occupancy, bus ridership and help to spread out

the peak demand levels for a given site.

It is recommended that the alternatives development process consider TDM components that can augment physical or operational improvements within the study area. Refer to Chapter 18 for more details about TDM options.

The effectiveness of these types of programs can be assessed based on surveys conducted for the Employee Commute Options Rule compliance. Typically, these measures can reduce commute travel demand for a given activity center by 1 to 10 percent or more, if the management takes aggressive measures. For more details, refer to the 1996 study[2] that assessed the marginal reduction in traffic generation associated with various TDM options.

Transportation System Management (TSM)

TSM are improvements that do not require additional right-of-way and are relatively low in cost. As such, TSM solutions are sometimes implemented as interim projects prior to construction of final solutions. Substandard performance at highway intersections can be addressed by adding capacity to critical movements or upgrading the traffic control schemes to serve higher demand levels. These types of improvements need to consider multiple time periods of the day instead of just a single peak period. For example, turn movement patterns could be different between the morning and afternoon so a given lane configuration may not work well in all cases. The range of potential solutions includes:

<u>Reconfiguring Lanes</u>

This involves revising existing lane designations. An example would be revising a two-lane approach, where you have a shared left/through lane and an exclusive right turn lane into an exclusive left turn lane and a shared through/right lane. This may or may not involve phasing changes at a signalized intersection.

<u>Signal Phasing</u>

This involves signal phasing changes such as adding a right turn overlap or adding a U-turn, converting left turn phasing Protected/permissive signal phasing, changing cycle lengths, split times. Effects on signal progression should be evaluated.

<u>Added Turn Lane Without Widening</u>

An example would be converting available shoulder or parking space for use as a turn lane. Potential impact on bicyclists needs to be evaluated.

<u>Roadway Reconfiguration</u>

A roadway reconfiguration is typically a conversion of a four-lane facility to a three-lane facility

[2] Guidance for Estimating Trip Reductions from Commute Options, Oregon Department of Environmental Quality, August 1996.

having one through travel lane in each direction plus a two-way left turn lane. This involves reallocating roadway space to improve safety, operations and encouraging multimodal. Roadway reconfigurations can range from simple restriping on a preservation project to a full reconfiguration of the roadway including hardscaping improvements such as curb extensions and other pedestrian and bicycle and parking improvements. A simple restriping may not achieve the desired goals of creating a more multimodal environment.

Some reconfigurations are built as interim projects prior to widening the facility to five lanes. The two-way left turn lane improves through movement flow by removing turn movements from the through lanes if there are a significant number of left turns. However, applying a reconfiguration to a roadway with few turning movements may adversely impact through movement flow. Many times there is enough leftover space to accommodate multimodal facilities such as bike lanes, wider sidewalks, bulb-outs, etc. Items to consider include:

- The evaluation needs to include a 20-year analysis to assess the design life of the facility. This needs to include a full predictive-level safety and multimodal analysis including parallel facilities. If the 20-year HDM mobility standards are not met, the project team may seek a design exception. Depending on the project scope and context, a 10-year interim analysis may be considered.
- Estimates of diverted volumes and evaluation of potential impacts from diversion onto parallel facilities. See Chapter 6 for analysis procedures.
- Assessment of the potential magnitude of a shift in modes, if possible, such as by using an MPO travel demand model.
- Reconfigurations through small cities need to consider effects on passing opportunities on facilities leading into or away from the community.
- If on a designated ORS 366.215 freight route, the evaluation process in the ORS shall be followed; see ORS 366.215 Guidance. Early involvement of freight and active transportation stakeholders is necessary.
- Parking considerations need to include the impacts of removal and/or replacement of parking either on both sides, on one side or even a few spaces, the impact on maneuvering into and out of parking spaces, and potential impacts on the safety of bicyclists.

Freeway Auxiliary Weaving Lanes

These are lanes added between closely spaced interchange on and off-ramps which improve operations by reducing impacts of weaving, entering and exiting traffic flows. These typically extend from one interchange to the next or through several interchanges. While auxiliary lanes can improve operations on freeways by keeping local trips off the freeway through lanes, longer auxiliary lanes may cause drivers to assume it is an additional travel lane. See Appendix 10A for analysis procedures on determining whether a weaving lane functions as an auxiliary or a mainline lane. These can result in problematic weaving forms, especially if lengths are short or if one or more lane changes are required which need to be evaluated using procedures in APM Chapter 11. Weaving sections that do not require lane changes to remain on the mainline are preferred. These may encourage short-hop local trips which are not desirable but may not be avoidable due to lack of parallel facilities. Shoulder width reductions should be evaluated for safety and capacity impacts.

Extension of Freeway Acceleration or Deceleration Lanes

These will generally improve operations and safety by allowing more room for vehicles to enter and exit the through traffic streams. The length should be sufficient to allow the design vehicle speed to be within 10 mph of the posted speed. Refer to the HDM for proper spacing of acceleration and deceleration lanes.

Active Transportation Demand Management (ATDM)

Chapter 18 and its appendix provide detail on specific ATDM solutions, strategies and considerations for application. Some of these strategies may require legislative changes such as hard shoulder running. Ongoing operations, maintenance, and staffing costs can be a significant portion of an ATDM-based solution.

10.12.3 Potential Land Use or Regulatory Changes

Land use and/or regulatory changes are considered in planning rather than project development, for example IAMPs and TIAs. In addition, other planning actions taken by the local jurisdiction may have substantial effects on the initial horizon year forecasts that would reduce the future demand and partially (or fully) mitigate the identified need. These actions could include:

- Re-zoning land to generate fewer motor vehicle trips.
- Restricting the intensity allowed within the current zoning by imposing trip caps/budgets that are regulated by local ordinance. The trip cap is based on the amount of traffic a facility can handle at a decided-upon v/c ratio level.
- Supporting mixed use development that minimizes trips onto the roadway system. A potential tradeoff is that mixed use development may reduce trips region-wide but may increase the number of trips in the local area. This may potentially reduce the capacity available to the auto mode since the capacity will be used by walk and bike.
- Designation of a multimodal mixed-use area (MMA) by a local jurisdiction. This is for areas meeting specific characteristics as defined in TPR OAR 660-012-0060 that, once adopted, allows a local jurisdiction to approve plan amendments without applying motor vehicle congestion-related performance standards. Other performance standards such as safety still apply. Plan amendments within MMAs are still subject to other transportation performance standards or policies that may apply including, but not limited to, safety for all modes, network connectivity for all modes (e.g. sidewalks, bicycle lanes) and accessibility for freight vehicles of a size and frequency required by the development.

These actions require coordination with local agencies that are responsible for land use review and approval, and it may require a separate review and approval process to be implemented.

10.12.4 Access Control and Local Circulation Improvements

State facilities should be reviewed to compare background access provisions on state highways according to adopted standards in ORS 374 and OAR 734-051. Local facilities should be reviewed against locally adopted access management standards. Consideration of access management solutions requires close coordination with the Region Access Management Engineer. See APM Chapter 4 for more information on access management and related solutions.

Access management in state highway facility plans is addressed in ORS 374.331 and OAR 734-051-7010. The location of county roads and city streets within the area described in the facility plan is determined through collaborative discussion and agreement between the department and the affected cities and counties. For state highway facility plans that propose to modify, relocate, or remove existing public or private connections to a state highway, a methodology is developed which balance the economic development objectives of real properties with safety, access management and mobility and which inform the affected real property owners of the potential for modification, relocation or closure of existing private connections. The department develops a methodology to weigh the benefits of a highway improvement or modernization project to public safety and mobility against local TSPs, and land uses permitted in local comprehensive plans and the economic development objectives of property owners who require access to the state highway. Affected property owners may request a review through a collaborative discussion process, and/or an Access Management Dispute Review Board Process.

Access management for highway improvement projects in the Statewide Transportation Improvement Program (STIP) is addressed in ORS 374.334, OAR 734-051-5120 and PD 03. An access management strategy is developed for the project by the department in collaboration with cities, counties and property owners abutting a state highway. The access management strategy identifies the location and type of public and private approaches and other necessary improvements that are planned to occur primarily in the highway right of way and that are intended to improve current conditions on the section of highway by moving in the direction of the objective standards in ORS 374.311 and OAR 734-051. The strategy establishes the methodology by which private approaches will be considered for modification, relocation or closure and which balances the economic development objectives of properties abutting the state highway with the transportation safety, access management objectives, and mobility. Affected property owners may request a review of the methodology through a collaborative discussion process, and/or an Access Management Dispute Review Board Process.

Consolidating (or eliminating) existing vehicular access can substantially improve travel speeds and reduce vehicle and pedestrian/bicycle conflicts along the highway, improving safety for all users. Improving safety will also reduce non-recurring delays and will improve reliability. Reduced access will typically increase capacity to some degree as well. This requires coordination with affected property owners and implementation of necessary permits and easements to develop an alternative local circulation plan. This approach is most effective on a site that is making a development application and has substandard existing access spacing provisions.

In addition, the local agency could implement alternative local circulation plans that reduce the volume of traffic using the highway and shifts a portion of the local vehicle trips onto local roadway facilities. This can be accomplished through connecting circulation routes within adjoining uses across parking lots or via alleys, frontage roads and backage roads.

10.12.5 Multimodal/Intermodal Improvements

Refer to Chapter 14 for the multimodal analysis methodologies. All the improvements below have related design elements (see Section 800 & 900 of the HDM) and freight/design vehicle impacts. Ongoing maintenance such as slab replacement, grinding, resurfacing, cleaning of debris, removal of obstructions such as sign poles, protruding vegetation, poor driveway cross slopes, non-standard corner ramps needs be considered as part of the solution evaluation process.

Pedestrians

<u>Pedestrian Segments</u>

- **Sidewalks –** Sidewalks should be provided on both sides of the road and connected to other facilities in urban areas. Wider sidewalks need to be provided where pedestrian volumes are higher such as in pedestrian-oriented areas such as CBDs, plazas, transit centers, etc. When filling gaps, locations near or that connect to pedestrian-oriented areas should be a higher priority.
- **Buffers –** should be provided where possible to improve the walking experience. A wide sidewalk can act as a buffer as well. Buffers can include bike lanes, parking, street furniture zones, landscape strips, retaining walls, ditches, drainage swales, etc. Where possible buffers should be wide enough to support landscaping. Landscaping can include trees as these provide the lowest-stress walking environment but tradeoffs with sight distance, potential for fixed-object collisions, clogged gutters, broken sidewalks, etc.

<u>Pedestrian Crossings</u>

For pedestrians it is important to consider the availability and potential of having regularly spaced crossing opportunities. Pedestrian travel can be diminished if out-of-direction travel is too large or is potentially unsafe if improved crosswalks are needed but spaced too far apart.

- **Traffic Signal Phasing/Right Turn on Red Restrictions –** Including left/right turn protected movements can increase the overall safety level by removing potential turn conflicts. Restricting right-turn-on-red at crossings with high pedestrian volumes should be considered when right-turns can be adequately served with a separate phase.
- **Protected/Enhanced Crossings –** NCHRP 562 provides methodologies to evaluate pedestrian crossing enhancements. The road diet mentioned previously can incorporate many of the crossing enhancements below.
  - **Pedestrian Activated Beacons (PAB) –** user-activated traffic control devices with yellow flashing lenses that require vehicles to stop and yield to pedestrians at midblock crossings and uncontrolled intersections. PABs provide increased motorist awareness of pedestrian crossings, makes a lower stress crossing, particularly where signalized crossings are widely spaced or out of direction

travel is excessive. A specific type of PAB is the rectangular rapid flashing beacon (RRFB), which is an option available under FHWA Interim Approval IA-21. The Traffic Manual provides specific criteria for installation including speed, volume and spacing. Installation of these devices requires approval from the State Traffic Engineer.

  - **Pedestrian Hybrid Beacons (PHB) –** user-activated traffic control devices that provide a red signal indication requiring vehicles to stop for pedestrians at midblock crossings and uncontrolled intersections. Vehicles cannot proceed until the red indication turns off. A WALK signal is provided for pedestrians. PHBs are generally used for higher pedestrian volume and higher speed locations than PABs. The Traffic Manual and MUTCD provide specific criteria for installation including speed, volume and spacing. The installation of these devices requires approval from the State Traffic Engineer.
  - **Raised Pedestrian Refuge Medians –** allows for two-stage crossing of wider roadways, providing a lower level of traffic stress and the ability to cross higher volume roadways. May be combined with other techniques such as illumination, bulb-outs and beacons.
  - **Curb bulb-outs –** can reduce pedestrian exposure while crossing the street. Could constrict freight movements, particularly with full-width lane bulb-outs. It is important to be aware of vehicular composition and heavy vehicle turn patterns. Where substantial truck volumes exist, investigation of alternate routes should be evaluated.
  - **Turn Radius Reduction –** provides for slower right turning vehicles, reduced crossing distance, and creates improved visibility between drivers and pedestrians. However, the radius needs to be large enough so that large trucks or buses do not overrun the curb, which is a safety concern for pedestrians.
- **Grade Separated Crossings –** limited to the highest volume and speed roadways such as freeways and expressways where at-grade crossings are not permitted or where safety is an issue. The extra distance required to access the overcrossing due to the length of ramps needs to be considered, particularly if the structure is over an arterial. If the extra time required is excessive, pedestrians may cross at grade. Undercrossings are generally preferred but need to be well lit and have sufficient clearance and width to provide a natural walkway. Culvert-style passages generally deter use especially in urban areas because of personal security concerns. Grade separated crossings are usually an order of magnitude greater in expense than at grade crossings.

Bicycles

Low stress tolerant riders require a high degree of separation between themselves and the adjacent traffic lanes. While the addition of a bike lane to a facility may accommodate the bike mode, to achieve the highest modal share, greater separation is desirable. Generally the higher the speed and greater the volume, the more separation is desired. Separate facilities can include (from most separation to least):

<u>Bicycle Segments</u>

- **Separated/Multi-Use Paths –** provides a wide separation from a parallel roadway or can be in a wholly separated right-of-way such as a creekside greenway or rails-to-trails corridor and can serve both commuter and recreational functions. Out-of-direction travel should be minimized for access on and off these paths. Additional considerations/analysis is necessary where these routes cross roadways and will likely require crossing enhancements such as median islands, beacons, or potential grade separation.

- **Separated/Protected Bike Lanes –** Similar to a buffered bicycle lane but with some sort of physical separators such as posts, planters, or parked cars. There can also be vertical separation from the roadway grade or sidewalk. These are typically located on higher speed and or volume streets that also have a significant bicycle volume. Separated/protected bike lanes also need to be connected to other bicycle facilities and not as an isolated facility.
- **Buffered Bike Lanes –** A bike lane with a painted non-physical buffer area. These should be used wherever possible where the right-of-way allows as they provide a lower stress experience resulting in greater usage than a standard bike lane. Any angle parking should be changed to parallel parking to be compatible. Back-in angle parking may be appropriate in some circumstances such as on a one-way street.
- **Bike Lanes** - Caution should be exercised where bike lanes are adjacent to right turn lanes, especially on high-speed facilities or if the length is longer than approximately 200 feet. Buffered bike lanes should be considered in these situations to avoid the 'sandwich' effect. Any angle parking should be changed to parallel parking to be compatible, although back-in angle parking may be appropriate in some circumstances such as on a one-way street.
- **Wider Shoulders –** Applied in rural conditions for higher speed/volume facilities.
- **Wider Outside Travel Lane**. Not preferred but may be considered where right of way does not permit installation of even a narrow bike lane. Parallel routes/bike boulevards should be considered if proper on-street accommodation cannot be provided. By law, vehicles must give adequate room to bicyclists when overtaking.
- **Bike Boulevards –** while these are local functionally-classed (not for collectors or arterials) on-street facilities, through traffic is usually restricted and can offer a low stress experience. These should be parallel with major arterials/higher stress facilities and should not have too much out-of-direction travel in proportion to their overall length (less than 25%). These can be a good solution if the major roadway cannot be significantly improved but may not be a good solution if the destinations are on the parallel arterial.
- **Sharrows –** shared bicycle lane markings for lower speed (25 mph or less) and volume environments such as CBDs where bicyclists can feel comfortable traveling within the traffic stream. Bicyclists are supposed to take the center of the lane when sharrow markings are present.
- **Green Paint –** used to delineate bicycle facilities through intersections or other complex roadway arrangements.
- **Bike Warning Beacons –** Warning systems for narrow roadways across significant bridges and through tunnels where motorists would not necessarily be expecting a bicyclist. These may be activated by a bicycle push button or by a passive detection system or both.

Bicycle Intersection Treatments

- **Bike Boxes –** Previously experimental, now approved for provisional use (FHWA Interim Approval dated October 12, 2016)). At signalized intersections where there is demonstrated high volume of bicyclists, to allow bikes to proceed ahead of the motor vehicles.

- **Two-Stage Turn Boxes –** Used to facilitate bicyclist left turns, typically used in locations where bicyclists may have difficulty with weaving over multiple lanes of through traffic. Two stage turn boxes may require Experimental approval from FHWA. Contact Region Traffic or TRS staff if there is interest in using this device.
- **Bike Signals –** to separate conflicting bicycle and vehicular traffic flows in certain complex situations such as when a right lane drop is to the left of a bike lane, or where the bike lane needs to weave through the intersection. May require analysis of additional signal phase. Bike signals can have a frequent violation rate where bike signals are uncommon and/or volumes are low as bicyclists may be less willing to wait for the bike phase, so use of these devices should be limited to complex situations with high volumes.

Transit

> *Transit improvements suggested in plans and projects are subject to the availability of funding from the transit provider (includes transit districts, cities, counties, non-profits, tribes, school districts, colleges and universities, the state of Oregon, and others). Capital projects (stop improvements, transit centers) are funded separately from vehicle purchases (typically included in the STIP) and from operating budgets. Budget limitations may prevent new routes from being added or frequencies shortened for example. However, local, regional, state and federal funding sources such as grant programs may be sought. Coordination with the transit agency is required for any alternatives involving transit. For more information and assistance in coordinating with local transit agencies contact the ODOT Public Transit Section. A good primer on public transit in Oregon is the publication Transit in Small Cities.*

- **Transit (Bus) Signal Pre-emption –** Buses are detected by the system which can allow for earlier and/or longer green indications which minimizes delays.
- **Bus Stop Pullouts** should be downstream (far side) of intersections to minimize impacts to through traffic. Mid-block pullouts can be considered especially where there is a mid-block pedestrian crossing. Some transit agencies prefer not to have pullouts. Pullouts should be considered in locations with high boardings (longer dwell time for passengers to pay for tickets from the bus driver) or bus transfer locations where a bus may wait for passengers transferring from another bus route.
- **Busways/Guideways (Bus Rapid Transit or BRT) –** Buses run in their own separated bus way either in the median, outside travel lane, or parallel to the roadway. Stops usually have higher platforms and are double-sided as both directions are separated. Stops are generally further apart and serve higher volume locations. Delay and interference with the rest of the traffic stream is minimized, which allows for a higher capacity and short headways (high frequency). These are for high passenger volume routes. Busways can also be completely on their own right-of-way and can be even grade-separated in places although the cost can approach the level of a light rail system in this case.

- **Bus-Only Lanes –** These are exclusive lanes for transit operations which might be found in central business districts, transit mall areas or even dedicated on/off ramps at interchanges for access to bus stops, park and ride lots, and transit centers.
- **Stop Improvements** - Benches, shelters, larger landing areas for mobility-impaired users, arrival/departure information, and illumination are amenities that can increase ridership or encourage riders to wait longer especially if frequencies are greater than 15 minutes. Most bus stops come in pairs and require crossing a street to access the opposite direction, so safe and nearby crossings are important.
- **Frequency Changes –** if the overall surrounding land use is supportive and if operating budgets can allow, additional frequency is one of the biggest improvements that can be made to improve transit service on a route to encourage/increase ridership.
- **Route Changes/Additions –** Routes are typically added either for coverage or for ridership. Coverage routes will generally loop through residential areas and likely will have lesser frequencies. Ridership routes will be on high ADT roadways and serve major pedestrian attractors and generators (downtown, schools, medical facilities, etc.). The analysis needs to consider the surrounding land use in estimating potential ridership.
- **Transit or Multimodal Hubs –** Creation of hubs with multiple transit routes, park and ride lots, bicycle racks, and good pedestrian and bike connections with the surrounding neighborhood can encourage more non-auto use in the community.

Freight

- **Local Truck Routes –** Used to reroute trucks out of a downtown or constricted areas to more suitable roadways. This may require improvements to the designated roadway. Requires coordination with the local jurisdiction as they usually establish the local truck routes by ordinance. If the local government is proposing to take trucks off a state highway they need to go through the ODOT Approval Procedure for Local Truck Routes.
- **Chain-Up Areas –** Used in areas with defined snow zones for large trucks and other vehicles to install or remove chains.
- **Climbing Lanes (Chapter 11, Appendix 10A) -** These are used on steep or long grades to maintain the traffic flow and speed by minimizing delay on the overall traffic stream. Unlike a passing lane, a climbing lane is not considered a capacity improvement. The length that extends over the crest needs to be long enough to accelerate the truck to within 10 mph of the posted speed. Driveways and intersections within the climbing lanes are highly discouraged. Evidence of heavy truck driving on the shoulder is a good indication that a climbing lane may be needed.
- **Truck Only Lanes –** These are exclusive lanes typically used to maintain truck speeds up when climbing grades in congested locations (an exclusive climbing lane).
- **Extending Green –** This is an operational improvement at signalized intersections to detect approaching trucks and extend the green time to reduce trucks stopping and re-starting, improving safety and efficiency.
- **Truck Aprons –** Paved areas used at roundabouts on approaches and on the center island and other locations to accommodate oversized vehicles. Curbs are of a mountable type that do not limit truck use but are uncomfortable for smaller vehicles. Broken signposts, bent signs, and broken curbs/sidewalks are indications that aprons may be needed.
- **Mountable Curbs –** Can be used on channelization islands, median barriers to allow for overrunning large vehicles or for emergency access. For example, an intersection may be

limited to traffic as a right-in-right-out but the mountable curbs allow for a fire truck to quickly access the side street without a lot of out-of-direction travel and a quicker response time.

- **Accommodations for Freight and Oversize/Overweight (OSOW) Vehicles** can include the above apron and curb allowances but should also consider needs for turning radius at intersections, impacts of curb extensions, painted medians in lieu of landscaped medians/ barriers, narrow lanes and other restrictions. Certain highway routes are covered by ORS 366.215 which requires coordination with freight stakeholders when a project proposes to reduce vehicle-carrying capacity, as described in Guidance for Implementation of ORS 366.215.
- Note that making freight accommodations (wider radius, limited curb extensions and the like) will have an impact on pedestrians and bicyclists, so these kinds of improvements need to be discussed in an open project team environment and coordinated with the various stakeholders and local jurisdictions.

Rail

> *Any alternative solutions within 500' of any rail line or railroad crossing (at/over/under) or where in-street rail running is present need to be coordinated with the Crossing Safety Team. This includes public or private roadway crossings, multi-use path crossings, and roadways or paths running parallel to rail lines. This also includes any ownership of the track including whether it is public such as ODOT or Tri-Met or a private railroad. It is important to coordinate as early as possible with the Rail Section as the potential rail crossing order coordination and application process can be very time consuming. Refer to the ODOT Rail website for contacts and more information.*

Public crossing improvements require rail crossing orders which can involve many elements. Some of the more common elements requiring rail crossing orders are listed below, although this is not an exhaustive list and any proposed improvements within 500 feet of the crossing need to be coordinated with the Crossing Safety Team. Most private crossings involve the property owner working directly with the railroad and do not require a crossing order, although coordination with the Crossing Safety Team is still necessary. In some cases a private crossing may need to become a public crossing which requires a crossing order. Usually to add a new public crossing, one or more existing public and/or private crossings may need to be closed.

- **Medians –** Prevent vehicles from going into the opposite lane to go around a down crossing gate. They can also be used to eliminate turning movements from streets and driveways that are too close to the railroad crossing or prevent movements that could create a standing queue across the tracks.
- **Quiet (Horn-Free) Zones –** These are zones which use of the train horn is prohibited except in emergency situations. These zones typically require improvements to crossings by adding gates, interconnects, and other safety improvements.

- **Signal Pre-Emption –** In order to clear vehicles potentially stopped on the tracks when a train arrives, interconnection is required for traffic signals located within 215 feet of the railroad crossing and should be considered for signals located further away depending on factors including traffic volumes, highway vehicle mix, highway vehicle and train approach speeds, frequency of trains, and queue lengths. For more information refer to Manual of Uniform Traffic Control Devices (MUTCD) standards.
- **Roadway Realignment –** Reducing the skew angle of the crossing which will reduce the potential of a bicyclist or motorcyclist from getting a wheel stuck in the flangeway and losing control. A ninety-degree crossing angle is recommended for better sight distance from the crossing user's perspective.
- **Street Closure** - Wherever possible it is preferred to eliminate at-grade crossings. This could be by construction of an over/under crossing or by closing the actual street crossing. Adding an at-grade crossing requires closure of one or more crossings elsewhere.
- **Spacing –** It is desirable to have adequate spacing from railroad crossings to nearby intersections and driveways to avoid queuing back into upstream intersections or roundabouts. Turn lanes at locations where the railroad crossing is at an intersection need to be long enough to accommodate the waiting queues, so they do not block into the non-stopped through lanes. An example of a roundabout near a crossing in a 25-mph speed zone would need to be approximately a minimum distance of 200 feet from a rail crossing.
- **Sidewalks –** Sidewalks and ramps must comply with ADA standards. Any deficiencies must be reviewed and addressed at crossings.
- **School Bus Pullouts –** Subject to Rail Section review and approval, it may be desirable for left-turn locations with school bus volumes where rail crossing exist on the receiving lane consider a school bus pullout if space permits, so vehicles behind the school bus in the left-turn lane do not get blocked in accessing the receiving lane when a bus has to stop in front of the track to open and close the doors as required by state law.

10.12.6 Safety Solutions

Many of the solutions listed in this chapter also have significant safety benefits. The All Roads Transportation Safety Program - Crash Reduction Factor (ARTS CRF) Appendix is an extensive toolbox that has specific safety solutions and considerations for both spot locations and systemic improvements. The ARTS CRF list should be the primary source for countermeasures on ODOT plans and projects. Systemic improvements really require application over a wider area (city, county, region, statewide) to have the full impact realized. Most APM analyses will be of spot locations as the traffic analysis process will indicate specific needs. The project context and site conditions will determine the overall impact of the safety benefit. Refer to Chapter 4 for safety analysis procedures.

10.12.7 System Improvements

A long-term vision for a corridor or system should be established, typically at the ODOT Region level. The OHP state highway and freight route classifications and designations should be considered. There may be more importance placed on mobility and freight movement for example. The long-term vision is used to determine the categories of improvements to be considered.

When evaluating potential alternatives in a corridor, the effects of potential changes to the facility type or function should be considered, such as whether the facility is interrupted flow or free flow. For example, for a rural high-speed corridor that is currently free flow, introducing interruption such as a stop sign or roundabout will change the facility type and may not meet driver expectations. Introducing any type of intersection control into a free flow section will reduce the capacity of the mainline dramatically – in some cases capacity can be cut in half.

Couplets

Couplets are one way to increase the capacity of a facility in an urban area without need to expand beyond the existing right-of-way. Typically two one-way opposing direction parallel roadways a block apart (can be more) designated as a single route. These can create better multimodal connections and facilities with lower stress levels as cross-sections are typically narrower. These can reduce the number of conflict points but may require more out-of-direction travel to access local destinations. The best implementation of this would be to use a second parallel street with compatible land uses (i.e. two commercial streets versus a commercial street and a residential street).

Bypasses

A bypass is a route that allows through traffic to pass a town center or other urban/congested area without interruption. These typically will have limited connections with other roadways, otherwise the bypass will no longer function as intended for the long term. Land use measures as part of a facility plan are generally needed to help preserve the function of the facility. Refer to the OHP 2003 Amendment on ODOT’s Bypass Policy for more information.

Street Grid

Creation of parallel streets to the state highway can improve connectivity, accessibility and allows local trips to occur off the main street. Walk and bike trips to destinations and to transit will be shorter than with street systems that have many dead-end streets. This will allow congestion on the main street to be reduced thus limiting capacity impacts. More urban areas may have a network of one-way streets in one or both directions that are usually in a central business district. These help to alleviate overall congestion and promote accessibility. Signal progression on one-way streets tends to have better results along with shorter cycle lengths, fewer phases, and less overall delay.

Signal Systems

A signal system is a series of signalized intersections that have coordinated timings that improve the efficiency of the traffic flow by reducing unnecessary starts and stops, fuel use and emissions. The system needs continual monitoring and periodic adjustments as traffic flows and patterns change. Systems come in a variety of configurations with increasing emphasis on demand responsive or adaptive systems that change based on current flows versus fixed time of day timing plans. These advanced systems require more detection and may be more complex and costly to install.

Roundabout Corridors

In urban or suburban areas a series of roundabouts may be considered in some cases as an alternative to a traditional corridor of coordinated signalized intersections. Potential benefits of a roundabout corridor include improved safety and reduced speeds as well as in conjunction with non-traversable medians to reduce driveway conflicts by providing indirect left turns. In rural areas where approach speeds are high and bike and pedestrian use is low, the design objectives are significantly different, so the addition of multiple roundabouts will reduce mobility for through travel which needs to be evaluated in the context of the vision and function of the highway.

Ramp Meters

Reduces the flow of on-ramps onto freeways to forestall total congestion on the mainline which increases travel speeds, improves safety, and reliability. For effectiveness, these need to be installed in groups of interchanges rather than at a single installation (unless filling in a system). Ramp meters break up platoons of traffic that can cause significant delays to mainline traffic. Ramp meters can have fixed on/off times or can respond dynamically to changing flows. These require ramps of adequate length for storage or may require ramp widening upstream of the meter to two or more lanes. Ramp meters can also have HOV or bus bypass lanes. These may create controversy (requires extensive public involvement) as it does delay time it takes to get onto the freeway and may create equity issues as lower income areas closer in are metered while higher-income suburban areas are not.

10.12.8 Segment Improvements

Additional lanes and roadways can improve capacity, reduce congestion, and improve flow, travel time, and reliability. Many new lane additions are special purpose such as auxiliary lanes, passing lanes, and high occupancy vehicle (HOV) lanes instead of full-length general-purpose expansions. However, capacity needs may still drive the need to expand the cross-section of a roadway especially for high volume or urban facilities. Solutions for other modes or operational strategies may serve to delay an expansion that will be likely needed in the future. Latent or induced demand will likely increase volumes on improved sections beyond the future no-build. These improvements can be combined with other multimodal improvements such as additional transit lines, or bus express lanes.

Multimodal needs may have a higher priority to be accommodated with the roadway cross-section depending on the facility type and project-area context, so new general purpose travel lanes may not be practical or desirable. The limits of the recommended improvements should consider operational and safety performance, study area intersections and the appropriate transitions back to the existing highway cross-section.

Added Travel Lane

The addition of travel lanes on a highway facility may be appropriate to serve forecasted travel demands. Within urban areas the cross-section requirements of the highway may be influenced by the approach and departure lane requirements at the major intersections, available right-of-way, the local environment, and other context-sensitive considerations (see Section 10.5). Outside of urban areas, added through lanes may be needed to serve forecasted long-range growth in nearby communities. It is preferable to avoid multiple lane additions and drops in succession by keeping a constant cross section instead. Trap (drop) lanes on mainline sections should be avoided where a travel lane unexpectedly becomes a turn lane.

Passing Lane

Lane additions on rural highways may be done to improve operations by enabling vehicles to pass by reducing delay and travel time. Safety is also improved as the chance for improper passing maneuvers, and the inherent risk of head-on crashes decreases particularly where passing opportunities are limited. Passing lanes should be approximately one-half to one mile in length as longer lengths tend to function like a multilane highway section (see Appendix 10A). The effects of a passing lane can improve operations like follower density up to several miles downstream depending on volumes (lower volumes have longer effective lengths). It is desirable to provide a passing lane every five miles as this is the typical downstream effective limit for a passing lane’s benefits. Driveways or intersections within the passing lane section are not desirable because drivers do not expect other vehicles to be slowing or stopping in the passing lane.

Collector-Distributor Roadways

Like auxiliary weaving lanes but these roadways run parallel to a freeway which connect entrance and exit ramps. These eliminate weaving maneuvers on the mainline by consolidating points to a single on and off connection to the mainline. These can be one or more lanes in width depending on the volume of ramps they connect to and can just span one or multiple interchanges.

Frontage/Backage Roads

Frontage and backage roads may be either one-way or two-way and are typically local access or service roads parallel to a highway. The purpose of these is to eliminate access points and related conflicts on the highway for safety. These are typically used as an access management improvement and will need to conform to the access management coordination process. A frontage road is an opportunity if there is plenty of right-of-way available, keeping in mind that intersections could be problematic if there is not adequate spacing from the highway. Backage roads may be more common at interchanges or in areas where the right-of-way is not restrictive or built up too much.

HOV (High Occupancy Vehicle) Lanes

HOV lanes are lanes designated for vehicles with two or more occupants to encourage carpooling and transit use. These may be tolled (High Occupancy Toll (HOT)) lanes to better control usage if allowed by legislation.

10.12.9 Intersection Improvements

An intersection traffic control study is needed when significant changes to an intersection are under consideration. The analyst should coordinate with Region Traffic and headquarters Traffic Engineering Section staff in preparing this study. Further guidance is provided in the Traffic Manual. Intersection safety performance should be a primary consideration in evaluating intersection control alternatives. The HSM may be used to evaluate safety performance of intersection control alternatives.

Improving Skew Angles

High intersection skew angles reduce sight distance and visibility for on-coming vehicles and pedestrians. Skew angles require drivers to look back over their shoulders. One set of turns is typically a shallow angle which creates a too-high turning speed and the possibility for encroachment as vehicles cut corners into other lanes and bicycle lanes which is very problematic for pedestrians or bicyclists . While the other corner set is very sharp and low speed, pedestrian visibility can still be a problem and may warrant a channelization island. Skewed crosswalks also increase the crossing time for pedestrians as well as creating longer delays to other movements in the cycle.

New Turn Lane

Turn lanes isolate the different turning movements from mainline volumes which can create additional capacity and improved safety by removing the turning vehicles from the through traffic stream. However, the additional width required can increase speeds, make sight distance more difficult, and increase the total crossing width for users. Guidance on right and left-turn lanes at unsignalized intersections is found in Chapter 12 and Appendix 10A. These are minimum thresholds only. It is important that all benefits and disbenefits of turn lanes are fully considered, including safety and multimodal performance. For planning level analysis at signalized intersections a turn lane should be considered as an option when turning volumes exceed roughly 150 to 200 vehicles per hour. On high-speed facilities right turn lanes at unsignalized intersections may create sight distance issues for vehicles on the stopped minor approach and should be buffered.

Channelized Right Turn Lanes

Typically, channelized turn lanes allow a right turn movement to be separated from the main signalized intersection to allow for an improved turning radius or to break up the total pedestrian crossing distance, thus limiting exposure. These right turn lanes can be yield or signal controlled (for dual right turns). Where possible, the channelized island should be a low-speed design to allow for maximum visibility of oncoming traffic and/or pedestrians.

Free-Flow Right Turn Lanes

When an exclusive right-turn lane volume approaches or exceeds 1,000 vehicles per hour and is not controlled by a traffic signal, the intersection can be modified to provide an exclusive receiving lane (add lane on the side street) that requires no merging with other movements. This results in a free-flow movement with no conflict points. This kind of arrangement is generally not recommended as it sets up a direct conflict with any pedestrians or through bicyclists unless it is in an area with no regular pedestrian or bicycle use such as may be the case in a rural highway-highway system connection.

Dual (Signalized) Left or Right Turn Lanes

If the volumes satisfy criteria, review the intersection geometry to determine if improvements are required on the receiving side of the intersection to adequately serve the extra approach lane. Typically a single left or right turn lane can carry about 300 vehicles per hour when intersecting another major cross-section. Higher volumes typically have major vehicle queue spillback and delay issues. It is preferred that the dual turn lanes are located on receiving streets that already have two lanes to avoid a lane drop. There should be no major driveways located just downstream from the dual turn lane to avoid creating a high lane imbalance and a poor operation not to mention the functional area overlap and related conflicts with closely spaced features.

Another example that can be less intuitive is when a left turn lane is suggested, the opposite side should also be considered for a turn lane since the cross-section on the receiving side needs to be widened anyway to align the through lanes. If the receiving roadway requires widening to accommodate the dual turn lane, then the downstream length of the receiving lanes must be considered for the lane utilization of the dual turn lane. These lanes could drop at the next major intersection or may merge at a certain downstream point. Coordination with the Traffic Engineering Section, Region Traffic/Roadway, or local jurisdiction is required to determine the termination point of the receiving lanes. This condition is undesirable and should be avoided where possible. Furthermore, the corridor needs of extra lanes between intersections may necessitate widening of the highway to add travel lanes to reduce merge/diverge and weaving issues between intersections. This is particularly the case in urban areas with closely spaced intersections. The approach and departure lanes at major intersections may dictate the cross-section of the highway between these major junctions.

When proposing dual turn lanes, impacts on crosswalks need to be considered. Adding a dual turn lane on the approach increases the exposure length for pedestrians. Closing a crosswalk on the receiving (exiting) leg may be necessary in some cases to eliminate multi-threat conflicts. However, closing crosswalks is not desirable as out-of-direction travel and delays is increased for pedestrians. If the crosswalk is left open, additional treatments such as signing, striping, signalization enhancements will need to be considered.

Typically a dual left or right turning lane at an intersection can carry up to 500 vehicles per hour. When forecasted volumes exceed this level, analysis of alternative solutions is needed. Alternative solutions may include improved adjacent accesses, better connecting linkages, interchange and signal phasing adjustments.

Excessive Intersection Size

When the width of an intersection leg starts to exceed approximately 110 feet curb to curb, further widening results in diminishing returns in terms of additional capacity, due to longer pedestrian crossing times and other factors. Pedestrian exposure is also unacceptable at this level even with multiple median refuges and channelized islands as total pedestrian delay and crossing distance makes the crossing undesirable. Bicyclists will also tend to avoid such an intersection as the overall number of lanes, distance and complexity will result in a high-stress environment which will deter all but the most tolerant riders. Also, when analysis appears to indicate a need for triple turn lanes, then other types of intersections or partial/full grade-separated solutions should be investigated.

Intersection Spacing

Intersections should be spaced adequately following the OHP spacing standards. At a minimum, there should be adequate distance to accommodate the upstream and downstream functional area requirements needed to minimize the potential of overlapping intersection conflicts and so that movements can be made legally. Spacing should also be large enough to accommodate excess demand to minimize queue spillback into upstream intersections. Signalized corridors need regularly spaced intersections to maintain progression and to allow for crossing opportunities for pedestrians.

Right-Turn Acceleration Lanes

Right-turn acceleration lanes are generally not allowed at at-grade intersections. In some situations can be considered where criteria in the HDM and Traffic Manual are satisfied, and approval is obtained from the State Traffic or Roadway Engineer through the design exception process. Used for right-turning vehicles joining the traveled way of the highway from a side street for the purpose of enabling drivers to make the necessary change between the speed of operation on the highway and the lower speed of the turning movement. These can also be used to help accelerate heavy vehicles to within at least 10 mph of the posted speed so adequate length is needed. These are not for use in urban areas as they are not compatible with nearby downstream intersections or driveways. Special considerations are required for cyclists and pedestrians. For more information and criteria refer to Section 500 of the HDM and Section 6 of the ODOT Traffic Manual.

Median Acceleration Lanes

Median acceleration lanes are used at an unsignalized intersection to allow left-turning vehicles to gain speed and merge with mainline traffic to improve intersection operations or safety, especially where no alternative routes exist (See Appendix 10A). Vehicles only must yield with one direction of through traffic and then accelerate to merge into the through traffic lane. These are typically installed where a signal is not desirable such as in a rural area. These are generally only used at a T-intersection. A roadway or access across from the minor leg would need to be converted to a right-in-right out to avoid conflicts with left-turning traffic. The left-turn acceleration lane needs to be of adequate length to accelerate the typical vehicle to within 10 mph of the posted speed. These also need considerable distance downstream to the next driveway or intersection.

Two-Way Left Turn Lanes (TWLTL)

TWLTLs are used to assist left turning movements into and out of driveways and to remove the potential of stopped vehicles from the through lanes. These are assumed to be a median technique instead of an auxiliary lane (see Appendix 10A). These can improve flow in the through lanes. These require properly spaced driveways so that turn movements cannot overlap or create the potential for a collision if two vehicles turn into the TWLTL at the same time for a two-stage movement. If there are no driveways in a section, then a TWLTL should not be used and a painted median used instead.

All-Way Stop Controls

If the side street approach to the highway carries roughly the same volume as the highway, an all-way stop control may be appropriate to reduce delays on the minor streets in cases where the existing controls are stop signs on the minor approaches only. This improvement needs to be coupled with a variety of advance warning devices such as signs, markings, beacons, rumble strips, etc. to avoid creating a safety issue with vehicles failing to stop. However, this solution should consider volume levels and any functional designations for priority freight movement on the highway. An all-way stop control is not recommended when freight movement is a priority, since it adds recurring delays on the highway regardless of volume levels. High volumes on all approaches generally will not have too much operational benefit as capacity will be exceeded. This can be an inexpensive interim solution for an operational issue especially if right-of-way or funding is constrained.

Roundabouts

The ODOT approval process and guidelines for consideration of roundabout facilities on state highways are contained in Section 509.3 of the HDM. In addition to the considerations shown in the HDM, the overall cost and right-of-way also need to be considered. Initial analysis (See Chapter 12) should start with a single-lane roundabout. Analysis needs to be progressive in adding right turn bypass lanes and additional partial or full circulatory lanes to avoid overdesigning. Multilane roundabouts are more complex and can lead to more driver confusion, improper left-turn exiting movements and additional conflict points.

Traffic Signal Controls

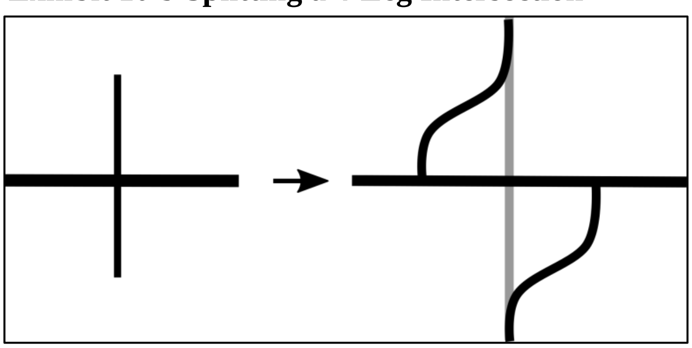
The ODOT standard intersection traffic control analysis (e.g. Intersection Control Evaluation) is required to justify new signal installations. It is important that all benefits and disbenefits of traffic signals are fully considered, including safety and multimodal performance. Issues to be considered include safety performance such as using methodologies from the HSM, traffic volumes, freight volumes, pedestrian volumes, and spacing relative to existing signal and the accepted standards for the highway facility. Traffic signals reduce the capacity by approximately half and increase delay of the mainline roadway allowing the side-street approaches to have more capacity and less delay. Traffic signals generally are not compatible in high-speed rural areas as they are not generally expected by drivers and could lead to high speed rear-end or angle crashes. Signals generally convert a lower number of fatal/serious injury high-speed angle and turning crashes into higher numbers of less serious rear-end crashes (still much higher than a roundabout). Queues from traffic signals may block upstream intersections and driveways impeding the flow of traffic onto or off the roadway. Signalized intersections also create lower stress locations for pedestrians and bicyclists to cross the roadway and ideally should be spaced so out-of-direction travel is minimized. However, complex intersections can make it more difficult for bicyclists to travel through, and long crossing distances can be intimidating to pedestrians. Shorter cycle lengths and fewer phases will have lesser delays and shorter queues versus more complex intersections with longer cycle lengths and more phases.

Other Intersection Types

<u>Offset T</u>

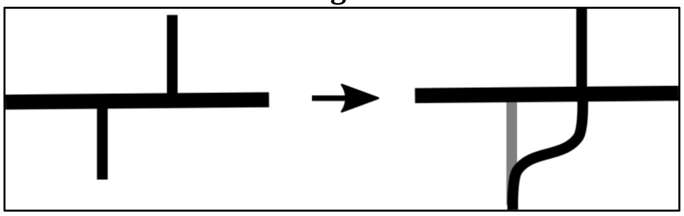
Offset T intersections may be formed by splitting a four-leg intersection into two three-leg “T” intersections to reduce the number of conflict points at each location as shown in Exhibit 10-9. The former side street straight through movement has additional out-of-direction travel as it now needs to turn left at one intersection and right at the other. The distance between the intersections needs to meet the functional area requirements and spacing standards.

Exhibit 10-9 Splitting a 4-Leg Intersection



This should not be confused with a pair of closely spaced T-intersections that may create overlapping turn movements and really should be combined into a single four-leg intersection, as shown in Exhibit 10-10.

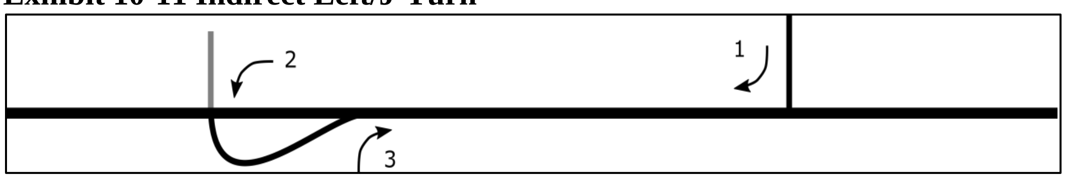
Exhibit 10-10 Combining Two Offset T-Intersections



Indirect Left/J-Turn

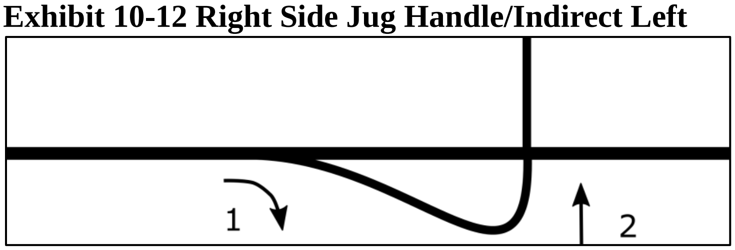
An indirect left turn or J-turn removes the conflict of the minor street left turn and through movements by creating a two-stage movement (one direction at a time) as shown in Exhibit 10-11. Minor street through movements would need to make an additional right turn plus potential mainline lane changes downstream from the two-stage left turn. The J-turn concept has vehicles crossing the second direction and then merging back into the traffic flow. J-turns are preferred when trucks are the design vehicle. Both types help facilitate installations of raised medians. The distance required downstream from the intersection to the indirect left or J-turn needs to be considered especially if there are multiple lanes and high speeds. An analysis using functional area (see Chapter 4) downstream values combined with uniform acceleration/deceleration equations can be used to estimate these distances. Vehicles will need to merge into, accelerate, change lanes and then decelerate into the left turn lane which could create substantial out-of-direction travel.

Exhibit 10-11 Indirect Left/J-Turn



A variation on the indirect left is the right-side jug handle. In this case, the major street left turn is replaced by an advance right turn followed by a through movement as shown in Exhibit 10-12.

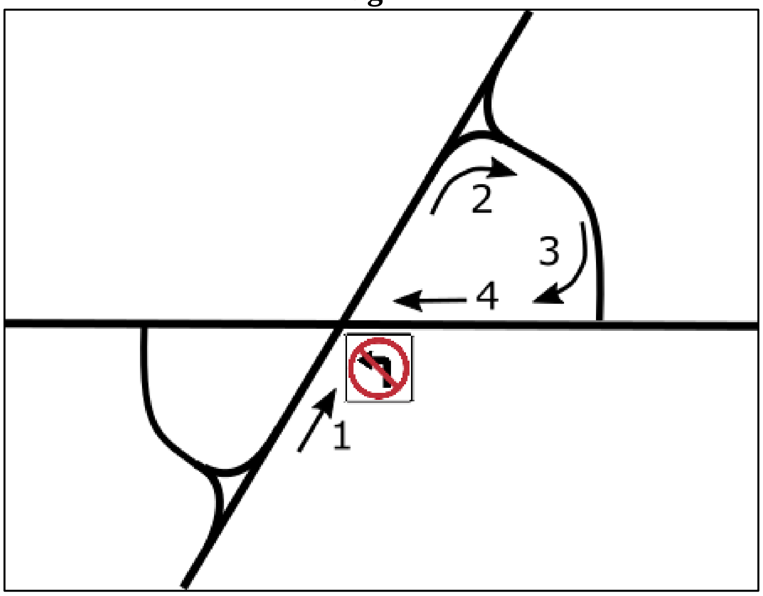
Exhibit 10-12 Right Side Jug Handle/Indirect Left



At-Grade Jughandle (One or More Quadrants)

This is a version of an indirect left as the left turn off the mainline is converted into a right turn followed by a second right turn and then a through movement as shown in Exhibit 10-13. It can also remove right and left turns from the main intersection turning the main intersection into a two-phase signalized intersection. The jughandle roadway could be one or two-way. If the mainline roadway had a median, then another jughandle would be required in the opposite quadrant. If both roadways had medians, then a jughandle would be required in all four quadrants. The main disadvantage of these is that the left turning traffic can travel twice through the intersection if right turns are only allowed onto the jughandle roadways from the mainline or if the jughandle is on the far side of the intersection. These are confusing to unfamiliar drivers who still may try to turn left at the main center intersection even with appropriate signing. These can also reserve right-of-way for a future interchange as these do take substantial room.

Exhibit 10-13 At-Grade Jughandle



10.12.10 Interchanges

When traffic volumes exceed these levels or if the functional integrity of the facility requires it, an interchange or grade-separated junction should be considered. This could take the form of an interchange or it could be a series of overcrossings on parallel routes to reduce the demand for the major arterials to a level that could be served by at-grade facilities.

Interchanges on highways are appropriate for all freeway facilities and most expressway facilities to reduce conflicts and to give priority to through movements on the state facility. ODOT and FHWA policies govern the different levels of interchanges which may be considered depending on whether a facility is an interstate, a non-interstate freeway or an expressway. Modifications to (or new) interstate freeway interchanges require a FHWA interchange modification request that is coordinated through the Traffic and Roadway Engineering Sections. For example, partial directional interchanges could be considered on expressways, but generally not on interstate freeways, although there may be specific locations where a partial directional interchange would be an appropriate treatment that would need to be approved by FHWA. In addition, some arterial locations may have grade-separated solutions when volume demands exceed the levels that can reasonably be served by an at-grade intersection.

Grade-separated configurations should be developed to serve the forecasted travel demands consistent with the layout and spacing standards recommended in the HDM. Refer to Section

600 of the HDM for more specific details that are useful in laying out interchange concepts. A planning level method for use in interchange type selection is available in HCM Chapter 23, as part of the analysis of interchange ramp terminals. The HCM method compares delay for signalized ramp terminals at diamond, SPUI, DDI, and partial cloverleaf interchanges, but does not evaluate two-way stop control or roundabout ramp terminals.

The following is a short review of the common elements of an interchange and a discussion of the conventional layout configurations that could be considered during alternative development as shown in Exhibits 10-14 through 10-23.

Ramp terminals should be developed to avoid spillback issues between the terminals or between the ramp terminal and nearby intersections.

Exit ramps and ramp terminals should avoid queueing onto the deceleration portion of the ramp. This should especially be avoided where sight distance is limited. Although not ideal, in some cases at signalized ramp terminals and on the crossroad special signal detection/timing ("dump loops") or other treatments may be appropriate.

Refer to HDM Section 600 and the Oregon Highway Plan for appropriate interchange spacing and spacing on the mainline between ramp junctions.

Ramp meters should provide for adequate storage of queued vehicles, avoiding spillback onto the crossroad ramp terminal. Dual lanes and/or bypass lanes may be appropriate in some cases.

Ramp Types

Jughandle Ramps (connection)

These ramps are generally used at low-level interchanges or grade-separated intersections, not for freeway connections and are characterized by low speeds. These generally start and end at an intersection and do not have any acceleration or deceleration areas. These have some sort of traffic control (signal, stop or yield) at the endpoints. They may be considered at major private approaches to a state highway. When used for non-interchange at-grade intersections they are termed connections as opposed to ramps.

Diagonal (Straight) Ramps

The carrying capacity of a ramp is determined by the conflicting movements at the ramp terminals. Typically a single lane straight ramp can carry 1,500 to 1,800 vehicles per hour. Ideally the ramp terminal spacing is great enough to allow for future loop ramps to avoid having to realign roadways in the future.

Loop Ramps

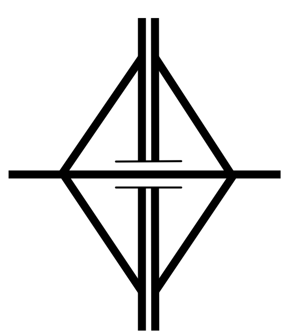
Typically a single lane loop ramp can carry 1,200 to 1,500 vehicles per hour. A loop ramp is appropriate to reduce left turning volumes at ramp terminal intersections. As noted above, when left turning volumes exceed 500 vehicles per hour, the typical at-grade intersection cannot generally accommodate it. For example, if a highway approach to a freeway interchange forecasted 700 left turns in the peak hour onto a freeway on-ramp, in most cases, the v/c ratio at this intersection would exceed guidelines. One solution would be to add a loop ramp so that this traffic demand could turn right at the intersection, in advance of the signal and loop onto the freeway rather than making a left turn, which requires a major share of the intersection capacity. These can be confusing to drivers as they turn in the opposite direction of expected travel. On-loops are generally preferred over off-loops, because of concerns regarding the speed differential between the off-loop and the mainline and difficulties encountered on off loops during adverse weather conditions. Loop ramps can also be problematic for pedestrians and bicyclists because of the higher speed diverge areas.

Directional Ramps

A directional ramp always bends toward the desired direction of travel. These are free-flow non-looping ramps that generally operate at high speeds for system movements. A semi-directional ramp exits a road in a direction opposite from the desired direction of travel but then turns toward the desired direction of travel. Many “flyover ramps” (as in a three-level interchange) are semi-directional.

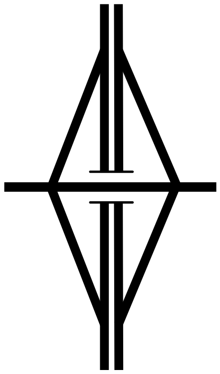
Interchange Types

Exhibit 10-14 Diamond Interchange



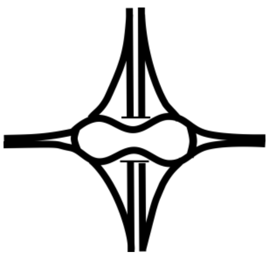
Diamond Interchange: An interchange that has straight ramps in all four quadrants is referred to as a diamond-shaped interchange. The capacity of this facility is typically determined by the operational analysis at the ramp terminals and merge/diverge areas on the mainline. The spacing of the intersections on the crossing street or highway will dictate the available vehicle storage and transition area. A standard diamond interchange has ramp terminal spacing greater than 800 feet. These would have adequate ramp terminal separation that could allow for future loop ramps. When volume forecasts are high at the terminal intersections and the spacing is limited, these could be factors that influence the need for an alternative layout concept. An operational analysis of the two ramp terminal intersections and any nearby intersections that could influence these locations will be required. The ramp terminals could also be separate roundabouts for interchanges. Some variations on the diamond interchange are described below.

Exhibit 10-15 Compressed Diamond Interchange



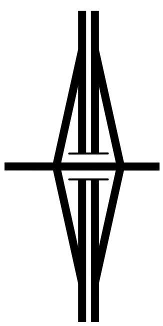
Compressed Diamond Interchange: A typically older interchange design where less than ideal ramp terminal spacing is present, between 400 and 800 feet. Sometimes the two ramp terminals can be operated with a single signal controller. Turn storage is done between the ramp terminals. Queue spillback between the ramp terminals is a common problem. Precludes any easy future construction of loop ramps. Some of these may have inadequate sight distance approaching and between the ramp terminals because of steep vertical curves.

Exhibit 10-16 Dog Bone Roundabout Interchange



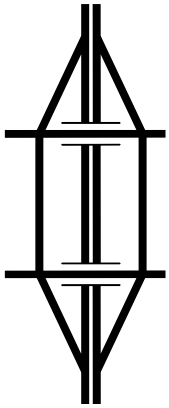
Dog Bone Roundabout Interchange: If roundabouts are desired as a ramp terminal treatment, then a connected "dog bone/peanut" style single roundabout as shown may be more feasible.

Exhibit 10-17 Tight Diamond Interchange



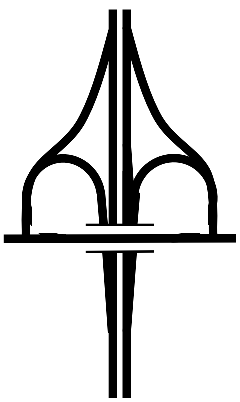
Tight Diamond Interchange: Typically found in urban areas, with ramp terminal spacing less than 400 feet. Requires signalized control. Usually the two ramp terminals can be operated with a single signal controller. Turn storage is done outside of the ramp terminals. Precludes any easy future construction of loop ramps.

Exhibit 10-18 Split Diamond Interchange



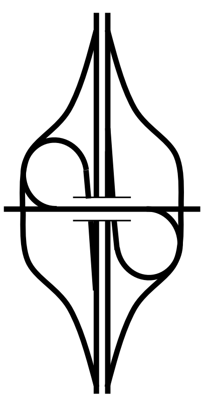
Split Diamond Interchange: Typically found on an urban grid system. Connections between each “half” of the interchange are one-way and are access controlled. Requires signalized control of all 4 intersections which must work together in both directions to avoid inordinate queuing effects.

Exhibit 10-19 Folded Diamond Interchange



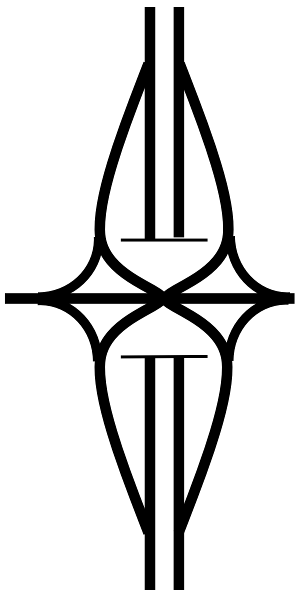
Folded Diamond Interchange: This interchange type “folds” one or two legs of the configuration to minimize impacts in one or two quadrants. Loop ramps can be located where topographical or environmental constraints adjacent to the interchange site do not favor the use of conventional straight ramps, e.g., where a railroad parallels the crossroad. Loop ramps that are located on the same side of the mainline facility can create weaving sections on the mainline or crossroad that may not be desirable.

Exhibit 10-20 Partial Cloverleaf Interchange



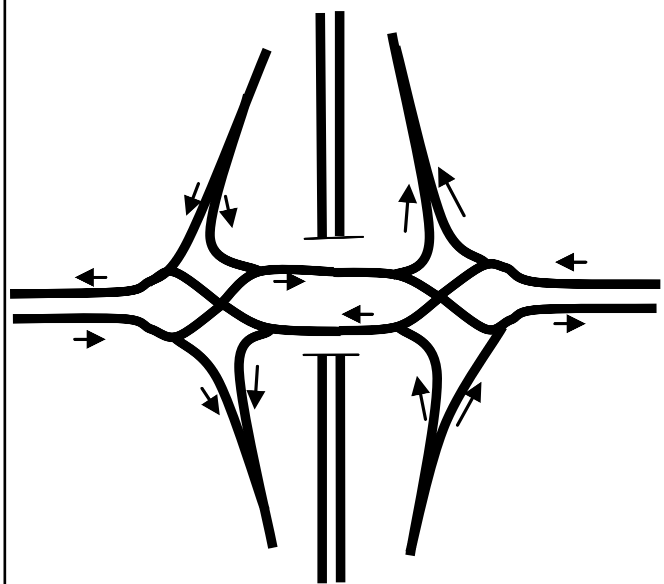
Partial Cloverleaf Interchange: A partial cloverleaf layout combines loop ramps and straight ramps to better serve areas with expected high turning volumes at the ramp terminals. In general, a partial cloverleaf configuration has a higher carrying capacity than a diamond interchange. The preferred configuration is where loop ramps are in opposite quadrants of the interchange. Loop ramps can also be recommended where topographical or environmental constraints adjacent to the interchange site do not favor the use of conventional straight ramps, e.g., where a railroad parallels the facility. Loop ramps that are located on the same side of a facility can create weaving sections on the mainline that may not be desirable.

Exhibit 10-21 Single Point Urban Interchange



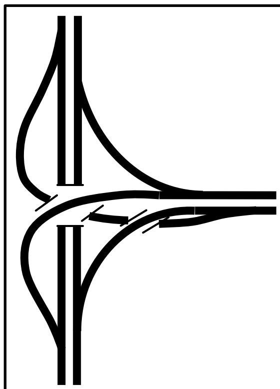
Single Point Urban Interchange (SPUI) also known as Single Point Urban Diamond (SPUD): The SPUI is a relatively recent development that evolved out of the need to limit ROW acquisition in built-up urban areas. SPUIs are a variation of the diamond interchange, which has two ramp terminals with the local arterial. A SPUI combines those two ramp terminal intersections into one larger intersection so that all turning movement to or from the freeway utilize the same intersection. The ramp terminal is typically signalized, although another option could be a roundabout. Having a single ramp terminal resolves the queue spillback issue that can congest standard diamond intersections and can be effective in serving high volumes of turning vehicle traffic. SPUI's need cross-street angles close to 90 degrees. High volume right turns may need to be signalized. SPUI's have nearly the same ROW costs as tight diamonds and the structure costs are often high. SPUIs are not very pedestrian-friendly as they do not allow for a crossing of the minor roadway. A crossing would have to occur at a downstream and/or upstream signalized intersection.

Exhibit 10-22 Diverging Diamond Interchange



Diverging Diamond Interchange: This is a new type of interchange design that has very few installations in the U.S. This form of diamond interchange has two directions of minor street traffic cross to the opposite side of the roadway under/over structure. This allows for two-phase signal operations since the left turns occur between the two signals in such a way that they do not cross the opposing through movements. Pedestrians are typically taken down the middle when the minor roadway is overcrossing the freeway and, on the outsides, when the minor roadway is an undercrossing. The advantage of the “down the middle’ approach is that pedestrians can stay on the same side of the roadway or cross to the other side in the interchange area efficiently.

Exhibit 10-23 Directional Interchange



Directional Interchange: This type of interchange is more common in urban areas or at junctions of freeways or expressways with other freeways or expressways. An example would be I-5 at I-205. They are high-speed high-volume connections with all free flow movements. There are configurations with full or partial trumpet or flyover.

11 SEGMENT AND FACILITY ANALYSIS

11.1 Purpose

Freeways and multilane highways serve more than half the vehicle miles travelled in Oregon (FHWA 2016). In urban areas, they serve as major commute routes and provide access to intermodal facilities, while in rural areas, they facilitate both intrastate and interstate movement of persons and freight. Recognizing their importance to Oregon's economy and quality of life, and the significant investment made in developing these facilities, the Oregon Highway Plan sets more restrictive mobility targets (i.e., lower volume-to-capacity [v/c] ratios) for these facilities than for other classes of roadways.

This chapter also provides procedures for the analysis of rural two-lane highways, which make up the largest percentage of the state highway mileage. These highways cover a wide range of geographical and topographical conditions and connect all parts of the state.

To assure that mobility targets and other key indicators of mobility are explicitly integrated into freeway and highway analyses in the state of Oregon, this chapter provides a range of procedures that are scaled to reflect analysis complexity, regional context, and study scope. These methods are anchored in national policy and guidance documents, including the *Highway Capacity Manual* (HCM) 7th Edition and its companion *Planning and Preliminary Engineering Applications Guide* (PPEAG). However, the methods have been customized through the incorporation of Oregon-specific default values and best practices that reflect the diverse nature of Oregon's freeway and highway operations, including differences between urban and rural facilities. The guidance in this chapter considers project context, project type, data availability, and level of effort needed to conduct an evaluation. The goal of the guidance is to balance resource needs and complexity with desired analysis outcomes and performance measures.

This chapter's methods can be used to evaluate the operations of freeways and the uninterrupted-flow portions of multilane and two-lane highways (i.e., roadway sections without traffic signals, roundabouts, or other forms of intersection control requiring highway traffic to potentially stop or yield). Two general categories of methods are provided: those applying to roadway *segments* and those applying to roadway *facilities.* Segments are sections of roadway with similar traffic demands and geometric characteristics, while facilities are composed of multiple contiguous segments.

Segment analysis is typically used to evaluate the v/c ratio of a given roadway section and as a prerequisite for performing a facility analysis. A facility analysis is typically used to evaluate other kinds of performance measures, such as travel time, travel speed, vehicle hours of delay, and measures of congestion, and to evaluate roadway operations when demand exceeds capacity, including identifying bottlenecks and the extent of queues. A facility analysis is also used for more specialized types of analyses involving work zones, managed lanes, travel time reliability, and active traffic management strategies.

To support planning for a future in which some of the motor vehicles on freeways are no longer human-driven, but instead are connected and automated vehicles (CAVs), this chapter includes guidance on adjusting the future capacity of freeway segments and facilities in planning scenarios where CAVs are assumed to be part of the traffic stream. This guidance is based on the capacity adjustment factors (CAFs) for freeway segments presented in the HCM, which were developed by a multi-state pooled-fund study led by ODOT. No guidance is presented for the effects of CAVs on multilane or two-lane highway capacity because no research has been conducted yet for these roadway types.

As of 2022, no vehicles were available commercially that met the definition of a CAV for the purposes of freeway analysis (i.e., a vehicle with an operating cooperative adaptive cruise control system that is capable of communicating with other vehicles and driving without human intervention in any freeway situation). The CAFs for CAVs are intended for use only in longer-range planning analyses at the broad brush or screening analysis levels (defined below). See Appendix 6B for more information about CAVs, including guidance on estimating the percentage of CAVs in the traffic stream in a future year, tables of CAF values, and example problems.

11.2 Overview of Analysis Levels, Applications, Methods, and Tools

11.2.1 Introduction

This section provides guidance on matching the appropriate level of analysis detail to the type of highway analysis being performed, along with guidance on the available tools that support different levels of analysis. Using a data-intensive analysis method for large-scale, long-range analyses may be an inefficient use of resources and may produce results that imply greater precision than is possible given the limitations of the available data. At the same time, using a method that is too generalized will not provide the detail needed to adequately evaluate alternatives and reach final decisions. Therefore, this section describes a range of methods that can be applied as a project progresses from identifying needs to developing, analyzing, and prioritizing alternatives, and finally to project development. It is recognized that the methods described in this section may not be applicable to every analysis situation that may be encountered; therefore, guidance is also provided for when alternative analysis tools, including traffic simulation, may be appropriate to supplement the analytical approaches presented in Chapter 11.

11.2.2 Analysis Levels

Section 2.3.2 introduced the concept of analysis levels. The analysis level used in a particular study depends on several factors, including the size of the study area, the amount of data already available or to be collected, the assumptions used to develop the input data (e.g., measured volumes versus 20-year traffic forecasts), the desired performance measures, and the intended use of the analysis results for decision making. The three levels applicable to freeway and multilane highway analysis are:

- **Broad brush,** a high-level roadway capacity analysis requiring minimal amounts of data and incorporating several assumptions or default values. The HCM and PPEAG refer to this analysis level as a *planning analysis* using *service volume tables.*
- **Screening,** a medium-level analysis applicable when more input data are available, but no final decisions about roadway design elements, traffic control, or project approval will be made because of the analysis. As the name implies, this level is often used to screen and prioritize several alternatives as part of a long-range planning effort. The PPEAG refers to this analysis level as a *planning analysis* using *simplified HCM methods* and/or the *HCM method with default values.*
- **Detailed,** a low-level analysis in which all or nearly all input data are known, and the analysis results will be used to make final decisions about roadway design elements, traffic control, and/or project approval. This type of analysis considers the widest number of factors that can influence roadway performance and generates the largest number of potential performance measures. The HCM and PPEAG refer to this analysis level as an *operations analysis when* the analysis is used to determine roadway performance. Other examples of detailed analyses include *design analysis,* where the HCM operations method is applied iteratively to determine the roadway geometry that achieves a desired roadway performance, and *analysis using detailed alternative tools,* such as simulation.

Exhibit 11-1 compares the ODOT and HCM/PPEAG analysis levels.

Exhibit 11-1 ODOT and HCM Analysis Levels Compared

ODOT Analysis Level	HCM/PPEAG Analysis Level	HCM/PPEAG Analysis Method
Broad Brush	Planning	Service volume tables
Screening	Planning	PPEAG: Simplified HCM method HCM/PPEAG: Operations method using default values
Detailed	Operations	HCM operations method (roadway performance, given known roadway/traffic characteristics)
	Design	HCM operations method (roadway geometry, given desired roadway performance)
	Alternative Tool	Simulation or other detailed alternative tool

11.2.3 Analysis Methods

This section summarizes the methods for highway analysis that are presented in detail in Section 11.3. The methods vary in their level of detail, the number of performance measures produced, and the roadway length analyzed. Alternative analysis tools, including simulation and tools that draw from national and private vendor databases, are also discussed briefly, because they can be used to address limitations of HCM-based analysis methods, to calibrate HCM methods, and/or to generate additional performance measures, among other potential uses.

Broad Brush

Tables providing generalized roadway capacity values are a broad-brush tool that can be used to quickly estimate a v/c ratio, requiring only knowledge of daily or peak-hour volumes, the number of directional lanes on the roadway, the generalized terrain (level, rolling, or mountainous), and the area type (urban or rural). These tables are developed directly from the detailed HCM method for determining capacity and apply default values for all required inputs not directly specified in the table. Thus, the accuracy of the results obtained from using such a table depends on how well the roadway's geometric and traffic characteristics match the assumptions used in developing the table. Results can be improved when more information is known about the facility—for example, the heavy vehicle percentage—as adjustment factors can be developed to reflect differences between known roadway characteristics and the assumed characteristics used to develop the table; this approach to using generalized capacity tables is taken in this chapter. Generalized capacity tables are provided for freeway and multilane highway facilities.

Screening

The screening methods presented in this chapter are simplified versions of detailed HCM methods, as presented in the PPEAG. Depending on the complexity of a given HCM method, the simplifications range from recommended default values to apply with the HCM method to separate methods that incorporate only the most important variables that influence the performance measure results. Screening methods are provided for all freeway and multilane highway segment and facility types.

Detailed

The detailed methods presented in this chapter are the methods given in the HCM 7th Edition. Detailed methods are provided for all freeway and multilane highway segment and facility types.

Alternative Tools

Alternative tools provide different approaches to estimating roadway performance than what is used in the HCM and PPEAG. They will be not discussed further in this chapter, which focuses on HCM-based methods, but the analyst should nevertheless be aware of their potential uses, including to address the limitations of HCM methods. Examples of alternative tools and their potential uses include:

- **Microsimulation.** Microsimulation models the behavior of individual vehicles in a traffic stream with the resolution of very short time intervals, as opposed to HCM-based methods, which model the aggregate behavior of the vehicles in the traffic stream over 15-minute periods. With appropriate calibration, microsimulation can also be used to evaluate conditions that are beyond the capability of HCM methods, such as complex interactions with a mix of driver behavior and vehicle types, or alternative performance measures. As a result, microsimulation can be a useful tool for addressing the limitations of HCM methods, for confirming the results of an HCM analysis, and for creating visualizations of roadway operations. At the same time, because of the effort required to set up and calibrate a simulation model, microsimulation frequently is not a cost-effective tool for planning and screening applications where many different facilities and/or alternatives must be analyzed. In addition, there are some applications that HCM-based methods can perform better than simulation, including estimating capacity for a given condition (simulation uses capacity as an input) and evaluating travel time reliability. Chapters 8 and 15 provide more information about mesoscopic and microscopic simulation.
- **Probe data analysis tools.** The range of tools available to evaluate existing or historical travel times/speeds and travel time reliability from commercial sources of probe data is described in Section 18.1.6. In general, each source of probe data has its own associated software tool for generating performance measures from archived travel time data. In addition, some tools exist that can process raw data obtained from any of the commercial sources to generate statistics and visualizations. These tools provide direct measurements of travel speeds and reliability (both for individual time periods and as averages), as opposed to the estimates produced by this chapter's methods. At the same time, the tools are incapable of directly measuring capacity (as only a sample of the overall traffic volume is used as probes), nor can they be used to forecast future performance. These tools work well in conjunction with HCM methods, providing existing condition information that can be used to calibrate HCM methods to local conditions, thus providing better future forecasts (see Appendix 11B for guidance on calibrating HCM methods). The use of probe data for calibration is particularly important for freeway facility and freeway reliability analyses.

11.2.4 Analysis Applications

Exhibit 11-2 summarizes the potential applicability of PPEAG methods, HCM methods, and simulation to five types of planning and engineering applications: regional transportation plans, transportation system plans (TSPs), corridor plans, refinement plans, and project development. These applications were first introduced in Section 2.3.1 and are described in more detail below.

Exhibit 11-2 Method Applicability to Common Transportation Planning and Engineering Applications

Analysis Level	Broad Brush	Screening		Detailed	
Analysis Tool / Application	Generalized Capacity Tables	PPEAG Simplified Segment	PPEAG Simplified Facility	HCM Operations	Simulation
Regional Transportation Plan	○	◐	●		
Transportation System Plan	○	◐	●		
Corridor Plan	○	◐	●		
Refinement Plan				◐ or ●	●
Project Development				◐	●

Note: ○ = problem identification, ◐ = evaluation, ● = detailed evaluation.

Regional transportation plans identify the long-term (20-year or longer horizon) transportation needs of metropolitan areas exceeding 50,000 population, develop potential projects or actions to address those needs, and prioritize recommended projects and actions in the process of developing a financially constrained plan. These areas typically have at least one (and often multiple) freeways and/or multilane highways within their planning area. Generalized capacity tables are useful for an approximate planning-level estimate of the capacity of these roadways; the future demand volumes generated by the regional travel demand model can be compared to these capacities to determine whether future problems may exist. The PPEAG's simplified segment methods can be used to evaluate identified problem areas, while the PPEAG's simplified facility method can be used to preliminary evaluate alternatives and to generate a broader range of performance measures than v/c ratios.

Transportation System Plans (TSPs) serve as the long-range transportation plan for a city or county, many of which do not have any freeways or multilane highways. Most cities that have a freeway within their planning area will only have one or two interchanges to study, but a county-wide TSP may have many miles of freeway or multilane highway to study. Like regional transportation plans, generalized capacity tables can be used to identify potential problems, the PPEAG simplified segment methods can be used to evaluate specific problem areas, and the PPEAG simplified facility method can be used for preliminary evaluation of projects being considered for the plan.

A **corridor plan** covers an entire highway (e.g., US 101) or a long-defined section of a route (e.g., Madras–Portland) to develop specific project recommendations to be implemented in the mid- to long-term. Like the previous two types of plans, generalized capacity tables can be used to identify potential problems, the PPEAG simplified segment methods can be used to evaluate specific problem areas, and the PPEAG simplified facility method can be used for preliminary evaluation of projects being considered for the plan.

A **refinement plan** studies a subarea in detail. The need for such a plan may have been identified through a TSP, where resource constraints did not permit the more-detailed analysis required to develop appropriate recommendations. Such a plan might include a single freeway interchange or a relatively short section of multilane highway but would be unlikely to study extended lengths of these roadway types (otherwise, a corridor plan would have been called for). HCM operations methods are appropriate for studying the interchange or multilane highway segment. Simulation can be considered to confirm draft recommendations, to address limitations of the HCM method, or to develop visualizations of key recommendations. But for many refinement plans, the HCM method may be sufficient even for a detailed evaluation.

At the **project development** stage, final decisions about roadway design features and traffic control are being made and planning methods are therefore inappropriate. HCM operations methods are appropriate for evaluating the project, with simulation used to confirm the results, develop visualizations, and/or address limitations of the HCM method.

11.2.5 Analysis Tools

Although a basic capacity analysis can be quickly performed by hand for an individual segment, analysis tools help speed up the analysis process, ensure accurate calculations (assuming accurate inputs), and generate additional useful performance measures. This section introduces the tools available (as of mid-2018) that implement this chapter's analysis methods, and compares the tools' capabilities, inputs and outputs, and limitations.

Tool Descriptions

The **PPEAG freeway planning tool** is a research-grade Excel spreadsheet that automates the PPEAG's freeway segment and facility methods. The spreadsheet was developed by NCHRP Project 07-22 and can be downloaded from the Applications Guides section of the HCM Volume 4 website (www.hcmvolume4.org, requires a free, one-time registration).

Highway Capacity Software (HCS) is commercial software for the Windows operating system that is developed, distributed, and supported by the McTrans Center at the University of Florida. HCS includes modules that implement the HCM's procedures for basic freeway and multilane highway segments, two-lane highways, freeway weaving segments, freeway merge and diverge segments, freeway facilities, and freeway and arterial travel time reliability.

FREeway EVALuation (FREEVAL) is open-source software developed by the Institute for Transportation Research and Education (ITRE) at North Carolina State University. It uses the Java programming language and therefore runs on both Windows and Mac computers. It implements the HCM freeway facilities method, which incorporates the methods for basic freeway segments, weaving segments, and merge and diverge segments. The tool also provides modules for the HCM's freeway travel time reliability and active traffic and demand management (ATDM) methods. The software and its users guide can be downloaded from the Technical Reference Library (Chapter 11) section of the HCM Volume 4 website (www.hcmvolume4.org, requires a free, one-time registration). FREEVAL-OR is a customized version of the tool that incorporates all the Oregon-specific default values identified in the APM. For more information on the use of FREEVAL-OR refer to Appendix 11E. Note: Java updates many introduce errors but the tool is still usable.

HCM CALC is software for the Windows operating system that was developed by SwashWare and the University of Florida Research Foundation. HCM CALC includes modules that implement the HCM's procedures for basic freeway and multilane highway segments, two-lane highways, freeway weaving segments, freeway merge and diverge segments, freeway facilities, and freeway travel time reliability. It is available for free through the Microsoft store for Windows 10 users.

Tool Comparison

While the tools described above automate some or all of this chapter's methodologies, they have different interfaces, outputs, and features. The differences between the tools are outlined in the tables below. Exhibit 11-3 compares the tools' general characteristics, including:

- Installation considerations such as cost, technical requirements, and level of national recognition.
- Staffing needs including level of training required, general complexity of the tool, and available resources and support; and
- User experience including the ease of entering inputs and exporting outputs.

Exhibit 11-3 Software Tool Overview, Resource Needs, and User Experience

Overview	PPEAG Tool	HCS	FREEVAL	HCM CALC
Tool Overview				
Source	hcmvolume4.org	McTrans	hcmvolume4.org	University of Florida
Cost	Free	License Fee	Free	Free
Operating system	Windows/Mac	Windows	Windows/Mac	Windows 10
Installation required	No (need Excel)	Yes	No (need Java)	Yes
Widespread use	Low	High	Medium	Low
Staff and Support Needs				
Learning curve	Low	Medium	Medium	Medium
Complexity	Low	Medium	Medium	Medium
Training available	○	●	◐	○
User guide	○	●	●	●
Instructional videos	○	○	●	○
Technical support	○	●	◐	◐
User Experience				
Copy/paste	●	○	◐	◐
Load/save	●	●	●	●
Import/export	○	●	●	○
Auto-fill	○	◐	●	○

Notes: ● = fully supported, ◐ = partially supported, ○ = not supported.

Exhibit 11-4 summarizes more detailed information about each tool that will impact the user, including:

- Available methodologies;
- How inputs are entered, including the ability to easily change the configuration;
- Available outputs, charts, and automated reports; and
- Specialized features that are not required for the HCM methodologies, but useful depending on the application, including maps, adjustment factors, different scenario analyses, and features that make analyses faster, such as built-in error handling and example files.

Exhibit 11-4 Software Tool Methodological Comparison

Features	PPEAG Tool	HCS	FREEVAL	HCM CALC
HCM Methodology				
PPEAG planning methods	●	○	○	○
HCM segment methods	○	●	◐	●
HCM facility methods	○	●	●	●
HCM travel time reliability method	○	●	●	●
HCM managed lane analysis	○	●	●	○
Input/Output Features				
Change spatial/temporal configuration	◐	●	●	●
Segment-level outputs	◐	●	●	●
Facility-level outputs	◐	●	●	●
HCM measures	◐	●	●	●
Charts/visualizations (core method)	◐	●	●	●
Charts/visualizations (reliability)	○	●	●	●
Charts/visualizations (ATDM)	○	○	●	○
Automated report generation	○	●	○	◐
Built-in scenario comparison	○	○	●	○
Additional/Specialized Features				
Load/save and shareable files	●	●	●	●
Facility graphic	○	●	●	○
Map interface (real-world coordinates)	○	○	○	○
Calibration (adjustment factors)	◐	●	●	●
CAV analysis	○	●	◐	◐
Built-in weather adjustments	○	●	●	◐
Incident scenario analysis	○	●	◐	◐
Work zone scenario analysis	○	●	◐	◐
ATDM method	○	○	●	○
Ramp metering	○	○	◐	○
Built-in error handling	◐	●	◐	◐
Example files included	○	●	●	○

Notes: ● = fully supported, ◐ = partially supported, ○ = not supported.

11.2.6 Matching Analysis Methods to Applications

The first consideration when selecting an analysis method is to think about how the results of the analysis will be used. If the analysis' conclusions and recommendations will be used in making final design decisions about roadway geometry or traffic control, or if the analysis will support a decision-making process such as issuing a permit or making a land-use decision, then a detailed analysis should be performed. On the other hand, if the purpose of the analysis is to identify future needs, develop potential alternatives, or prioritize a list of potential projects, then broad brush and/or screening (i.e., planning) levels of analysis may be more appropriate.

Once the general level(s) of analysis have been determined, the next consideration is the specific analysis tasks required during a project. Here, the goal is to use project resources efficiently by matching the analysis method for a given task to the precision required for the end result. For example, for a corridor study, a first step might involve analyzing many roadway segments along the corridor to identify existing and potential future problem areas. Later steps will confirm the problems and start to identify potential solutions. Therefore, a broad-brush method might be appropriate to start with, to rule out segments from further analysis that clearly have no operational problems.

Because broad-brush methods produce more approximate results than other methods, the operational threshold selected for ruling out a segment should be more conservative than with a screening or detailed method. In other words, when there is doubt about whether a segment would meet an operational standard, the segment should be retained in the analysis.

As the analysis progresses, more detailed methods can be used to confirm the initial results and to begin to identify potential solutions. Although these methods require more input data and more time per segment to perform, there are also fewer segments to analyze at this stage. Continuing with the example of a corridor study, a screening method could be applied to the segments identified as potential problem areas. Some of the “borderline” segments retained from the initial broad-brush method may be dropped after this stage, while the degree of the operational problem will be quantified for the remaining segments and potential solutions can begin to be developed. If the operation of a given segment cannot be analyzed due to a limitation of the analysis method (e.g., over-capacity operation or the inability of the method to analyze a potential solution), then increasingly detailed methods can be applied as required.

A key advantage of using the HCM-based methods described in Section 11.3 is that they produce consistent results across different analysis levels, if consistent inputs are used. Therefore, as a project progresses from identifying problems to identifying potential solutions to preliminary design, forecasted project outcomes should stay reasonably consistent. Of course, specific performance estimates will change somewhat over time as more detailed information is incorporated into the analysis.

11.3 Freeways and Multilane Highways

This section presents analysis methods for freeway and multilane highway segments and facilities. It begins by summarizing the types of data required for each method, along with potential sources of those data. The section then continues by describing segment-based methods; these methods produce v/c ratios and are a necessary first step for performing a facility analysis. The segment methods are followed by descriptions of facility-based methods; these methods can analyze over-capacity conditions over an extended analysis period and can generate additional performance measures, such as average travel speed.

Each segment or facility type includes methods for both screening and detailed analysis; the facility sections also provide a broad-brush analysis method. Each method is accompanied by an example demonstrating its use.

Exhibit 11-5 summarizes the input data required to calculate a v/c ratio for each of the methods presented in Section 11.3. Following the table, each input is defined, and one or more data sources identified for it.

Exhibit 11-5 Input Data Required to Calculate *v/c* Ratios for Freeway and Multilane Highway Analysis Methods

Input Data	Generalized Capacity Tables	PPEAG Simplified Segment*	PPEAG Simplified Facility	HCM7 Basic Segment	HCM7 Merge & Diverge	HCM7 Weave	HCM7 Facility
Volume-Related Inputs							
Mainline volume**	●	●	●	●	●	●	●
Peak hour factor		○	○	●	●	●	
Percent heavy vehicles		○	○	●	●	●	●
Terrain class or specific grade	●	●	●	●	●	●	●
Area type (urban/rural)	●						
Section/segment length			●				●
Ramp volume(s)**		●	●		●		●
Acceleration/deceleration lane length					●		●
Distances to adjacent ramps***					●		●
Volumes on adjacent ramps***					●		●
Weaving volumes**		●	●			●	●
Capacity-Related Inputs							
Mainline free-flow speed (FFS)†		●	●	●	●	●	●
Number of mainline lanes	●	●	●	●	●	●	●
Driver population		○	○	○	○	○	○
Ramp metering		○	○		○	○	○
Ramp FFS					●		●
Number of ramp lanes					●		●
Number of weaving lanes						●	●
Weave length		●	●			●	●
Jam density							○
Queue discharge capacity drop							○
CAV-Related Input††							
Percent CAVs	○	○	○	○	○	○	○

Notes: ● = required input, ○ = optional input (can be defaulted), empty = not required.
*Ramp inputs only apply to ramp sections; weave inputs only apply to weave sections.
**HCM facility method requires volumes by 15-minute analysis period.
***Six-lane facilities only (three lanes in each direction).
†If not observed in the field or estimated from the speed limit, will require additional geometric data to estimate, such as lane width, shoulder clearance, and interchange density.
††Freeways only.

Volume-Related Inputs

- **Mainline and ramp volumes.** Segment-based analyses require hourly volumes, which can be derived from annual average daily traffic (AADT) volumes, design hour (*K*) factors, and directional (*D*) factors for broad-brush and screening analyses, but preferably from shorter-duration (e.g., 5-or 15-minute) counts for detailed analyses. ODOT's TransGIS system (http://gis.odot.state.or.us/transgis/) provides AADTs and *K* and *D* factors for roadway and ramp sections; see Section 3.4 for additional sources of traffic count data. Freeway facility analyses require volumes for each 15-minute period of an analysis hour. The screening method for freeway facilities can apply a default traffic profile to generate 15-minute volumes from a known hourly volume.

- **Peak hour factor (PHF).** The hourly volume divided by four times the peak 15-minute volume. Screening analyses can apply a default value (see Appendix 11C), while detailed analyses will preferably calculate a PHF from a location-specific count.

- **Percent heavy vehicles.** The volume of trucks and buses, divided by the total traffic volume. If a segment contains a steep upgrade (see terrain class discussion below), then the mix of single-unit and tractor-trailer trucks (e.g., 30%/70%) is also required. The percent heavy vehicles can be obtained by adding the truck and bus percentages from TransGIS. The HCM 7th Edition contains lookup tables for grade adjustment factors as a function of percent grade, length of grade, heavy vehicle percentage, and different single-unit and tractor-trailer fleet mixes. Note that the HCM calls out recreational vehicles (RVs) as a separate type of heavy vehicle, but that ODOT uses the FHWA vehicle classification system, in which RVs are classified as either light trucks or single-unit trucks, depending on the number of axles.

- **Terrain class (level, rolling, mountainous) or specific grade.** Level terrain is any terrain (including downgrades) where trucks operate at or near the posted speed. Rolling terrain contains upgrades that cause trucks to slow, but not to their crawl speed. Mountainous terrain is only used in broad-brush and screening applications and contains upgrades where trucks operate at their crawl speed. For detailed analyses, any segment that contains a grade that is either (1) between 2–3% and longer than ½ mile, or (2) steeper than 3% and longer than ¼ mile, should be analyzed as a specific grade (where the slope and grade length are required) rather than as a general terrain class. Grade information can be obtained from ODOT's Vertical Grade Report.

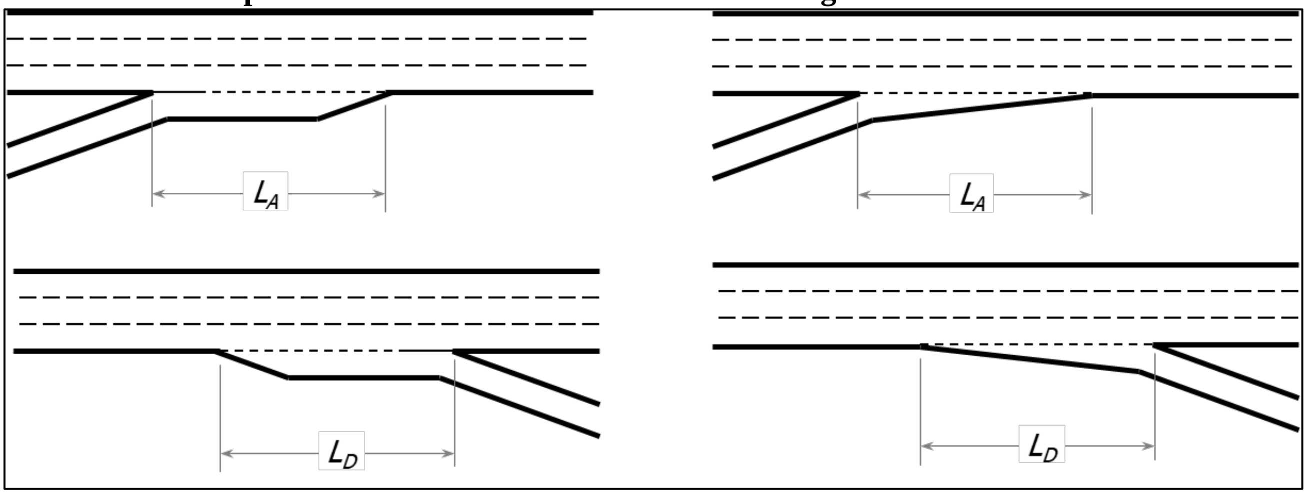
- **Area type (urban/rural).** The area type is determined from the FHWA's functional class for the highway segment. This information is available in TransGIS or in the detailed highway inventory.

- **Section or segment length.** See the individual methods for definitions. Potential sources of length data, depending on the segment type, include recent scaled aerial photos and the detailed highway inventory.

- **Ramp acceleration/deceleration lane length.** The distance from the ramp gore point to the ending/starting point of the taper (see Exhibit 11-6). This information can be determined from recent scaled aerial photos or design drawings.

- **Distances to adjacent ramps.** The distance from the ramp gore point to the gore points of the next upstream and downstream ramp. This information can be obtained from recent scaled aerial photos and the detailed highway inventory.
- **Weaving volumes.** See Section 11.3.3 for guidance.

Exhibit 11-6 Ramp Acceleration and Deceleration Lane Length Definitions



Source: HCM 7th Edition, Exhibit 14-5.

Notes: L_A = acceleration lane length, L_D = deceleration lane length.

Capacity-Related Inputs

- **Mainline free-flow speed (FFS).** See Appendix 11A for guidance.
- **Number of mainline and ramp lanes.** This information can be obtained from aerial photos, TransGIS, and the detailed highway inventory.
- **Driver population.** This is an optional input that describes the mix of drivers in the overall traffic stream, ranging from all familiar drivers to all unfamiliar drivers. See Appendix 11B for guidance.
- **Ramp metering.** This is an optional input that describes whether ramp meters are in operation during the analysis period and, if so, the ramp metering rate.
- **Ramp FFS.** For off-ramps, the posted ramp advisory speed can be used as the ramp FFS. For on-ramps, the smaller of the mainline FFS minus 10 mph or the design speed of the controlling curve on the ramp can be used as the ramp FFS.
- **Number of weaving lanes and weaving length.** See Section 11.3.3 for guidance.
- **Jam density and queue discharge capacity drop.** See Section 11.3.4 for guidance.

CAV-Related Input

- **Percent CAVs.** The percentage of vehicles in the freeway traffic stream with operating cooperative adaptive cruise control systems**.** See Appendix 6B for guidance.

11.3.1 Basic Freeway and Multilane Highway Segments

Screening Analysis Method

<u>Definition of a Basic Section</u>

Section H6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a simplified method for estimating the v/c ratio of a basic freeway section. A basic freeway section is defined as the portion of a freeway between an off-ramp and the next downstream ramp.

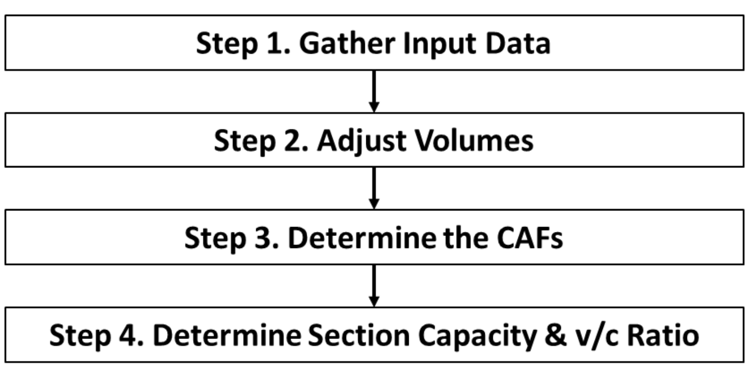
The same basic method, with a slightly different capacity equation, also applies to multilane highway sections. Section boundaries can include signalized intersections, major unsignalized intersections where highway volumes change significantly, lane adds or drops, and changes in terrain type.

<u>Applicability</u>

The method can be applied to basic freeway sections with a FFS ≤75 mph and to multilane highways with a FFS ≤70 mph. Multilane highways with signal spacing less than 2 miles should be analyzed as urban streets. The capacity adjustment for CAVs can only be applied to freeway sections and not multilane highways.

<u>Method for Calculating the v/c Ratio</u>

The following steps are used to calculate the v/c ratio for a basic freeway or multilane highway segment using the screening method:



Step 1: Gather Input Data. The following data are required; inputs in *italics* can be defaulted when not available:

- Hourly demand.
- The *peak hour factor.*
- The *heavy vehicle percentage.*
- The number of lanes.
- The *free-flow speed.*
- The terrain class (level, rolling, mountainous).

See Appendix 11C for Oregon-specific default values. The FFS can be estimated using the "roadway characteristics" method described as part of the detailed analysis method below, or as the speed limit plus 5 mph. See Appendix 11A for adjusting the FFS for differential truck speed limits or for mountainous terrain.

Step 2. Adjust Volumes. The traffic volume is converted to a 15-minute flow rate by dividing the volume by the peak hour factor.

Step 3. Determine the Capacity Adjustment Factors. Optionally, a capacity adjustment factor for driver population CAF_{pop} can be applied, as described in Appendix 11B. Otherwise, a default value of 1.00 is used for this factor. An optional capacity adjustment factor for CAVs CAF_{CAV} can also be applied to freeway sections, as described in Appendix 6B. Otherwise, a default value of 1.00 is used for this factor.

Step 4. Determine the Section Capacity and v/c Ratio. The capacity of a basic freeway section is determined using the following equation (derived from PPEAG Equation 16):

$$c = \frac{\left(2{,}200 + 10 \times (\min(70, FFS) - 50)\right)}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{pop} \times CAF_{CAV}$$

where

c = basic freeway section capacity (veh/h);

FFS = section free-flow speed (mph);

E_T = truck equivalency = 2 (level terrain), 3 (rolling terrain), or 5 (mountainous terrain);

$\%HV$ = heavy vehicle percentage (e.g., 6% = 6);

N = number of lanes (integer);

CAF_{pop} = driver population capacity adjustment factor; and

CAF_{CAV} = CAV capacity adjustment factor.

The corresponding equation for a basic multilane highway section is:

$$c = \frac{\left(1{,}900 + 20 \times (\min(65, FFS) - 45)\right)}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{pop}$$

The *v/c* ratio is then the flow rate divided by the section capacity.

Example 11-1 Multilane Highway Analysis (Screening Method)

Step 1. Gather Input Data. The multilane highway section being analyzed is located in a rural area with level terrain and has a FFS of 70 mph. There are two lanes in the analysis direction. The AADT is 26,900 with K=10.0 and D=55, and the volume includes 9.2% heavy vehicles. The driver population is familiar with the facility.

The AADT must be converted into a peak-hour volume by multiplying by the decimal version of the facility's *K*- and *D*-factors, resulting in a (rounded) volume of 1,480 veh/h.

Step 2. Adjust Volumes. The peak-15-minute demand flow rate is determined by dividing the peak hour volume by the peak hour factor. The PHF is unknown; therefore, the default value of 0.88 for rural multilane highways is used (see Appendix 11C or HCM 7). The resulting demand flow rate is 1,480 / 0.88 = 1,682 veh/h.

Step 3. Determine the Capacity Adjustment Factor. Because this section has a population of drivers familiar with the facility, CAF_{pop} = 1.00.

Step 4. Determine Section Capacity and v/c Ratio. Because the section is located in level terrain, a truck equivalency of 2 is used. The capacity of the basic multilane highway section is then:

$$c = \frac{\left(1{,}900 + 20 \times (\min(65, FFS) - 45)\right)}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{pop}$$

$$c = \frac{\left(1{,}900 + 20 \times (\min(65{,}70) - 45)\right)}{1 + (2 - 1)(9.2/100)} \times 2 \times 1.00 = 4{,}212 \text{ veh/h}$$

The *v/c* ratio is then 1,682 / 4,212 = 0.40.

Example 11-2 Freeway Analysis (Screening Method)

Step 1. Gather Input Data. The freeway segment being analyzed is located in an urban area with mountainous terrain, with a FFS of 55 mph. There are three lanes in each direction. The AADT is 160,000 with *K* = 8.2 and *D* = 52; the volume includes 4.1% heavy vehicles and no CAVs. The driver population is familiar with the facility.

The AADT must be converted into a peak-hour volume by multiplying by the decimal version of the facility's *K*- and *D*-factors, resulting in a (rounded) volume of 6,820 veh/h.

Step 2. Adjust Volumes. The peak-15-minute demand flow rate is determined by dividing the peak hour volume by the peak hour factor. The PHF is unknown; therefore, the default value of 0.94 for freeways is used (see Appendix 11C or HCM 7). The resulting demand flow rate is 6,820 / 0.94 = 7,255 veh/h.

Step 3. Determine the Capacity Adjustment Factors. Because this section has a population of drivers familiar with the facility, CAF_{pop} = 1.00. No CAV analysis is being performed; therefore, CAF_{CAV} = 1.00.

Step 4. Determine Section Capacity and v/c Ratio. Because the section is located in mountainous terrain, a truck equivalency of 5 is used. The capacity of the basic freeway section is then:

$$c = \frac{\left(2{,}200 + 10 \times (\min(70, FFS) - 50)\right)}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{pop} \times CAF_{CAV}$$

$$= \frac{\left(2{,}200 + 10 \times (\min(70{,}55) - 50)\right)}{1 + (5 - 1)(4.1/100)} \times 3 \times 1.00 \times 1.00 = 5{,}799 \text{ veh/h}$$

The *v/c* ratio is then 7,255 / 5,799 = 1.25.

Detailed Analysis Method

Definition of a Basic Segment

Chapter 12 of the HCM provides methods for estimating the v/c ratios of basic freeway and multilane highway segments. Basic segments are any portion of a facility outside the influence area of ramp merges, diverges, and weaving areas, and (for multilane highways) signalized intersections. The influence area of a merge segment extends 1,500 feet downstream from the merge gore point, the influence area of a diverge segment extends 1,500 feet upstream from the diverge gore point, while the influence area of a weaving segment extends 500 feet upstream and downstream from the ramp gore points. The influence area of an isolated signalized intersection on a multilane highway is 1 mile upstream and downstream of the signal, while the influence area of periodic signalized intersections along a multilane highway is 2 miles upstream and downstream. Note that when the signal spacing is less than 2 miles, the roadway is analyzed as an urban street and not as a multilane highway.

Applicability

The method can be applied directly to general-purpose freeway and multilane highway basic segments. Some aspects of the method (e.g., free-flow speed estimation) are also used as part of the analysis of merge, diverge, and weaving segments.

Chapter 12 of the HCM provides an extension to the method for evaluating managed lane basic segments; consult the HCM for details on performing a managed lane analysis.

The method can potentially be applied to extended bridge and tunnel segments by calibrating capacity and speed adjustment factors for those segments (see Appendix 11B).

The method is not applicable to facilities with free-flow speeds greater than 75 mph (freeways) or 70 mph (multilane highways), nor is it applicable to freeway segments near toll plazas.

Because detailed analysis is intended for near-term situations when all or nearly all inputs are known, and because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in detailed analyses.

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a basic freeway or multilane highway segment using the detailed method:

Step 1. Gather Input Data
↓
Step 2. Adjust Volumes
↓
Step 3. Determine CAFs & SAFs
↓
Step 4. Determine Capacity & v/c Ratio

Step 1: Gather Input Data. The following data are required for the segment:

- Hourly demand.
- Peak hour factor.
- Heavy vehicle percentage.
- Free-flow speed (FFS). See Appendix 11A for guidance on determining the FFS.
- Number of lanes.
- Terrain type or specific grade.

Step 2. Adjust Volumes. Traffic volumes are converted from vehicles per hour (veh/h) to passenger cars per hour (pc/h) by applying the following equations, derived from HCM Equation 14-1 and HCM Equation 12-10, respectively:

$$v = \frac{V}{PHF \times f_{HV}}$$

with

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

- v = demand flow rate under equivalent base conditions (pc/h),
- V = hourly volume under prevailing conditions (veh/h),
- PHF = peak hour factor (decimal),
- f_{HV} = heavy vehicle factor (decimal),
- P_t = proportion of heavy vehicles in the traffic stream (decimal) (for example: 6% = 0.06), and
- E_T = passenger car equivalent (PCE) of one heavy vehicle in the traffic stream.

The PCE of a heavy vehicle can be assumed to be 2 in level terrain and 3 in rolling terrain. However, if an analysis segment contains a grade that is either (1) between 2–3% and longer than ½ mile, or (2) steeper than 3% and longer than ¼ mile, then Exhibits 12-26 through 12-28 in the HCM 7h Edition should be used to determine an appropriate PCE value (see Appendix 11D).

Step 3. Determine Capacity and Speed Adjustment Factors. The HCM provides the ability to adjust capacity and speed to account for non-ideal conditions, through the use of capacity adjustment factors (CAFs) and speed adjustment factors (SAFs). These may be applicable in the following situations:

- To reflect a driver population that includes unfamiliar drivers (see Appendix 11B),
- To calibrate HCM analysis results to match local conditions (see Appendix 11B), or
- To account for the effects of severe weather, incidents, and work zones as part of a travel time reliability analysis (see Section 11.3.7).

Multiple CAFs/SAFs can be included in an analysis (e.g., for driver population and work zones) by multiplying them together to determine an overall CAF/SAF. If no adjustment is to be made, then both CAF and SAF are assigned values of 1.00.

For a capacity analysis of a basic freeway segment, the only CAFs and SAFs that would normally be used initially (i.e., pre-calibration) would be those for driver population.

Step 4. Determine Capacity and v/c Ratio. The per-lane capacity of a basic freeway segment is determined by the following equation, derived from HCM Equations 12-6 and 12-8:

$$c = (2{,}200 + 10 \times [(FFS \times SAF) - 50]) \times CAF$$

where

c = capacity (pc/h/ln),

FFS = free-flow speed (mph),

SAF = speed adjustment factor (decimal), and

CAF = capacity adjustment factor (decimal).

Similarly, the per-lane capacity of a basic multilane highway segment is determined by the following equation, derived from HCM Equations 12-7 and 12-8:

$$c = (1{,}900 + 20 \times [FFS - 45]) \times CAF$$

Multilane basic segment capacity is capped at 2,300 pc/h/ln, while freeway basic segment capacity is capped at 2,400 pc/h/ln.

where all variables are as defined previously.

The v/c ratio is then:

$$v/c = \frac{v}{c \times N}$$

where N is the number of directional lanes.

Example 11-3 Multilane Highway Analysis (Detailed Method)

Step 1. Gather Input Data. The multilane highway being analyzed has four lanes (two in each direction), level terrain, 9.2% heavy vehicles, a peak-direction volume of 1,480 veh/h, and a PHF of 0.88. The driver population consists of drivers familiar with the facility. The speed limit for autos is 65 mph, while the speed limit for trucks is 60 mph.

Because the highway has different auto and truck speed limits, a weighted average FFS must be calculated, as described in Appendix 11A. In the absence of detailed roadway characteristics, the auto FFS is estimated as the auto speed limit (65 mph) plus 5 mph, or

70 mph. The truck FFS is estimated as the auto FFS (70 mph) minus the difference in the auto and truck speed limits (5 mph), or 65 mph. The weighted average FFS is then:

$$FFS = (1 - P_T)FFS_{auto} + (P_T)FFS_{truck} = (1 - 0.092)(70) + (0.092)(65) = 69.5 \text{ mph}$$

Step 2. Adjust Volumes. First, calculate the heavy vehicle factor. In level terrain, a heavy vehicle's PCE is 2:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.092(2 - 1)} = \frac{1}{1.092} = 0.916$$

The demand flow rate is then:

$$v = \frac{V}{PHF \times f_{HV}} = \frac{1{,}480}{0.88 \times 0.916} = 1{,}836\ pc/h$$

Step 3. Determine Capacity Adjustment Factors. No capacity adjustment is required, because the driver population is familiar with the facility; therefore CAF is set at 1.00. SAFs do not apply to multilane highways.

Step 4. Determine Capacity and v/c Ratio. The per-lane capacity of the basic multilane highway segment is:

$$c = (1{,}900 + 20 \times [FFS - 45]) \times CAF$$

$$c = (1{,}900 + 20 \times [69.5 - 45]) \times 1.00 = 2{,}390 \rightarrow 2{,}300 \text{ pc/h/ln}$$

Because the calculated value of 2,390 pc/h/ln exceeds the maximum allowed value of 2,300 pc/h/ln, the per-lane capacity is set at 2,300 pc/h/ln.

The *v/c* ratio is then:

$$\frac{v}{c} = \frac{v}{c \times N} = \frac{1{,}836}{2{,}300 \times 2} = 0.40$$

Example 11-4 Freeway Analysis (Detailed Method)

Step 1. Gather Input Data. The freeway being analyzed climbs a winding 3¼-mile, 6% grade. The design speed of the most severe curve on the grade is 50 mph and there are three lanes in the uphill direction. The uphill volume is 790 veh/h, with 47.7% heavy vehicles (20% single-unit and 80% tractor-trailers) and a PHF of 0.88. The driver population is a balanced mix of familiar and unfamiliar drivers.

Step 2. Adjust Volumes. First, the heavy vehicle factor is calculated. Because this segment contains a specific grade, the PCE is determined from HCM Exhibits 12-26 through 12-28. Exhibit 12-26 is selected because its truck mix (30% single-unit, 70% tractor-trailer) is closest to the observed conditions. For the combination of grade slope, grade length, and heavy vehicle percentage, the PCE is 3.14.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.477(3.14 - 1)} = \frac{1}{2.021} = 0.495$$

The demand flow rate is then:

$$v = \frac{V}{PHF \times f_{HV}} = \frac{790}{0.88 \times 0.495} = 1{,}814\ pc/h$$

Step 3. Determine Capacity and Speed Adjustment Factors. Because this segment has a mix of familiar and unfamiliar drivers, Exhibit 26-9 from Volume 4 of HCM 7 is used to obtain the following adjustment factors for driver population: CAF_{pop} = 0.939 and SAF_{pop} = 0.950. Also refer to Figure 1 of Appendix 11B.

Furthermore, because this segment is considered mountainous terrain, the procedure in Section 3 of Chapter 26 is required to estimate a mixed-flow CAF_{mix} and SAF_{mix}. CAF_{mix} is determined using HCM Equations 26-1 through 26-4. The required inputs are the following:

- The auto-only CAF, CAF_{ao}, which equals CAF_{pop} if no other capacity adjustments (e.g., weather, incidents) are being used in the analysis.
- The heavy-vehicle percentage, expressed as a decimal (e.g., 0.477).
- The grade length (in miles) and the grade percent (as a decimal).

The resulting CAF_{mix} is 0.473.

SAF_{mix} is determined with HCM Equations 26-6 through 26-15, using the following inputs:

- The demand flow rate v and CAF_{mix}, previously calculated.
- The base FFS, in this case, the design speed of the most severe curve, 50 mph.
- The percentage of non–heavy vehicles; the combined percentage of single-unit trucks (SUTs), buses, and RVs; and the percentage of tractor trailers (TTs) (decimal). These values should total 1.000.
- The grade length (in miles) and the grade percent (as a decimal).

The resulting SAF_{mix} is 0.789.

Step 4. Determine Capacity and v/c Ratio. The per-lane capacity of the basic freeway segment is:

$$c = (2{,}200 + 10 \times [(FFS \times SAF) - 50]) \times CAF \leq 2{,}400$$

$$c = (2{,}200 + 10 \times [(50 \times 0.789) - 50]) \times 0.473 = 991\ pc/h/ln$$

The *v/c* ratio is then:

$$\frac{v}{c} = \frac{v}{c \times N} = \frac{1{,}814}{991 \times 3} = 0.61$$

11.3.2 Merge and Diverge Segments

Screening Analysis Method

<u>Definition of a Merge–Diverge Section</u>

Section H6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a simplified method for estimating the v/c ratio of a merge–diverge section (called a "ramp section" in the PPEAG). A merge–diverge section is defined as the portion of a freeway between an on-ramp and the next downstream ramp, where the two ramps are <u>not</u> connected by an auxiliary lane.

<u>Applicability</u>

The method can be applied directly to merge–diverge sections on a freeway or multilane highway mainline. Ramp metering can be addressed through a capacity adjustment factor, as described in the detailed method above. The capacity adjustment for CAVs can only be applied to freeway sections and not to multilane highways.

<u>Method for Calculating the v/c Ratio</u>

The following steps are used to calculate the v/c ratio for a merge–diverge section using the screening method:

Step 1. Gather Input Data

↓

Step 2. Adjust Volumes

↓

Step 3. Determine CAFs

↓

Step 4. Determine Section Capacity & v/c Ratio

Step 1: Gather Input Data. The following data are required; inputs in *italics* can be defaulted when not available:

- Hourly demands for the freeway entering the merge–diverge section, the on-ramp, and (if present) the off-ramp.
- The *peak hour factor*.
- The *heavy vehicle percentage* within the merge–diverge section.
- The number of lanes on the freeway entering, exiting, and within the merge–diverge section; the number of lanes on the on-ramp; and (if present) the number of lanes on the off-ramp.
- The *free-flow speed* of the freeway mainline.
- The terrain type (level, rolling, mountainous).

See Appendix 11C for Oregon-specific default values.

Step 2. Adjust Volumes. Freeway and ramp volumes are converted to 15-minute flow rates by dividing the volumes by the peak hour factor.

Step 3. Determine Capacity Adjustment Factors. As a default, the capacity of a merge–diverge section is about 95% that of the corresponding basic freeway or multilane highway section; therefore, CAF_{ramp} = 0.95. The analyst can substitute a different value if field observations indicate lower capacities. If applicable, capacity adjustment factors for driver population CAF_{pop} (Appendix 11B) and/or ramp metering CAF_{meter} (a value of 1.03) can also be applied. Otherwise, default values of 1.00 are used for these capacity adjustment factors. Finally, an optional capacity adjustment factor for CAVs CAF_{CAV} can be applied to freeway sections, as described in Appendix 6B. Otherwise, a default value of 1.00 is used for this factor.

Step 4. Determine the Merge–Diverge Section Capacity. Capacity is determined using the following equation (derived from PPEAG Equation 16):

$$c = \frac{\left(2{,}200 + 10 \times (\min(70, FFS) - 50)\right)}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{ramp} \times CAF_{pop} \times CAF_{meter} \times CAF_{CAV}$$

where

c = merge–diverge section capacity (veh/h);

FFS = section free-flow speed (mph);

E_T = truck equivalency = 2 (level terrain), 3 (rolling terrain), or 5 (mountainous terrain);

$\%HV$ = heavy vehicle percentage (e.g., 6% = 6);

N = number of lanes in the ramp section (integer);

CAF_{ramp} = ramp section capacity adjustment factor (decimal);

CAF_{pop} = driver population capacity adjustment factor;

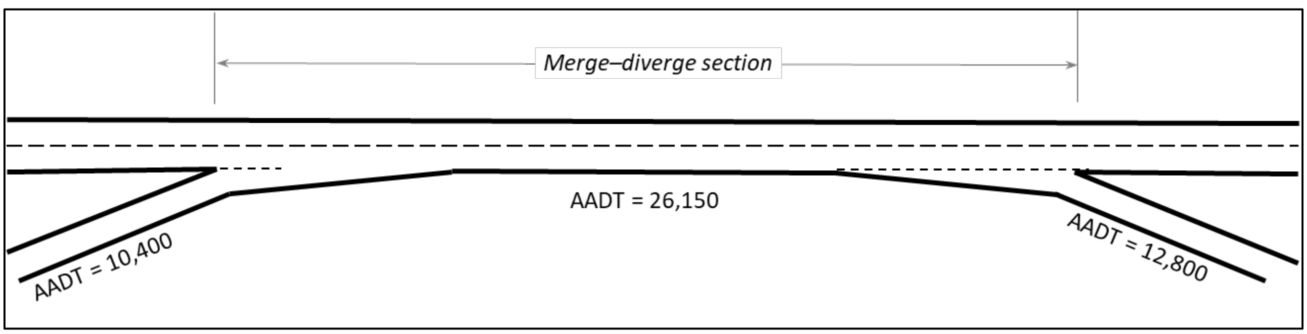
CAF_{meter} = ramp metering capacity adjustment factor; and

CAF_{CAV} = CAV capacity adjustment factor.

The *v/c* ratio is then the total flow rate in the merge–diverge section divided by the section capacity. Conditions external to the merge–diverge section may also create capacity constraints. The capacity of the freeway mainline can be compared to the entering and exiting demand flows, using the screening procedure in Section 11.3.2. The PPEAG recommends using a value of 2,000 veh/h/ln as a planning-level ramp capacity.

Example 11-5 Merge–Diverge Section Analysis (Screening Method)

Step 1. Gather Input Data. The merge–diverge section has the lane configuration and directional AADTs shown below:



This section of freeway is level and has 16.8% heavy vehicles, a *K*-factor of 9.3, and a FFS of 60 mph. The ramps have *K*-factors of 10.0. The on-ramp has 10.2% heavy vehicles, while the off-ramp has 3.5% heavy vehicles. No ramp metering is in use, drivers are familiar with the facility, and there are no CAVs.

The directional AADTs must be converted into peak-hour volumes by multiplying by the decimal version of the facility's *K*-factor. This results in a (rounded) freeway merge–diverge section volume of 2,430 veh/h, an on-ramp volume of 1,040 veh/h, and an off-ramp volume of 1,280 veh/h.

Step 2. Adjust Volumes. The peak-15-minute demand flow rates are determined by dividing the peak hour volumes by the PHF. The PHF is unknown; therefore, the freeway default value of 0.95 is used. For the freeway ramp section, this is 2,430 / 0.95 = 2,558 veh/h. Similarly, the on-ramp flow rate is 1,095 veh/h and the off-ramp flow rate is 1,347 veh/h.

Step 3. Determine Capacity Adjustment Factors. The merge–diverge section capacity adjustment factor CAF_{ramp} is 0.95. Because the driver population consists of familiar drivers, CAF_{pop} = 1.00. There is no ramp metering; therefore, CAF_{meter} = 1.00. No CAV analysis is being performed; therefore CAF_{CAV} = 1.00.

Step 4. Determine the Section Capacity and v/c Ratio. The section capacity is calculated as:

$$c = \frac{\left(2{,}200 + 10 \times (\min(70{,}60) - 50)\right)}{1 + (2 - 1)(16.8/100)} \times 2 \times 0.95 \times 1.00 \times 1.00 \times 1.00 = 3{,}741 \text{ veh/h}$$

The section's *v/c* ratio is then 2,558 / 3,741 = 0.68.

The on-ramp *v/c* ratio is 1,095 / 2,000 = 0.55, while the off-ramp *v/c* ratio is 1,347 / 2,000 = 0.67.

Detailed Analysis Method

Definition of Merge and Diverge Segments

Chapter 14 of the HCM provides a method for estimating the v/c ratio of merge and diverge segments—that is, sections of freeway containing either an on-ramp (merge) or an off-ramp (diverge), and where adjacent on- and off-ramps are not connected by an auxiliary lane (those are analyzed as weaving segments). Interchanges where mainline roadways join or split are also treated as merge or diverge segments; these are known as *major merge* and *major diverge segments*.

Merge segments generally extend 1,500 feet downstream from the on-ramp gore point, while diverge segments generally extend 1,500 feet upstream from the off-ramp gore point. If another ramp is located within this influence area, the segment length is reduced and the lower of the two capacity results from the two ramps is applied to the overlap area.

Applicability

The method can be applied directly to:

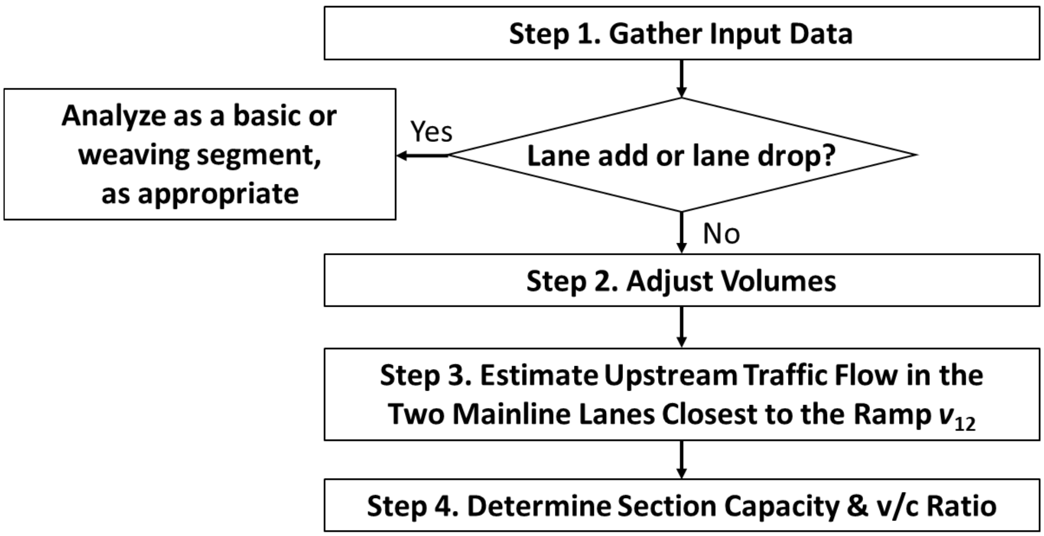
- Ramps on a freeway mainline;
- Ramps on a multilane highway, if located sufficiently far away from a traffic signal so as not to experience platooning effects; and
- Ramps on collector–distributor roads.

Guidance is provided later in this section on applying this method to on-ramps where ramp metering is in operation and to major merge/diverge areas. Chapter 14 of the HCM also provides an extension to the method for evaluating direct ramps to and from managed lanes (e.g., high-occupancy vehicle or high-occupancy toll lanes). Consult the HCM for details on performing a managed lane merge or diverge analysis.

Because detailed analysis is intended for near-term situations when all or nearly all inputs are known, and because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in detailed analyses.

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a merge or diverge segment using the detailed method:



Step 1: Gather Input Data. The following data are required for both the freeway (or multilane highway) mainline and the ramp:

- Hourly demands.
- Peak hour factor.
- Heavy vehicle percentage.
- Free-flow speed (FFS). See Appendix 11A for how to determine the mainline FFS and Section 11.3.1 for how to determine the ramp FFS.
- Terrain type (level, rolling) or specific grade.

The following additional data are also required:

- Ramp acceleration/deceleration lane length, as described in Section 11.3.1.
- Number of lanes on the mainline entering the segment, on the ramp, and at the merge/diverge point.
- For six-lane facilities only (three lanes per direction), the distances to the next upstream and downstream ramp, and the volumes on those ramps.

If an on-ramp acceleration lane exceeds 1,500 ft in length, a lane-add situation exists. Similarly, if an off-ramp deceleration lane exceeds 1,500 ft, a lane-drop situation exists. In either situation, the remaining steps in this section are not performed. Instead, the segment is analyzed either as a weaving segment or as a basic segment according to the following rules:

- An added lane that is dropped at the next off-ramp, with no other ramps located in between, is treated as a weaving segment (see Section 11.3.4).
- All other lane-add and lane-drop situations are analyzed as basic segments, with the lane being added or dropped being counted as one of the freeway lanes. Although analyzed as a basic segment, the segment should still be coded in software as a merge or diverge segment, as appropriate, to properly account for the change in volume at the location.
- An added lane that is dropped at a nearby downstream interchange, with other ramps located between the add and drop points, may form a multiple weaving segment. These are discussed in Section 11.3.4. Note that weaving turbulence may exist within a multiple weaving segment that is not fully accounted for by the methodology.

Step 2. Adjust Volumes. Mainline and ramp volumes are converted from vehicles per hour (veh/h) to passenger cars per hour (pc/h) by applying the following equations, derived from HCM Equation 14-1 and HCM Equation 12-10, respectively:

$$v_i = \frac{V_i}{PHF \times f_{HV}}$$

with

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

v_i = demand flow rate for movement *i* under ideal conditions (pc/h),

V_i = hourly volume for movement *i* under prevailing conditions (veh/h),

PHF = peak hour factor (decimal),

f_{HV} = heavy vehicle factor (decimal),

P_t = proportion of heavy vehicles in the traffic stream (decimal) (for example: 6% = 0.06), and

E_T = passenger car equivalent (PCE) of one heavy vehicle in the traffic stream.

The PCE of a heavy vehicle is 2 in level terrain and 3 in rolling terrain. Exhibits 12-26 through 12-28 in the HCM 7th Edition (see also Appendix 11D) can be used to determine an appropriate PCE value when either (1) a grade is 2–3% and longer than ½ mile, or (2) a grade is steeper than 3% and longer than ¼ mile.

Step 3. Estimate Upstream Traffic Flow in the Two Mainline Lanes Closest to the Ramp v_{12}. In most cases, the ramp will be on the right side of the freeway or multilane highway, and the two mainline lanes closest to the ramp will be the two rightmost mainline lanes. For a left-hand ramp, these will be the two leftmost mainline lanes. On a four-lane facility (two lanes in each direction), v_{12} is simply the total demand flow on the mainline just upstream of the ramp influence area. On wider facilities, the process described in the HCM 7th Edition starting on page 14-15 is used to determine v_{12}, with the potential modifications described in the Special Cases section of the chapter (starting on page 14-30) for two-lane ramps, left-side ramps, and 10-lane freeways.

Step 4. Determine Capacity and v/c Ratio. Capacity is determined for both the ramp segment and the ramp roadway. For a ramp junction, the capacity of the ramp influence area (the two mainline lanes closest to the ramp plus the ramp acceleration or deceleration lane) is 4,600 pc/h for a merge segment and 4,400 pc/h for a diverge segment. For ramp roadways, the capacity values given in Exhibit 14-12 in the HCM 7th Edition are used.

Optionally, the capacities of the mainline and the ramp (but not the ramp influence area) can be adjusted using a CAF, as described previously in Section 11.3.1, Step 3. For an initial (i.e., pre-calibration) capacity analysis, the only CAFs that might normally be used are those for driver population CAF_{pop} (see Appendix 11B) and ramp metering CAF_{meter} (described below). Appendix C further provides defaults for merge, diverge, and weaving segment capacities, which are implemented in software in the form of CAFs. For basic freeway segments, the default CAF is 1.0. All applicable CAFs are multiplied together to create an overall CAF, which is applied as follows (HCM Equation 14-21):

$$c_{mda} = c_{md} \times CAF$$

where

c_{mda} = adjusted merge/diverge segment capacity (pc/h),

c_{md} = unadjusted merge/diverge capacity (pc/h), and

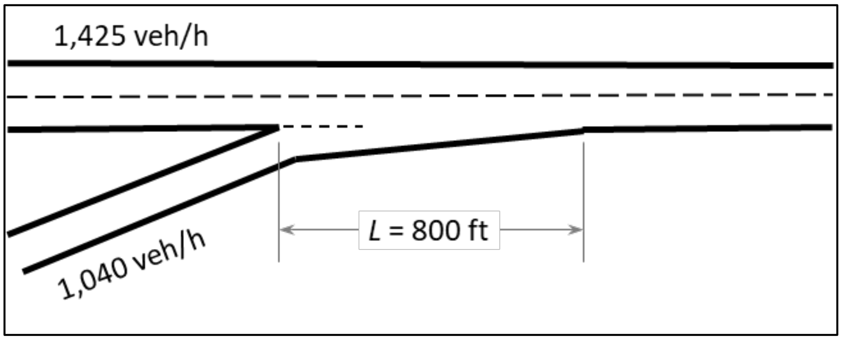
CAF = capacity adjustment factor (decimal), default = 1.00 for basic segments; see Appendix C for other segment types.

The v/c ratio for each location is then determined as follows:

- For a merge ramp junction, the *v/c* ratio is the sum of v_{12} and the ramp demand flow, divided by 4,600.
- For a diverge ramp junction, the *v/c* ratio is v_{12} divided by 4,400.
- For the mainline basic segments before or after the merge/diverge segment, the *v/c* ratio is the mainline demand flow in the upstream or downstream basic segment, divided by the adjusted mainline basic segment capacity.
- For ramp roadways, the *v/c* ratio is the ramp demand flow divided by the adjusted ramp capacity.

Example 11-6 Merge Segment Analysis, 4-Lane Freeway (Detailed Method)

Step 1. Gather Input Data. The merge segment has the lane configuration and entering volumes shown below:



The freeway mainline is level, with a 70-mph FFS in the merge segment, 16.8% heavy vehicles, a PHF of 0.90, and a driver population familiar with the roadway. The ramp has a free-flow speed of 55 mph, level terrain, 10.2% heavy vehicles, and a PHF of 0.93. There is no ramp meter.

Step 2. Adjust Volumes. First, the heavy vehicle factor is calculated. In level terrain, a heavy vehicle's PCE is 2. Then, for the freeway:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.168(2 - 1)} = \frac{1}{1.168} = 0.856$$

The freeway mainline demand flow rate is then:

$$v_f = \frac{V_f}{PHF \times f_{HV}} = \frac{1{,}425}{0.90 \times 0.856} = 1{,}850 \text{ pc/h}$$

Similarly, for the on-ramp, the heavy vehicle factor is 0.907 and the ramp demand flow rate is 1,233 pc/h.

Step 3. Estimate Upstream Traffic Flow in the Two Mainline Lanes Closest to the Ramp v_{12}. This is a four-lane freeway; therefore v_{12} equals the freeway mainline demand flow rate, 1,850 pc/h.

Step 4. Determine Capacity and v/c Ratio. Capacity is determined for the following locations:

- Ramp junction: This is a merge segment, so the ramp junction capacity is 4,600 pc/h.
- Ramp: From HCM Exhibit 14-12, the capacity of a single-lane ramp with a FFS of 55 mph is 2,200 pc/h.

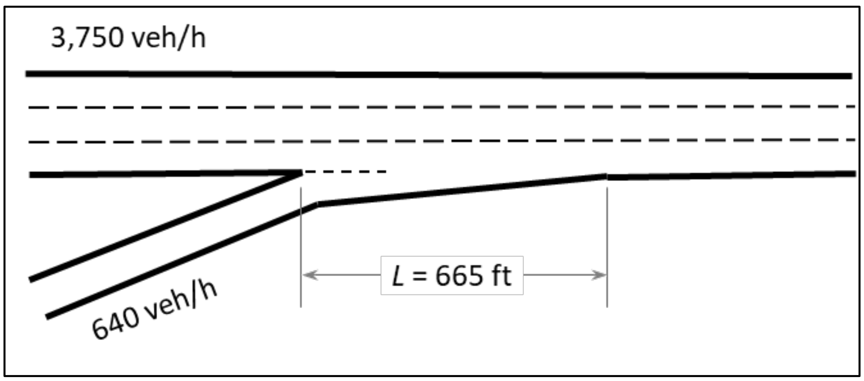
There is no ramp metering and the driver population consists of familiar drivers; therefore, no capacity adjustment is made to either the ramp or the ramp junction.

The *v/c* ratios are then calculated as follows:

- Ramp junction: The demand flow is the sum of v_{12} and the ramp demand flow. Then, v/c = (1,850 + 1,233) / 4,600 = 0.67.
- Ramp: The demand flow is the ramp demand flow. Then, v/c = 1,233 / 2,200 = 0.56.

Example 11-7 Merge Segment Analysis, 6-Lane Freeway (Detailed Method)

Step 1. Gather Input Data. The merge segment has the lane configuration and entering volumes shown below:



The freeway mainline is level, with a 70-mph FFS, 17.6% heavy vehicles, a PHF of 0.90, and a driver population familiar with the roadway. The ramp has a FFS of 55 mph, level terrain, 6.5% heavy vehicles, a PHF of 0.88, and no ramp meter. There is an on-ramp 1,800 feet upstream, with a peak-hour volume of 645 veh/h, 6.5% heavy vehicles, level terrain, a PHF of 0.94, and a 45-mph FFS. The next downstream ramp is an off-ramp 9,300 feet away, with a peak-hour volume of 790 veh/h, 14.5% heavy vehicles, level terrain, a PHF of 0.94, and a 45-mph FFS.

Step 2. Adjust Volumes. Volumes are adjusted for heavy vehicles and peak-15-minute conditions in the same manner as in Example 11-6 , resulting in following demand flow rates:

- Freeway mainline upstream of the merge: 4,900 pc/h
- On-ramp: 775 pc/h
- Upstream on-ramp: 731 pc/h
- Downstream off-ramp: 962 pc/h

The upstream and downstream ramp flow rates are required because the freeway has a six-lane cross-section and these ramps may influence vehicle positioning in the two right-hand freeway lanes.

Step 3. Estimate Upstream Traffic Flow in the Two Mainline Lanes Closest to the Ramp v_{12}. Because the merge segment has three lanes, the procedures in Chapter 14 of the HCM 7th Edition are applied to determine the traffic flow in the two right-hand freeway lanes. First, the formula used to estimate the proportion of traffic in the two right-hand lanes is determined from HCM Exhibit 14-8. For this situation, where the subject ramp is an on-ramp, the upstream ramp is an on-ramp, and the downstream ramp is an off-ramp, the exhibit says to use either HCM Equation 14-3 or 14-5. The choice of equation is determined by whether the downstream off-ramp is close enough to affect traffic operations. This determination is made by applying HCM Equation 14-7 with the downstream ramp flow rate v_D and the length of the acceleration lane in the merge segment L_A to determine the equilibrium separation distance L_{EQ} of the two ramps:

$$L_{EQ} = \frac{v_D}{0.1096 + 0.000107 L_A} = \frac{962}{0.1096 + 0.000107(665)} = 5{,}322 \text{ ft}$$

The actual distance to the downstream off-ramp, 9,300 ft, is larger than the equilibrium distance; therefore, the HCM instructs the analyst to apply HCM Equation 14-3 to determine the proportion of traffic in the two right-hand lanes P_{FM}:

$$P_{FM} = 0.5775 + 0.000028 L_A = 0.5775 + 0.000028(665) = 0.596$$

Finally, HCM Equation 14-2 is used to determine the freeway traffic flow in the two right-most lanes v_{12}:

$$v_{12} = v_F \times P_{FM} = 4{,}900 \times 0.596 = 2{,}920 \text{ pc/h}$$

Step 4. Determine Capacity and v/c Ratio. Capacity is determined for the following locations:

- Ramp junction: This is a merge segment, so the ramp junction capacity is 4,600 pc/h.
- Ramp: From HCM Exhibit 14-12, the capacity of a single-lane ramp with a FFS of 55 mph is 2,200 pc/h.

There is no ramp metering and the driver population consists of familiar drivers; therefore, no capacity adjustment is made to either the ramp or the ramp junction.

The *v/c* ratios are then calculated as follows:

- Ramp junction: The demand flow is the sum of v_{12} and the ramp demand flow. Then, *v/c* = (2,920 + 775) / 4,600 = 0.80.
- Ramp: The demand flow is the ramp demand flow. Then, *v/c* = 775 / 2,200 = 0.35.

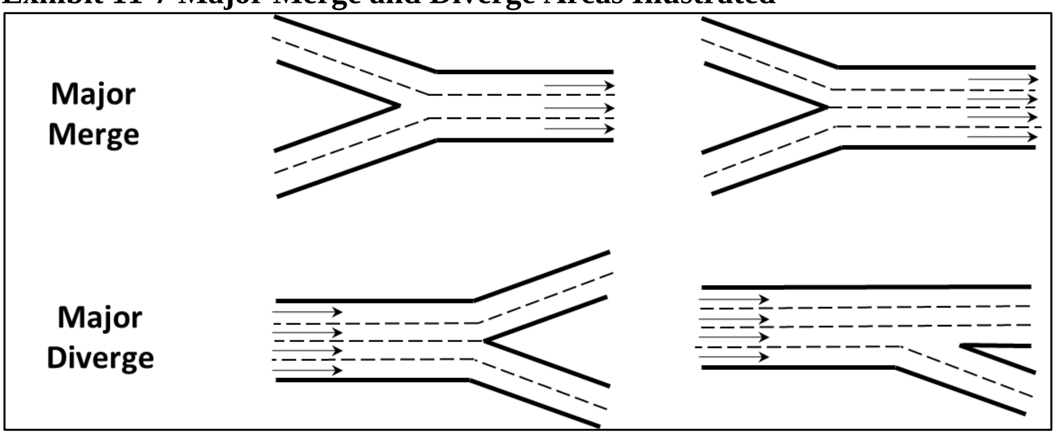
Ramp Metering

The effects of ramp metering on merge and diverge segment capacity can be addressed through a capacity adjustment factor CAF_{meter}. The HCM 7th Edition (Chapter 37, Section 4) recommends a value of 1.03 when ramp metering is in operation. Note that ramp metering will also reduce the per-lane on-ramp capacity to a level equivalent to the metering rate, and therefore the ramp volume entering the freeway may need to be adjusted.

Major Merge and Diverge Areas

The HCM defines a *major merge area* as "one in which two primary roadways, each having multiple lanes, merge to form a single freeway segment," such as when two freeways join to form a single freeway or a high-speed multilane ramp joins a freeway. The HCM defines a *major diverge area* as "one in which two primary roadways, each having multiple lanes, diverge from a single freeway segment," such as when a freeway splits into two separate freeways or high-speed multilane ramp diverges from a freeway. Key characteristics of major merge and diverge areas are: (1) all of the roadways involved are at or near freeway design standards, (2) all roadways have two or more lanes, and (3) there is no clear acceleration/deceleration lane. Exhibit 11-7 illustrates potential configurations of major merge and diverge areas.

Exhibit 11-7 Major Merge and Diverge Areas Illustrated



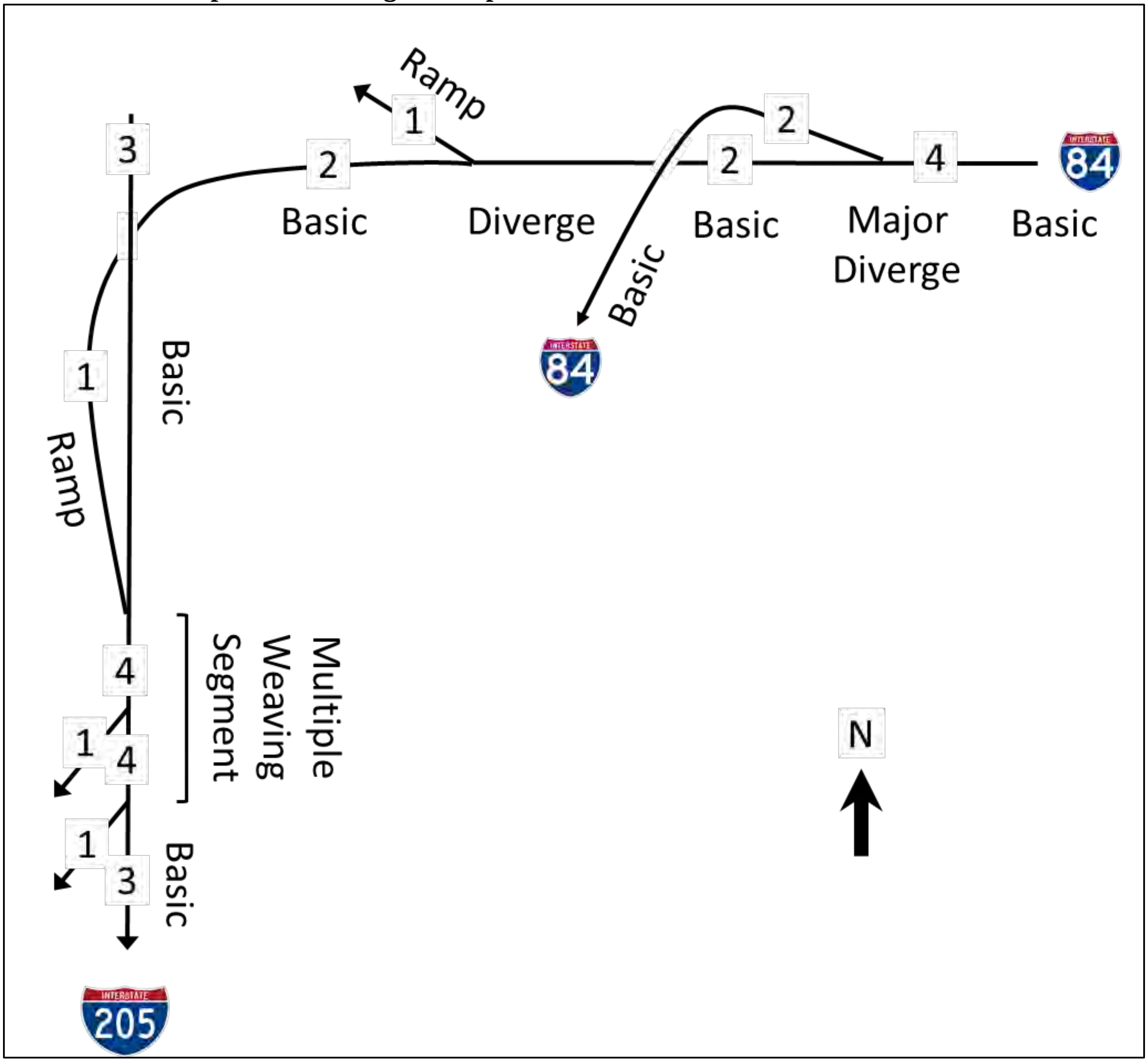
Source: Derived from HCM 7th Edition, Exhibits 14-20 and 14-21.

The HCM recommends evaluating the capacity of major merge and diverge areas by the checking the basic segment capacities of the entry and exit legs. Section 11.3.2 describes how to determine basic freeway segment capacities.

Complex Interchanges

Complex interchanges, with a series of on- and off-ramps, should be broken into smaller basic freeway, (major) merge, (major) diverge, weave, and ramp segments for analysis. Exhibit 11-8 shows an example of I-84 westbound at I-205 and the roadways leading to I-205 southbound.

Exhibit 11-8 Complex Interchange Example: Westbound I-84 to I-205 Southbound



I-84 westbound has four lanes approaching the interchange, which split into two lanes to stay on I-84 and two lanes to connect to both directions of I-205. Because both exit legs of the split have multiple lanes and because both exit legs are designed to freeway standards, the split would be analyzed as a major diverge, involving capacity checks of the entry and exit legs as basic freeway segments.

The exit leg to I-205 has an exit of its own, to I-205 north, with the lanes continuing to I-205 south constructed to freeway standards at the exit. Therefore, the 1,500 feet of roadway preceding the off-ramp would be analyzed as a freeway diverge segment.

Continuing toward I-205 south, one of the lanes is dropped. Following this point, the single-lane roadway would be treated as a ramp roadway, with a lower capacity than a basic freeway segment. The ramp joins I-205 south as an added lane; therefore, the junction is analyzed as a basic segment rather than as a merge. However, because the added lane is dropped two off-ramps (about 2,800 ft) downstream, a multiple weaving segment exists, which may contain weaving turbulence not accounted for in the methodology. See Exhibit 11-10 in Section 11.3.4 for an example of segmenting a multiple weaving segment into separate merge, diverge, and basic segments.

11.3.3 Weaving Segments

Screening Analysis Method

<u>Definition of a Weaving Section</u>

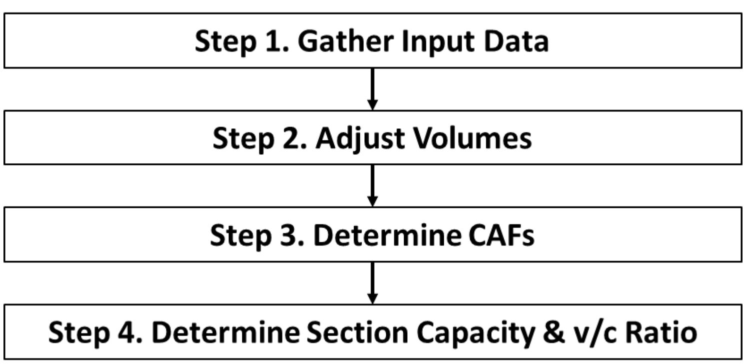
Section H6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a simplified method for estimating the v/c ratio of a weaving section. A weaving section is defined as the portion of a freeway between an on-ramp and an off-ramp, where the two ramps are connected by an auxiliary lane. Any other combination of ramps should be treated as one or more ramp sections.

<u>Applicability</u>

The method can be applied directly to weaving sections on a freeway or multilane highway mainline. Ramp metering can be addressed through a capacity adjustment factor, as described in the detailed method below. The capacity adjustment for CAVs can only be applied to freeway sections and not to multilane highways.

<u>Method for Calculating the v/c Ratio</u>

The following steps are used to calculate the v/c ratio for a weaving section using the screening method:



Step 1: Gather Input Data. The following data are required; inputs in *italics* can be defaulted when not available:

- Hourly demands for the following: freeway entering the weaving section, freeway exiting the weaving section, on-ramp, off-ramp. Weaving volumes should also be used, when available, but for many planning applications they will not be available. Instead, it can be conservatively estimated that there is no ramp-to-ramp volume and that all of the on- and off-ramp traffic must weave. A regional travel demand model, if available, can be a potential source of future weaving volumes.

- The *peak hour factor*.
- The *heavy vehicle percentage* within the weaving section.
- The weaving section length (i.e., the distance between the ramp gore points).
- The number of lanes on the freeway entering, exiting, and within the weaving section; the number of lanes on the on-ramp; and the number of lanes on the off-ramp.
- The *free-flow speed* of the freeway mainline.
- The terrain type (level, rolling, mountainous).

See Appendix 11C for Oregon-specific default values.

Step 2. Adjust Volumes. The four section entry and exit volumes are converted to 15-minute flow rates by dividing the volumes by the peak hour factor.

Step 3. Determine Capacity Adjustment Factors. The capacity of a weaving section is less than that of a basic freeway section. The following equation (PPEAG Equation 23) is used to estimate the reduction in capacity in a weaving section CAF_{weave}:

$$CAF_{weave} = 0.884 - 0.0752VR + 0.0000243L_{weave}$$

where

CAF_{weave} = weaving section capacity adjustment factor (decimal), always ≤1.00;

VR = volume ratio (decimal) = weaving flow rate divided by total flow rate in the section; and

L_{weave} = weaving section length (ft).

If applicable, a capacity adjustment factor for driver population CAF_{pop} (Appendix 11B) and/or ramp metering CAF_{meter} (a value of 1.03) can also be applied. Otherwise, a default value of 1.00 is used for these adjustment factors. In addition, an optional capacity adjustment factor for CAVs CAF_{CAV} can be applied to freeway sections, as described in Appendix 6B. Otherwise, a default value of 1.00 is used for this factor.

Step 4. Determine the Weaving Section Capacity. Capacity is determined using the following equation (derived from PPEAG Equation 16):

$$c = \frac{\left(2{,}200 + 10 \times (\min(70, FFS) - 50)\right)}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{weave} \times CAF_{pop} \times CAF_{meter} \times CAF_{CAV}$$

where

c = weaving section capacity (veh/h);

FFS = section free-flow speed (mph);

E_T = truck equivalency = 2 (level terrain), 3 (rolling terrain), or 5 (mountainous terrain);

$\%HV$ = heavy vehicle percentage (e.g., 6% = 6);

N = number of lanes in the weaving section (integer);

CAF_{weave} = weaving section capacity adjustment factor (decimal);

CAF_{pop} = driver population capacity adjustment factor;

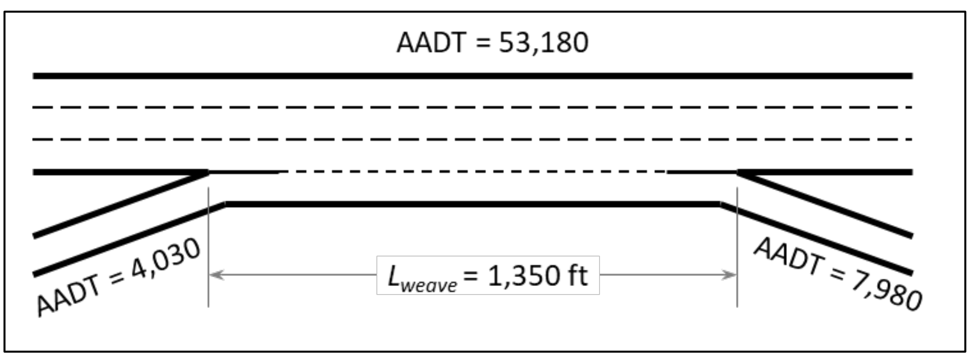
CAF_{meter} = ramp metering capacity adjustment factor; and

CAF_{CAV} = CAV capacity adjustment factor.

The *v/c* ratio is then the total flow rate in the weaving section divided by the weaving section capacity. Conditions external to the weaving section may also create capacity constraints. The capacity of the freeway mainline can be compared to the entering and exiting demand flows, using the screening procedure in Section 11.3.1, while the capacity of the ramps can be compared to their demand flows using the screening procedure in Section 11.3.2.

Example 11-8 Weaving Section Analysis (Screening Method)

Step 1. Gather Input Data. The weaving section has the lane configuration, length, and directional AADTs shown below:



No weaving volumes are available; therefore, it is assumed conservatively that all ramp volumes must weave. This section of freeway is level and has 6.5% heavy vehicles, no CAVs, a *K*-factor of 7.6, and a driver population of regular commuters. The freeway has a free-flow speed of 65 mph, the on-ramp has a free-flow speed of 45 mph, and the off-ramp has a free-flow speed of 30 mph. No ramp metering is in use.

The AADTs must be converted into peak-hour volumes by multiplying by the decimal version of the facility's *K*-factor. This results in a (rounded) on-ramp volume of 305 veh/h, an off-ramp volume of 605 veh/h, and a total weaving section volume of 4,040 veh/h.

Step 2. Adjust Volumes. The peak-15-minute demand flow rates are determined by dividing the peak hour volumes by the PHF. The PHF is unknown; therefore, the freeway default value of 0.95 is used. For the on-ramp, this calculation is 305 / 0.95 = 321 veh/h. Similarly, the off-ramp flow rate is 637 veh/h and the total section volume is 4,253 veh/h. The volume ratio *VR* is (321+637) / 4,253 = 0.223.

Step 3. Determine Capacity Adjustment Factors. The weaving section capacity adjustment factor is:

$$CAF_{\text{weave}} = 0.884 - 0.0752VR + 0.0000243L_{weave}$$

$$CAF_{\text{weave}} = 0.884 - 0.0752(0.223) + 0.0000243(1{,}350)$$

$$CAF_{\text{weave}} = 0.900$$

Because the driver population consists of regular commuters, CAF_{pop} = 1.00. There is no ramp metering; therefore, CAF_{meter} = 1.00. No CAV analysis is being performed; therefore, CAF_{CAV} = 1.00.

Step 4. Determine the Weaving Section Capacity and v/c Ratio. The section capacity is calculated as:

$$c = \frac{\left(2{,}200 + 10 \times (\min(70{,}65) - 50)\right)}{1 + (2-1)(6.5/100)} \times 4 \times 0.900 \times 1.00 \times 1.00 \times 1.00 = 7{,}944 \text{ veh/h}$$

The section's *v/c* ratio is then 4,253 / 7,944 = 0.54.

The demand entering the weaving section is the section demand minus the on-ramp demand, or 3,932 veh/h, while the capacity of the upstream basic segment is:

$$c = \frac{\left(2{,}200 + 10 \times (\min(70{,}65) - 50)\right)}{1 + (2-1)(6.4/100)} \times 3 \times 1.000 \times 1.00 \times 1.00 = 6{,}626 \text{ veh/h}$$

Therefore, there is no capacity constraint on the upstream segment. Similarly, the demand on the downstream basic segment is 3,616 veh/h, compared to a capacity of 6,626 veh/h, and therefore no capacity constraint exists on the downstream segment. Ramp *v/c* ratios can be checked using the same process as illustrated in Example 11-9 for the detailed method.

Detailed Analysis Method

Definition of a Weaving Segment

Chapter 13 of the HCM provides a method for estimating the v/c ratio of a weaving segment. The most common form of weaving segment is a *one-sided weave,* where an on-ramp is closely followed by an off-ramp on the same side of the freeway and:

- The two ramps are connected by an auxiliary lane that drops at the off-ramp; and
- No weaving maneuver requires more than two lane changes.

An uncommon form of weaving segment also addressed by the method is a *two-sided weave,* where either (1) an on-ramp on one side of the freeway is closely followed by an off-ramp on the opposite side of the freeway, or (2) at least one weaving maneuver requires three or more lane changes.

A weaving segment extends 500 feet upstream from the on-ramp gore point and 500 feet downstream from the off-ramp gore point.

Applicability

The method can be applied directly to:

- Weaving segments on a freeway mainline;
- Weaving segments on a multilane highway, if located sufficiently far away from a traffic signal so as not to experience platooning effects; and
- Weaving areas on collector–distributor roads.

The method is potentially applicable, with analyst modifications, to:

- Weaving segments where ramp metering is in operation on the on-ramp, and
- Multiple weaving segments (e.g., an on-ramp connected by an auxiliary lane to two closely spaced downstream off-ramps).

Guidance on these potentially applicable situations is provided below, and simulation is a potential supplemental or alternative tool that can be applied in these cases.

The method cannot be used to evaluate weaving on urban streets, including freeway frontage roads.

Chapter 13 of the HCM also provides an extension to the method for evaluating weaving associated with managed lanes (e.g., high-occupancy vehicle or high-occupancy toll lanes). These situations include weaving between the managed and general-purpose lanes at managed lane access points, weaving across the general-purpose lanes between general-purpose ramps and the managed lanes, and weaving associated with direct ramps into and out of the managed lanes. Consult the HCM for details on performing a managed lane weaving analysis.

> *Because detailed analysis is intended for near-term situations when all or nearly all inputs are known, and because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in detailed analyses.*

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a weaving segment using the detailed method:

Step 1. Gather Input Data
↓
Step 2. Adjust Volumes
↓
Step 3. Determine Number of Weaving Lanes
↓
Step 4. Determine Maximum Weaving Length
↓
Step 5. Determine Section Capacity & v/c Ratio

Step 1: Gather Input Data. The following data are required:

- Hourly demands for the following movements: freeway-to-freeway, freeway-to-ramp, ramp-to-freeway, and ramp-to-ramp. Most often, only volumes entering and exiting the weaving segment will be available. However, if just one of the four weaving movements is counted or estimated from a model (ramp-to-ramp volumes are generally lowest and easiest to count by various methods), the other three movements can be determined from the entry and exit volumes (see Example 11-9 for an illustration of the process).
- The peak hour factor.
- The heavy vehicle percentage within the weaving segment.

- The "short length" of the weaving area, defined as the distance where broken pavement striping permits lane-changing movements.
- The number of lanes on the freeway entering, exiting, and within the weaving segment; the number of lanes on the on-ramp; and the number of lanes on the off-ramp.
- The free-flow speeds of the freeway mainline (see Appendix 11A) and the ramps (see Section 11.3.1).
- The terrain type (level, rolling) or the specific grade.

Step 2. Adjust Volumes. The four movement volumes are converted from vehicles per hour (veh/h) to passenger cars per hour (pc/h) by applying the following equations, derived from HCM Equation 13-1 and HCM Equation 12-10, respectively:

$$v_i = \frac{V_i}{PHF \times f_{HV}}$$

with

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

v_i = demand flow rate for movement *i* under ideal conditions (pc/h),

V_i = hourly volume for movement *i* under prevailing conditions (veh/h),

PHF = peak hour factor (decimal),

f_{HV} = heavy vehicle factor (decimal),

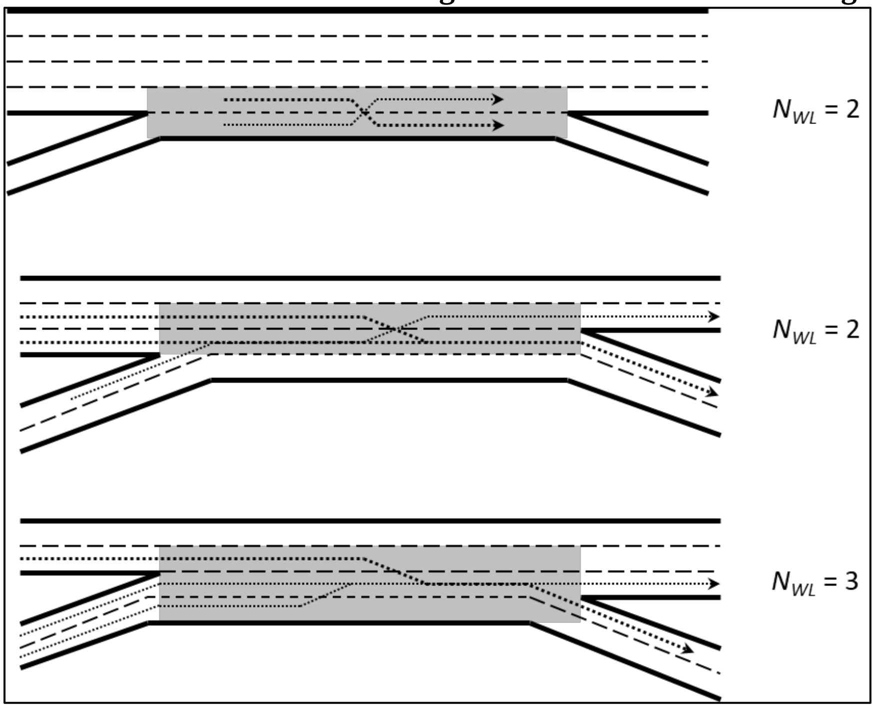
P_t = proportion of heavy vehicles in the traffic stream (decimal) (for example: 6% = 0.06), and

E_T = passenger car equivalent (PCE) of one heavy vehicle in the traffic stream.

The PCE of a heavy vehicle is 2 in level terrain and 3 in rolling terrain. Exhibits 12-26 through 12-28 in the HCM 7th Edition (see also Appendix 11D) can be used to determine an appropriate PCE value when either (1) a grade is 2–3% and longer than ½ mile, or (2) a grade is steeper than 3% and longer than ¼ mile.

Step 3. Determine the Number of Weaving Lanes. The number of weaving lanes N_{WL} in a two-sided weaving segment is 0 by definition. In one-sided weaving segments, N_{WL} is the total number of lanes from which a weaving maneuver can be made with zero or one lane changes. Exhibit 11-9 shows values of N_{WL} for common weaving configurations.

Exhibit 11-9 Number of Weaving Lanes for Common Weaving Configurations



Source: Derived from HCM 7th Edition, Exhibit 13-5.

Step 4. Determine the Maximum Weaving Length. The maximum length of a weaving segment L_{MAX} is determined using the following equation (HCM Equation 13-4):

$$L_{MAX} = [5{,}728(1 + VR)^{1.6}] - (1{,}566N_{WL})$$

where

L_{MAX} = maximum weaving segment length (ft),

VR = volume ratio (decimal) = weaving demand flow rate divided by the total demand flow rate in the weaving segment, and

N_{WL} = number of weaving lanes.

If $L_S > L_{MAX}$, segment should be analyzed as separate merge and diverge segments, possible with a basic segment between them depending on distances, follow procedure in Section 11.3.2; otherwise, continue with weaving analysis.

Step 5. Determine the Weaving Segment Capacity and v/c Ratio. The weaving segment reaches capacity when either its density exceeds 43 pc/mi/ln or when the weaving demand flow exceeds a specified value. The capacity of a weaving segment based on density is determined from the following equation (HCM Equation 13-5):

$$c_{IWL} = c_{IFL} - [438.2(1 + VR)^{1.6}] + (0.0765L_S) + (119.8N_{WL})$$

where

c_{IWL} = per-lane weaving segment capacity under equivalent ideal conditions (pc/h/ln),

c_{IFL} = per-lane capacity of a basic freeway (or multilane highway, as appropriate) segment with the same free-flow speed as the weaving segment, under equivalent ideal conditions (pc/h/ln) (see Section 11.3.2),

VR = volume ratio,

L_S = short length of the weaving segment (ft), and

N_{WL} = number of weaving lanes.

The capacity of a weaving segment based on weaving flow c_{IW} (pc/h) is either (2,400 / VR) when N_{WL} = 2, or (3,500 / VR) when N_{WL} = 3. No capacity value for weaving flow is defined for two-sided weaving segments; the capacity based on density is used instead.

These two capacity values (based on ideal conditions) are then converted into capacities under prevailing conditions as follows (HCM Equations 13-6 and 13-8):

$$c_W = c_{IWL} \times N \times f_{HV}$$
$$c_W = c_{IW} \times f_{HV}$$

where c_W is the weaving segment capacity under prevailing conditions (veh/h), N is the number of lanes within the weaving area, and all other variables are as defined previously. The lower of the two values of c_W is taken as the capacity of the weaving segment under prevailing conditions.

As described previously in Section 11.3.2, Step 3, capacity can be optionally adjusted using a CAF. For an initial (i.e., pre-calibration) capacity analysis, the only CAFs that might normally be used are those for driver population CAF_{pop} (see Appendix 11B) and ramp metering CAF_{meter} (described below). All applicable CAFs are multiplied together to create an overall CAF, which is applied as follows (HCM Equation 13-9):

$$c_{wa} = c_w \times CAF$$

where

c_{wa} = adjusted weaving segment capacity (veh/h),

c_w = unadjusted weaving segment capacity (veh/h), and

CAF = capacity adjustment factor (decimal), default = 1.00.

Finally, the volume-to-capacity ratio v/c is determined as follows (HCM Equation 13-10):

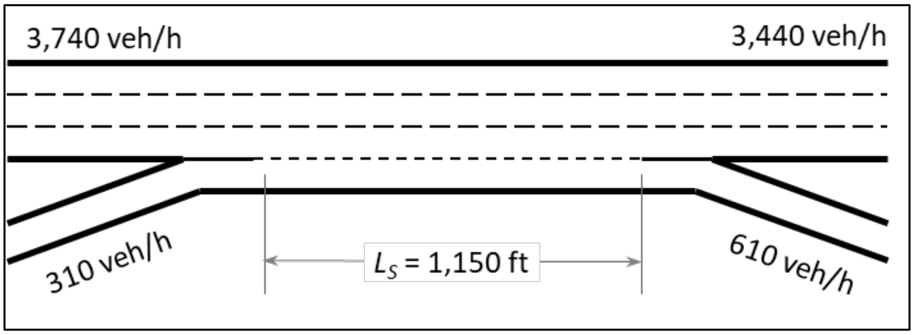
$$v/c = \frac{v \times f_{HV}}{c_{wa}}$$

where all variables have been defined previously. The heavy vehicle adjustment factors are used multiple times because v reflects equivalent ideal conditions, while c_{wa} and any capacity adjustment factors reflect prevailing conditions.

Conditions external to the weaving segment may also create capacity constraints. Therefore, the capacity of the freeway mainline should be compared to the entering and exiting demand flows (see Section 11.3.2) and the capacity of the ramps should be compared to their demand flows (see Section 11.3.3).

Example 11-9 Weaving Segment Analysis (Detailed Method)

Step 1. Gather Input Data. The weaving segment has the lane configuration and entering and exiting volumes shown below:



In addition, 50 veh/h are counted making ramp-to-ramp movements. The remaining weaving movements are determined as follows:

- Ramp-to-freeway = 310 – 50 = 260 veh/h
- Freeway-to-ramp = 610 – 50 = 560 veh/h
- Freeway-to-freeway = 3,740 – 560 = 3,180 veh/h

This section of freeway is a downgrade and has 6.3% heavy vehicles, a driver population of regular commuters, and a PHF of 0.94. The freeway has a free-flow speed of 65 mph, the on-ramp has a free-flow speed of 45 mph, and the off-ramp has a free-flow speed of 30 mph. The on-ramp has 2.7% heavy vehicles, while the off-ramp has 9.1% heavy vehicles.

Step 2. Adjust Volumes. First, the heavy vehicle factor is calculated. In level terrain (which includes most downgrades), a heavy vehicle's PCE is 2. Then:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.063(2 - 1)} = \frac{1}{1.063} = 0.941$$

The freeway-to-freeway demand flow rate is then:

$$v_{FF} = \frac{V_{FF}}{PHF \times f_{HV}} = \frac{3{,}180}{0.94 \times 0.941} = 3{,}595 \text{ pc/h}$$

Similarly, the other flow rates are:

- v_{RR} = 55 pc/h

- v_{RF} = 284 pc/h
- v_{FR} = 650 pc/h
- on-ramp = 339 pc/h
- freeway mainline entering = 4,229 pc/h
- freeway mainline exiting = 3,890 pc/h
- off-ramp = 708 pc/h

The total demand flow rate in the weaving segment is 3,595 + 55 + 284 + 650 = 4,584 pc/h. The volume ratio is the weaving flow divided by the total flow: (284 + 650) / 4,584 = 0.204.

Step 3. Determine the Number of Weaving Lanes. By comparison with the uppermost weaving area depicted in Exhibit 11-9, N_{WL} is 2.

Step 4. Determine the Maximum Weaving Length. The maximum length of a weaving segment with this configuration and volume ratio is:

$$L_{MAX} = [5{,}728(1 + VR)^{1.6}] - (1{,}566N_{WL}) = [5{,}728(1 + 0.204)^{1.6}] - (1{,}566 \times 2) = 4{,}577 \text{ ft}$$

The short length of this weaving area, 1,150 ft, is less than the maximum length. Therefore, it is correct to treat the weaving area as a weaving segment and the process continues to Step 5.

Step 5. Determine the Weaving Segment Capacity and v/c Ratio. First, the ideal capacity of an equivalent basic segment with a 65-mph free-flow speed must be determined. From Exhibit 12-7 in the HCM 7th Edition, this value is 2,350 pc/h/ln. The weaving segment's ideal capacity based on density is then:

$$c_{IWL} = c_{IFL} - [438.2(1 + VR)^{1.6}] + (0.0765L_S) + (119.8N_{WL})$$
$$c_{IWL} = 2{,}350 - [438.2(1 + 0.204)^{1.6}] + (0.0765 \times 1{,}150) + (119.8 \times 2) = 2{,}088 \text{ pc/h/ln}$$

The weaving segment's ideal capacity based on weaving flow is 2,400 / 0.204 = 11,765 pc/h.

These two capacities are then converted into capacities under prevailing conditions as follows:

$$c_W = c_{IWL} \times N \times f_{HV} = 2{,}088 \times 4 \times 0.941 = 7{,}859 \text{ pc/h}$$
$$c_W = c_{IW} \times f_{HV} = 11{,}765 \times 0.941 = 11{,}071 \text{ pc/h}$$

The lower of these two values, 7,859 pc/h, is taken as capacity. No adjustment to capacity is needed for driver population, therefore $c_{WA} = c_W$. Finally, the v/c ratio is determined to be:

$$v/c = \frac{v \times f_{HV}}{c_{wa}} = \frac{4{,}584 \times 0.941}{7{,}859} = 0.55$$

The capacity of the freeway mainline entering and exiting the weaving segment is (3 × 2,350) = 7,050 pc/h, which is greater than the entering and exiting demands. From Exhibit 14-12 in the HCM 7th Edition, the capacity of a single-lane ramp with a 45-mph free-flow speed is 2,100 pc/h, which is greater than the on-ramp demand. Similarly, the capacity of a single-lane ramp with a 30-mph free-flow speed is 1,900 pc/h, which is greater than the off-ramp demand. Therefore, there are no external capacity constraints to address.

<u>Ramp Metering</u>

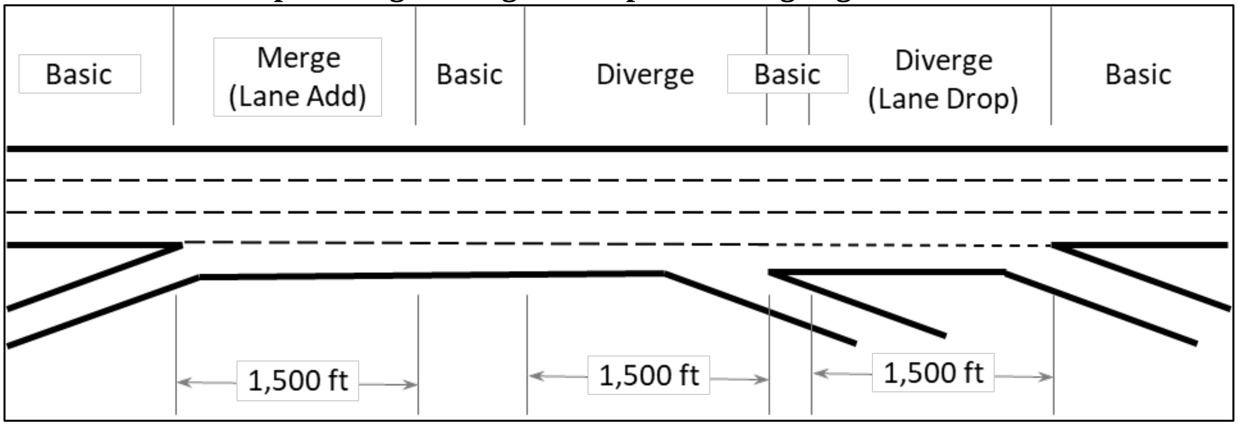
The effects of ramp metering on weaving area capacity can be addressed through a capacity adjustment factor. No specific guidance exists in the HCM for the capacity effects of ramp metering in a weaving segment. In the absence of local data, the HCM capacity adjustment factor CAF_{meter} for merge segments can be used to approximate the effect on weaving segments. The HCM 7th Edition (Chapter 37, Section 4) recommends a value of 1.03 when ramp metering is in operation. Ramp metering will reduce the on-ramp capacity to a level equivalent to the metering rate.

<u>Multiple Weaving Segments</u>

Multiple weaving segments exist when the combination of on- and off-ramps creates weaving movements between multiple sets of origins and destinations. The HCM 7th Edition recommends analyzing these as a series of merge, diverge, basic, and simple weaving segments.

Exhibit 11-10 shows an example of a multiple weaving segment with one on-ramp followed by two off-ramps, all connected by an auxiliary lane. The 1,500 feet downstream of the on-ramp is a merge segment with a lane add. Because of the added lane, the segment is analyzed the same as a basic segment; however, it is coded in software as a merge segment because of the need to account for the volume entering the freeway at this point. The portion of the freeway 1,500 feet upstream of the first off-ramp is analyzed as a diverge segment, while the portion of the freeway between the merge and diverge segments is analyzed as a basic segment. The 1,500 feet of the freeway upstream of the second off-ramp is a diverge segment with a lane drop; it is analyzed as a basic segment but is coded in software as a diverge segment to account for the volume leaving the freeway. Finally, the portion of the freeway between the two diverge segments is analyzed as a basic segment.

Exhibit 11-10 Example of Segmenting a Multiple Weaving Segment



11.3.4 Freeway Facilities

Freeway facility methods evaluate the operations of an extended stretch of freeway. The detailed and screening analysis methods aggregate the results of individual segment or section analyses to generate a range of additional useful performance measures, including average travel speed and vehicle-hours of delay, as well as measures of travel time reliability (discussed in Section 11.3.7). These two methods can be applied to study periods more than one hour long and to freeways operating over capacity. This section also provides a broad-brush facility analysis method, consisting of a table of generalized capacity values that can be adjusted to local conditions, to the extent data (e.g., heavy-vehicle percentage, peak hour factor) are available.

Broad-Brush Analysis Method

Exhibit 11-11 contains a table providing estimates of design-hour, peak-direction freeway capacities. If only AADTs are available, they can be converted to a directional design-hour volume *V* by applying *K*- and *D*-factors as follows:

$$V = AADT \times \frac{K}{100} \times \frac{D}{100}$$

Exhibit 11-11 Generalized Freeway Capacities (veh/h)

Area Type	Terrain	Posted Automobile Speed Limit (mph)				
		50	55	60	65	70
Urban	Level	3,825	3,910	3,995	4,080	4,165
	Rolling	3,655	3,735	3,815	3,895	3,980
	Mountainous	3,350	3,425	3,500	3,570	3,645
Rural	Level	3,215	3,285	3,360	3,430	3,500
	Rolling	2,680	2,740	2,800	2,860	2,915
	Mountainous	2,010	2,055	2,100	2,145	2,190

Note: Assumptions used in this table are: HV% = 5 (urban), 25 (rural); PHF = 0.94; 2 lanes per direction; free-flow speed = posted speed + 5 mph; driver population familiar with the facility; CAF_{ramp} = 0.95.

When specific heavy vehicle percentages and/or peak-hour factors are known, the number of lanes is greater than two, or driver population or CAV effects are desired to be included, the values in Exhibit 11-11 can be adjusted as follows to provide a better estimate of capacity reflecting the local conditions:

$$c_{adj} = c_{table} \times \frac{PHF_{local}}{PHF_{table}} \times \frac{1 + (E_T - 1)(\%HV_{table}/100)}{1 + (E_T - 1)(\%HV_{local}/100)} \times \frac{N_{local}}{2} \times CAF_{pop} \times CAF_{CAV}$$

where

c_{adj} = adjusted facility capacity (veh/h);

c_{table} = capacity value from Exhibit 11-14 (veh/h);

PHF_{local} = local peak hour factor (decimal);

PHF_{table} = peak hour factor assumed in Exhibit 11-14 (decimal);

E_T = truck equivalency = 2 (level terrain), 3 (rolling terrain), or 5 (mountainous terrain);

$\%HV_{table}$ = heavy vehicle percentage assumed in Exhibit 11-14 (e.g., 25% = 25);

$\%HV_{local}$ = local heavy vehicle percentage (e.g., 25% = 25);

N_{local} = number of directional travel lanes;

CAF_{pop} = local capacity adjustment factor for driver population; and

CAF_{CAV} = optional capacity adjustment factor for CAVs from Appendix 6B (default = 1.00).

Example 11-10 Freeway Capacity Analysis (Broad-Brush Method)

A six-lane urban freeway (three lanes in each direction) is located in rolling terrain and has a 50-mph speed limit. The AADT is 121,400, the *K*-factor is 7.7, the *D*-factor is 54, the PHF is 0.92, the heavy-vehicle percentage is 9.1, and there are no CAVs.

The design-hour volume *V* is:

$$V = AADT \times \frac{K}{100} \times \frac{D}{100} = 121{,}400 \times \frac{7.7}{100} \times \frac{54}{100} = 5{,}050 \text{ veh/h}$$

The capacity obtained from Exhibit 11-14, which assumes 5% heavy vehicles, a PHF of 0.94, and two travel lanes, is 3,655 veh/h. An adjusted local capacity can be determined as follows by substituting the local heavy-vehicle percentage, PHF, and number of lanes, while keeping the table values for all other inputs that are unknown or unchanged:

$$c_{adj} = c_{table} \times \frac{PHF_{local}}{PHF_{table}} \times \frac{1 + (E_T - 1)(\%HV_{table}/100)}{1 + (E_T - 1)(\%HV_{local}/100)} \times \frac{N_{local}}{2} \times CAF_{pop} \times CAF_{CAV}$$

$$c_{adj} = 3{,}655 \times \frac{0.92}{0.94} \times \frac{1 + (3 - 1)(5/100)}{1 + (3 - 1)(9.1/100)} \times \frac{3}{2} \times 1.00 \times 1.00 = 4{,}994 \text{ veh/h}$$

The v/c ratio is then (5,050 / 4,994) = 1.01.

Screening Analysis Method

Definition of a Freeway Facility

Section H6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a simplified method for evaluating the performance of a freeway facility or "supersection." A supersection consists of multiple contiguous freeway sections, extending up to the distance that an automobile can drive at the posted speed in 15 minutes—typically 9 to 12 miles in urban areas and up to 15 miles in rural areas. In addition to the maximum length criterion, other criteria to be considered when defining the endpoints of a supersection include: freeway-to-freeway interchanges, urbanized area boundaries, major intersecting routes, major trip generators (e.g., central city downtowns, major airports), and the state border. Unlike the detailed method, it is not necessary that the first and last segments of the supersection operate below capacity throughout the study period, although the mainline demand entering the first section should be below the section's capacity throughout the study period. One of the method's simplifications is that all unserved demand to a section is stored as a vertical queue at the section entrance, rather than being propagated into the upstream section.

Study Period Length

The method was designed for a 1-hour study period, although longer study periods can be studied if the analyst has (or can generate) 15-minute traffic demands for the analysis periods beyond the peak hour. To capture the full effects of any congestion that occurs, the entire facility should operate below capacity during the first and last 15-minute analysis periods.

Applicability

The method is applicable to general-purpose freeway lanes. All of the screening method limitations described in Sections 11.3.2 through 11.3.4 also apply to freeway facilities. In particular, if the capacity of an off-ramp roadway or terminal is exceeded, causing queues to spill back onto the mainline, the facility method cannot be used and an alternative analysis tool will be required.

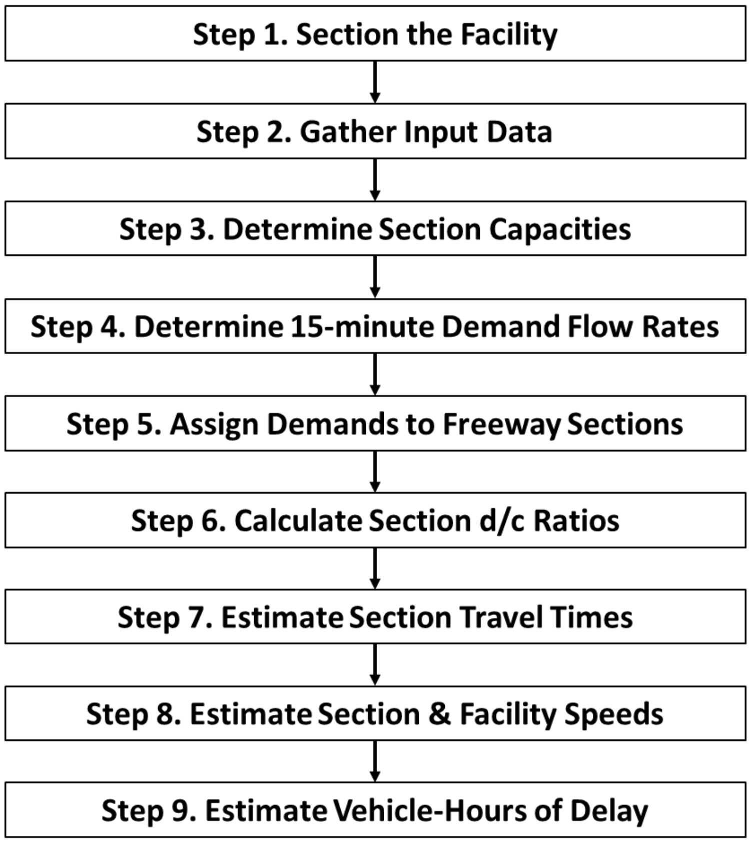
Calculation Process

The method estimates the average travel time required to traverse each freeway section during each 15-minute analysis period, with appropriate adjustments for the effects of over-capacity conditions within a given segment. The method also meters demand to downstream segments when a segment’s capacity is reached. The travel time results are aggregated to the facility level to produce the following performance measures for each analysis period:

- Average travel time
- Average speed
- Vehicle hours of delay

The PPEAG also describes how to estimate average density, level of service, and queuing at a section level once each section’s demand-to-capacity ratio has been calculated.

The following steps are involved in performing a freeway facility analysis at a screening level:



Although it is feasible to perform a screening-level facility analysis by hand, it is more efficient to automate the process in a spreadsheet, such as the freeway planning tool developed for the PPEAG that was described in Section 11.2.5. The analysis process is as follows.

Step 1. Section the facility. Determine the start and end points of the facility and the number of 15-minute periods to include in the analysis are determined, following the guidelines given above. Next, divide the facility into sections, with section boundaries at each ramp gore point. Categorize each section as a basic, ramp, or weave section, following the guidance in Sections 11.3.2 through 11.3.4. Basic freeway segments should be split in two when a lane add or drop occurs, when a change in the terrain classification occurs, or when the free-flow speed changes as a result of changes in the roadway geometry or posted speed limit.

Step 2. Gather input data. All of the data required for a screening analysis of the individual freeway sections are required, plus the length of each section (see Section 11.3.1 for details).

Step 3. Determine section capacities. Determine the capacity of each freeway section by following the screening methods described in Sections 11.3.2 through 11.3.4.

Step 4. Determine 15-minute demand flow rates. This step is only required if 15-minute volumes are unavailable and is performed for the mainline volume entering the first section and for all ramp entering and exiting volumes. First, if hourly volumes are unavailable, convert each AADT to an hourly volume. The demand flow rate in each of the four 15-minute analysis periods is then synthesized using the following equation, derived from PPEAG Equation 17:

$$v_t = \begin{cases} V & t = 1,3 \\ V \times \left(\dfrac{1}{PHF}\right) & t = 2 \\ V \times \left(2 - \dfrac{1}{PHF}\right) & t = 4 \end{cases}$$

where

v_t = demand flow rate during analysis period t (veh/h),

V = hourly volume (veh/h), and

PHF = peak hour factor (decimal).

This process assigns the highest volume to the second analysis period and the lowest volume to the last analysis period, while the first and third analysis periods are assigned flow rates equivalent to the average volume during the peak hour.

The mainline demand entering the first section during analysis period 2 should be compared to the section capacity. If the demand exceeds capacity, the study area should be expanded upstream.

The demand on each on-ramp during each analysis period should be compared to a capacity value of 2,000 veh/h/ln. If demand exceeds capacity, the excess demand is stored and added to the on-ramp demand for the following analysis period (note that this condition may mean that queues will spill back into the ramp terminal intersection and beyond).

The demand during analysis period 2 on each off-ramp during each analysis period should also be compared to a capacity value of 2,000 veh/h/ln. If demand exceeds capacity, queues will spill back from the ramp onto the mainline and the screening procedure is not applicable. If this situation happens, the analysis stops at this point and a more detailed analysis method should be considered.

Step 5. Assign demands to freeway sections. This process starts with the first freeway section during the first analysis period and works downstream to the end of the facility. It then repeats for each of the remaining analysis periods.

The entry demand to a given section consists of the mainline entering demand plus the on-ramp demand (if an on-ramp exists in the section), plus any unserved demand in the section carried over from the previous analysis period. This demand is compared to the section's capacity as follows:

- If demand is less than or equal to capacity, the demand for the off-ramp (if present) is used as-is. The mainline exiting demand is then the entry demand minus any off-ramp demand; this demand becomes the mainline entering demand in the next downstream section.
- If demand is greater than capacity, the section's entry volume is set to capacity and the excess demand is carried over to the next analysis period. The off-ramp volume is reduced in the same proportion as the mainline segment volume. The mainline exiting demand is then the section capacity minus the adjusted off-ramp demand; this demand becomes the mainline entering demand in the next downstream section.

Step 6. Calculate section d/c ratios. These are calculated for each section for each analysis period. The d/c ratio is the section entry demand divided by the section capacity.

Step 7. Estimate section travel times. The section travel time is estimated from three components: (1) the time to travel the section at the free-flow speed, (2) extra delay occurring during undersaturated (under-capacity) conditions, and (3) extra delay occurring as a result of oversaturated (over-capacity) conditions. The following equations are used, derived from Equations 20 through 22 in the PPEAG:

$$T_{i,t} = \frac{3{,}600L_i}{FFS_i} + L_i\left(\Delta_{RU_{i,t}} + \Delta_{RO_{i,t}}\right)$$

with

$$\Delta_{RU_{i,t}} = \begin{cases} 0 & X_{i,t} < E \\ A\left(X_{i,t}\right)^3 + B\left(X_{i,t}\right)^2 + C\left(X_{i,t}\right) + D & E \le X_{i,t} \end{cases}$$

$$X_{i,t} = \min(1, d_{i,t}/c_i)$$

$$\Delta_{RO_{i,t}} = \frac{900}{2L_i}\left(\max\left[1, d_{i,t}/c_i\right] - 1\right)$$

where

$T_{i,t}$ = travel time for section *i* during analysis period *t* (s),

L_i = length of section *i* (mi);

FFS_i = free-flow speed of section *i* (mph),

$\Delta_{RU_{i,t}}$ = undersaturated delay rate for section *i* during analysis period *t* (s/mi),

$\Delta_{RU_{i,t}}$ = oversaturated delay rate for section i during analysis period t (s/mi),

A, B, C, D, E = parameters from Exhibit 11-12,

$d_{i,t}$ = demand in section i during analysis period t (veh/h),

c_i = capacity of section i (veh/h), and

900 = analysis period length (s) = 15 minutes.

Exhibit 11-12 Parameters for Screening-Level Freeway Speed Estimation Equation

FFS (mph)	A	B	C	D	E
75	**68.99**	**−77.97**	**34.04**	**−5.82**	**0.44**
70	**71.24**	**−85.48**	**35.58**	**−5.44**	**0.52**
65	**92.45**	**−127.33**	**56.34**	**−8.00**	**0.62**
60	**121.35**	**−184.84**	**83.21**	**−9.33**	**0.72**
55	**156.43**	**−248.99**	**99.20**	**−0.12**	**0.82**

Step 8. Estimate section and facility speeds. The average speed $S_{i,t}$ in each section i during analysis period t is calculated as follows:

$$S_{i,t} = \frac{3{,}600 L_i}{T_{i,t}}$$

where all variables are as defined previously. The average facility speed $S_{F,t}$ during analysis period t is then:

$$S_{F,t} = \frac{3{,}600 \sum_i L_i}{\sum_i T_{i,t}}$$

Step 9. Estimate vehicle-hours of delay. The vehicle-hours of delay $VHD_{i,t}$ in section i during analysis period t, based on a threshold speed S_{TH} (mph) when delay is considered to begin, is calculated as follows:

$$VHD_{i,t} = \max\left(0, \frac{0.25 v_{i,t} \times L_i}{S_{i,t}} - \frac{0.25 v_{i,t} \times L_i}{S_{TH}}\right)$$

where all variables are as defined previously. The total facility vehicle-hours of delay is then simply the sum of all section VHD values across all analysis periods. The threshold speed is normally set as the posted speed for automobiles.

Detailed Analysis Method

Definition of a Freeway Facility

Chapter 10 of the HCM provides methods for evaluating the performance of freeway facilities. A facility consists of multiple contiguous freeway segments, extending up to the distance that an automobile can drive at the posted speed in 15 minutes—typically 9 to 12 miles in urban areas and up to 15 miles in rural areas. In addition to the maximum length criterion, other criteria to be considered when defining the endpoints of a facility include freeway-to-freeway interchanges, urbanized area boundaries, major intersecting routes, major trip generators (e.g., central city downtowns, major airports), and the state border. It is important that the first and last segments of the facility operate below capacity throughout the defined analysis period, so that all the effects of any congestion that may occur are captured by the analysis.

For longer facilities, the analyst needs to carefully consider the demand inputs and the interpretation of results. The method assumes 15-minute demands to be applied instantaneously across the entire facility, even if travel times are longer than 15-minutes. In these cases, the delay, congestion, and queuing impacts are valid, but the temporal onset of congestion may be estimated to occur too early.

Study Period Length

The method accommodates study periods of 1 hour and longer, with an upper limit being set by (1) the needs of the analysis and (2) the capabilities of the software being used to apply the method. At a minimum, the entire facility should operate below capacity during the first and last 15-minute analysis periods. The FREEVAL software allows for up to 96 15-minute analysis periods to be analyzed, resulting in a full 24-hour study period.

Applicability

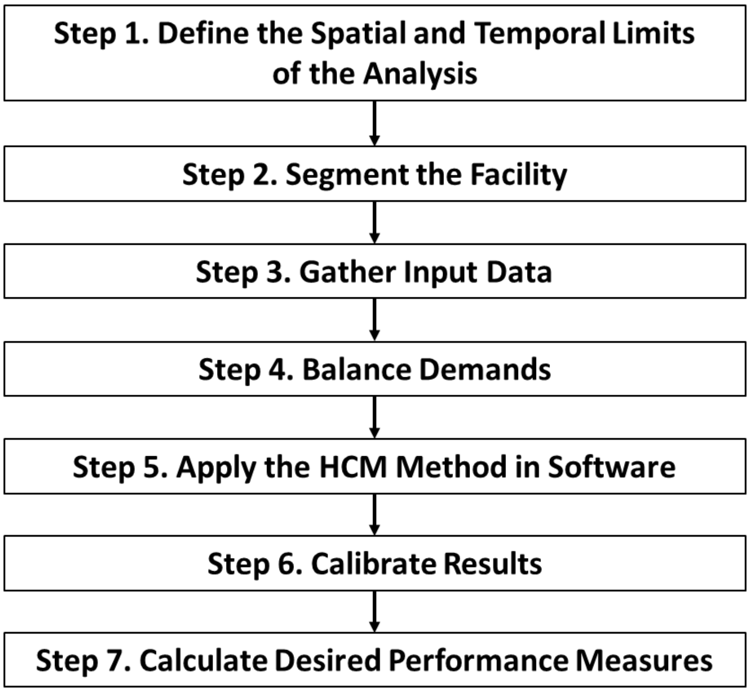
The method can be applied directly to general-purpose freeway lanes. Some aspects of the method (e.g., free-flow speed estimation) are also used as part of the analysis of merge, diverge, and weaving segments. Chapter 10 of the HCM provides extensions to the method for evaluating managed lane facilities and freeway work zones.

All of the detailed method limitations described in Sections 11.3.2 through 11.3.4 also apply to freeway facilities. In particular, if the capacity of an off-ramp roadway or terminal is exceeded, causing queues to spill back onto the mainline, the facility method cannot be used, and an alternative analysis tool will be required. The method does not directly estimate changes in demand resulting from congestion (e.g., motorists choosing alternative routes or modes), but the analyst can manually provide estimates of these changes with a demand adjustment factor. A demand adjustment factor (DAF) is a factor that is used to multiply all entering and/or exiting demands on the facility. The DAF can be less than 1.0 to estimate diversion effects to alternate facilities, or greater than 1.0 to estimate impacts of diversion onto the subject facility, or to evaluate future traffic growth scenarios.

STOP *Because detailed analysis is intended for near-term situations when all or nearly all inputs are known, and because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in detailed analyses.*

Calculation Process

The following steps are involved in performing a freeway facility analysis at a detailed level:



The method estimates the average travel time required to traverse each freeway segment during each 15-minute analysis period, with appropriate adjustments for the effects of upstream and downstream bottlenecks (e.g., queue spillback, demand metering). The results are then aggregated to the facility level to produce the following performance measures for each analysis period:

- Average travel time
- Average speed
- Average density
- Vehicle miles traveled (VMT)
- Vehicle hours traveled (VHT)
- Vehicle hours of delay (VHD)
- Level of service

Since the freeway facility analysis captures multiple segments over multiple time periods, the results may be displayed in the form of contour plots or heat maps. The example in Exhibit 11-13 below shows a heat map of facility speeds, with each cell representing the result of a 15-minute time period for one segment. Cells are formatted to show high (free-flow) speeds in green and slow (congested) speeds in red. The analysis shows is for a 34-segment facility that was analyzed over seven hours (28 time periods) from 2pm (14:00) to 9pm (21:00).

Exhibit 11-13 Speed Contour Example for Freeway Facilities Method

Analysis Period	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6	Seg. 7	Seg. 8	Seg. 9	Seg. 10	Seg. 11	Seg. 12	Seg. 13	Seg. 14	Seg. 15	Seg. 16	Seg. 17	Seg. 18	Seg. 19	Seg. 20	Seg. 21	Seg. 22	Seg. 23	Seg. 24	Seg. 25	Seg. 26	Seg. 27	Seg. 28	Seg. 29	Seg. 30	Seg. 31	Seg. 32	Seg. 33	Seg. 34
#1 14:00 - 14:15	70	65	65	69	63	63	63	68	69	63	70	63	69	63	70	65	70	70	70	69	70	65	69	69	70	65	65	70	69	70	63	62	61	62
#2 14:15 - 14:30	69	65	65	69	63	63	63	68	69	63	70	63	69	63	70	65	69	70	70	68	70	65	69	68	70	65	65	70	68	70	63	60	60	60
#3 14:30 - 14:45	69	65	65	69	63	62	62	68	69	63	70	63	68	63	70	65	69	70	70	68	70	65	69	68	70	65	65	70	68	70	63	57	60	57
#4 14:45 - 15:00	69	65	65	69	62	62	62	68	69	63	70	63	68	63	70	65	69	70	70	68	70	65	69	68	70	65	64	70	68	70	62	55	59	55
#5 15:00 - 15:15	68	65	65	69	62	62	62	68	69	62	70	62	68	62	70	64	69	70	70	68	70	64	69	68	70	64	64	70	68	70	38	58	53	58
#6 15:15 - 15:30	67	65	65	69	62	62	62	68	69	62	70	62	68	62	70	64	69	70	70	68	70	64	69	68	70	64	64	70	68	70	21	58	48	58
#7 15:30 - 15:45	66	65	65	68	62	61	61	67	69	62	70	62	68	62	70	64	69	70	70	68	70	64	69	68	70	64	64	70	68	70	13	58	43	58
#8 15:45 - 16:00	66	65	65	68	62	61	61	67	69	61	70	61	68	61	70	64	69	70	70	68	70	64	69	68	70	64	64	69	68	70	8	58	39	58
#9 16:00 - 16:15	65	65	65	67	61	61	61	67	69	61	70	61	68	61	70	64	69	70	70	68	70	64	69	68	70	64	64	57	33	14	7	58	35	58
#10 16:15 - 16:30	64	64	65	67	61	61	61	67	69	61	70	61	68	61	70	64	69	70	70	68	70	64	69	46	27	20	17	20	14	12	7	58	32	58
#11 16:30 - 16:45	64	64	65	66	61	61	61	67	69	61	70	61	68	61	70	64	69	70	48	33	19	11	9	11	10	11	13	21	15	15	7	58	29	58
#12 16:45 - 17:00	64	64	65	66	61	60	60	67	69	61	70	61	68	61	55	32	23	11	8	9	8	9	10	13	12	13	15	21	15	15	7	58	27	58
#13 17:00 - 17:15	64	64	65	66	61	60	60	67	69	61	69	37	28	12	9	9	8	7	7	11	9	9	10	13	12	13	15	22	15	15	7	58	25	58
#14 17:15 - 17:30	66	65	65	67	62	61	61	66	52	16	10	7	11	10	10	12	10	9	10	14	10	11	11	16	12	11	15	22	15	15	7	58	24	58
#15 17:30 - 17:45	67	65	65	69	54	54	44	44	21	12	13	9	12	10	11	12	8	7	7	10	10	9	10	15	12	13	15	21	16	15	7	58	24	58
#16 17:45 - 18:00	68	65	65	68	45	35	20	20	12	9	11	8	12	9	11	12	11	11	8	11	10	10	11	14	12	13	15	21	16	15	7	58	25	58
#17 18:00 - 18:15	69	65	65	68	49	43	20	20	14	10	12	9	12	10	10	10	7	8	8	14	11	10	11	15	12	12	15	22	16	15	7	58	26	58
#18 18:15 - 18:30	70	65	65	69	63	63	51	51	18	9	10	7	11	11	10	11	9	8	8	12	10	11	12	16	13	14	16	21	17	15	7	58	27	58
#19 18:30 - 18:45	70	65	65	69	63	63	63	68	69	16	11	9	12	11	13	13	11	9	8	14	11	11	11	15	13	15	16	22	16	15	7	58	29	58
#20 18:45 - 19:00	70	65	65	69	64	64	64	68	70	56	41	39	46	15	11	11	11	10	9	13	12	10	12	16	14	13	17	22	16	15	7	58	32	58
#21 19:00 - 19:15	70	65	65	69	64	64	64	68	70	65	70	65	69	58	37	23	12	10	9	15	12	11	11	16	15	16	13	21	16	16	7	58	37	58
#22 19:15 - 19:30	70	65	65	69	64	64	64	68	70	65	70	65	69	65	70	66	70	63	31	24	19	13	12	17	14	13	14	22	17	16	7	58	43	58
#23 19:30 - 19:45	70	65	65	69	64	64	64	68	70	65	70	65	69	65	70	66	70	70	70	69	70	66	69	43	30	26	21	25	16	16	7	58	51	58
#24 19:45 - 20:00	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	37	28	7	58	65	58
#25 20:00 - 20:15	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	69	70	18	58	70	58
#26 20:15 - 20:30	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	69	70	68	70	67	70
#27 20:30 - 20:45	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	69	70	66	70	65	70
#28 20:45 - 21:00	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	69	70	66	70	65	70

The speed contours show several hours of congestion from roughly 3:30pm to 8pm, and a queue that extends at its maximum to segment 5. The boundaries of the analysis (first and last time period and first and last segment) are not congested, suggesting a well-defined analysis period. For a freeway facility analysis, all congestion should preferably be fully contained within the specified time-space domain.

Due to the number of calculations involved, software is necessary to perform a detailed freeway facilities analysis. Analysis tools that were available at the time of writing for performing a freeway facilities analysis were described in Section 11.2.5. The analysis process is as follows.

Step 1. Define the spatial and temporal limits of the analysis. The start and end points of the facility and the number of 15-minute periods to include in the analysis are determined, following the guidelines given above.

Step 2. Segment the facility. The facility is first divided into sections between ramp gore points. Next, merge, diverge, and weave segments are identified, following the guidance in Sections 11.3.2 and 11.3.3. The remaining unassigned portions of the facility then become basic freeway segments. Basic freeway segments should be split when a lane add or drop occurs, when a change in the terrain classification occurs, or when the free-flow speed changes as a result of changes in the roadway geometry.

Some adjustments to segment boundaries may be required in the case of closely spaced ramps (where ramp influence areas overlap each other) and very long weaving segments (where the segment length exceeds the maximum weaving length). Step A-2 of the HCM methodology, starting on page 10-24 of the HCM 7th Edition, details the segmentation process, including these special cases.

Step 3. Gather input data. All of the data required for a detailed analysis of the individual freeway segments are required, except for the peak hour factor (see Section 11.3.1 for details). In lieu of a peak hour factor, 15-minute volumes must be provided for each segment for each analysis period.

The length of each segment is also required, along with two parameters not used by segment-based analyses: jam density and queue discharge capacity drop. These parameters can initially be defaulted to 190 pc/h/ln and 7%, respectively, if local values are not available. Appendix 11B describes how these parameters can be used later in the process to calibrate the method to match field conditions.

Step 4. Balance demands. Both the total entering and the total exiting demand for the facility (considering both the freeway mainline and all on- and off-ramps located along the facility) should be determined for each analysis period. If the entering demand does not equal the exiting demand (for example, because traffic counts are used to estimate demand and congestion is occurring along the facility that prevents entering demand from reaching its desired exit), the demands in that time period will need to be adjusted. Step A-4 of the HCM methodology, starting on page 10-28 of the HCM 7th Edition, describes the balancing process.

Step 5. Apply the HCM methodology in software. All of the information about the facility is coded into the software and the software is used to determine individual segment capacities, v/c ratios, and locations and extent of congestion.

Step 6. Calibrate results. The initial software results should be compared to available information about existing conditions. If necessary, stepwise adjustments can be made first to free-flow speed, then capacity, and (as a last resort) to demand to match existing conditions. Appendix 11B describes this process.

Step 7. Calculate desired performance measures. Once the facility has been satisfactorily calibrated, the software can then be used to report performance measures for both existing conditions and desired future scenarios that the analyst subsequently codes. Vehicle-hours of delay are calculated by comparing actual travel times to the travel time at the posted speed limit. The portion of the travel time greater than the travel time at the speed limit is considered to be delay.

11.3.5 Multilane Highway Facilities

No guidance is presented for the effects of CAVs on multilane highway capacity because no research has been conducted yet for these roadway types. However, it may be reasonable to apply the capacity adjustments for basic freeway segments provided in Appendix 6B to multilane highways for broad-brush and screening level analyses, given the similar operating characteristics and methodologies for freeways and multilane highways.

Broad-Brush Analysis Method

Exhibit 11-14 contains a table providing estimates of design-hour, peak-direction capacities along multilane highways without traffic signals. It is applied in the same way that Exhibit 11-11 in Section 11.3.4 is applied.

Exhibit 11-14 Generalized Multilane Highway Capacities (veh/h)

Area Type	Terrain	Posted Automobile Speed Limit (mph)				
		45	50	55	60	65
Urban	Level	3,620	3,800	3,980	4,160	4,345
	Rolling	3,455	3,625	3,800	3,975	4,145
	Mountainous	3,165	3,325	3,485	3,640	3,800
Rural	Level	2,815	2,955	3,100	3,240	3,380
	Rolling	2,345	2,465	2,580	2,700	2,815
	Mountainous	1,760	1,850	1,935	2,025	2,110

Note: Assumptions used in this table are: HV% = 5 (urban), 25 (rural); PHF = 0.95 (urban), 0.88 (rural); 2 lanes per direction; free-flow speed = posted speed + 5 mph; driver population familiar with the facility.

Example 11-11 Multilane Highway Capacity Analysis (Broad-Brush Method)

A rural four-lane multilane highway (two lanes in each direction) is located in rolling terrain with a 55-mph speed limit. The AADT is 21,700, the K factor is 16.2, the D factor is 62, and the heavy-vehicle percentage is 18.6.

The design-hour volume V is:

$$V = AADT \times \frac{K}{100} \times \frac{D}{100} = 21{,}700 \times \frac{16.2}{100} \times \frac{62}{100} = 2{,}180 \text{ veh/h}$$

The capacity obtained from Exhibit 11-14, which assumes 25% heavy vehicles, is 2,580 veh/h. An adjusted local capacity can be determined as follows by substituting the local heavy-vehicle percentage and keeping the table values for all other inputs that are unknown or unchanged:

$$c_{adj} = c_{table} \times \frac{PHF_{local}}{PHF_{table}} \times \frac{1 + (E_T - 1)(\%HV_{table}/100)}{1 + (E_T - 1)(\%HV_{local}/100)} \times \frac{N_{local}}{2} \times CAF_{pop}$$

$$c_{adj} = 2{,}580 \times \frac{0.88}{0.88} \times \frac{1 + (3 - 1)(25/100)}{1 + (3 - 1)(18.6/100)} \times \frac{2}{2} \times 1.00 = 2{,}820 \text{ veh/h}$$

The v/c ratio is then (2,180 / 2,820) = 0.77.

Screening Analysis Method

Definition of a Multilane Highway Facility

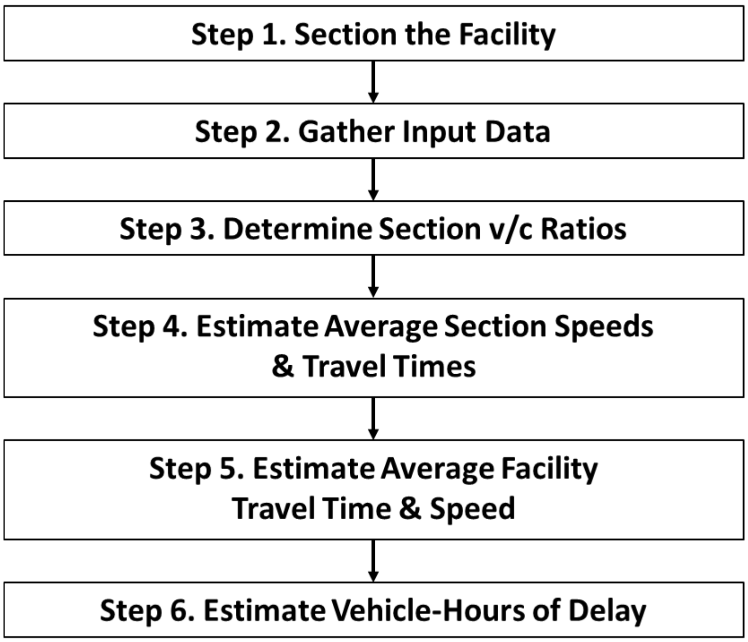
Section I6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a method for evaluating the performance of a multilane highway facility. A facility consists of one or more contiguous multilane highway sections, along with the signalized intersection (if present) at the end of the facility. The maximum facility length should be the distance that an automobile can drive at the posted speed in 15 minutes. A facility boundary should also be established at a location where the highway transitions to a different facility type (e.g., two-lane highway, freeway, urban street). In addition to these criteria, other criteria that could be considered when establishing the endpoints of a facility include urbanized area boundaries and major intersecting routes.

Applicability

The method can be used to evaluate highway performance during the design hour on multilane highway facilities with free-flow speeds between 45 and 70 mph.

Calculation Process

The process described below estimates average travel time and average speed by section and facility and vehicle-hours of delay for the facility as a whole. The PPEAG also describes how to calculate average density and level of service for multilane highways. The following steps are involved:



Step 1. Section the facility. Determine the start and end points of the facility, following the guidance above. Next, divide the facility into one or more basic sections. Basic section boundaries should be established at signalized intersections, major unsignalized intersections where highway demand changes significantly, lane adds or drops, locations where a change in the terrain classification occurs, and locations where the free-flow speed changes as a result of changes in the roadway geometry or posted speed limit. If interchanges are located along the highway, ramp or weaving sections (as appropriate) will also need to be established, following the guidance for freeways in Section 11.3.5.

Step 2. Gather input data. All of the data required for a screening analysis of a basic multilane highway sections are required, plus the length of each section (see Section 11.3.1 for details). In addition, if the facility ends at a signalized intersection, the average control delay for the highway mainline is a required input. Refer to Chapter 13 for procedures to calculate control delay at signalized intersections.

Step 3. Determine section v/c ratios. Determine the v/c ratio for each multilane highway section by following the screening methods described in Sections 11.3.1 through 11.3.3.

Step 4. Estimate average section speeds and travel times. The following equation, derived from PPEAG Equation 40, is used to estimate average speeds (not including any intersection delay) along a multilane highway section:

$$S_{u,i} = \frac{FFS_i}{1 + a \times (v_i/c_i)^b}$$

where

$S_{u,i}$ = uninterrupted-flow average travel speed for section *i* (mph),

FFS_i = free-flow speed of section *i* (mph),

a, b = parameters from Exhibit 11-15,

v_i = demand volume in section *i* (veh/h), and

c_i = capacity of section *i* (veh/h).

Exhibit 11-15 Parameters for Multilane Highway Speed Estimation

Free-Flow Speed (mph)	*a*	*b*
70	0.37	6.9
65	0.27	7.3
60	0.23	7.5
55	0.18	7.7
50	0.13	8.1
45	0.07	8.9

The average travel time for a multilane highway section, including control delay at a signalized intersection (if any) at the end of the section is then:

$$TT_\mathrm{i} = \frac{3{,}600 L_\mathrm{i}}{S_{u,i}} + d_i$$

where

TT_i = average travel time in section *i* (s),

L_i = length of section *i* (mi),

S_i = uninterrupted-flow average travel speed for section *i* (mph), and

d_i = average control delay at the downstream signal (s).

The average travel speed S_i for section *i*, including any signalized intersection delay, is then (derived from PPEAG Equation 42):

$$S_i = \frac{3{,}600 L_i}{TT_i}$$

where all variables are as defined previously.

Step 5. Estimate average facility travel time and speed. The average facility travel time TT_F (in seconds) is the sum of the individual section travel times, while the average facility speed S_F (in mph) is (PPEAG Equation 42):

$$S_F = \frac{3{,}600L}{TT_F}$$

where L is the facility length in miles.

Step 6. Estimate vehicle-hours of delay. The vehicle-hours of delay VHD_i for section i, based on a threshold speed S_{TH} (mph) when delay is considered to begin, is calculated as follows:

$$VHD_i = \max\left(0, \frac{v_i \times L_i}{S_i} - \frac{v_i \times L_i}{S_{TH}}\right)$$

where all variables are as defined previously. The total facility vehicle-hours of delay is then the sum of all section VHD values. The threshold speed is normally set as the posted speed for automobiles.

Detailed Analysis Method

The HCM does not provide a detailed method for evaluating multilane highway facility performance. Capacity problems usually arise at signalized intersections along the highway and not along basic segments. Other performance measures can be estimated using the screening method described above or by using alternative analysis tools.

11.4 Two-Lane Highways

This section is planned to be completely updated or possibly just replacing the Class III highway section with the methodology from the report Improved Analysis of Two-Lane Highway Capacity and Operational Performance (NCHRP Web-Only Document 255) which is contained in the HCM 7th Edition. This uses the same follower density performance measure as below but with a somewhat differing methodology.

No guidance is presented for the effects of CAVs on two-lane highway capacity because no research has been conducted yet for these roadway types.

Two-lane highway operations are characterized by passing maneuvers, formation of platoons within the traffic stream, and delay experienced by trailing vehicles while unable to pass lead vehicles. For increased passing demand, passing capacity decreases due to limited passing opportunities. Quality of service becomes unacceptable even for lower volume-to-capacity (v/c) ratios. Hence, use of volume-to-capacity ratio may not be a good performance measure for two-lane highway analysis. In addition, the v/c ratio calculation for two lane highways is very basic as it is just a flow rate divided by a fixed capacity value. This creates a misleading result as it does not reflect any of the driver behavior present (platooning, inability to maintain desired speed, etc.) on a two-lane highway.

The 6th Edition of the HCM uses Percent-Time Spent Following (PTSF), Average Travel Speed (ATS), and Percent Free-flow Speed (PFFS) as a measure to assess two-lane highways operations. In general, any segment that is two to three miles from the nearest signalized intersection on rural highways exhibits uninterrupted flow (HCM 6). Two-lane highways are classified into Class I, Class II and Class III highways based on wide range of functions. As per the HCM, arterials are considered to be Class I highways, and most collectors and local roads are considered to be Class II. Class III highways are a special case and may be any functional class. Definitions of the three classes are (HCM 6):

- **Class I two-lane highways** are highways where motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary connectors of major traffic generators, daily commuter routes, or major links in state or national highway networks are generally assigned to Class I. These facilities serve mostly long-distance trips or provide the connections between facilities that serve long-distance trips. Rural Principal Arterials (Functional Class 02 highways) mostly act as Class I highways. Coos Bay-Roseburg Highway-OR 42 (No. 35) is an example of a Class I highway.
- **Class II two-lane highways** are highways where motorists do not necessarily expect to travel at high speeds. Two-lane highways functioning as access routes to Class I facilities, serving as scenic or recreational routes (and not as primary arterials), or passing through rugged terrain (where high-speed operation would be impossible) are assigned to Class II. Class II facilities most often serve relatively short trips, the beginning or ending portions of longer trips, or trips for which sightseeing plays a significant role. Rural Minor Arterials (Functional Class 06 highways) and Rural Major Collectors (Functional Class 07) mostly act as Class II highways. For instance, West Diamond Lake Hwy- OR 230 (No. 233) that connects Crater Lake Hwy (OR 62) and Diamond Lake Hwy (OR 138) primarily serves recreational trips and passes through undeveloped, rugged terrain.
- **Class III two-lane highways** are special cases serving moderately developed areas. They may be portions of a Class I or Class II highway that pass through small towns, unincorporated communities, or developed recreational areas. On such segments, local traffic often mixes with through traffic, and the density of unsignalized roadside access points is noticeably higher than in a purely rural area. Class III highways may also be longer segments passing through more spread-out recreational areas, also with increased roadside densities. Such segments are often accompanied by reduced speed limits that reflect the higher activity level. Any signalized, all-way stop, or roundabout intersections in these

areas convert the section to an urban street and this method no longer applies. Some example sections:

- Gearhart to Warrenton section on Oregon Coast Hwy-US 101 (No. 9)
- Detroit city section on N Santiam Hwy-OR 22 (No. 162)
- Richland city section on Baker – Copperfield Highway-OR 86 (No. 12)

The rural US 101 section from Gearhart to Warrenton is a spread-out recreational area with substantial development along the highway. The Detroit and Richland sections of the highways pass through small towns having speed restrictions, significant road side developments and unsignalized access points.

ATS is a mobility indicator on two-lane highways. PTSF represents the freedom to maneuver and is defined as percent time spent following in platoon behind a slow moving vehicle while unable to pass. PFFS reflects the percent of travel at or near the posted speed limit. On Class I highways, both ATS and PTSF represents quality of service. While, PTSF defines LOS on Class II highways, PFFS is used to define LOS on Class III highways. LOS criteria for two-lane highways are summarized in Exhibit 11-1616.

Exhibit 11-16 LOS for Two-Lane Highways

LOS	Class I Highways		Class II Highways	Class III Highways
	ATS (mi/h)	PTSF (%)	PTSF (%)	PFFS (%)
A	>55	≤35	≤40	>91.7
B	>50–55	>35–50	>40–55	>83.3–91.7
C	>45–50	>50–65	>55–70	>75.0–83.3
D	>40–45	>65–80	>70–85	>66.7–75.0
E	≤40	>80	>85	≤66.7

Source: HCM 6th Edition, Exhibit 15-3

The HCM 6 manual presents only directional segment analysis and that is considered acceptable on ODOT two-lane highway facilities. The capacity of two-lane highways under based conditions is 1,700 passenger cars per hour (pc/h), with a limit of 3,200 pc/h for both directions. The limit is due to the interactions between directional flows; when a capacity of 1,700 pc/h is reached in one direction, the maximum opposing flow is limited to 1,500 pc/h. For a complete description of the methodology, refer to Chapter 15 of the HCM 6.

The PTSF performance measure used in the HCM 6 manual is difficult to measure in the field. The HCM also recommends use of a surrogate measure, percent followers, defined as the percentage of vehicles in the traffic stream with time headways smaller than 3.0 seconds. However, development of alternative performance measure for two-lane operations has attracted increasing interest. For instance, average travel speed, percent followers, and follower densities are key alternative measures tested for two-lane highway operations.

11.4.1 Follower Density Models for Class I and Class II Highways

The Oregon Department of Transportation (ODOT) has conducted studies to develop alternative LOS criteria for two-lane highway analysis[1]. The studies were based on the framework adopted for empirical investigation of two-lane rural highway performance indicators in Montana[2]. The study uses follower density as a performance measure to describe two-lane highways operations. Follower density is the number of followers in a directional traffic stream over a unit length of a highway. Followers are vehicles travelling with headway less than 3.0 seconds (*HCM 6*). The argument behind using this performance indicator is that a road with low average daily traffic (ADT) and high PTSF should have a lower LOS than the same road with a higher ADT and equal PTSF[3]. Unlike other performance measures, follower density takes into consideration the effect of the traffic level on highway performance[2]. Generally, density measures are difficult to directly measure in the field, but it can be estimated at point locations from volume and speed measurements from permanent or temporary traffic count detectors.

Similar to the HCM 6 methodology, the ODOT study developed LOS criteria for Class I and II two-lane highways. The study developed relationships between follower density (veh/mile/lane) and platooning variables for the best statistical significance. Exhibit 11-17 lists the follower density models.

The platooning variables included in the follower density models are:

- Traffic flow in the direction of travel (veh/h),
- Opposing traffic flow (veh/h),
- Percent heavy vehicles (%),
- Percent no-passing zones (%),
- Rolling Terrain[4] (1 = Rolling Terrain, 0 = Otherwise), and
- Mountainous Terrain[4] (1 = Mountainous Terrain, 0 = Otherwise).

[1] Modeling Follower Density on Two-Lane Rural Highways; and Modeling Performance Indicators on Two-Lane Rural Highways: The Oregon Experience

[2] Al-Kaisy, A., and Karjala, S. (2008). Indicators of Performance on Two-Lane Rural Highways: Empirical Investigation, Transportation Research Record, No. 2071, pp. 87–97.

[3] Van As, C. (2003).The Development of an Analysis Method for the Determination of Level of Service on Two-Lane Undivided Highways in South Africa. South African National Roads Agency, Pretoria.

[4] Terrain is: Level for grades less than 3% ; Rolling for grades between 3 to 6%; and Mountainous for grades greater than 6 %

Exhibit 11-17 Follower Density Models by Two-Lane Highway Class

Functional Class	Model Form	R^2
Class I Highways	Follower Density = -0.1917 + 0.005953 (Traffic Volume) + 0.0005167 (Opposing Volume) + 0.0006739 (% Heavy Vehicles) + 0.0002392 (% No Passing) + 0.05248 (Rolling Terrain)	0.81
Class II Highways	Follower Density = -0.1784 + 0.006189 (Traffic Volume) - 0.0001607 (Opposing Volume) + 0.0006163 (%Heavy Vehicles) + 0.0006055 (% No Passing) + 0.0168 (Rolling Terrain) + 0.03994 (Mountainous Terrain)	0.75

Follower density acts as a surrogate measure to assess operations of rural two-lane highways. Example 11-12 and Example 11-13 outline the application of these procedures. Follower density is most significantly affected by traffic volume and opposing volume. Percent heavy vehicles, percent no-passing zones and terrain type have a much lesser effect. The effect of these variables on Class II highways is somewhat greater than on Class I highways. The overall effect of these variables will not affect the Level of Service unless near a boundary condition. Percent heavy vehicles and terrain type are readily available. Percent no-passing zone data is typically collected from videologs which may be somewhat time consuming, and could be defaulted to a rough estimate such as 25%, 50%, 75% etc. With the help of follower density models and PTSF LOS boundaries, follower density thresholds are established at each LOS category as listed in Exhibit 11-18.

Exhibit 11-18 LOS Criteria by Two-Lane Highway Class

LOS	Class I Highways	Class II Highways
	Follower Density (veh/mile/lane)	Follower Density (veh/mile/lane)
A	<= 2	<= 2.5
B	> 2 - 3.5	> 2.5 - 4.0
C	> 3.5 - 6.0	> 4.0 - 6.5
D	> 6.0 - 9.0	> 6.5 - 10.0
E	> 9.0	> 10.0

The HCM 6 manual emphasizes estimation of capacity conditions, especially, for evacuation planning, special event planning, and evaluation of the downstream impacts of incident bottlenecks once cleared. However, use the capacity estimation for judging events, not for a volume-capacity calculation. For a complete description of the capacity estimation, refer to Chapter 15 of the HCM 6 Manual.

For the purposes of reporting, the volume-to-capacity measure should still be shown with the caveats noted in the introductory paragraph of this section for consistency with established mobility targets and design guidelines. This LOS-based measure should be used as a supplement, not as a replacement for v/c ratio. Both measures need to be reported but the follower-density based LOS measure is a better representation of highway performance.

This LOS-based methodology should be used for the analysis of rural state two-lane highways and can be used for county and other jurisdiction roadways. This methodology can be used on a planning analysis basis for corridor plans using AADT, K30 and D30 factors from ATR's to develop 30th highest hour volumes as well as information from databases (i.e. Highway Economic Reporting System (HERS)). Using directional and classification tube counts, analysis can be created for more detailed refinement/facility plans and projects.

The subject roadways need to be segmented by HCM roadway class, major intersections, passing/climbing lanes, and terrain type. Segments need to be at least two miles from any signalized intersection to avoid platooning effects. Class III segments need to be two lanes, so any two-way left turn lane segments are not included (need to use an urban street methodology for these).

Class I and II sections with resulting poor LOS may indicate that a slow-moving vehicle turnout, passing lane, climbing lane, or multilane section is needed. For passing and climbing lanes and the multilane sections follow the HCM normal procedures. Class III sections with resulting poor LOS may indicate that turn lanes or additional through lanes may be necessary.

Example 11-12 Class I Highway LOS

This example demonstrates the application of follower density based LOS criteria for Class I highway to find the expected LOS in each direction on the two-lane highway segment as described below:

Input Data

- Albany-Corvallis Highway (No. 31), MP 6.41
- Peak hour volume = 1833 veh/h (both directions; 2012 data)
- Directional split (during analysis period) = 63% EB and 37% WB
- PHF = 0.92
- 2 % trucks EB ; 2 % trucks WB
- 1-mile segment length
- 34% no-passing zones EB ; 50% no-passing zones WB

- Level terrain

LOS by Follower Density Model

This highway section is a rural principal arterial (Functional Class 02) which links two major cities (Albany and Corvallis) and is an important commuter corridor. Therefore, this segment is Class I as per HCM 6.

For a directional split of 63/37, analysis will be conducted for both 63 % direction of flow and 37 % direction of flow.

Follower density model:

Follower Density = -0.1917 + 0.005953 (Traffic Volume) + 0.0005167 (Opposing Volume) + 0.0006739 (% Heavy Vehicles) + 0.0002392 (% No Passing) + 0.05248 (Rolling Terrain)

LOS criteria:

LOS	**Class I Highways**
	Follower Density (veh/mile/lane)
A	<= 2.0
B	> 2.0 - 3.5
C	> 3.5 - 6.0
D	> 6.0 - 9.0
E	> 9.0

Analysis on 63 % direction of flow (EB)

Traffic flow rate = volume / PHF = (1833 x 0.63) / 0.92 = 1255 veh/hr

Opposing traffic flow rate = opposing volume / PHF = (1833 x 0.37) / 0.92 = 737 veh/hr

Percent Heavy Vehicles = 2 %

Percent No Passing zone = 34%

Rolling Terrain = 0 as terrain is considered “Level”

Follower Density = -0.1917 + 0.005953 (1255) + 0.0005167 (737) + 0.0006739 (2) + 0.0002392 (34) + 0.05248 (0)

= 7.3 veh/mile/lane

LOS is D for the analysis direction.

Analysis on 37 % direction of flow (WB)

Traffic flow rate = volume / PHF = (1833 x 0.37) / 0.92 = 737 veh/hr

Opposing traffic flow rate = opposing volume / PHF = (1833 x 0.63) / 0.92 = 1255 veh/hr

Percent Heavy Vehicles = 2 %

Percent No Passing zone = 50 %

Rolling Terrain = 0 as terrain is considered “Level”

Follower Density = -0.1917 + 0.005953 (737) + 0.0005167 (1255) + 0.0006739 (2) + 0.0002392 (50) + 0.05248 (0)

= 4.2 veh/mile/lane

LOS is C for the analysis direction.

Example 11-13 Class II Highway LOS

This example demonstrates the application of follower density based LOS criteria for a Class II highway to find the expected LOS in each direction on the two-lane highway segment.

Input Data

- West Diamond Lake Hwy (No. 233) at MP 5.86
- Peak hour volume = 109 veh/h (total in both directions; 2013 data)
- Directional split (during analysis period) = 69 % EB and 31 % WB
- PHF = 0.74
- 26 % trucks EB ; 27 % trucks WB
- 1-mile segment length
- 45% no-passing zones EB ;5% no-passing zones WB
- Rolling terrain

<u>LOS by Follower Density Model</u>

This highway section is a rural minor arterial which serves primarily scenic and recreational destinations (i.e. Crater Lake National Park), passes through rugged terrain, and high travel speeds are not expected in all places. This highway best fits into the HCM Class II designation.

For a directional split of 69/31, analysis will be conducted for both 69 % direction of flow and 31 % direction of flow.

Follower density model:

Follower Density = -0.1784 + 0.006189 (Traffic Volume) - 0.0001607 (Opposing Volume) + 0.0006163 (%Heavy Vehicles) + 0.0006055 (% No Passing) + 0.0168 (Rolling Terrain) + 0.03994 (Mountainous Terrain)

LOS criteria:

LOS	Class II Highways Follower Density (veh/mile/lane)
A	<= 2.5
B	> 2.5 - 4.0
C	> 4.0 - 6.5
D	> 6.5 - 10.0
E	> 10

Analysis on 69 % direction of flow (EB)

Traffic flow rate = volume / PHF = (109 x 0.69) / 0.74 = 102 veh/hr

Opposing traffic flow rate = opposing volume / PHF = (109 x 0.31) / 0.74 = 46 veh/hr

Percent Heavy Vehicles = 26 %

Percent No Passing zone = 45%

Rolling Terrain = 1 as terrain is considered “Rolling”

Mountainous Terrain = 0 as terrain is considered “Rolling” type

Follower Density = -0.1784 + 0.006189 (102) - 0.0001607 (46) + 0.0006163 (26)
+ 0.0006055 (45) + 0.0168 (1) + 0.03994 (0)

= 0.51 veh/mile/lane

LOS is A for the analysis direction.

Analysis on 31 % direction of flow (WB)

Traffic flow rate = volume / PHF = (109 x 0.31) / 0.74 = 46 veh/hr

Opposing traffic flow rate = opposing volume / PHF = (109 x 0.69) / 0.74 = 102 veh/hr

Percent Heavy Vehicles = 27 %

Percent No Passing zone = 5%

Rolling Terrain = 1 as terrain is considered “Rolling”

Mountainous Terrain = 0 as terrain is considered “Rolling”

Follower Density = -0.1784 + 0.006189 (46) - 0.0001607 (102) + 0.0006163 (27)
+ 0.0006055 (5) + 0.0168 (1) + 0.03994 (0)

= 0.13 veh/mile/lane

LOS is A for the analysis direction.

11.4.2 Class III Highways Methodology

Preliminary models developed for Class III highways were limited because of limited sample size. However, the percent free flow speed (PFFS) LOS measure suggested in the HCM can easily be obtained from field data. Users are advised to use the HCM 6 methodology and LOS criteria for Class III highways. Motorists are expected to travel at or near the posted speed limit on these facilities. Neither higher speeds nor concerns about passing restrictions are expected. Instead, the ability to travel near the free flow speed (measured by PFFS) is a LOS measure. The PFFS is the ratio of average travel speed to free-flow speed. The LOS criteria for two-lane Class III highways are shown in Exhibit 11-1616. For a complete description of the methodology, refer to Chapter 15 of the HCM 6 Manual. However, the following steps provide a brief summary of Class III highways methodology.

Step 1: Estimation of FFS

After gathering input data, the first step in the analysis is to find the free flow speed (FFS). The HCM 6 manual suggests three methodologies to estimated FFS.

- **Direct Field Measurement:** Mean speed of 100 random vehicle speeds at low traffic conditions (i.e., two-way flow rate is less than or equal to 200 veh/h) for each analysis direction.

Field Measurements at Higher Flow Rates: If the observed total flow rate exceeds 200 veh/h, find the mean speed of a random sample of 100 vehicle speeds in each analysis direction. The measured mean speed is then adjusted as (Equation 15-1, HCM 6):

$$FFS = S_{FM} + 0.00776\left(\frac{v}{f_{HV,ATS}}\right)$$

$$f_{HV,ATS}$$

Where

FFS = free-flow speed (mi/h),

S_{FM} = mean speed of sample (v > 200 veh/h) (mi/h),

v = total demand flow rate (both directions), during period of speed measurements (veh/h),

$f_{HV,ATS}$ = heavy vehicle adjustment factor for ATS, from Equation 15-4 or Equation 15-5.

- **Estimating FFS:** If the field data is not available, FFS can be estimated as (Equation 15-2, HCM 6):

$$FFS = BFFS - f_{LS} - f_A$$

Where

FFS = free-flow speed (mi/h),

$BFFS$ = base free-flow speed (mi/h),

f_{LS} = adjustment for lane and shoulder width (mi/h) (Exhibit 15-7), and

f_A = adjustment for access-point density (mi/h) (Exhibit 15-8).

The BFFS is the speed that would be expected on the basis of the facility's horizontal and vertical alignment, if standard lane and shoulder widths were present and there were no roadside access points. A rough estimate of BFFS might be taken as the posted speed limit plus 10 mi/h (HCM 6).

Step 2: Demand Adjustment for ATS

Demand volumes in both directions (analysis direction and opposing direction) are converted to flow rates under equivalent base conditions as (Equation 15-3, HCM 6):

$$v_{i,ATS} = \frac{V_i}{PHF \times f_{g,ATS} \times f_{HV,ATS}}$$

where

$v_{i,ATS}$	=	demand flow rate *i* for ATS estimation (pc/h);
i	=	"d" (analysis direction) or "o" (opposing direction);
V_i	=	demand volume for direction *i* (veh/h);
$f_{g,ATS}$	=	grade adjustment factor, from Exhibit 15-9 or Exhibit 15-10 in HCM 6
$f_{HV,ATS}$	=	heavy vehicle adjustment factor, from Equation 15-4 or Equation 15-5

Step 3: Estimate the ATS

Average Travel Speed (ATS) is estimated from the FFS, the demand flow rate, the opposing flow rate, and the percentage of no-passing zones in the analysis direction as (Equation 15-6, HCM 6):

$$ATS_d = FFS - 0.00776\left(v_{d,ATS} + v_{o,ATS}\right) - f_{np,ATS}$$

where

ATS_d	=	average travel speed in the analysis direction (mi/h);
FFS	=	free-flow speed (mi/h);
$v_{d,ATS}$	=	demand flow rate for ATS determination in the analysis direction (pc/h);
$v_{o,ATS}$	=	demand flow rate for ATS determination in the opposing direction (pc/h); and
$f_{np,ATS}$	=	adjustment factor for ATS determination for the percentage of no-passing zones in the analysis direction, from Exhibit 15-15 in the HCM 6th Edition manual.

Step 4: Estimate the Percent Free-Flow Speed (PFFS)

PFFS is the ratio of Average Travel Speed (ATS) in the analysis direction and Free Flow Speed (FFS), Equation 15-11, HCM 6.

$$PFFS = \frac{ATS_d}{FFS}$$

Step 5: Capacity Estimation

Under base conditions, capacity of two-lane highways in one direction is 1,700 pc/h. However, capacity is limited to 3,200 pc/h for both directions, because of interaction between directional flows. It is important to note that two-lane highways quality of service deteriorates even at low volume-to-capacity ratios (HCM 6). For Class III highways, only the ATS-based capacity is computed (Equation 15-12, HCM 6):

$$c_{dATS} = 1{,}700 f_{g,ATS} f_{HV,ATS}$$

where

c_{dATS} = capacity in the analysis direction under prevailing conditions based on ATS (pc/h),

$f_{g,ATS}$ = grade adjustment factor, and

$f_{HV,ATS}$ = heavy vehicle adjustment factor.

The adjustment factors in the capacity estimation are based on a flow rate greater than 900 veh/h to avoid an iterative solution. Flow rates of less than 900 veh/h will require iteration. If the directional distribution is other than 50/50 (in level and rolling terrain), the two-way capacity may be more than the 3,200 pc/h limit. If the limit is exceeded, then the base capacity is restricted to 1,700 pc/h in the heaviest demand direction. Capacity in the opposing direction is found by using the directional distribution of opposing flow, with an upper limit of 1,500 pc/h. The capacity estimation is for judging potential bottlenecks caused by high travel periods or special events, not for a volume-to-capacity ratio calculation. Example 11-14 provides an example for assessing LOS on Class III Highways.

Example 11-14 Class III Highway LOS

This example demonstrates the application of follower density-based LOS criteria for a Class III highway to find the expected LOS in each direction on the two-lane highway segment as described below:

Input Data

- Baker – Copperfield Highway (No. 12), MP 42.27
- Peak hour volume = 104 veh/h (total in both directions; 2010 data)
- Directional split (during analysis period) = 53% EB and 47% EB
- PHF = 0.83
- 24 % trucks EB; 24 % trucks WB
- 1-mile segment length
- 100% no-passing zones EB; 100% no-passing zones WB
- Level terrain
- Field measured FFS = 35 mph

LOS by HCM Methodology

This highway segment is a rural major collector (Functional Class 07) and travels through the town of Richland. This will have local traffic mixing with the through traffic, have a higher amount of access points, no signalized intersections, and has reduced speed limits. Since there are no signalized intersections, the two-lane methodology still applies and would be a HCM Class III section.

Step 1: Estimation of FFS

Field measured FFS = 35 mph

Step 2: Demand Adjustment for ATS

Separate analysis is done for both directions. Demand volume is converted to flow rate under equivalent base conditions using (Equation 15-3 in HCM 6):

$$v_{i,ATS} = \frac{V_i}{PHF \times f_{g,ATS} \times f_{HV,ATS}}$$

Total demand volume in both directions is:

$$V_{\text{EB}} = (104 \times 0.53) = 55 \text{ veh/h}$$

$$V_{\text{WB}} = (104 \times 0.47) = 49 \text{ veh/h}$$

Demand flow rate for both directions is:

$$v_{\text{EB}} = (104 \times 0.53)/0.83 = 67 \text{ veh/h}$$

$$v_{\text{WB}} = (104 \times 0.47)/0.83 = 59 \text{ veh/h}$$

Value of $f_{g,ATS}$, and E_T for both directions (see Exhibit 15-10 and Exhibit 15-11 HCM 6):

Value	EB	WB
$f_{g,ATS}$	1.00	1.00
E_T	1.9	1.9

Then, $f_{HV\,ATS}$ is calculated using (Equation 15-4 in HCM 6)

$$f_{HV,ATS} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV,ATS(EB)} = \frac{1}{1 + 0.24(1.9 - 1)} = 0.82$$

$$f_{HV,ATS(WB)} = \frac{1}{1+0.24(1.9-1)} = 0.82$$

Note that the recreational vehicle term P_R is not used since RVs are included in the truck percentage.

Demand adjusted flow rates are:

$$v_{EB,ATS} = \frac{(104 \times 0.53)}{0.83 \times 1.00 \times 0.82} = 81 \text{ pc/h}$$

$$v_{WB,ATS} = \frac{(104 \times 0.47)}{0.83 \times 1.00 \times 0.82} = 72 \text{ pc/h}$$

Step 3: Estimate the ATS

The ATS is estimated using (Equation 15-6 in HCM 6)

$$ATS_d = FFS - 0.00776(v_{d,ATS} + v_{o,ATS}) - f_{np,ATS}$$

The $f_{np,ATS}$ adjustment factor for no-passing zones is taken from Exhibit 15-15 (HCM 6). The adjustment factor is based on a 35-mph FFS, opposing demand flow rate of 81 pc/h EB and 72 pc/h WB, and 100% no-passing zones.

$$f_{np,ATS(\mathrm{EB})} = 2.4 \text{ mi/h}$$

$$f_{np,ATS(\mathrm{WB})} = 2.4 \text{ mi/h}$$

The ATS in each direction of analysis is:

$$ATS_{\mathrm{EB}} = 35.0 - 0.00776(81 + 72) - 2.4 = 31.4 \text{ mi/h}$$

$$ATS_{\mathrm{WB}} = 35.0 - 0.00776\,(72 + 81) - 2.4 = 31.4 \text{ mi/h}$$

Step 4: Estimate the Percent Free-Flow Speed (PFFS)

The LOS for Class III facilities is based on PFFS achieved, or ATS/FFS. For this segment PFFS is as follows (Equation 15-11 HCM 6):

$$PFFS_{\mathrm{EB}} = 31.4/35.0 = 89.7\%$$

$$PFFS_{\mathrm{WB}} = 31.4/35.0 = 89.7\%$$

From Exhibit 11-16, the LOS for EB direction is B, while the LOS for WB direction is also B.

Step 5: Capacity Estimation

Capacity in the analysis direction under prevailing conditions is given by Equation 15-12 HCM 6:

$$c_{dATS} = 1{,}700 f_{g,ATS} f_{HV,ATS}$$

The adjustment factors in the capacity estimation ($f_{g,ATS}$, $f_{HV\,ATS}$) are based on a flow rate greater than 900 veh/h. Capacity in either direction is as follows:

$$c_{\mathrm{EB},ATS} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

$$c_{\mathrm{WB},ATS} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

The implied values of capacity are

1,700/0.53 = 3,208 veh/h (EB) and

1,700/0.47 = 3,617 veh/h (WB).

As the capacity is limited to 3,200 pc/h, the prevailing capacity would be 3,200 × 1.00 × 1.00 = 3,200 veh/h.

With a 53/47 directional split,

EB capacity would be 3,200 × 0.53 = 1,696 veh/h and,

WB capacity would be 3,200 × 0.47 = 1,504 veh/h.

11.4.3 Passing and Climbing Lanes

Both passing and climbing lanes are low-cost improvements that can be very effective in improving the operation of two-lane highways and can reduce the need to widen highways to four lanes. The *HCM* includes methodologies for analyzing these types of facilities in Chapter 20. Appendix 10A includes criteria for helping determining if these sections operate as auxiliary lanes (no increase in systemic capacity) or as multilane highways.

When analyzing either passing or climbing lanes it must be determined whether a no-passing restriction will be placed on opposing traffic in the area of the added lane. If passing by opposing traffic will not be allowed, the operations of opposing traffic must be reanalyzed to include this restriction.

While the methodologies described below can be used to evaluate the operations of passing and climbing lanes, the appropriate locations and lengths to use for design should be determined through the use of ODOT's HDM.

Passing Lanes

Passing lanes are typically used where there may be inadequate passing opportunities, either because of sight distance limitations or as traffic volumes approach capacity. By providing a safe place to pass, passing lanes tend to reduce unsafe passing maneuvers. In addition to improving operations in the segment containing the passing lane, operations of the highway downstream of the passing lane may also be improved for up to several miles before queues begin to reform. Exhibit 20-23 in the *HCM* shows the general relationship between the directional flow rate and the length of the downstream roadway affected. The *HCM* methodology is applicable to directional segments of two-lane highways that include the entire passing lane, and should also include the full effective downstream length (Exhibit 20-23), if possible.

A critical part of passing lane analysis using the *HCM* methodology includes dividing the analysis segment into four regions.

- Upstream of the passing lane.
- The passing lane, including tapers.
- Downstream of the passing lane, but within its effective length.
- Downstream of the passing lane, but beyond its effective length.

When using the Highway Capacity Software (HCS) to perform calculations, only the total segment length, length upstream of the passing lane and length of the passing lane are needed for input. The program will automatically calculate the other lengths based on these lengths and the directional flow rate. As with the Two-Lane Highway analysis, a volume to capacity ratio for a directional segment must be obtained by dividing the passenger car equivalent peak 15-minute flow rate by the appropriate capacity. For a complete description of the remaining analysis assumptions and methodology, see Chapter 20 in the *HCM*.

The analysis methodology in the *HCM* for passing lanes is intended to be applied to highways on level or rolling terrain only. Added lanes on mountainous terrain or on specific grades should be analyzed as climbing lanes.

Climbing Lanes

Climbing lanes are similar to passing lanes, but are generally used where grades cause unreasonable reductions in operating speeds of some vehicles. An unreasonable reduction in operating speeds is typically considered to occur where speed differentials of more than 10 mph are created. These lanes increase the local capacity of a two-lane highway section (but not the whole highway) by providing a specific lane for slower vehicles to travel in while climbing an extended grade. This enables faster vehicles to pass these slower vehicles safely without having to leave the main travel lane. While climbing lanes are typically thought of as being associated with upgrades, they can also be applied to downgrades where heavy vehicles must drive in a low gear to avoid speeding out of control.

When analyzing the downgrade direction, passenger car equivalents for trucks operating at crawl speeds are available in Exhibit 20-18 of the *HCM*. For all other heavy vehicles, the passenger car equivalents in the *HCM* for level terrain should be used (Exhibit 20-9).

11.5 Travel Time Reliability

As described in APM Section 9.3, travel time reliability (or simply, *reliability*) considers (1) the range of potential travel times roadway users may experience, (2) the consistency of travel times, and (3) the ability of a roadway to provide a desired travel time. APM Section 9.3.5 described resources available for evaluating and reporting reliability under existing conditions. This section presents methods for forecasting future travel time reliability.

11.5.1 Overview of Travel Time Reliability Methods

APM Section 9.3.6 introduced the primary types of methods for forecasting reliability:

- Planning-level methods based on the SHRP 2 C11 equations for estimating common reliability performance measures;
- Detailed macroscopic methods, which develop a travel time distribution from the results of hundreds of scenarios modeling different demand levels, weather conditions, incidents, work zones, and special events; and
- Microscopic simulation.

This subsection summarizes the methods presented in Section 11.5 and describes potential applications for them.

Planning-Level Method

PPEAG-Based Screening Method

The screening method described in Section 11.5.3 also uses the SHRP 2 C11 equations to estimate common reliability performance measures but uses PPEAG-based methods to develop the v/c ratios and average speeds required as inputs to the equations. The method is well-suited to preliminary evaluations of the reliability of individual facilities, as well as for testing the reliability effect of projects that change roadway demand or capacity. It can be applied to any roadway type. Because screening methods are not sensitive to the effects of most roadway operations strategies, nor are they sensitive to weather conditions or facility-specific crash or incident rates, the detailed macroscopic methods described below should be used for detailed evaluations and alternatives analysis.

Detailed Macroscopic Methods

HCM Reliability Methods

HCM methods also define hundreds of scenarios and combine the results into a travel time distribution. The HCM methods offer more flexibility for defining a reliability reporting period and for testing roadway operations strategies, and they produce travel time estimates based on state-of-the-art HCM 7th Edition methods. The HCM method requires considerable work to gather and enter data about demand volumes and roadway characteristics (if not already performed as part of a traditional roadway operations analysis), but has a reasonable process for defining scenarios and generating a travel time distribution. HCM methods can only be used at present to evaluate the reliability of freeways and urban streets.

Simulation

As discussed in Section 9.3.6, although the FHWA has sponsored case studies demonstrating the potential use of microsimulation to forecast reliability, it is not practical to do so at present because of the time required to develop, code, run, and analyze the many different scenarios required to generate an accurate travel time distribution. Although it is relatively easy to batch-run changes in demand through a simulation model, other factors that influence reliability, such as weather, incidents, work zones, and roadway operations strategies require coding and running individual models. The FHWA case studies involved 8 or 9 common scenarios, whereas the HCM recommends a minimum of 200 scenarios for generating a travel time distribution and generating reliability performance measures. In particular, it is the effect of relatively rare scenarios that has the greatest influence on the overall reliability results. Consequently, simulation is not recommended at present as a tool for forecasting reliability.

Reliability Method Comparison

Exhibit 11-19 summarizes the analysis capabilities, data needs, and potential applications for the primary methods available for forecasting reliability.

Exhibit 11-19 Reliability Forecasting Method Comparison

	Screening	HCM	Simulation
Analysis Needs and Data Availability			
Preferred roadway types	Any	Freeways, urban streets	Any
Reliability reporting period	Weekday any hours, 1 year	Any	Weekday peak period, 1 year
Reliability performance measures	Common	Any	Difficult
Maximum study area size	Facility	Facility	Facility
Input data needs	Low	High	Very High
Defaults available for inputs	No	Yes	No
Applications			
Regional Transportation Plan	◐		
Transportation System Plan	◐		
Corridor Plan	◐	●	
Refinement Plan	◐	●	●
Project Development	◐	●	●

Note: ○ = problem identification, ◐ = evaluation, ● = detailed evaluation.

11.5.2 Data Needs and Sources

Exhibit 11-20 summarizes the input data requirements for the primary methods available for forecasting reliability. In some cases, the methods calculate or obtain certain values directly without requiring the analyst to provide them. See Section 11.3 for suggested data sources for roadway section data, Appendix 11C for default values for reliability-related inputs, and Appendix 11F for guidance on developing local values for reliability inputs.

Exhibit 11-20 Input Data Required for Reliability Forecasting Methods

Input Data	Screening	HCM (Freeway)	Simulation
Roadway Section Data			
Free-flow speed	●	●	●
Posted speed	●	●	●
Volume-to-capacity ratio	●	*	
Number of lanes	●	●	●
Average speed	●	*	*
Section length	○	●	●
Detailed demand data		●	●
Detailed roadway geometry		●	●
Simulation parameters			●
Reliability Factor Data			
Demand levels	X	○	●
Incident types & durations	X	○	●
Severe weather types & durations	X	○	X
Adjustments for special events	X	○	●

Notes: ● = required input, ○ = optional input (can be defaulted), * = calculated value, X = reliability factor not directly considered by method, empty = not required.

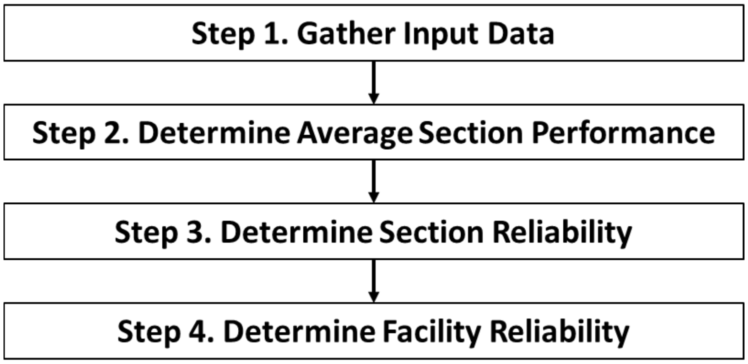
11.5.3 Screening Analysis Method

Introduction

The screening method can be applied to any roadway type. It is based on the “data poor” reliability forecasting equations developed by SHRP 2 Project C11, along with Oregon-specific modeling.[5] Input values used by these equations are developed using PPEAG methods, which gives the screening method a greater sensitivity to local roadway conditions than does the basic SHRP 2 C11 method. The screening method first forecasts the reliability of individual roadway sections. The section results are then aggregated into a forecast of the reliability of a longer roadway facility.

[5] Cambridge Systematics, Inc. Oregon SHRP2 C11 Reliability Analysis Implementation Plan, Task 2, Final Technical Memorandum #1. Medford, MA, Oct. 2018.

Calculation Process
The following steps are involved in estimating travel time reliability:



The method begins by determining average roadway performance on a section-by-section basis, following the screening methods described previously in APM Sections 11.3.1 through 11.3.4. Once each section's volume-to-capacity ratio and average peak-hour speed are determined, the method can then estimate any of the following performance measures for the section:

- Average (mean) travel time index (TTI_m, TTI_{Pm})
- 50th percentile TTI (TTI_{50}, TTI_{P50})
- 80th percentile TTI (TTI_{80}, TTI_{P80})
- 95th percentile TTI (TTI_{95}, TTI_{P95})
- Percent trips occurring at less than 45 mph
- Percent trips occurring at less than 30 mph
- Person hours of delay based on the posted speed

Step 1: Gather Input Data. The input data requirements are the same for performing a facility analysis. For freeway facilities, the following data are required for each section (items in italics can be defaulted):

- Section type (basic, merge–diverge, weaving)
- Hourly demand
- *Peak hour factor*
- *Percent heavy vehicles*
- Number of lanes
- *Free-flow speed*
- Posted speed
- Terrain class (level, rolling, mountainous)
- Section length

Step 2: Determine Average Section Performance. In this step, two key performance measures are determined for each section: volume-to-capacity ratio and average travel time without incidents. For freeway sections, the screening methods described in APM Sections 11.3.1 through 11.3.3 for basic freeway sections, merge–diverge segments, and weaving sections, respectively, are used to determine volume-to-capacity ratio. For other roadway types, use the corresponding PPEAG method to determine the volume-to-capacity ratio.

For freeway facilities, average travel time without incidents is estimated using a process similar to Step 7 of the screening method for freeway facilities (APM Section 11.3.4). However, because the reliability performance measure equations work with hourly volumes instead of 15-minute volumes, the Step 7 process is simplified to work with hourly volumes as follows:

$$T_{FFS,i} = \frac{3{,}600 L_i}{FFS_i}$$

$$T_i = T_{FFS,i} + L_i\left(\Delta_{RU_i} + \Delta_{RO\,i}\right)$$

with

$$\Delta_{RU_{i,t}} = \begin{cases} 0 & X_i < E \\ A(X_i)^3 + B\left(X_{i,t}\right)^2 + C(X_i) + D & E \le X_i \end{cases}$$

$$X_i = \min(1, d_i/c_i)$$

$$\Delta_{RO\,i} = \frac{900}{2L_i}(\max[1, d_i/c_i] - 1)$$

where

$T_{FFS,i}$ = travel time to traverse section *i* at the free-flow speed (s),

L_i = length of section *i* (mi);

FFS_i = free-flow speed of section *i* (mph),

T_i = average travel time to traverse section *i* without incidents (s),

Δ_{RU_i} = undersaturated delay rate for section *i* (s/mi),

Δ_{RU_i} = oversaturated delay rate for section *i* (s/mi),

A, B, C, D, E = parameters from Exhibit 11-12 (see Section 11.3.4),

d_i = demand flow rate in section *i* (veh/h) = hourly demand divided by the peak hour factor, and

c_i = capacity of section *i* (veh/h).

For all other roadway types, use the corresponding PPEAG method to determine the average travel time without incidents.

Step 3: Determine Section Reliability. This step begins by estimating each roadway's mean travel time index as a function of the free-flow speed, the recurring delay rate, and the incident delay rate, as follows:

$$TTI_m = 1 + FFS \times (RDR + IDR)$$

$$RDR = \frac{1}{S} - \frac{1}{FFS}$$

$$IDR = [0.020 - (N - 2) \times 0.003] \times X^{12} \qquad X \leq 1.00$$

where

TTI_m = mean travel time index (unitless)

FFS = free-flow speed (mph),

RDR = recurring delay rate (h/mi),

IDR = incident delay rate (h/mi),

S = average peak-hour speed (mph),

N = number of lanes (N = 2, 3, or 4; if there are more than 4 lanes, set N to 4), and

X = volume-to-capacity ratio (if $X > 1$, set X to 1).

Note that the estimate of incident delay rate used by the screening method is based on national averages and not individual roadway characteristics. If there is interest in evaluating the effect of safety countermeasures on roadway reliability, the detailed method should be used instead.

Once TTI_m is determined, additional reliability performance measures can be calculated from it. The choice of equations depends on the type of roadway being studied, and more performance measures can be estimated for some roadway types than for others. Four roadway categories are defined:

- Urban freeways and multilane highways,
- Rural freeways and multilane highways,
- Rural two-lane highways, and
- Urban arterials.

"Urban" and "rural" are determined using the FHWA definitions for federal-aid highways.

Urban freeways and multilane highways:

$$TTI_{50} = \frac{0.8701 \times 80.9980 + 14.0785 \times TTI_m{}^{2.2141}}{80.9980 + TTI_m{}^{2.2141}}$$

$$TTI_{80} = \frac{14.8892 \times TTI_m{}^{1.6443}}{5.0817^{1.6443} + TTI_m{}^{1.6433}}$$

$$TTI_{95} = 16.7754 \times \exp\left(-\frac{2.8221}{TTI_m}\right)$$

$$CONG_{30} = -9.1128 + 140.3250 \times \ln(TTI_m) \quad \text{if } TTI_m \geq 1.07; \text{0 otherwise}$$

$$CONG_{45} = -3.0184 + 205.3288 \times \ln(TTI_m) \quad \text{if } TTI_m \geq 1.02; \text{0 otherwise}$$

Rural freeways and multilane highways:

$$TTI_{50} = 1 + 0.5383 \times \ln(TTI_m)$$

$$TTI_{80} = 0.2834 \times e^{(1.2631 \times TTI_m)}$$

$$TTI_{95} = \frac{(0.9941 \times 21.0911 + 2.2971 \times TTI_m{}^{17.5709}}{(21.0911 + TTI_m{}^{17.5709})} \quad \text{with a minimum value of 1}$$

Rural two-lane highways:

$$TTI_{50} = 0.6836 \times \exp(0.3996 \times TTI_m)$$

$$TTI_{80} = \frac{30.0787}{1 + 75.7094 \times \exp(-0.9778 \times TTI_m)}$$

$$TTI_{95} = \frac{0.3691 \times 8.3171 \times 6.0980 \times TTI_m{}^{4.2633}}{8.3171 + TTI_m{}^{4.2633}}$$

Urban arterials:

$$TTI_{50} = \frac{0.5580 + 0.2236 \times TTI_m}{1 - 0.2618 \times TTI_m + 0.0307 \times TTI_m{}^2}$$

$$TTI_{80} = 0.5161 \times (TTI_m + 0.5105)^{1.6694}$$

$$TTI_{95} = \frac{9.1585}{1 + (\frac{TTI_m}{2.1327})^{-2.8021}}$$

$$CONG_{20} = 7424.8705 \times \exp\left(\frac{-9.4124}{TTI_m}\right) \quad \text{for } TTI_m \geq 1.06; \text{0 otherwise}$$

where

TTI_{50} = 50th-percentile travel time index (unitless),

TTI_m = mean travel time index (unitless),

TTI_{80} = 80th-percentile travel time index (unitless),

TTI_{95} = 95th-percentile travel time index (unitless),

$CONG_{40}$ = duration of congestion, 40-mph threshold (minutes),

$CONG_{30}$ = duration of congestion, 30-mph threshold (minutes), and

$CONG_{20}$ = duration of congestion, 20-mph threshold (minutes).

ODOT uses a policy TTI based on the posted speed rather than the free-flow speed. The mean TTI is converted into a mean policy TTI as follows:

$$TTI_{Pm} = TTI_m \times \frac{PSL}{FFS} \qquad \text{with a minimum value of } 1.00$$

where

TTI_{Pm} = mean policy travel time index (unitless),

PSL = posted speed limit (mph), and

all other variables are as previously defined. Similarly, TTI_{P50}, TTI_{P80}, and TTI_{P95} are found by multiplying TTI_{50}, TTI_{80}, and TTI_{95}, respectively, by the ratio of the posted speed to the free-flow speed.

Person hours of delay can also be determined for each roadway section as described in APM Section 9.3. The analyst first calculates the time to travel through the section at the posted speed. The portion of the estimated travel time that exceeds the travel time at the posted speed is then counted as delay. The number of persons experiencing the delay is then determined from vehicle volumes by type (e.g., passenger car, bus, truck) and an assumed occupancy for each vehicle type.

Step 4: Determine Facility Reliability. In this step, the mean travel time including incidents determined in Step 2 for each section is summed for the entire facility. Next, the mean TTI is determined for the facility and used to determine other reliability measures of interest for the facility, such as the 95th-percentile TTI for the facility. Finally, all TTI values are converted into the policy TTI values for facility.

The mean travel time to traverse the facility (including incidents) is determined as follows:

$$T_f = \sum_{i=1}^{n} T_{FFS,i} \times TTI_i$$

where

T_f = mean travel time to traverse the facility (s),

$T_{FFS,i}$ = time to traverse section i at the free-flow speed (s),

TTI_i = mean travel time index for section i (unitless), and

n = number of sections in the facility.

The mean facility travel time index $TTI_{m,f}$ is then:

$$TTI_{m,f} = \frac{T_f}{\sum_{i=1}^{n} T_{FFS,i}}$$

where all other variables have been defined previously.

The value of $TTI_{m,f}$ is then substituted for TTI_m in the equations in Step 3 to estimate other reliability performance measures of interest for the facility. Finally, all TTI results are multiplied by the ratio of the posted speed to the free-flow speed to develop the equivalent policy TTIs.

Example 11-15 Freeway Reliability Analysis (Screening Method)

The study facility is I-5 southbound through the Eugene urban area. The existing travel time reliability of I-5 is being assessed to provide a baseline condition for a transportation system plan that will eventually also include future-conditions assessments.

Step 1. Gather input data. The facility boundaries are set at the points where the automobile speed limit drops from 65 mph to 60 mph at the north end of the urban area, and where the speed limit changes back to 65 mph at the south end. On- and off-ramps and lane drops are located at the milepoints shown in the diagram below; none of the ramps are metered.

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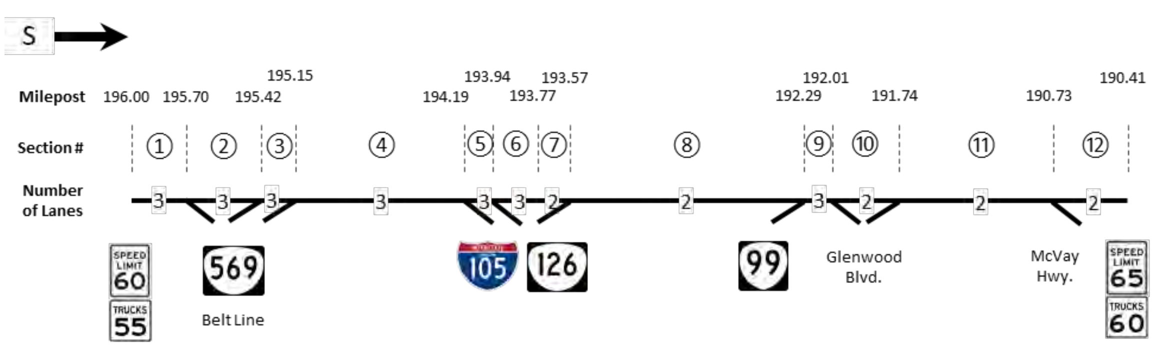


The freeway is divided into sections, with section boundaries located at ramp junctions. In addition, a section boundary is established at the point where the number of basic freeway lanes drops from 3 to 2. The section between the on-ramp from Highway 99 and the off-ramp to Glenwood Blvd. has an auxiliary lane and is therefore assigned as a weaving section. All other sections starting with an on-ramp are assigned as merge–diverge sections. All remaining sections are assigned as basic sections.

ODOT's *Traffic Volume Tables* are used to find directional AADTs for the I-5 mainline and ramp AADTs in the weaving section. ODOT's TransGIS tool (https://gis.odot.state.or.us/transgis/) is used to obtain K-factors, heavy vehicle percentages, and number of lanes in each section. The length of each section is determined from the milepoints of the section boundaries. Appendix 11C is used to obtain the following default values:

- Peak hour factor: 0.94 (urban freeway)
- Terrain: level (Figure 3)
- Ramp-to-ramp AADT in the weaving section: 285 veh = (2,470 / 33,210) × 3,830

Figure 1 in Appendix 11B is used to obtain a capacity adjustment factor for driver population CAF_{pop} for the facility. That table gives a value of 0.968 (mostly familiar drivers) for I-5 in the Willamette Valley. There are no CAVs; therefore, the default capacity adjustment factor for CAVs CAF_{CAV} of 1.00 will be used.

The method provided in Appendix 11A for differential auto and truck speed limits is used to estimate the free-flow speed. For example, in section 1, the auto speed limit is 60 mph, the truck speed limit is 55 mph, and the heavy vehicle percentage is 24.4%. In the absence of a measured value, the auto free-flow speed is estimated as the auto speed limit plus 5 mph, or 65 mph. The truck free-flow speed is then estimated as the auto free-flow speed (65 mph) minus the difference in the auto and truck speed limits (5 mph), or 60 mph. Finally, a weighted average free-flow speed is calculated as follows:

$$FFS = (1 - P_T)FFS_{auto} + (P_T)FFS_{truck}$$
$$FFS = (1 - 0.244)65 + (0.244)60 = 63.8 \text{ mph}$$

The following table summarizes the input data by section.

Section	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
Type	B	B	MD	MD	B	B	B	MD	W	B	MD	B
# of lanes	3	3	3	3	3	3	2	2	3	2	2	2
AADT	24,010	17,190	23,990	36,110	29,470	20,070	20,070	30,740	33,210	29,380	30,640	26,640
K-factor	10.1	9.8	9.8	9.8	9.8	9.6	9.6	9.6	9.5	9.5	9.5	9.8
PHF	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Heavy vehicle %	24.4	17.9	17.9	17.9	17.9	17.9	17.9	17.9	17.9	17.9	17.9	17.9
Length (mi)	0.30	0.28	0.27	0.96	0.25	0.17	0.20	1.28	0.28	0.27	1.01	0.32
FFS (mph)	63.8	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1
Posted speed (mph)	60	60	60	60	60	60	60	60	60	60	60	60
Terrain	Level	Level	Level	Level	Level	Level	Level	Level	Level	Level	Level	Level

Note: B = basic, MD = merge–diverge, W = weave.

Step 2. Determine Average Section Performance. Each section's volume-to-capacity ratio and average speed are determined following the procedures given in APM Sections 11.3.1 through 11.3.3. The process is illustrated for the weaving section (section 9).

- *Step 2.1: Develop hourly volumes.* AADTs are converted to hourly volumes by applying the section's K-factor. For example, the weaving section has an AADT of 33,210 and a K-factor of 9.5. The section's hourly volume is then 33,210 × (9.5 / 100) = 3,155 veh/h. Similarly, the on-ramp volume is 235 veh/h, the off-ramp volume is 364 veh/h, and the ramp-to-ramp volume is 27 veh/h. Because the ramp-to-ramp volume is known, the weaving volumes can also be determined. The ramp-to-freeway volume is the on-ramp volume minus the ramp-to-ramp volume: 235 − 27 = 208 veh/h. The freeway-to-ramp volume is the off-ramp volume minus the ramp-to-ramp volume: 364 – 27 = 337 veh/h.

- *Step 2.2: Adjust volumes.* The hourly volumes are next converted into 15-minute demand flow rates by dividing by the peak hour factor (PHF). For the weaving section, the PHF is 0.94 and the 15-minute flow rate is 3,155 / 0.94 = 3,356 veh/h. Similarly, the ramp-to-freeway flow rate is 221 veh/h and the freeway-to-ramp flow rate is 359 veh/h.

- *Step 2.3: Determine capacity adjustment factors.* The CAFs for population (0.968) and CAVs (1.00, as there are no CAVs) have already been determined. There are no ramp meters; therefore, CAF_{meter} = 1.00. From APM Section 11.3.2, CAF_{ramp} = 0.95 for merge–diverge sections and 1.00 elsewhere. From APM Section 11.3.3, CAF_{weave} = 1.00 for all sections except the weaving section. The value of CAF_{weave} for the weaving section is a function of the weaving flows and the section length. The volume ratio *VR* is the sum of the ramp-to-freeway and freeway-to-ramp flow rates divided by the total flow rate in the weaving section:

  $$VR = \frac{(221 + 359)}{3{,}356} = 0.173$$

  The value of CAF_{weave} for the weaving section can now be determined using the equation found in APM Section 11.3.3:

  $$CAF_{weave} = 0.884 - 0.0752VR + 0.0000243L_{weave}$$

  $$CAF_{weave} = 0.884 - 0.0752(0.173) + 0.0000243(0.28 \text{ x } 5280)$$

  $$CAF_{weave} = 0.907$$

- *Step 2.4: Determine section capacity and v/c ratio.* Each section's capacity can now be determined from the capacity equation given in APM Section 11.3.1 (basic sections), 11.3.2 (merge–diverge sections), or 11.3.3 (weaving sections).

For example, the weaving section's capacity is determined as follows, using a truck equivalency factor E_T of 2 for level terrain:

$$c = \frac{\left(2{,}200 + 10 \times (\min(70, FFS) - 50)\right)}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{weave} \times CAF_{pop} \times CAF_{meter} \times CAF_{CAV}$$

$$c = \frac{\left(2{,}200 + 10 \times (\min(70{,}64.1) - 50)\right)}{1 + (2 - 1)(17.9/100)} \times 3 \times 0.907 \times 0.968 \times 1.00 \times 1.00$$

$$c = 5{,}230 \text{ veh/h}$$

The v/c ratio is then the weaving section's flow rate divided by the capacity: 3,356 / 5,230 = 0.64.

- *Step 2.5: Determine average section speed.* This step starts by calculating the average section travel time without incidents, as described in APM Section 11.3.4. The travel time is a function of the section length, free-flow speed, oversaturated delay rate, and undersaturated delay rate. Continuing with the example of the weaving section, the oversaturated delay rate is 0 because the section's v/c ratio of 0.64 is less than or equal to 1. The calculation of the undersaturated delay rate requires looking up as many as five parameters from Exhibit 11-12. First, the value of the *E* parameter for a free-flow speed of 65 mph is compared to the section's v/c ratio. The v/c ratio is greater than the *E* value of 0.62; therefore, there is some undersaturated delay and the other four parameters must be looked up. The undersaturated delay rate for the weaving section is calculated as:

$$\Delta_{RU_{i,t}} = A\left(X_{i,t}\right)^3 + B\left(X_{i,t}\right)^2 + C\left(X_{i,t}\right) + D$$

$$\Delta_{RU_{i,t}} = 92.45(0.64)^3 - 127.33(0.64)^2 + 56.34(0.64) - 8.00$$

$$\Delta_{RU_{i,t}} = 0.14 \text{ s/mi}$$

The mean travel time through the weaving section, without incidents, is then:

$$T_{i,t} = \frac{3{,}600 L_i}{FFS_i} + L_i\left(\Delta_{RU_{i,t}} + \Delta_{RO_{i,t}}\right)$$

$$T_{i,t} = \frac{3{,}600(0.28)}{64.1} + (0.28)(0.14 + 0)$$

$$T_{i,t} = 15.8 \text{ s}$$

Finally, the mean speed through the weaving section, without incidents, is:

$$S_{i,t} = \frac{3{,}600 L_i}{T_{i,t}} = \frac{3{,}600(0.28)}{15.8} = 63.8 \text{ mph}$$

The following table provides travel times and mean speeds for each section along the study facility.

Section	1	2	3	4	5	6	7	8	9	10	11	12
T (s)	16.9	15.7	15.2	54.5	14.0	9.5	11.2	78.4	15.8	15.7	61.4	18.3
S (mph)	63.8	64.1	64.1	63.4	64.1	64.1	64.1	58.8	63.9	61.8	59.2	62.9

Step 3. Determine Section Reliability. This step begins by determining each section's mean travel time index TTI_m, accounting for the effects of incidents on the average section speed. The mean TTI is a function of the free-flow speed, the recurring delay rate, and the incident delay rate.

The incident delay rate is a function of the mean speed and the free-flow speed. For section #8, it is calculated as follows:

$$IDR = [0.020 - (N-2) \times 0.003] \times X^{12}$$

$$IDR = [0.020 - (2-2) \times 0.003] \times 0.86^{12}$$

$$IDR = 0.00327 \text{ s/mi}$$

For section #8, the recurring delay rate is:

$$RDR = \frac{1}{S} - \frac{1}{FFS} = \frac{1}{56.1} - \frac{1}{64.1} = 0.00141 \text{ s/mi}$$

Finally, the mean TTI for section #8 is calculated as follows:

$$TTI_m = 1 + FFS \times (RDR + IDR)$$

$$TTI_m = 1 + 64.1 \times (0.00141 + 0.00327)$$

$$TTI_m = 1.30$$

This result means that during an average peak hour, the travel time through section #8 is 61% longer than the free-flow travel time. Once the mean TTI has been determined, it can be used to estimate other reliability performance measures. For example, the 95th-percentile TTI for section #8 can be calculated using the formula for an urban freeway:

$$TTI_{95} = 16.7754 \times \exp\left(-\frac{2.8221}{TTI_m}\right) = 16.7754 \times \exp\left(-\frac{2.8221}{1.30}\right) = 1.91$$

The policy 95th-percentile TTI (i.e., a TTI based on the posted speed limit) is then:

$$TTI_{P95} = TTI_{95} \times \frac{PSL}{FFS} = 1.91 \times \frac{60}{64.1} = 1.79$$

This result means that in 1 of every 20 peak hours on average, the travel time through section #8 is 79% longer than the travel time at the posted speed. The following table provides mean TTI, policy mean TTI, 95th-percentile TTI, and policy 95th-percentile TTI for each section along the study facility.

Section	1	2	3	4	5	6	7	8	9	10	11	12
TTI_m	1.00	1.00	1.00	1.02	1.00	1.00	1.00	1.30	1.01	1.09	1.27	1.04
TTI_{Pm}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.22	1.00	1.02	1.18	1.00
TTI_{95}	1.00	1.00	1.00	1.07	1.00	1.00	1.00	1.91	1.02	1.27	1.80	1.12
TTI_{P95}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.79	1.00	1.19	1.69	1.05

Step 4. Determine Facility Reliability. This step aggregates the results from the individual sections to produce estimates of overall facility reliability. First, the travel times at the free-flow speed and the posted speed are determined for each section. Mean travel times (including incidents) are determined for each section by multiplying a section's free-flow travel time at the posted speed by its mean TTI. Similarly, 95th-percentile travel times are determined for each section by multiplying its free-flow travel time by its 95th-percentile TTI. The following table provides free-flow, posted speed, mean, and 95th-percentile travel times by section.

Section	1	2	3	4	5	6	7	8	9	10	11	12
T_{FFS}	16.9	15.7	15.2	53.9	14.0	9.5	11.2	71.9	15.7	15.2	56.7	18.0
T_{PSL}	18.0	16.8	16.2	57.6	15.0	10.2	12.0	76.8	16.8	16.2	60.6	19.2
T_m	16.9	15.7	15.2	55.2	14.0	9.5	11.2	93.5	15.8	16.6	71.8	18.8
T_{95}	16.9	15.7	15.2	57.4	14.0	9.5	11.2	137.6	16.0	19.2	102.3	20.2

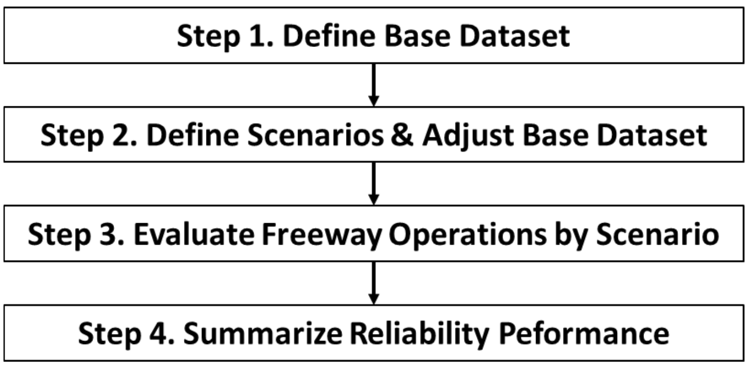
Next, the individual section travel times at the posted speed are summed to produce the facility's travel time at the posted speed (335.4 s). Similarly, the individual section mean and 95th-percentile travel times are summed to produce the facility's mean (354.3 s) and 95th-percentile (435.5 s) travel times, respectively. The mean facility policy TTI is then the mean facility travel time divided by the travel time at the posted speed: 354.3 / 335.4 = 1.06. The facility 95th-percentile policy TTI is the 95th-percentile travel time divided by the travel time at the posted speed: 435.5 / 335.4 = 1.30. Thus, during 5% of peak hours, it takes 30% longer to travel the length of the facility (approximately 1 minute, 40 seconds longer) than the time required to travel at the posted speed.

11.5.4 Detailed Analysis Method

Overview

The detailed analysis method uses the HCM 7th Edition freeway reliability analysis methodology described in HCM Chapters 11 and 25. The method predicts travel times for hundreds of scenarios over the course of a reporting period, which are subsequently assembled into a travel time distribution used to develop reliability performance measures of interest. The method can estimate the impacts of nonrecurring congestion due to demand variability, weather, incidents, work zones, and special events (e.g., football games at UO or OSU, Bridge Pedal), and is sensitive to countermeasures that affect these factors (e.g., safety countermeasures that reduce a roadway's crash rate). The method can also be used to quantify the contribution of each type of congestion factor, as an aid to prioritizing measures to improve roadway reliability. The reliability reporting period is flexible and can incorporate either weekday (e.g., commuter route) or weekend (e.g., recreational route) study periods over the course of a month, a season (multiple months), or an entire year, depending on the analysis needs.

The detailed method involves the following steps:



This process requires software to implement because evaluating freeway operations by itself for one scenario is too computationally intensive to calculate by hand, let alone for hundreds of scenarios. Consequently, the remainder of the section describes the steps the analyst will need to follow to evaluate reliability, but not the specific calculations involved. For more details about the calculation process incorporated into software, see Chapters 11 and 25 of the HCM 7th Edition.

Calculation Process

<u>Step 1: Define the Base Dataset</u>

- **Step 1a. Define the study facility.**
  - The facility length should no greater than the distance a vehicle can travel in 15 minutes when the facility operates under capacity.
- **Step 1b. Define the study period length.**
  - The study period length can be between 1 and 24 hours. The minimum study period length should be sufficiently long to capture the formation and dissipation of queues.
- **Step 1c. Define the seed day.**
  - Fifteen-minute demand data for a single day (the "seed day") will need to be provided for each analysis period within the study period. The specific day used is not particularly important, as long as demand on that day was unaffected by severe weather, incidents, work zones, or special events. Demand volumes for all other days within the reliability reporting period

will be generated relative to the seed day. The seed day should be reported as either the specific month and day when the demand data were collected, or as an "average day" if demands are generated by factoring an AADT.

- **Step 1d. Collect input data required for the freeway facilities core method.**
  - This process is summarized in Section 11.3. Required data include peak hour factors, truck percentages, terrain and area types, base free-flow speeds, lane and shoulder widths, and segment types (e.g., basic, merge, weaving) and lengths.

<u>Step 2: Define Scenarios and Adjust the Base Dataset</u>

- **Step 2a. Define the Reliability Reporting Period**
  - *Define the duration of the reliability reporting period* (RRP). The RRP is defined in Section 9.3.4. It typically covers a calendar year, but shorter timeframes may be appropriate for certain analyses (e.g., Memorial Day weekend to Labor Day weekend for routes connecting the Willamette Valley to the Oregon Coast). A minimum of month is recommended.
  - *Decide which days of the week to include in the RRP.* This decision will typically be driven by demand patterns on the highway and the specific analysis questions to be answered. For example, an urban freeway would typically evaluate weekdays, while a recreational route might focus on weekend days. A rural highway without strong commuting patterns might evaluate all days.
  - *Decide which days to exclude.* Depending on the needs of the analysis, non-typical days may be excluded from the RRP. For example, holidays could be excluded from an analysis of a commute route, but might be important to retain in an analysis of a recreational route.

- **Step 2b. Gather Reliability Inputs**
  - *Demand variability data.*
    1. Option 1: Calculate demand adjustment factors (DAFs) for each day in the RRP using data from an Automatic Traffic Recorder (ATR) located on the study facility (see Appendix 11F). The DAF is equal to the demand on the given day divided by the demand on the seed day.
    2. Option 2: Apply Oregon default DAFs calculated for different facility types (see Appendix 11C).
  - *Weather data.*
    1. Option 1: Calculate weather event probabilities using data from a weather station with a similar climate as the study facility (see Appendix 11F). Ten-year weather datasets are recommended to capture highly impactful weather events that do not occur every year.

ii. Option 2: Apply Oregon default weather event probabilities for the study facility (see Appendix 11C).
iii. Option 3: Apply the HCM default weather event probabilities for the Portland metropolitan area.
iv. The analyst can optionally define DAFs associated with different types of severe weather events. No default DAFs are available, although extreme weather events are understood to affect traffic demands. Analyst judgment is required.

- *Incident data.*
    i. Option 1: Calculate incident frequencies, severity distributions, and durations using local data (see Appendix 11F). Local data should only be used when incident logs are complete and accurate over the entire RRP.
    ii. Option 2: Identify the crash rate for the study facility from ODOT's published crash rates (https://www.oregon.gov/ODOT/Data/Pages/Crash.aspx). Estimate an incident rate using the default Oregon incident-to-crash ratio and apply Oregon default incident severity distributions and durations (see Appendix 11C).
    iii. The analyst can optionally define DAFs associated with different incident types. No default DAFs are available, although traffic management strategies to provide information about incidents can result in shifts in demand. Analyst judgment is required.
- *Short-term work zone and special event data.*
    i. Local data must be used to account for short-term work zones and/or special events.
        1. Required work zone data include the schedule of days and times the work zone is in effect, start and end locations, reductions in the posted speed, lanes affected, and means of separating the work area from traffic. The analyst can optionally specify a demand adjustment associated with the work zone. It is recommended that long-term work zones be analyzed separately, with the freeway facility analysis (seed) file reflective of demand and roadway characteristics present during the work zone.
        2. Required special event data (e.g., football games, music festivals) include the schedule of days and times, duration, changes in demand, and changes in traffic control.

- **Step 2c. Define or Refine Global Inputs**
    - A well-calibrated seed file is preferred; changing global inputs is not recommended.

- If necessary, facility-wide jam density and queue discharge capacity drop values can be calibrated, as described in Appendix 11B.

- **Step 2d. Define the Number of Scenario Replications for Reliability Analysis**
  - A minimum of 200 scenarios is recommended to generate a travel time distribution. This minimum number of scenarios allows relatively rare, but potentially highly impactful, weather and incident conditions to be modeled. Replications of an individual day (with different randomly generated weather and incident events) are used to generate the variety of conditions that might be experienced during the RRP.
  - The default number of replications for a 12-month RRP is four. The combination of five weekdays, 12 months, and four replications results in 240 scenarios.
  - Shorter RRPs will require a greater number of replications to produce a minimum 200 scenarios. Exhibit 11-9 in the HCM 7th Edition recommends the minimum number of replications to use for a given combination of number of months of the year and number of days in a week included in the RRP.

- **Step 2e. Assign Demand Variability by Day and Month to Scenarios**
  - The software applies the appropriate DAF determined in Step 2b to each scenario to account for differences in demand between the day represented by the scenario and the demand on the seed day.

- **Step 2f. Assign Weather Effects to Scenarios**
  - The software randomly assigns weather events (weather type, starting time, and duration) to scenarios in software, based on their probability of occurrence on the day represented by the scenario.
  - The software applies capacity adjustment factors (CAFs) and speed adjustment factors (SAFs), and (if provided by the analyst) DAFs to account for the effects of severe weather on roadway operations during the analysis periods when the weather occurs.

- **Step 2g. Assign Incidents to Scenarios**
  - The software randomly assigns incidents (incident severity, starting time, and duration) to scenarios in software, based on their probability of occurrence.
  - The software applies the capacity adjustment factors (CAFs) and speed adjustment factors (SAFs), and (if provided by the analyst) DAFs determined in Step 2b to account for the effects of the incident on roadway operations during the analysis periods affected by the incident.

- **Step 2h. Assign Work Zones to Scenarios**
    - If work zones have been specified by the analyst, the software will apply the CAFs, SAFs, and DAFs determined in Step 2b to the affected analysis periods to any scenario occurring on a day when a work zone is in effect.
- **Step 2i. Assign Special Events to Scenarios**
    - If special events have been specified by the analyst, the software will substitute the demands and roadway characteristics specified in Step 2b in place of the values used in the seed file. This substitution will occur for the affected analysis periods for any scenario occurring on a day when a special event occurs.

<u>Step 3: Evaluate Freeway Operations by Scenario</u>

- **Analyze Each Scenario**
    - The software performs the HCM freeway facilities core methodology (see Section 11.3.4) and determines the average facility travel time for each scenario, along with all other performance measures normally generated by the core methodology.

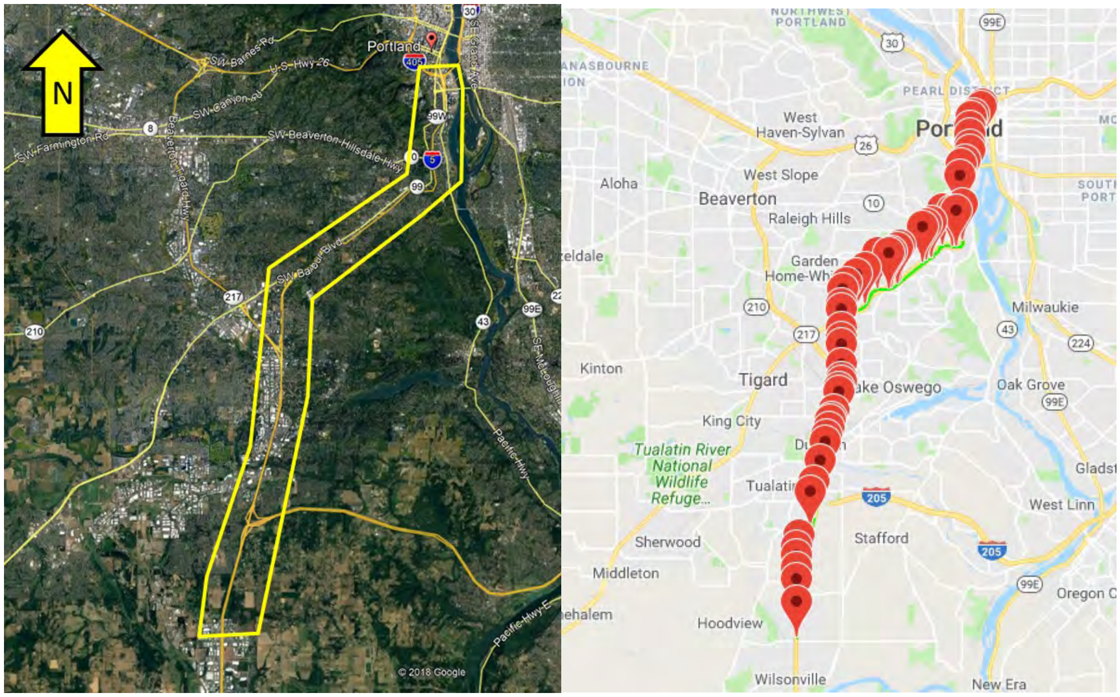
<u>Step 4: Summarize Facility Performance</u>

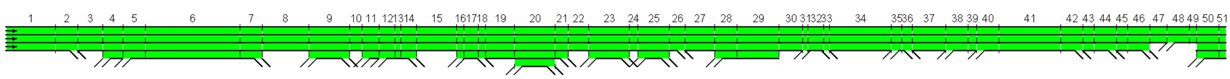
- **Step 4a. Assemble the Travel Time Distribution**
    - The software will compile the travel times associated with each scenario into a cumulative travel time distribution.

- **Step 4b. Compute Reliability Performance Measures**
    - The software will generate reliability performance measures from the travel time distribution. The specific measures automatically reported will vary by the software package used; however, the cumulative travel time distribution can be used to generate any other desired reliability measure not reported automatically. Typical reliability performance measures are described in Section 9.3.3. If the software does not generate policy TTIs (i.e., TTIs based on the posted speed limit rather than the free-flow speed), these can be calculated by multiplying the TTI by the ratio of the posted speed to the free-flow speed.

- **Step 4c. Report Reliability Performance Measures**
    - Only a selection from the range of available reliability performance measures should be reported and shared with decision-makers. Reporting too many measures could distract attention from key indicators. Section 9.3.3 provides recommendations on reliability performance measures to report.

Example 11-16 Freeway Reliability Analysis (Detailed Method)





Step 1: Define the Base Dataset

- **Step 1a. Define the study facility.**
  - Interstate I-5, MP 286 – 300
- **Step 1b. Define the study period length.**
  - 12:00 am – 11:59 pm (24 hours)
- **Step 1c. Define the seed day.**
  - 11/7/2017
- **Step 1d. Collect input data required for the freeway facilities core method.**
  - The geometry was created using FREEVAL's map-based segmentation interface, and demands were identified from 2017 AADT records. Oregon default capacities were applied for all merge, diverge, and weave segments.

Step 2: Define Scenarios and Adjust the Base Dataset

- **Step 2a. Define the Reliability Reporting Period**
  - Duration: 01/01/2017 – 12/31/2017
  - Days of the week: Monday–Friday
  - Excluded days: None

- **Step 2b. Gather Reliability Inputs**
    a. Demand variability data: Oregon default for Interstate - Urban (from Appendix 11C)

Month	Monday	Tuesday	Wednesday	Thursday	Friday
January	1.00	1.42	1.49	1.29	1.48
February	1.27	1.62	1.68	1.65	1.71
March	1.44	1.67	1.74	1.77	1.84
April	1.56	1.78	1.8	1.82	1.89
May	1.63	1.79	1.83	1.89	1.94
June	1.73	1.93	1.95	1.99	2.02
July	1.79	1.97	1.84	2.03	2.1
August	1.75	1.99	2.0	2.02	2.09
September	1.53	1.70	1.75	1.77	1.82
October	1.57	1.81	1.81	1.84	1.90
November	1.44	1.74	1.79	1.83	1.74
December	1.23	1.52	1.71	1.76	1.81

    o Weather data: HCM default for the Portland metropolitan area

Month	Medium Rain	Heavy Rain	Light Snow	Light/ Medium Snow	Medium/ Heavy Snow	Heavy Snow	Severe Cold	Low Visibility	Very Low Visibility	Minimum Visibility	Non-severe weather
January	1.2%	0.3%	1.0%	0.0%	0.1%	0.0%	0.0%	1.0%	0.0%	2.5%	93.9%
February	0.5%	0.1%	0.2%	0.0%	0.0%	0.0%	0.0%	0.6%	0.0%	2.3%	96.3%
March	0.8%	0.2%	0.1%	0.0%	0.0%	0.0%	0.0%	0.4%	0.0%	0.6%	97.8%
April	0.4%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.0%	0.3%	99.1%
May	0.5%	0.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	99.3%
June	0.5%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	99.4%
July	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	99.9%
August	0.1%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	99.8%
September	0.3%	0.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%	0.0%	0.2%	99.1%
October	0.7%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.5%	0.0%	1.3%	97.3%
November	1.2%	0.3%	1.0%	0.0%	0.1%	0.0%	0.0%	1.0%	0.0%	2.5%	93.9%
December	0.5%	0.1%	0.2%	0.0%	0.0%	0.0%	0.0%	0.6%	0.0%	2.3%	96.3%
Average Duration (Minutes)	37.9	18.3	112.6	20.5	21.9	6.6	0.0	37.4	0.0	131.2	37.9

- Incident data: The crash rate from ODOT tables varies by segment from 0.44 to 1.57 crashes per million vehicle miles. The default Oregon incident-to-crash ratio is 4.35.[6] Other incident data are defaulted from values in Appendix 11C:

Incident Severity	**Distribution %**	**Mean Duration (min)**	**Standard Deviation**	**Minimum Duration**	**Maximum Duration**
Shoulder Closure	75.4	34.0	15.1	8.7	58.0
1-Lane Closure	19.6	34.6	13.8	16.0	58.2
2-Lane Closure	3.1	53.6	13.9	30.5	66.9
3-Lane Closure	1.9	67.9	21.9	36.0	93.3
4-Lane Closure	0.0	67.9	21.9	36.0	93.3

- Short-term work zones: None
- Special events: None

- **Step 2c. Define or Refine Global Inputs**
  - No changes made.

- **Step 2d. Define the Number of Scenario Replications for Reliability Analysis**
  - Number of months = 12
  - Days per week = 5
  - Number of replications = 4 (default)
  - Check number of scenarios: 12 × 5 × 4 = 240 > 200 (OK)

- **Step 2e. Assign Demand Variability by Day and Month to Scenarios**
  - DAF relative to the seed date

Month	**Monday**	**Tuesday**	**Wednesday**	**Thursday**	**Friday**
January	0.575	0.816	0.856	0.741	0.851
February	0.730	0.931	0.966	0.948	0.983
March	0.828	0.960	1.000	1.017	1.057
April	0.897	1.023	1.034	1.046	1.086
May	0.937	1.029	1.052	1.086	1.115
June	0.994	1.109	1.121	1.144	1.161
July	1.029	1.132	1.057	1.167	1.207
August	1.006	1.144	1.149	1.161	1.201
September	0.879	0.977	1.006	1.017	1.046
October	0.902	1.040	1.040	1.057	1.092
November	0.828	1.000	1.029	1.052	1.000
December	0.707	0.874	0.983	1.011	1.040

[6] Cambridge Systematics, Inc. Oregon SHRP2 C11 Reliability Analysis Implementation Plan, Task 2, Final Technical Memorandum #1. Medford, MA, Oct. 2018.

- **Step 2f. Assign Weather Effects to Scenarios**
  - CAF, SAF, and DAF

	Medium Rain	Heavy Rain	Light Snow	Light/ Medium Snow	Medium/ Heavy Snow	Heavy Snow	Severe Cold	Low Visibility	Very Low Visibility	Minimum Visibility	Nor-severe Weather
CAF	0.93	0.86	0.96	0.91	0.89	0.78	0.92	0.90	0.88	0.90	1.00
SAF	0.93	0.92	0.87	0.86	0.84	0.83	0.93	0.94	0.92	0.92	1.00
DAF	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

- **Step 2g. Assign Incidents to Scenarios**
  - SAFs and DAFs assigned default values of 1.00.
  - CAFs taken from Oregon default values (Appendix 11C).

# of Lanes	Shoulder Closure	1-Lane Closure	2-Lane Closure	3-Lane Closure	4-Lane Closure
2	0.81	0.70			
3	0.83	0.74	0.51		
4	0.85	0.77	0.50	0.52	
5	0.87	0.81	0.67	0.50	0.50

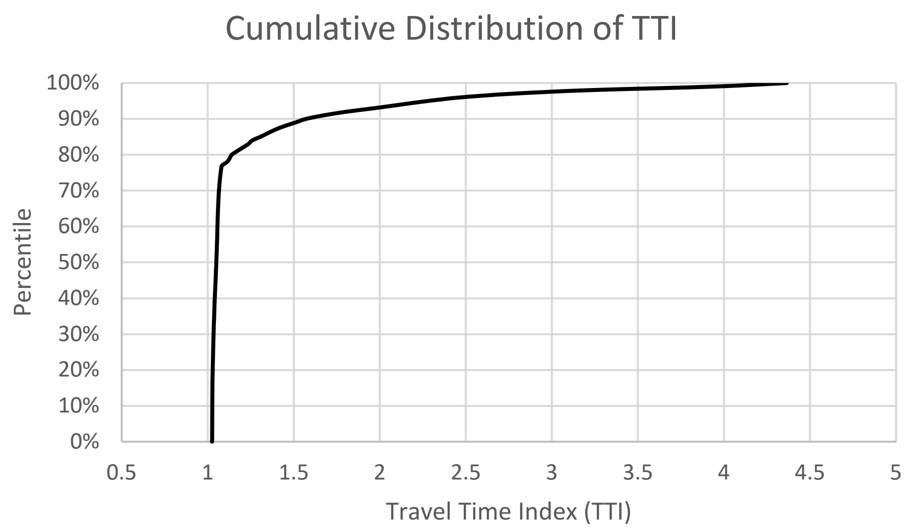
- **Step 2h. Assign Work Zones to Scenarios**
  - Skipped (no work zones identified)
- **Step 2i. Assign Special Events to Scenarios**
  - Skipped (no special events identified)

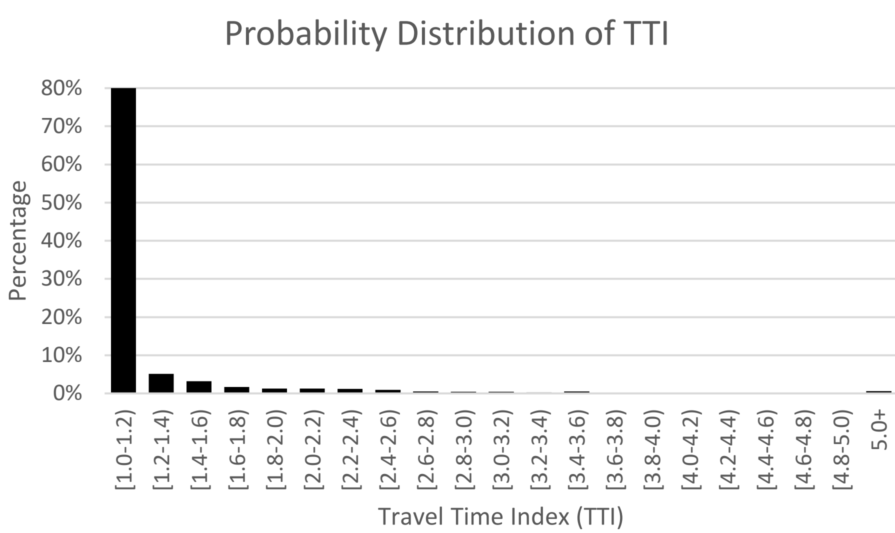
Step 3: Evaluate Freeway Operations by Scenario

This step is performed in software.

Step 4: Summarize Facility Performance

- **Step 4a. Assemble the Travel Time Distribution**





- **Step 4b. Compute Reliability Performance Measures and Step 4c. Report Reliability Performance Measures**
    - All of these measures are reported by the software
    - TTI percentiles can be observed from the cumulative TTI graph
        - 50th-percentile TTI =1.05
        - 80th-percentile TTI = 1.14
        - 95th-percentile TTI = 2.28
    - Mean TTI = 1.26
    - Misery index = average of the highest 5% of TTI values = 3.97
    - Reliability rating = % of TTIs at or below 1.33 = 85.57%
    - Vehicle miles of travel occurring at a TTI > 2 = 6.75%

11.5.5 TSMO Evaluation

Oregon DOT and other state agencies apply a range of Transportation Systems Management and Operations (TSMO) strategies to improve traffic operations on freeways. In the HCM, these strategies are commonly referred to as Active Travel and Demand Management (ATDM) strategies. Common TSMO/ATDM strategies for freeways include:

- Ramp Metering (static or dynamic),
- Part Time Shoulder Use (static or dynamic),
- Traveler information systems,
- Variable Speed Limit, and
- Managed Lanes.

The evaluation of TSMO strategies is possible as part of a detailed analysis methodology but is beyond the scope of the screening and sketch-planning methods. Within the detailed analysis method, there are two principal ways to conduct a TSMO strategy:

1. **Average TSMO performance –** evaluating the operational effects of the strategy on an average day using the core freeway facility method described in Section 11.3.4
2. **Reliability TSMO performance –** evaluating the effects of the strategy on whole-year performance using the detailed travel time reliability procedure described in Section 11.5.4.

While both analysis methods are viable, a reliability-based TSMO evaluation is generally preferred for the following reasons:

- TSMO strategies tend to impact reliability more than average-day performance (see Section 9.3),
- Strategies without capacity improvements won’t impact performance of the average day,
- Several TSMO strategies are targeted at specific sources of unreliable travel (e.g. incident management or road weather management), and

- Several TSMO strategies respond dynamically to changing conditions (e.g. dynamic ramp metering or dynamic part time shoulder use).

In evaluating the viability of a specific TSMO strategy (or a combination of strategies), it is recommended to initially complete a reliability analysis using the method described in Section 11.5.4. Alternatively, probe-based travel time data for the subject facility can be used to explore variability of travel times, as well as underlying sources of unreliable travel. The TSMO strategy should be targeted to counteract specific causes of unreliable travel to maximize the return of investment. For example:

- A facility with unreliable winter travel may benefit from a road weather management system;
- A facility with high incident occurrence and long incident clearance times may benefit from safety improvements or a freeway service patrol system;
- A facility with high occurrence of tourist or special traffic resulting in demand fluctuations may benefit from traveler information systems, or
- A facility with unreliable travel due to fluctuations in on-ramp and mainline demand may benefit from a dynamic ramp metering or dynamic part-time shoulder use system.

The evaluation of a TSMO strategy in an HCM reliability context occurs through adjustments of the HCM calibration factors for free-flow speed (SAF), capacity (CAF), and demand (DAF). At the present time, limited guidance exists for these TSMO adjustment factors, which is summarized below.

Managed Lanes

The HCM 7th Edition provides procedures for evaluating freeways with managed lanes, also referred to as high-occupancy vehicle (HOV) lanes. The HCM managed lane method provides speed–flow relationships and maximum observed flow rates (not true capacities) for managed lane segments as a function of number of lanes (one vs. two lanes) and separation type (paint, buffer, or barrier). Managed lanes can further be evaluated in a facility context, as well as in a whole-year reliability analysis.

Part Time Shoulder Use

A part time shoulder use (PTSU) system results in the (time-limited) addition of an additional lane to selected segments. The shoulder lane has a typical capacity of approximately 1,600–1,800 passenger cars per hour per lane, based on FHWA guidance. Because the HCM method does not allow for lane-by-lane capacity values to be entered, a blended cross-section capacity is calculated following guidance in HCM Chapter 37:

$$AveCap(s) = \frac{CapShldr(s) + CapMFlanes(s) \times MFlanes(s)}{1 + MFlanes(s)}$$

where

AveCap(s) = average capacity per lane for section *s* (veh/h/ln),
CapShldr(s) = capacity per shoulder lane for section *s* (veh/h/ln),
CapMFlanes(s) = capacity per mixed-flow lane in section *s* (veh/h/ln), and

$MFlanes(s)$ = number of mixed-flow lanes in section s (integer).

Ramp Metering
Ramp metering limits the allowable on-ramp demands to enter the freeway mainline, and is implemented through a ramp capacity limit in the HCM and FREEVAL. In addition, HCM Chapter 37 recommends the use of a capacity adjustment factor (CAF) of 1.03 to the freeway merge segment, to reflect reduced turbulence due to the ramp metering strategy. In addition, HCM Chapter 37 provides guidance for dynamic ramp metering evaluation.

Incident and Road Weather Management
An incident management program or freeway service patrol is implemented through a reduction in incident rate, incident severity, or incident duration. No default values are available for these adjustment, and local data or judgment should be used. Similarly, a road weather management system is likely to reduce the duration of weather impacts or weather-related incidents, which need to be obtained locally.

Traveler Information System
Traveler Information Systems primarily impact the demands on a facility. No defaults are available, but demand adjustment factors (DAFs) can be calibrated locally or entered based on assumptions (or objectives) of the specific strategy.

11.6 Truck Level of Service

[Reserved for future use.]

Appendix 11A – Determining Free-Flow Speed

Appendix 11B – Freeway Facility Calibration

Appendix 11C – Oregon Default Values

Appendix 11D – Passenger Car Equivalents on Specific Grades

Appendix 11E – Software

Appendix 11F – Reliability Data Guidance

12 UNSIGNALIZED INTERSECTION ANALYSIS

12.1 Purpose

This chapter presents commonly used unsignalized intersection deterministic analysis procedures and identifies specific methodologies and input parameters to be used on ODOT projects. Simulation procedures are covered in APM Chapter 15. Software settings are covered in Appendix 12A/13A. Topics covered include:

- Turn Lane Criteria
- Unsignalized Intersection Capacity Analysis
- Traffic Signal Warrants
- Estimating Vehicle Queue Lengths at Unsignalized Intersections

> *The scope of this chapter is limited to auto mode analysis at unsignalized intersections. A complete evaluation of unsignalized intersections requires a broader evaluation including of non-auto modes. Refer to APM Chapter 10 for modal considerations such as for left and right turn lanes, and to Chapter 14 for multimodal analysis procedures such as MMLOS. The need for other evaluations such as per the Traffic Manual and HDM should be coordinated with Region Roadway/Traffic or Traffic Engineering/Roadway Engineering Section.*

For software-specific settings and parameters for unsignalized intersection analysis, refer to Appendix 12A/13A.

12.2 Turn Lane Criteria

Proposed left or right turn lanes at unsignalized intersections and private approach roads must meet the installation criteria contained in the Highway Design Manual (HDM). Meeting the criteria does not require a turn lane to be installed. Engineering judgment must be used to determine if an installation would be safe and practical. The ODOT Traffic Manual provides further guidance on the use of right and left turn lanes.

12.2.1 Left Turn Lane Criteria

Purpose: A left turn lane improves safety and increases the capacity of the roadway by reducing the speed differential between the through and the left turn vehicles. Furthermore, the left turn lane provides the turning vehicle with a potential waiting area until acceptable gaps in the opposing traffic allow them to complete the turn. Installation of a left turn lane must be consistent with the access management strategy for the roadway.

Left Turn Lane Evaluation Process

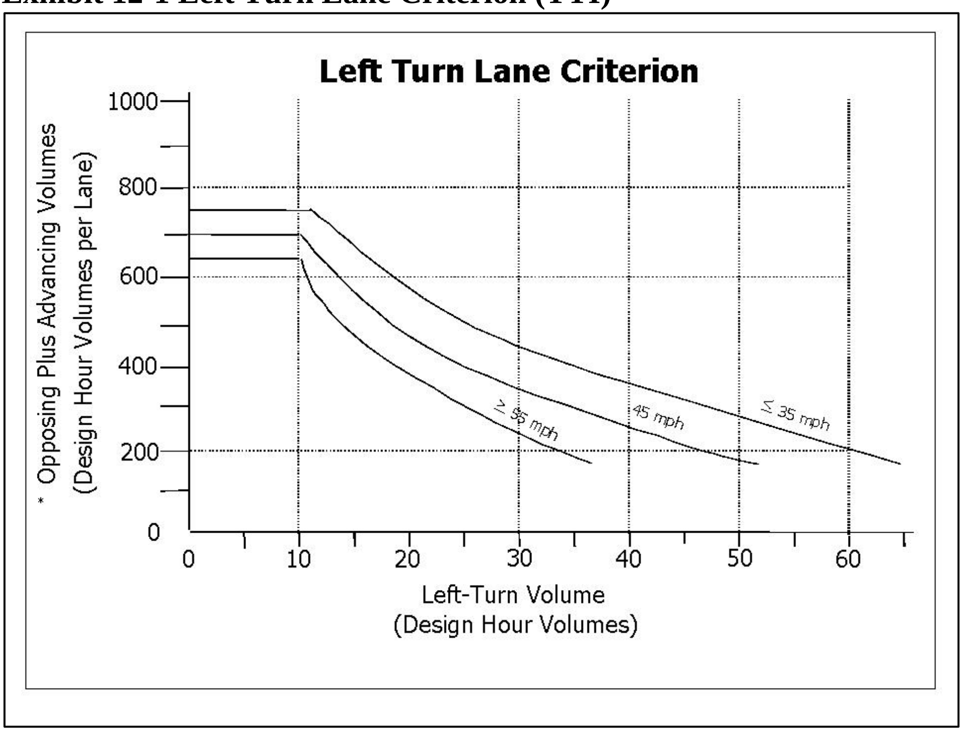
- A left turn lane should be installed, if criterion 1 (Volume) or 2 (Crash) or 3 (Special Cases) are met, unless a subsequent evaluation eliminate it as an option; and
- The Region Traffic Engineer must approve all proposed left turn lanes on state highways, regardless of funding source; and
- Left turn lane complies with Access Management Spacing Standards; and
- Left turn lane conforms to applicable local, regional, and state plans.

Criterion 1:   Vehicular Volume

The vehicular volume criterion is intended for application where the volume of intersecting traffic is the principal reason for considering installation of a left turn lane. The volume criterion is determined by the Texas Transportation Institute (TTI) curves in Exhibit 12-1.

The criterion is not met from zero to ten left turn vehicles per hour but indicates that careful consideration be given to installing a left turn lane due to the increased potential for rear-end collisions in the through lanes. While the turn volumes are low, the adverse safety and operations impacts may require installation of a left turn. The final determination will be based on a field study.

Exhibit 12-1 Left Turn Lane Criterion (TTI)



*(Advancing Volume/Number of Advancing Through Lanes) + (Opposing Volume/Number of Opposing Through Lanes)
Opposing left turns are not counted as opposing volumes

Criterion 2: Crash Experience

The crash experience criterion is satisfied when:

1. Adequate trial of other remedies with satisfactory observance and enforcement has failed to reduce the accident frequency; and
2. There is a history of crashes of the type susceptible to correction by a left turn lane (such as where a vehicle waiting to make a left turn from a through lane was struck from the rear); and
3. The safety benefits outweigh the associated improvement costs; and
4. The installation of the left turn lane does not adversely impact the operations of the roadway.

Criterion 3: Special Cases

1. **Railroad Crossings**: If a railroad is parallel to the roadway and adversely affects left turns, a worst-case scenario should be used in determining the storage requirements for the left turn lane design. The left turn lane storage length depends on the amount of time the roadway is closed, the expected number of vehicle arrivals and the location of the crossing or other obstruction. The analysis should consider all of the variables influencing the design of the left turn lane and may allow a design for conditions other than the worst-case storage requirements, providing safety is not compromised.
2. **Passing Lane**: Special consideration must be given to installing a left turn lane for those locations where left turns may occur and other mitigation options are not acceptable.
3. **Geometric/Safety Concerns**: Consider sight distance, alignment, operating speeds, nearby access movements and other safety related concerns.
4. **Non-Traversable Median**: As required in the Median Policy, a left turn lane must be installed for any break in a non-traversable median (OHP Action 3B.4).
5. **Signalized Intersection**: Consideration shall be given to installing left turn lanes at a signalized intersection. The State Traffic-Roadway Engineer shall review and approve all proposed left turn lanes at signalized intersection locations on the state highway system.
6. **Other Conditions**: Other surrounding conditions, such as a drawbridge, could adversely affect left turns and must be treated in a manner similar to that for railroad crossings.

Evaluation Guidelines

1. The **evaluation** should indicate the installation of a left turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the left turn lane should not be installed or, if already in place, not allowed to remain in operation.
2. **Alternatives Considered**: List all alternatives that were considered, including alternative locations. Briefly discuss alternatives to the left turn lane considered to diminish congestion/delays resulting in criteria being met.
3. **Access Management**: Address access management issues such as the long-term

access management strategy for the state roadway, spacing standards, other accesses that may be located nearby, breaks in barrier/curb, etc.

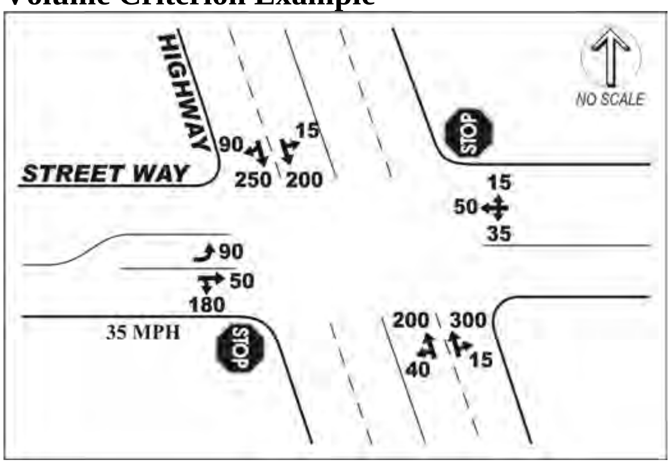
4. **Land Use Concerns**: Include how the proposed left turn lane addresses land use concerns and transportation plans.
5. **Plan**: Include a plan or diagram of proposed location of left turn lane.
6. **Operational Requirements**: Consider storage length requirements, deceleration distance, desired alignment distance, etc. For signalized intersections, installing a left turn lane must be consistent with the requirements in the Traffic Signal Guidelines.

Example 12-1 Left Turn Lane Criterion

Left Turn Volume Criterion

Volume Criterion Example shown below shows an unsignalized intersection with a shared through-right lane and a shared through-left lane on the Highway. The peak hour volumes and lane configurations are included in the figure. The 85th percentile speed is 45 mph and the intersection is located in a city with a population of 60,000. Do the NB and SB left turn movements meet the volume criterion?

Volume Criterion Example



- **Southbound Left**: The southbound advancing volume is 90 + 200 + 250 + 15 = 555, and the northbound opposing volume is 515 vehicles (the opposing left turns are not counted as opposing volumes). The volume for the y-axis on Exhibit 12-1 is determined using the equation:

  y-axis volume = ((Advancing Volume/Number of Advancing Lanes) + (Opposing Volume/Number of Opposing Lanes)) y-axis = (555/2 + 515/2) = 535

  To determine if the southbound left turn volume criterion is met, use the 45-mph curve in Exhibit 12-1, 535 for the y-axis and 15 left-turns for the x-axis. The volume criterion is not met in the southbound direction.

- **Northbound Left**: The northbound advancing volume is 40 + 300 + 200 + 15 = 555, and the southbound opposing volume is 540 vehicles (the opposing left turns are not counted as opposing volumes). The volume for the y-axis on Exhibit 12-1 is (555/2+ 540/2) = 548. To determine if the northbound left turn volume criterion is met, use the 45-mph curve in Exhibit 12-1, 548 for the y-axis and 40 left-turns for the x-axis. The volume criterion is met in the northbound direction.

12.2.2 Right Turn Lane Criteria – Unsignalized Intersections

Not all intersections that meet the siting criteria below should have a right turn lane installed. Refer to APM Chapter 10 for modal considerations for right turn lanes, and to Chapter 14 for multimodal analysis procedures such as MMLOS. The need for other evaluations such as per the Traffic Manual and HDM should be coordinated with Region Roadway/Traffic or Traffic Engineering/Roadway Engineering Sections.

Purpose

The purpose of a right turn lane at an unsignalized intersection is to improve safety and to maximize the capacity of a roadway by reducing the speed differential between the right turning vehicles and the other vehicles on the roadway.

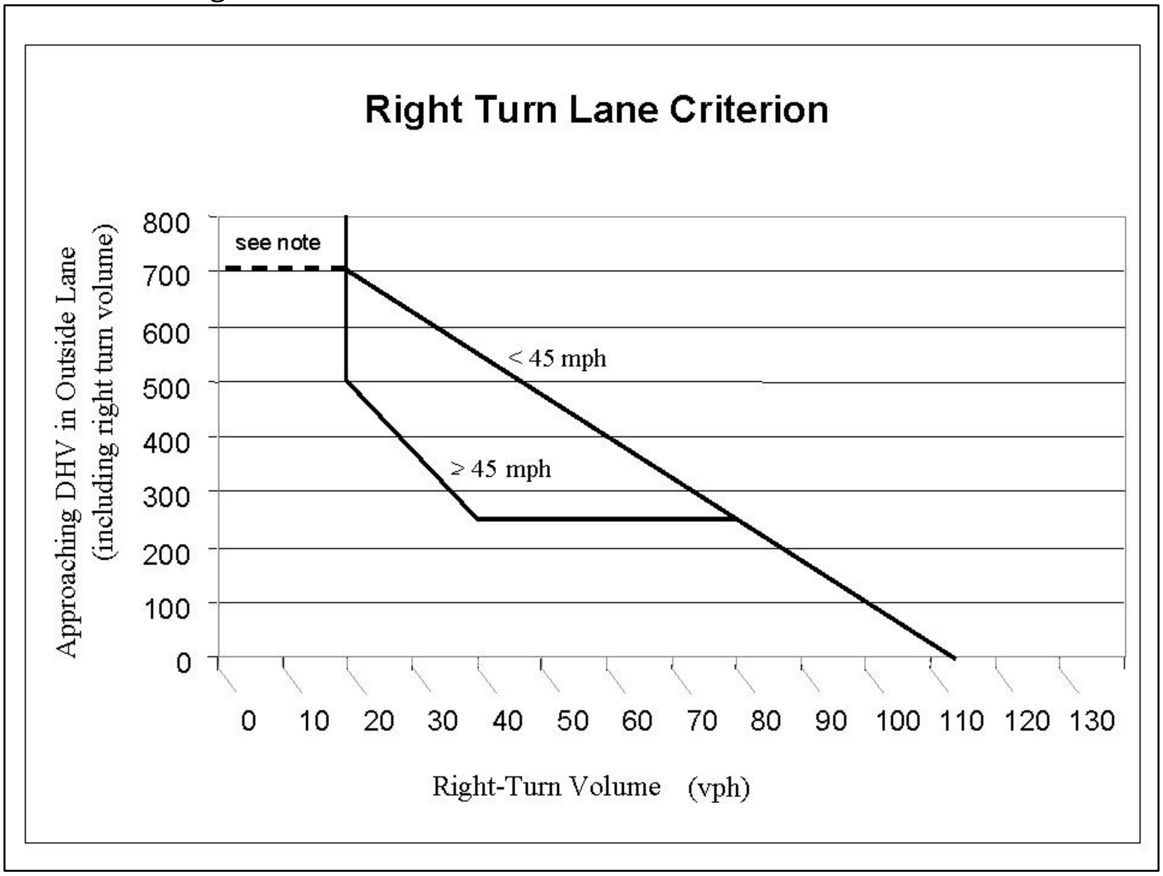
Right Turn Lane Evaluation Process

1. A right turn lane should be installed, if criterion 1 (Volume) or 2 (Crash) or 3 (Special Cases) are met, unless a subsequent evaluation eliminates it as an option; **and**
2. The Region Traffic Engineer must approve all proposed right turn lanes on state highways, regardless of funding source; **and**
3. The right turn lane complies with Access Management Spacing Standards; **and**
4. The right turn lane conforms to applicable local, regional and state plans.

<u>Criterion 1: Vehicular Volume</u>

The vehicular volume criterion is intended for application where the volume of intersecting traffic is the principal reason for considering installation of a right turn lane. The vehicular volume criterion is determined using the curve in Exhibit 12-2.

Exhibit 12-2 Right Turn Lane Criterion



Note: If there is no right turn lane, a shoulder needs to be provided. If this intersection is in a rural area and is a connection to a public street, a right turn lane is needed.

Criterion 2: Crash Experience

The crash experience criterion is satisfied when:

1. Adequate trial of other remedies with satisfactory observance and enforcement has failed to reduce the accident frequency; **and**
2. A history of crashes of the type susceptible to correction by a right turn lane; **and**
3. The safety benefits outweigh the associated improvements costs; **and**
4. The installation of the right turn lane minimizes impacts to the safety of vehicles, bicycles or pedestrians along the roadway.

Criterion 3: Special Cases

1. **Railroad Crossings**: If a railroad is parallel to the roadway and adversely affects right turns, a worst-case scenario should be used in determining the storage requirements for the right turn lane design. The right turn lane storage length depends on the amount of time the roadway is closed, the expected number of vehicle arrivals and the location of the crossing or other obstruction. The analysis should consider all of the variables influencing the design of the right turn lane and may allow a design for conditions other than the worst-case storage requirements, providing safety is not

compromised.
2. **Passing Lane**: Special consideration must be given to installing a right turn lane for those locations where right turns may occur and other mitigation options are not acceptable.
3. **Geometric/Safety Concerns**: Consider sight distance, alignment, operating speeds, nearby access movements and other safety related concerns.
4. **Other Conditions**: Other surrounding conditions, such as a drawbridge, could adversely affect right turns and must be treated in a manner like that for railroad crossings.

Evaluation Guidelines

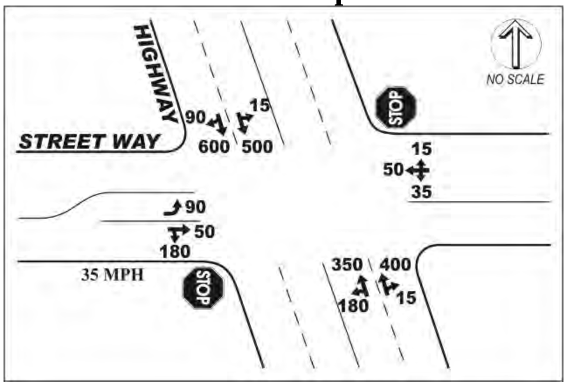
1. The **evaluation** should indicate the installation of a right turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the right turn lane should not be installed or, if already in place, should be reevaluated for continued use.
2. **Alternatives Considered**: List all alternatives that were considered, including alternative locations. Briefly discuss alternatives to the right turn lane considered to diminish congestion/delays resulting in criteria being met.
3. **Access Management**: Address access management issues such as the long-term access management strategy for the state roadway, spacing standards, other accesses that may be located nearby, breaks in barrier/curb, etc.
4. **Land Use Concerns**: Include how the proposed right turn lane addresses land use concerns and transportation plans.
5. **Plan**: Include a plan or diagram of proposed location of right turn lane.
6. **Operational Requirements**: Consider storage length requirements, deceleration distance, desired alignment distance, etc. For signalized intersections, installing a right turn lane must be consistent with the requirements in the Traffic Signal Guidelines.

Example 12-2 Right Turn Lane Criterion

Right Turn Vehicular Volume Criterion

Volume Criterion Example shown below shows an unsignalized intersection with a shared through-right lane and a shared through-left lane on the Highway. The peak hour volumes and lane configurations are included in the figure. The 85th percentile speed is 45 mph, and the intersection is located in a city with a population of 60,000. Determine if a NB or SB right turn lane meets the criterion.

Volume Criterion Example



The northbound outside lane has 400 through vehicles and 15 right turning vehicles for a total of 415 vehicles. Using the 45-mph curve in Exhibit 12-2, along with 415 approaching vehicles and 15 right turning vehicles we find that the vehicular volume criterion is not met.

The southbound outside lane has 600 through vehicles and 90 right turning vehicles for a total of 690 vehicles. Using the 45-mph curve in Exhibit 12-2, along with 690 approaching vehicles and 90 right turning vehicles we find that the vehicular volume criterion is met.

12.3 Unsignalized Intersection Capacity

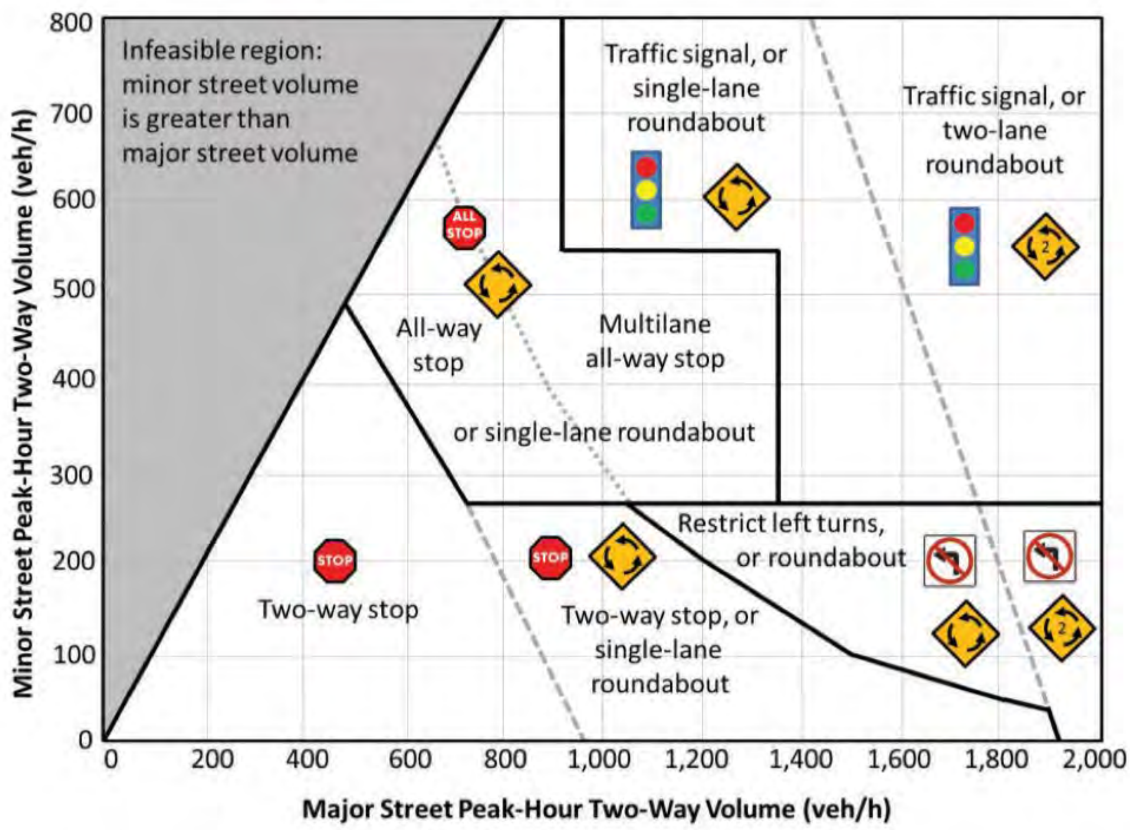
Capacity analysis for unsignalized intersections should generally follow the established methodology of the current HCM for both two-way and all-way stop control.. Many jurisdictions require delay and level of service as the actual threshold performance measures.

Refer to OHP Action 1F.1 for clarification on how the OHP mobility targets are applied at different segment and intersection facilities. Different OHP v/c ratio targets apply to mainline versus minor approaches. HDM v/c ratios apply to all approaches as they do not specify minor or mainline.

If operational performance measure targets or criteria indicate a need, such as where the minor approach exceeds the v/c ratio target, the analyst needs to investigate multiple traffic control type solutions from lowest impact to highest. Potential solutions that could be considered range from, but are not limited to additional channelization, changes to lane alignments/designations, conversion to all-way stop, realignment, roundabout, and j turns as well as more intensive solutions such as signals and grade separations. Supplemental operational measures and considerations also come into play. Volume to capacity ratio is just one factor. Other operational factors to be investigated include multimodal considerations, safety and crash history, Level of Service and delay, sight distance, conflict points, functional area adequacy, and availability of alternate routes. The decision-making process may involve an intersection control evaluation and/or a design exception process. For further guidance on solution development, refer to Chapter 10.

For a sketch planning level estimation of future traffic control needs, the Planning & Preliminary Engineering Applications Guide (PPEAG) provides a graphical method as shown in Exhibits 12-3 and 12-4. Refer to the PPEAG for guidance on appropriate use of this method.

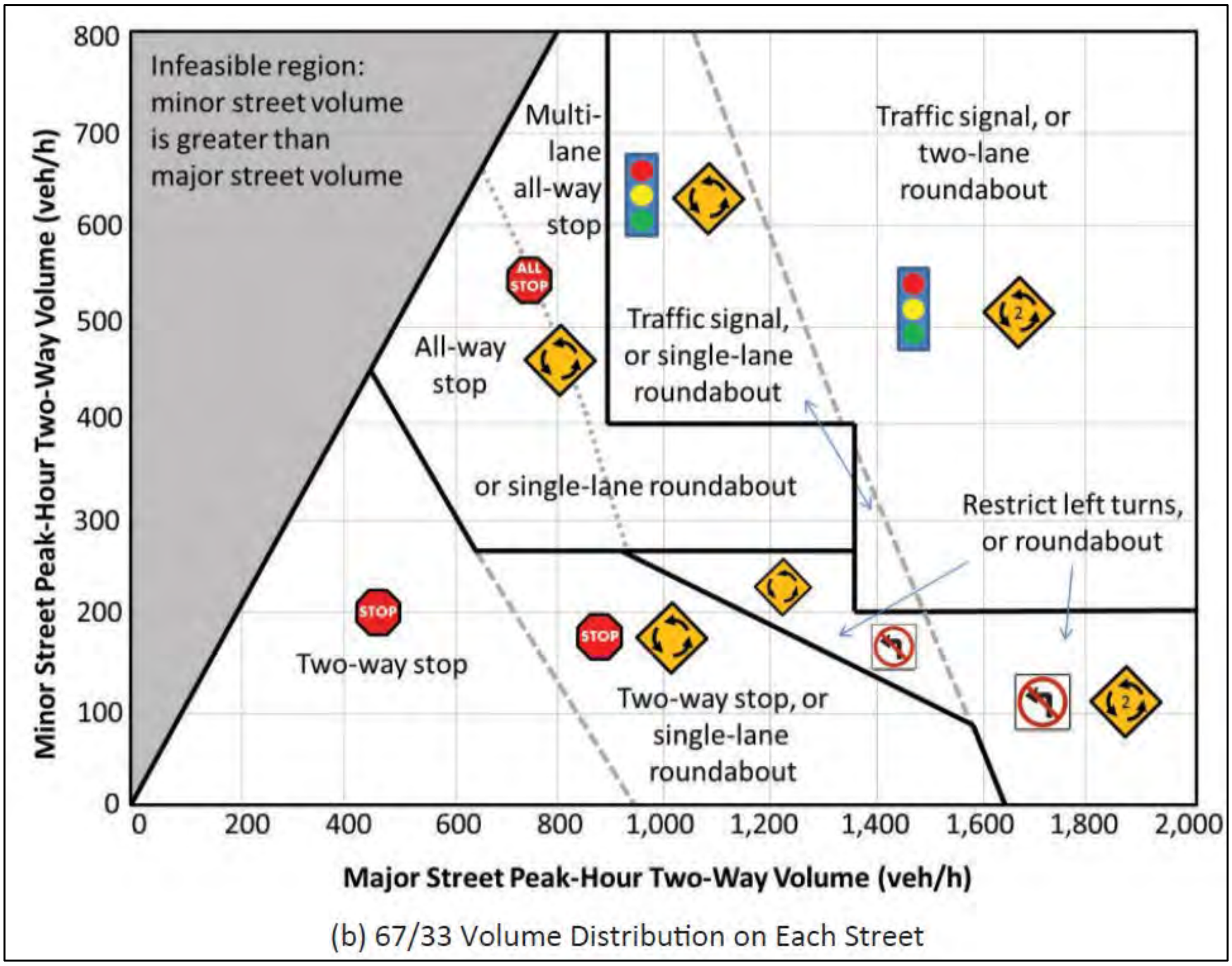
Exhibit 12-3 Planning Level Estimate of Traffic Control Needs - 50/50 Directional Volume Distribution



(a) 50/50 Volume Distribution on Each Street

Source: PPEAG Exhibit 17

Exhibit 12-4 Planning Level Estimate of Traffic Control Needs - 67/33 Volume Distribution



Source: Calculated from MUTCD 8-hour signal warrant, MUTCD all-way STOP warrant, and HCM methods for roundabout capacity and STOP-controlled intersection delay.
Notes: Assumes eighth-highest-hour volumes = 55% of peak hour volumes, peak hour factor = 0.92, 10% left turns and 10% right turns on each approach, and a single lane on each approach as the base case.
See text for an explanation of how boundaries between regions in the graphs were determined.

Source: PPEAG Exhibit 17

12.3.1 Two-Way Stop Control

For two-way stop control, the HCM employs a procedure for analyzing unsignalized intersections that is primarily based on an established hierarchy of intersection movements (based on assigned ROW) and a gap acceptance model. The major components of the gap acceptance model include the critical gap and follow-up time; where the critical gap is the minimum time interval in the major street traffic stream that allows intersection entry for one minor street vehicle and the follow-up time is the time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major street gap under a condition of continuous queuing on the minor street. A simplified planning level analysis method is available in the PPEAG, including a simplified spreadsheet tool.

Substitution for the default values of critical gap and follow-up times used in the HCM shall only be permitted after conducting a thorough field investigation and obtaining ODOT approval.

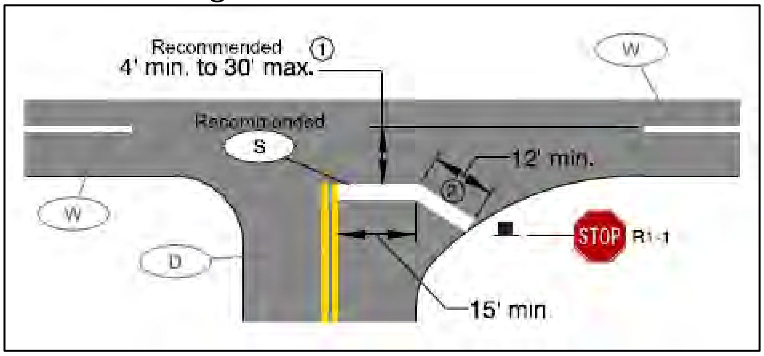
At two-way stop intersections, the controlling movement (usually a minor street left turn) often controls the overall intersection performance. Therefore, the v/c ratio for that movement will typically be the one reported and evaluated against the adopted mobility standard. This is especially important to recognize when analyzing two-way stop-controlled intersections where the very low v/c ratios for the unimpeded, high-volume major street movements will overshadow the higher v/c ratios for the lower-volume minor street movements. In these situations the unimpeded v/c ratio is often very low, even though the minor street movements are near or over capacity. However, as there may be times when the mainline v/c ratio is near the mobility standard, it should always be acknowledged before deferring to minor street movements. Both the mainline through and left movement v/c ratios should be reported along with the minor road approaches as programs generally only report out minor road v/c and mainline left. For analysis result tables, the highest controlling major and minor v/c should be reported.

The analyst should also check for heavy traffic flows that may occur in the opposite direction of peak hour volumes. For example, a high volume right turn movement in the pm peak period can be an indicator of a paired high volume left turn movement in the am peak period.

Right Turn Flares

A right turn flare is where, on the stop-controlled approach at a two-way stop-controlled intersection, a shared lane allows right-turning vehicles to complete their movement while other vehicles are occupying the lane (see Exhibit 12-5). Current analysis procedures/processes/software differ as to how a right turn flare on the minor street is analyzed at unsignalized intersections.

Exhibit 12-5 Right Turn Flare



Source: ODOT Traffic Line Manual 2018 Figure 150-B

The HCM, HCS and Vistro provide a method for directly coding and evaluating the capacity of a flared right turn lane. Synchro/SimTraffic and SIDRA do not allow directly for a flare, so in some cases it may be appropriate to code in a separate (short) turn lane. However, Synchro and SIDRA both see the added lane as having full capacity which is not the case as a flare is limited in its capacity. Therefore the capacity of an intersection is over-estimated when a flare is coded as a separate short turn lane regardless of the “storage” length of the flare.

If SimTraffic is being used, it may still be appropriate to include a scenario with a short turn lane with an appropriate length taper measured from field conditions or from design guidance (i.e., HDM). This will reflect the impact of a flare in SimTraffic when modeling driver behavior and vehicle characteristics for determining measures such as queuing and stop delay.

Engineering judgment is needed to determine when a right turn flare should be coded. There is not a single way to analyze/report the v/c ratios since the factors above vary widely across analysis areas. The analyst should observe operations in the field to understand existing usage. Considerations include:

- Purpose: What is the purpose of the analysis – broad versus specific? Plan versus project? What measures are needed? What is the correct effort for the work?
- Physical Conditions: Width, length, curbed section or not, available sight distance. A flare may be created by a large radius to accommodate trucks. Are there other constraints such as parking?
- Volumes: Total, turn moves
- Characteristics: Drivers, vehicles, traffic flow volumes, bicycles, pedestrians
- Operations – Are vehicles observed using/creating a flare? Is access to the flare blocked by queues?

There are three ways to handle right turn flares for reporting v/c ratio:

1. The most conservative is to not code any flare (the outside lane is a full shared lane).
2. The most correct mathematical way is to input the data (directly or by importing) into the HCM/HCS/Vistro unsignalized processes and coding the flare to account for the partial increase in capacity.

3. A third approach is to code the intersection both ways in HCM/HCS/Vistro and see if the values are different enough to warrant reporting the difference (perhaps as a range).

When the decision is made to include the effects of a flare on an unsignalized intersection, the analyst must use the HCM/HCS/Vistro process to report v/c ratios.

Shared Major Street Left Turn Approaches

There is a limitation of the HCM unsignalized intersection methodology for shared left turn approaches. Major street left turns are always treated as exclusive turn lanes regardless of how they are coded. This can result in very low shared left turn v/c ratios (like 0.01) on an approach that should be over capacity.

Shared major left turn vehicles are approximated in the HCM methodology by adjusting the potential for a "queue-free state" in the case of delaying through movement vehicles. This calculation ratchets down the through lane capacity (1700 for an unstopped lane) to reflect the capacity for the left turning vehicles. Note that this value is for the left turns not for the through movement (which is ignored).

The resulting reported HCM v/c is the left turn volume divided by the capacity of the shared lane for the lefts only. The v/c of the major street (non-stopped) left turn only reflects the left turn volume regardless of if it is in a shared or an exclusive lane (v/c = volume of left turns / shared lane capacity). Other through movements and the stopped movements use the total lane volume divided by the shared lane capacity to obtain v/c.

In most cases this won't make a difference as the minor approaches will tend to control. However, in cases of small minor leg movements and a high volume on the mainline, the major through or the major left will control.

To calculate the correct shared approach v/c requires that you add the through v/c (volume of through vehicles divided by 1700) to the left turn v/c.

Example 12-3 Shared Through-Left V/C Calculation

Software programs that follow HCM 2010/6th Edition report out a value for the v/c ratio for the major left turn movement. However, this v/c is only of the left turn and does not include the through movement.

From a HCM software report: NBL v/c = 0.02

Divide the major street flow rate (pcph) by 1700 pcph to obtain the v/c of the through movement.
In this case the northbound major street flow rate is 191 pcph.

NBT v/c = 191 pcph / 1700 pcph = 0.11

Add the major left to the through movement to obtain the total reportable v/c:

NBLT v/c = NBL v/c + NBT v/c = 0.02 + 0.11 = **0.13**

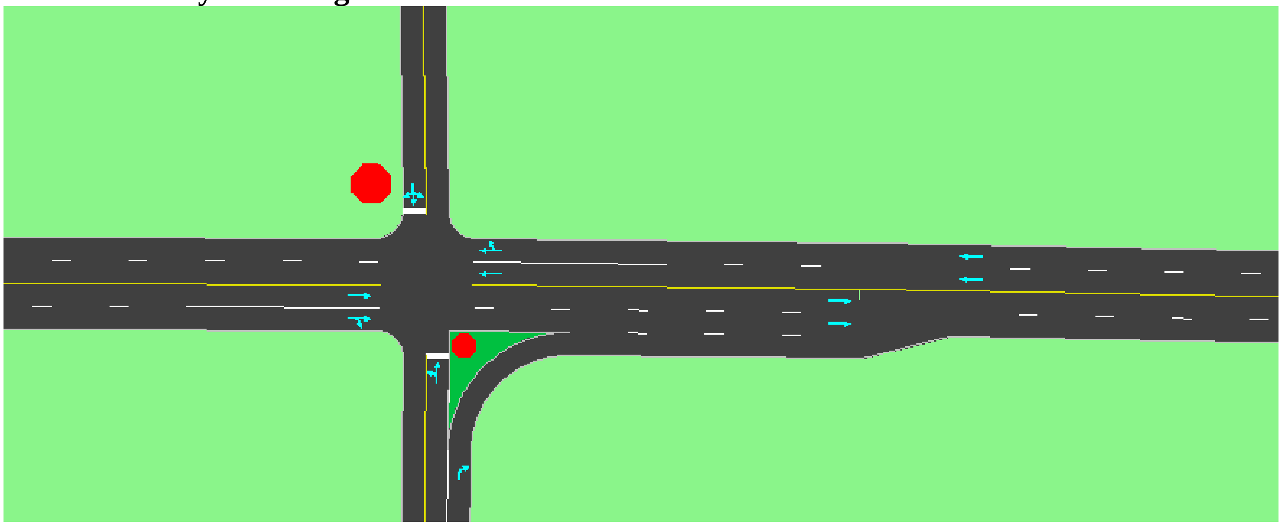
Unsignalized Intersection Acceleration Lanes

An unsignalized intersection acceleration lane is an added lane for vehicles turning from a side street at an at-grade intersection that allows the turning vehicle to accelerate from the turning speed to highway speed, typically on rural limited access highways. The v/c ratio of intersection acceleration lanes is performed using segment analysis. The worst v/c ratio is reported out of either the upstream segment before the merge point, or of the downstream segment after the merge point. Refer to Chapter 11 for segment v/c ratio calculation procedures. Additional analysis of intersection acceleration lane operations may be performed using microsimulation. Refer to Chapter 10 for general considerations on intersection acceleration lanes. An engineering study, Roadway Design Exception, and State Traffic-Roadway Engineer approval is required for acceleration lanes from at-grade intersections on state highways. Refer to Section 500 of the HDM and Section 6 of the ODOT Traffic Manual for more information.

Right-Turn Acceleration Lanes

A right turn acceleration lane is created in Synchro by coding a minor stop-controlled approach right turn movement with one Add Lane, entering the curb radius, and designating the sign control as Free, Stop or Yield as appropriate. If the acceleration lane is a drop lane, a bend node is coded at the end of the lane drop. This will draw an add lane on the departure side of the intersection that will merge with the through travel lanes downstream. In the simulation window, the lane alignment for through traffic is coded as L-NA so through vehicles do not enter the right turn acceleration lane. Likewise, right turning traffic is coded as R-NA so right turn vehicles turn into the acceleration lane and not the through lane. See Exhibit 12-6.

Exhibit 12-6 Synchro Right Turn Acceleration Lane

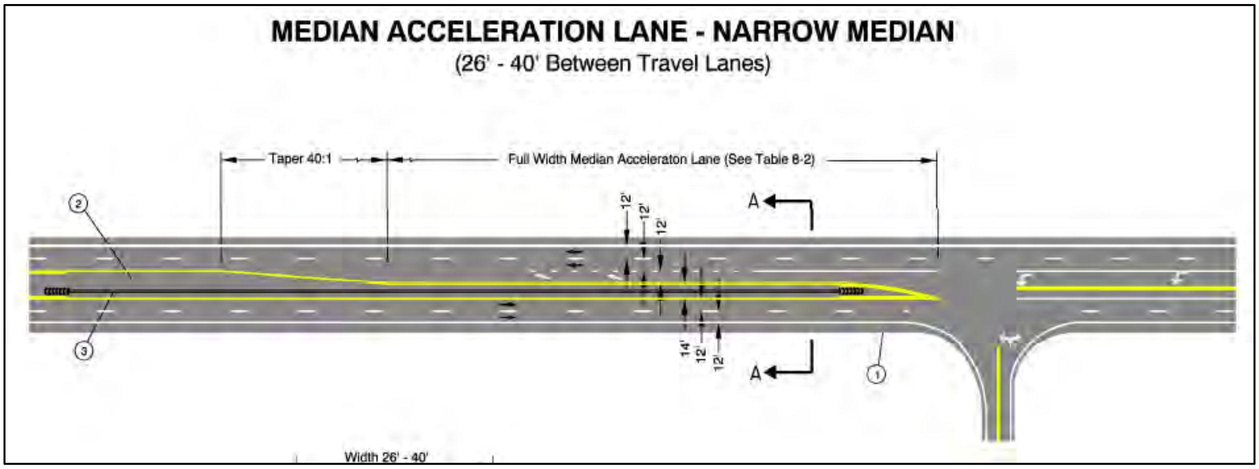


This coding does not provide a v/c ratio of the right turn acceleration lane. The v/c ratio analysis is performed using segment analysis for a two-lane highway. The worst v/c ratio is reported out of either the upstream segment before the merge point, or of the downstream segment after the merge point. For a multilane highway a merge analysis would be performed.

<u>Median Acceleration Lanes and Left Turn Add Lanes</u>

A median acceleration lane is shown in Exhibit 12-7. The acceleration lane drops downstream of the intersection.

Exhibit 12-7 Median Acceleration Lane



Source: 2012 ODOT Highway Design Manual

A left turn add lane is shown in Exhibit 12-8. This differs from the median acceleration lane in that the added lane does not drop downstream of the intersection. This design requires a barrier separating the through lane from the add lane.

Exhibit 12-8 Left Turn Add Lane

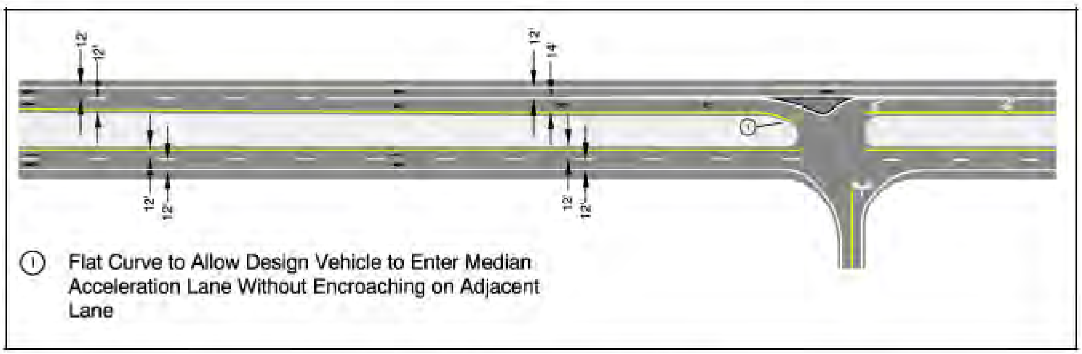


Figure 8-14: Left Turn Add Lane

Source: 2012 ODOT Highway Design Manual

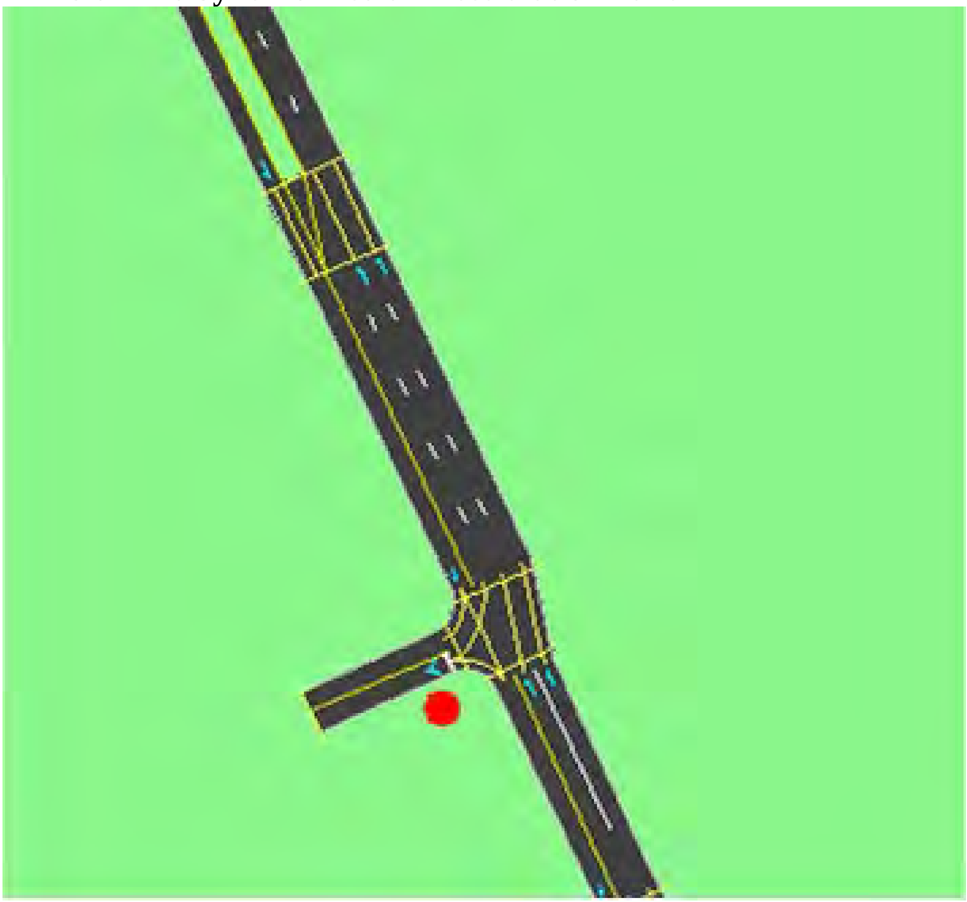
A median acceleration lane or left turn add lane can be created in Synchro by coding the movement as an Add Lane. Synchro provides a v/c ratio for the left turn into the median acceleration lane using a non-HCM methodology. The segment downstream of the merge point still needs to be evaluated using segment analysis, unless it is a left turn add lane where there is no merge point.

For simulation of an add lane, Synchro includes a Lane Alignment setting to establish whether vehicles are allowed to enter the added lane as they pass through an intersection or where through movements need to stay in their own lane. For a median acceleration lane, to prevent through vehicles from entering the median acceleration lane the movement is coded as R-NA. The left-out movement is coded as L-NA to force those vehicles to turn into the median acceleration lane only. See Exhibit 12-9 and Exhibit 12-10.

Exhibit 12-9 Synchro Lane Alignment Settings

SIMULATION SETTINGS	EBL	EBR	NBL	NBT	SBT	SBR
Lanes and Sharing (#RL)						
Traffic Volume (vph)	70	35	0	1125	905	55
Future Volume (vph)	70	35	0	1125	905	55
Storage Length (ft)	0	0	0	—	—	0
Storage Lanes (#)	—	—	—	—	—	—
Taper Length (ft)	—	—	—	—	—	—
Lane Alignment	L-NA	Right	Left	R-NA	Left	Right
Lane Width (ft)	12	12	12	12	12	12
Enter Blocked Intersection	1 veh	1 veh	No	Yes	Yes	Yes
Median Width (ft)	12	—	—	0	0	—

Exhibit 12-10 Synchro Median Acceleration Lane

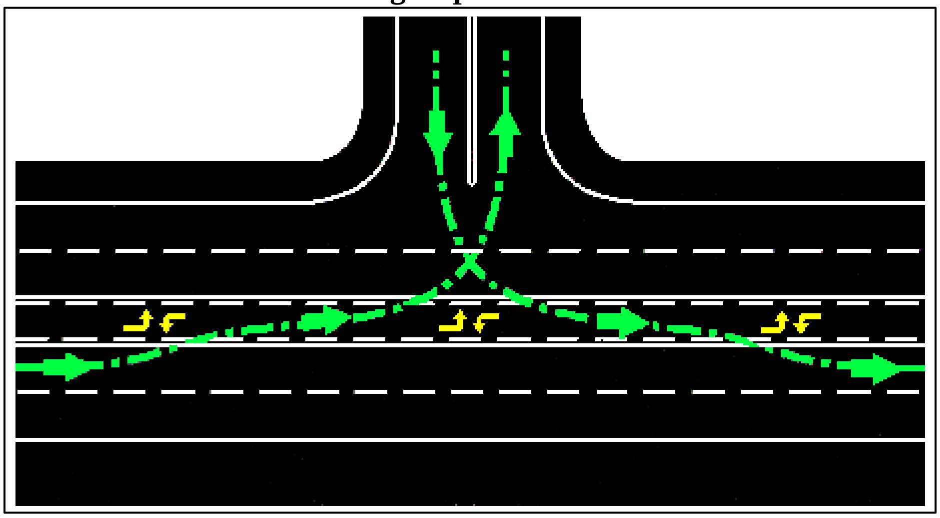


Two-Way Left Turn Lanes (TWLTLs)

Synchro provides a TWLTL feature. With this feature Synchro assumes two-stage left turn out from the minor approach at an intersection, as shown in Exhibit 12-11. This is coded by inputting a Median Width and checking the TWLTL option. Two vehicles can be stored in the median. This does not model driveway operations along a TWLTL. Synchro allows coding of TWLTL operation at four-leg intersections – this configuration is only allowed at minor crossroads. Consult with Region Traffic on whether TWLTL striping is appropriate. SimTraffic does not model two-stage gap acceptance.

Refer to Chapter 10 and Appendix 10A for general considerations on TWLTLs.

Exhibit 12-11 TWLTL 2-Stage Operation



Connected and Automated Vehicles

No conclusive research has been conducted yet on the potential future effects of connected and automated vehicles (CAVs) at two-way stop controlled (TWSC) intersections. However, operational issues at TWSC intersections typically arise on the stop-controlled approaches and CAVs are unlikely to improve the capacity of those approaches. This is because capacity improvements due to CAVs primarily arise because of CAVs being able to form platoons of closely spaced vehicles. In contrast, stop control disrupts side-street platoons, particularly if CAVs obey the legal requirement to come to a full and complete stop before proceeding. Similar to signalized intersections (see Chapter 13), at high percentages (>60–80%) of CAVs in the traffic stream, CAV platoons on the main street may create larger gaps that can be utilized by major-street left-turning traffic and by side-street traffic. However, assuming 100% human-driven vehicles, even for planning analyses, is a conservative approach that is recommended to be applied until further research occurs.

No guidance is presented for the effects of CAVs on TWSC intersections, and it is recommended that no adjustments to capacity are made until further research becomes available.

12.3.2 All-Way Stop Control (AWSC)

Under low volume conditions, two-way stop control (TWSC) is sufficient at most intersections. However, in some circumstances AWSC may be justified, for example as a safety treatment or as an interim improvement such as prior to installation of a roundabout or traffic signal. An Intersection Traffic Control Study is required for multi-way stop installation. The ODOT Traffic Manual contains guidance on the engineering study required for AWSC as well as the approval process. AWSC requires approval of the State Traffic-Roadway Engineer.

The MUTCD contains threshold criteria for AWSC based on crashes or volumes. These are guidelines rather than mandatory requirements. They should not be regarded as an absolute minimum that must be met to consider AWSC.

For AWSC intersection operational analysis, the HCM procedure is based on an analysis of each approach independently. The procedure determines the capacity of each approach, which is used to calculate v/c ratios. The highest v/c ratio approach will be the one reported and evaluated against the adopted mobility target or standard. For multiple lane approaches, report out the highest lane v/c on the controlling approach. Some programs report out only degree of saturation, which should be assumed equivalent to v/c ratio. A simplified planning level analysis method is available in the Planning & Preliminary Engineering Applications Guide. Refer to Chapter 10 for guidance on the consideration of AWSC as a solution.

No research has been conducted yet on the potential future effects of connected and automated vehicles (CAVs) at AWSC intersections. However, CAVs are unlikely to improve the capacity of an AWSC intersection because the all-way stop will disrupt any CAV platoons that might exist on any intersection approach.

No guidance is presented for the effects of CAVs on AWSC intersections, and it is recommended that no adjustments to capacity are made until further research becomes available.

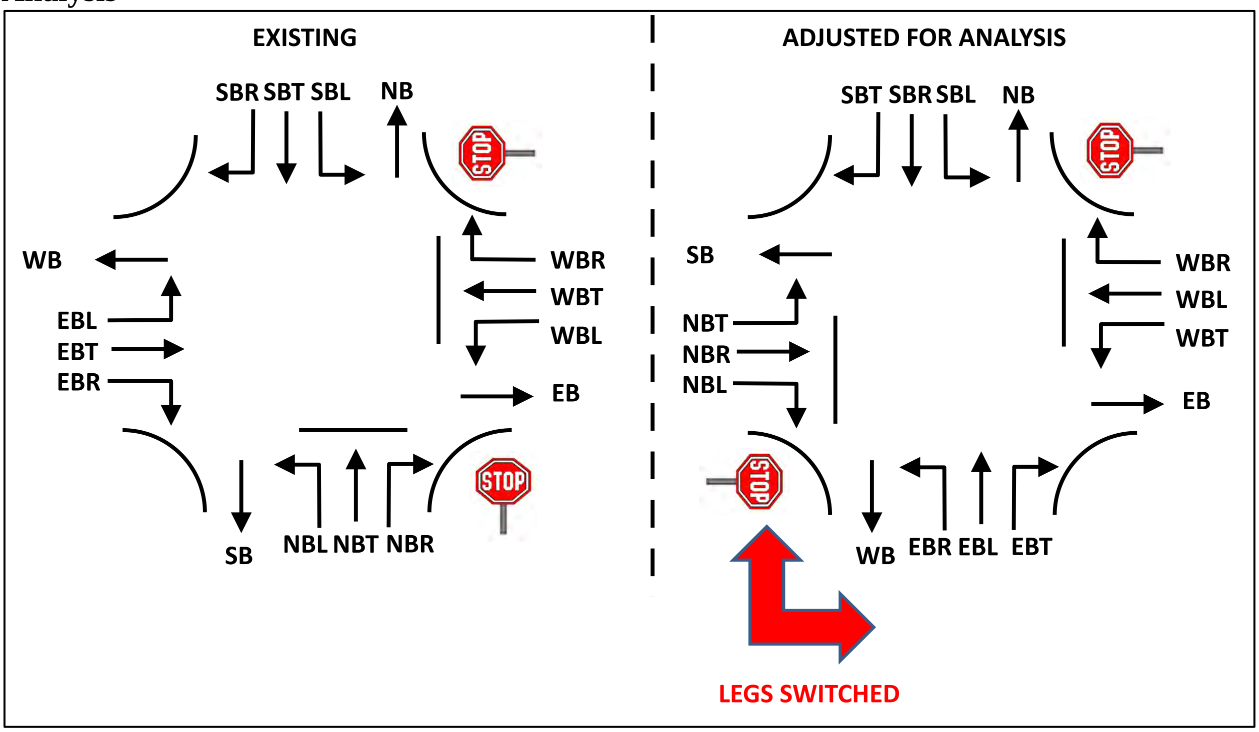
12.3.3 Non-HCM Compatible Stop Configurations

The HCM does not include methods for analysis of certain unsignalized intersections with unusual configurations. These configurations can be analyzed using microsimulation, but simulation does not produce a v/c ratio. SIDRA is the only program able to compute v/c ratios for configurations such as all-way stops with more than 4 legs, or non-standard stop sign placement such as where the mainline turns at an intersection.

An HCM based workaround procedure to obtain an approximate v/c ratio can be done for a 4-leg two-way stop intersection where the stop signs have non-standard placement by moving the volumes to mimic an HCM analyzable configuration. An example is shown in Exhibit 12-12. In this example, the highest volumes occur between the SB and EB approaches which are not stopped, as shown in the Existing configuration. The WB and NB approaches are low volume and are stopped. The workaround to analyze this configuration is to model the approaches having the major flow as if they were opposite each other. This was done in the Adjusted for Analysis configuration shown in Exhibit 12-12, where the EB and NB approaches were switched with each other. Note that Exhibit 12-12 shows turn movements rather than actual lane configurations. All movements still go to the same departure leg as in the Existing configuration. In other words, the directional approach and departure volumes on each leg of the intersection remain unchanged. The Adjusted for Analysis configuration can then be analyzed using HCM TWSC methodology.

The v/c ratios resulting from this method should be considered as approximate only. This method can also be used to estimate preliminary signal warrants. It should be noted that the resulting volumes are only for approximating the analysis and should not be shown on flow diagram figures.

Exhibit 12-12 Non-HCM Compatible Intersection with Directions Adjusted for Analysis



The workaround described above does not work for T intersections where the stem leg is not stopped while the other two legs are stopped. The v/c ratio for such a configuration can be obtained using SIDRA or can be approximated by analyzing the intersection under all-way stop control, or by taking the average v/c ratio between AWSC and TWSC.

12.3.4 Roundabouts

Roundabouts are a safe and efficient intersection option with more free flow than a stop sign or signal provides. Roundabouts can be a gateway or transition feature, roadway connection point, or key element of an access management project. Research has shown roundabouts generally reduce crashes and vehicle delay as compared to signals. Roundabouts have fewer conflict points and severe injury crashes in comparison to other intersection designs.

For roundabouts on state highways, refer to the ODOT Traffic Manual and HDM for roundabout guidelines, standards, siting criteria and the approval process. The State Traffic-Roadway Engineer has been delegated the authority to approve the installation of roundabouts on State Highways, which is divided into two phases: Conceptual Approval and Design Approval.

Unlike traffic signals, there are no roundabout warrants because roundabouts are intersection designs and not traffic control devices. As such the decision to convert an intersection to a roundabout is an engineering design decision and not a traffic control device decision. Roundabout automobile capacity analysis generally follows the current HCM method. For further information, refer to Roundabouts: An Informational Guide (i), Second Edition, also known as NCHRP Report 672. A simplified planning level analysis method is available in the Planning & Preliminary Engineering Applications Guide, including a simplified spreadsheet tool.

Studies have shown that U.S. drivers use roundabouts more conservatively than international drivers. Therefore, U.S. roundabout capacities are generally lower than international values.

ODOT HCM Roundabout Automobile Methodology

HCM 7 Exhibit 22-10 shows 12 steps in the HCM analysis

Step 1: Flow rates from demand volumes

Step 2: Passenger car equivalents (bicycle, medium trucks, and heavy trucks)

Step 3: Circulating and exiting flow rates, addition of movements

Step 4: Entry flow rates by lane

Step 5: Capacity of entry lanes

Step 6: Pedestrian impedance to vehicles

Step 7: Vehicles /hour /lane from capacities and factors

Step 8: Volume/capacity ratio for each lane

Step 9: Average control delay, similar to unsignalized intersections

Step 10: LOS for each lane on each approach

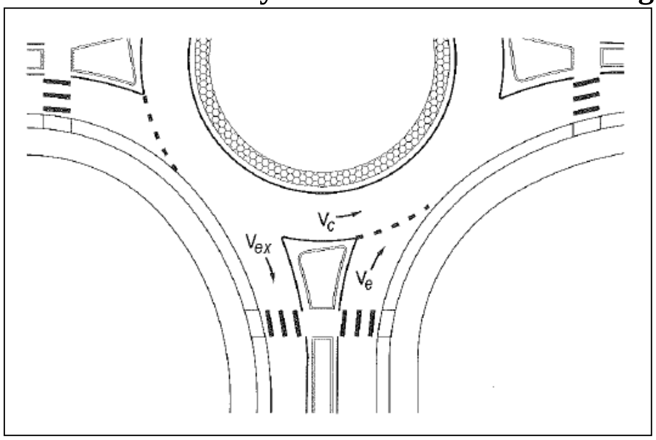
Step 11: Average Control Delay and LOS for entire roundabout

Step 12: Queues for each lane

Exhibit 12-13 (HCM 7 Exhibit 22-12) shows a single lane roundabout with an entry flow conflicting with a circulatory flow. Please note the subscripts: “c” is for circulatory, “e” is for entry, and “ex” is for exiting flow. Entry vehicles yield to circulatory vehicles.

Bicycles that enter the roundabout as a vehicle should be included in the intersection volumes for each movement (including U-turns).

Exhibit 12-13 Analysis on One Roundabout Leg



Source: HCM 7 Exhibit 22-1

Step 1: Flow rates from demand volumes, as per count

Use HCM 7 Equation 22-8 to find the demand flow rate for each movement.

$$v_i = \frac{V_i}{PHF}$$

Where:

v_i = demand flow rate for movement (veh/h)
V_i = demand volume recorded for movement, include bicycles as a vehicle (veh/h)
PHF = peak hour factor

Step 2: Passenger car equivalents (bicycle, medium trucks, and heavy trucks)

Flow rates in vehicles per hour (veh/h) are converted to equivalent passenger cars per hour (pc/h) using vehicle factors. The bicycle equivalent factor should be 1.0, rather than 0.5 as suggested in HCM 7 (Exhibit 12-14).

Exhibit 12-14 Recommended Passenger Car Equivalents

Vehicle Type	Passenger Car Equivalents (E)
Passenger Car	1.0
Bicycle	1.0
Medium truck (two axles, UPS truck)	1.5
Heavy vehicle	2.0

Demand volumes (vph) are converted to equivalent passenger cars per hour (pc/h) using a heavy vehicle factor equation similar to that found in HCM 6. E_m and E_h are the equivalent factors for medium and heavy vehicles, 1.5 and 2, respectively. Heavy vehicles should be WB-67 or long trucks, such as fire engines. This designation is the engineer's judgment and also dependent on the counting methodology. The proportion that these vehicle types occur in a count is designated as P_m and P_h.

An adjusted heavy vehicle adjustment factor equation:

$$f_{HV} = \frac{1}{1 + P_m(E_m - 1) + P_h(E_h - 1)}$$

Where:

f_{HV} = heavy vehicle adjustment factor
P_m = proportion of demand volume that consists of medium trucks (decimal)
P_h = proportion of demand volume that consists of heavy vehicles (decimal)
E_m = passenger car equivalent for medium trucks (Passenger Car Equivalents given)
E_h = passenger car equivalent for heavy vehicles (Passenger Car Equivalents given)

This f_{HV} is then used in HCM 7, Equation 22-9.

$$v_{i,pce} = \frac{v_i}{f_{HV}}$$

Where:

$v_{i,pce}$ = demand flow rate for movement (passenger cars per hour; pc/h)
v_i = demand flow rate for movement (veh/h)
f_{HV} = heavy vehicle adjustment factor

Step 3: Circulating and exiting flow rates; addition of movements

The circulating flow rates in front of each entry are summed in terms of passenger car equivalents. See HCM 7 Equation 22-11 below.

$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

Where:

v_c = circulating flow rates in front of specified entry; in passenger car equivalents

$v_{WBU,pce}$ = flow rates of a specified movement

Step 3B: If considering a bypass lane, calculate the conflicting flow rates. The conflicting flow rates for where the bypass lane merges into the exiting lane can be calculated with HCM 7 Equation 22-12, similar to Equation 22-11.

Step 4: Entry flow rates by lane, if more than one lane

This step is for a multi-lane roundabout approach with more than one entry lane.

For approaches with multiple lanes, lane utilization must be estimated. If field data are not available, HCM 7 Exhibit 22-9 provides guidance on potential default values for different lane configurations.

For approaches with movements that may use more than one lane, follow HCM 7 Exhibit 22-14 to determine the assumed lane assignment.

Using the assumed lane assignment, assign flow rates to each lane using the formulas provided in HCM 7 Exhibit 22-15.

Step 5: Capacity of entry lanes; uses value from step 3

For roundabouts without a capacity and headway study, one should use HCM 7 Equation 33-1 with Exhibit 12-15 below to find the capacity for each entry lane using the circulatory flow rate calculated in Step 3.

$$C = Ae^{(-B \times V_c)}$$

Where:

C = roundabout entry lane capacity (pc/h)

A = intercept parameter, from Exhibit 12-15

V_c = circulating (conflicting) flow (pc/h)

B = slope parameter, from Exhibit 12-15

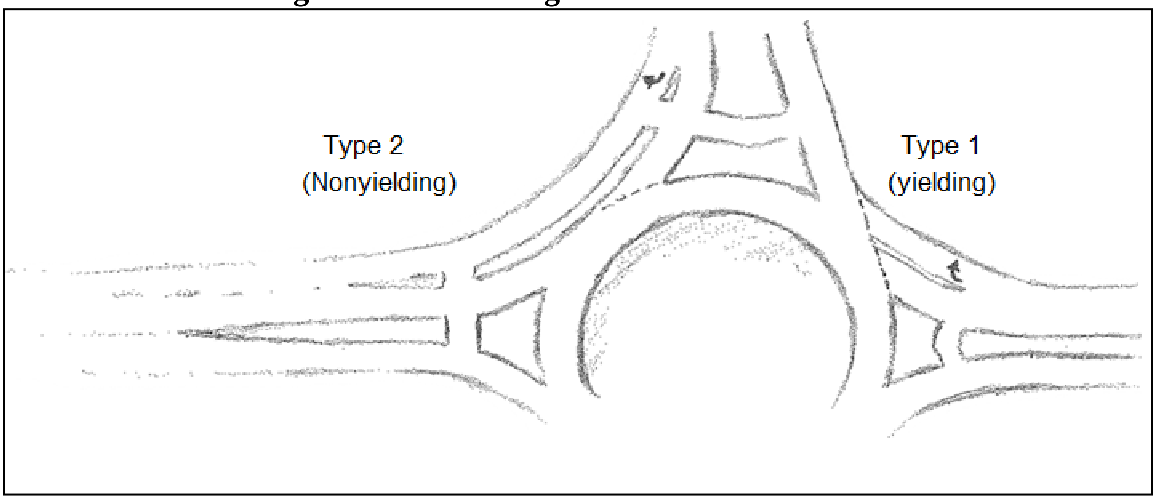
Exhibit 12-15 Roundabout Entry Lane Capacity Model Parameters

Entry Lane Type	*A*	*B*
One-lane entry conflicted by one circulating lane	1,380	0.00102
Two-lane entry conflicted by one circulating lane (both entry lanes)	1,420	0.00091
One-lane entry conflicted by two circulating lanes	1,420	0.00085
Two-lane entry conflicting by two circulating lanes (right entry lane)	1,420	0.00085
Two-lane entry conflicting by two circulating lanes (left entry lane)	1,350	0.00092

Source: HCM 7, Exhibit 33-12

For a Type 1 Yielding Bypass lane as shown in Exhibit 12-16, the capacity of the bypass lane should also be calculated. The exiting flow is used as the circulating or conflicting flow and the bypass lane volume must yield as the entry flow. Use of the single or multilane capacity equation (HCM 7 Step 5, Equation 22-6 or 22-7) depends on the number of opposing exit lanes. No calculation is necessary if the bypass lane is a Type 2, non-yielding bypass entering an add-lane. The capacity of an add-lane is expected to be high.

Exhibit 12-16 Yielding and Non-Yielding



<u>Step 6: Pedestrian impedance to vehicles</u>

Step 6A: The following procedure is for analysis of single lane roundabouts; for two entry lanes, see Step 6B below. For one entry lane, use one of the following three equations, similar to HCM 7 Exhibit 22-18, to find the entry capacity adjustment factor for pedestrians.

1. If the conflicting flow rate exceeds 881 pc/h, or if the number of conflicting pedestrians per hour is less than 40, the entry capacity adjustment factor for pedestrians is 1.0

$$\text{If } v_{c,pce} > 881 \text{ or } n_{ped} < 40,\ \ f_{ped} = 1$$

2. If the number of conflicting pedestrians per hour is equal to or greater than 40, but less than 101, use the following formula to calculate the entry capacity adjustment factor for pedestrians.

$$\text{Else, if } 40 \leq n_{ped} \leq 101,\ f_{ped} = 1 - 0.000137 n_{ped}$$

3. If either of the above two conditions are not met, use the following formula to calculate the entry capacity adjustment factor for pedestrians.

Else,

$$f_{ped} = \frac{1{,}119.5 - 0.715 v_{c,ped} + 0.00073 v_{c,pce} n_{ped}}{1{,}068.6 - 0.654 v_{c,pce}}$$

Where:

f_{ped} = entry capacity pedestrian adjustment factor
v_c = conflicting flow (pc/h)
n_{ped} = conflicting pedestrians (p/h)

An adjustment factor for pedestrians of 1.0 is recommended if there are fewer than 40 pedestrians crossing a leg in an hour. Less than 40 pedestrians crossing a leg in an hour do not have a significant effect on single lane roundabout operation.

If the hourly number of passenger car equivalent vehicles circulating in front of an entrance is over 881, then the adjustment factor for pedestrians is a factor of 1.0. If that is not the case and the number of pedestrians crossing at a crosswalk is greater than 40 and less than or equal to 101, then the second equation determines the adjustment factor for pedestrians.

Step 6B: If considering more than one entry lane, see HCM 7 Step 6 including Exhibits 22-20 and 22-21 for the entry capacity adjustment factor for pedestrians.

<u>Step 7: Vehicles /hour /lane from capacities and factors</u>

Step 7A: A weighted average of the heavy vehicle adjustment factor is created for each entry lane with HCM 6 Equation 22-15.

$$f_{HVe} = \frac{f_{HV,U} v_{U,PCE} + f_{HV,L} v_{L,PCE} + f_{HV,T} v_{T,PCE} + f_{HV,R,e} v_{R,e,PCE}}{v_{U,PCE} + v_{L,PCE} + v_{T,PCE} + v_{R,e,PCE}}$$

Where:

f_{HVe} = averaged heavy vehicle adjustment factor for entry lane
f_{HVi} = heavy vehicle adjustment factor for movement *i*
$v_{i,PCE}$ = demand flow for movement i (pc/h)

The entry lane flow rate is converted back to vehicles per hour with HCM 7 Equation 22-13, a rearrangement of Equation 22-9.

$$v_i = v_{i,PCE} f_{HV,e}$$

Where:

$v_{i,PCE}$ = demand flow rate for lane *i* (pc/h)
v_i = demand flow rate for lane *i* (veh/h)
$f_{HV,e}$ = heavy vehicle adjustment factor

Step 7B: The capacity of a lane is converted back to vehicles per hour in Equation 22-14.

$$c_i = c_{i,PCE} f_{HVe} f_{ped}$$

Where:

$c_{i,PCE}$ = demand flow rate for movement (Epc/h)
c_i = demand flow rate for movement (veh/h)
f_{HVe} = heavy vehicle adjustment factor
f_{ped} = pedestrian adjustment factor

<u>Step 8: Volume/capacity ratio for each lane</u>

The volume/capacity ratio of a lane is calculated in Equation 22-16.

$$x_i = \frac{v_i}{c_i}$$

Where:

x_i = volume-to-capacity ratio of the subject lane *i*
v_i = demand flow rate of the subject lane *i* (veh/h)
c_i = capacity of the subject lane *i* (veh/h)

The v/c ratio is calculated for each lane on each approach. The highest lane v/c ratio calculated should be reported. An approach with a v/c ratio exceeding a standard, such as the applicable OHP/HDM v/c ratio, calls for further analysis and potential improvement, such as a bypass lane.

The decision to build a roundabout is determined by the State Traffic-Roadway Engineer (with consultation from Region Traffic). Considerations for further study may include highway classification, traffic characteristics, and system continuity.

Step 9: Average control delay, similar to unsignalized intersections

HCM 7 states the delay to be similar to unsignalized intersections, per United Sates roundabout data. The HCM makes a good point about delay at peak hour or design hour:

> "At higher volume-to-capacity ratios, the likelihood of coming to a complete stop increases, thus causing behavior to resemble STOP control more closely."

At higher volumes, it is likely that motorists may make stops before the crosswalk as well as the yield/stop that HCM 7 describes as resembling STOP control.

The average control delay of a lane is calculated in HCM 7 Equation 22-17.

$$d = \frac{3600}{c} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3600}{c}\right)x}{450T}}\right] + 5 \times \min[x, 1]$$

Where:

d = average control delay (s/veh)
x = volume-to-capacity ratio of the subject lane
c = capacity of the subject lane (veh/h)
T = time period (h) (T = 0.25 for a 15-min analysis)

The delay is calculated for each lane on each approach.

Step 10: LOS for each lane on each approach

The delay from Step 9 and the v/c ratio from Step 8 are used with Exhibit 12-17 (HCM 7 Exhibit 22-8) to determine the LOS of each lane on each approach.

Exhibit 12-17 HCM Unsignalized LOS Table

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a]	
	$v/c \leq 1.0$	$v/c > 1.0$
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: [a] For approaches and intersectionwide assessment, LOS is defined solely by control delay.

Source: HCM Exhibit 22-8

Step 11: Average Control Delay and LOS for entire roundabout

The average control delay of a roundabout is calculated in HCM 7 equations 22-18 and 22-19. For a single lane roundabout with single entry lanes, these equations will reduce to an average of approach (HCM 7 Equation 22-19):

$$d_{intersection} = \frac{\sum d_i v_i}{\sum v_i}$$

Where:

$d_{intersection}$ = average control delay for entire intersection (s/veh)
d_i = control delay for approach *i* (s/veh)
v_i = flow rate for approach *i* (veh/h)

With the average intersection delay, the intersection LOS is found from Exhibit 12-16 (HCM 7 Exhibit 22-8).

For multilane approaches and approaches with bypass lanes, the full Equation 22-18 is used, which calculates a weighted average delay for the approach. An overall intersection delay and LOS can also be determined using Equation 22-19.

Step 12: Queues for Each Lane

The 95th percentile queue of a roundabout entry lane is calculated in HCM 7 Equation 22-20.

$$Q_{95} = 900T\left[x - 1 + \sqrt{(1-x)^2 + \frac{\left(\frac{3600}{c}\right)x}{150T}}\right]\left(\frac{c}{3600}\right)$$

Where:

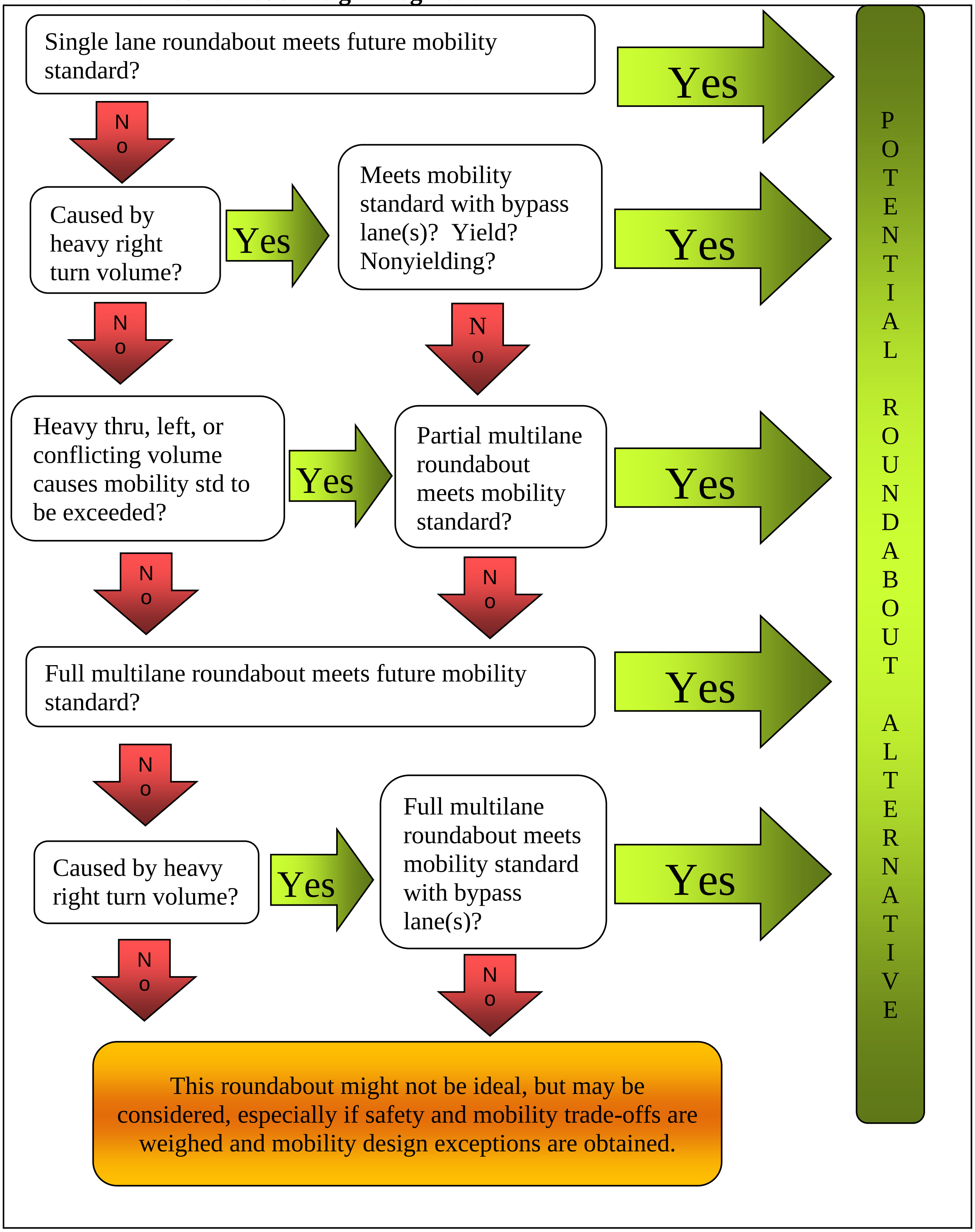
Q_{95} = 95th percentile queue (veh)
x = volume-to-capacity ratio of the subject lane
c = capacity of the subject lane (veh/h)
T = time period (h) (T = 0.25 for a 15-min analysis)

Logical Design Progression

Start analysis of a single lane roundabout with existing and future volumes. If an entry lane exceeds the mobility standard, then analyze a bypass lane for that approach. The bypass lane volume is subtracted out of the roundabout entry lane volume. This affects flow rate calculations of Steps 1 through 5. This may also affect capacity, v/c, delay, LOS, or 95th percentile queue. If a bypass lane merges into an existing lane (Yielding

Type 1), then calculate the capacity of the bypass lane (HCM 7 Chapter 33, Example Problem 1). If not due to a heavy right turn movement, then a multilane roundabout should be considered (not all of the circulating lanes must have more than one lane). If a multilane roundabout entry lane exceeds the mobility standard, then again consider a bypass lane. A flow chart showing this process is shown in Exhibit 12-18.

Exhibit 12-18 Roundabout Design Progression



Reporting

ODOT required outputs:

- Highest entry lane V/C
- Each bypass lane V/C
- Predicted queue lengths

Other jurisdictions may require:

- Intersection LOS and delay
- Bypass LOS
- Lane capacities
- Delay and LOS on each leg
- Entry and conflicting flows

Connected and Automated Vehicles

Chapter 33 of HCM 7 provides a process for adjusting the capacity of a roundabout entry lane to account for the effects of connected and automated vehicles (CAVs) in the traffic stream. This process, described in Appendix 6B, adjusts the parameters *A* and *B* used in the entry lane capacity equation in Step 5, resulting in somewhat higher entry lane capacities, depending on the percentage of CAVs in the traffic stream. No CAV capacity adjustment is yet available for roundabout bypass lanes, due to a lack of research.

As of 2023, no vehicles were available commercially that met the definition of a CAV for the purposes of the capacity adjustments provided for roundabout analyses in the HCM (i.e., a vehicle with an operating cooperative adaptive cruise control system that can communicate with other vehicles and driving without human intervention in any situation). The capacity adjustment process for CAVs presented in Appendix 6B is intended for use only in longer-range planning analyses. That appendix also provides guidance on estimating the percentage of CAVs in the traffic stream in a future year and example problems.

Because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in near-term analyses such as traffic impact studies.

12.4 Traffic Signal Warrants

Because the presence of traffic signals can degrade some aspects of overall traffic operations on a highway in addition to the improvements they provide, traffic signal warrants are used to determine when installation may be justified by identifying conditions where the benefits may outweigh the costs. The Manual on Uniform Traffic Control Devices (MUTCD) provides a set of 9 warrants to be used in determining if the installation of a traffic signal should be considered. In addition to these, the ODOT Transportation Planning Analysis Unit has also developed a set of “preliminary” traffic signal warrants, which are based on a portion of the MUTCD warrants but require less data for analysis. The preliminary warrants are generally not accepted as a basis for approving the installation of a traffic signal but are useful for projecting signalization needs for future years. Full warrants are evaluated later as part of the engineering study required by the MUTCD. Many other considerations go into determining whether a signal should be installed. For example, a signal installation is generally not appropriate in a rural area. The MUTCD and Preliminary Signal Warrant (PSW) methodologies are described below.

> *The MUTCD warrants are part of the Traffic Engineering Section signal approval process. For all other applications/projections, only the ADT-based Preliminary Signal Warrant process can be used.*

When evaluating signal warrants (preliminary or MUTCD), it is important to include only the appropriate lane configurations and traffic volumes. Incorrect modeling of intersections is a very common mistake and can make a significant difference to the outcome of the analysis. There may be times when minor streets need to be modeled as major streets because of high side-street volumes (e.g., rural interchange) or left turns behave as right turns when dealing with one-way streets. In such cases, sound engineering judgment is critical to obtaining accurate analysis. Direction for proper modeling of intersections when analyzing signal warrants is included in the next section.

Traffic signal warrants must be met and the State Traffic Engineer’s approval obtained before a traffic signal can be installed on a state highway. However, approval of a signal depends on more than just a warrant analysis. Meeting a warrant is necessary to install a signal, but it does not mean a signal should be recommended or guarantee its installation. Considerations to be evaluated include safety concerns, alternatives to signalization, signal systems, delay, queuing, bike and pedestrian needs, railroads, access, consistency with local plans, local agency support and others. The engineering investigation, conducted or reviewed by the Region Traffic Engineer, must demonstrate a reduction in delay, improvements in safety, improved connectivity or some other "benefit" and why a signal is the best solution as compared to other alternatives, such as listed in MUTCD Section 4B.04a. During the consideration, input from the Region Traffic Engineer and from the Traffic Engineering Section must be obtained prior to reaching any conclusions. Coordination should occur early in the project process to allow sufficient time to develop and evaluate alternatives to signalization if deemed necessary. Once the investigation and recommendation is reviewed, the section will act on the request.

If preliminary signal warrants are met, project analysts need to forward a copy of the PSW form and analysis to Region Traffic and coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. If Region Traffic supports the concept of a signal installation, they will forward the analysis to the Traffic Engineering Section.

12.4.1 Preliminary Signal Warrants

Introduction

The single most important criterion for preliminary signal warrant analysis is engineering judgment. In the following procedures only the fundamental parameters of volumes and approach lanes are provided.

Background

There are 9 traffic signal warrants found in the MUTCD, listed in Part 4. The signal warrants are:

- Warrant 1, Eight-Hour Vehicular Volume
    - Condition A – Minimum Vehicular Volume
    - Condition B – Interruption of Continuous Traffic
- Warrant 2, Four-Hour Vehicular Volume
- Warrant 3, Peak Hour
- Warrant 4, Pedestrian Volume
- Warrant 5, School Crossing
- Warrant 6, Coordinated Signal System
- Warrant 7, Crash Experience
- Warrant 8, Roadway Network
- Warrant 9, Intersection Near a Grade Crossing

OAR 734-020-0460 (1) stipulates that only MUTCD Warrant 1 Condition A and Condition B may be used to project future needs for traffic signals beyond three years from the present time. Condition A deals primarily with high volumes on the intersecting minor street. Condition B addresses high volumes on the major street and the delays and hazards to vehicles on the minor street trying to either access or cross the major street. The preliminary warrant is considered satisfied if either Condition A or Condition B is met for either 100% of the thresholds or 70% of the thresholds when the major-street speed exceeds 40 mph or in an isolated community with a population of less than 10,000. The 80% and 56% thresholds that may apply after adequate trial of other remedial measures are not used for preliminary signal warrants. MUTCD Warrant 3, Peak Hour cannot be used to project needs for future traffic signals.

Information for Narrative

The following statement should be included in the Analysis Methodology section of the Narrative:

> TPAU uses Signal Warrant 1, Condition A and Condition B (MUTCD), which deal primarily with high volumes on the intersecting minor street and high volumes on the major-street. Meeting preliminary signal warrants does not guarantee that a signal shall be installed. Before a signal can be installed a field warrant analysis is conducted by the Region. If warrants are met, the State Traffic Engineer will make the final decision on the installation of a signal.

Analysis

In MUTCD Warrant 1 the eighth highest hour of an **average** day is used to determine whether a warrant is met. At the analysis stage in TPAU, ADT is used for preliminary signal warrant analysis. A conversion factor of 5.65% is applied to the ADT to reach the eighth highest hour. The conversion factor of 5.65% was developed based on a study of 1991 to 1994 manual counts and as agreed on by TPAU and the Traffic Section. This factor was used to convert MUTCD hourly volumes to ADT volumes (divided the MUTCD volume by the factor .0565). This equals the target ADT volume to meet MUTCD Warrant 1. As an example, for Condition A to be met the MUTCD requires a minimum total of 500 vehicles per hour on both approaches of the major street, where the major and minor streets both have only one lane for moving traffic (at 100%, assuming no reductions). To convert this to ADT volumes, the following calculations are made:

$$ADT = \frac{500}{0.0565} = 8{,}850$$

These calculations of ADT thresholds have already been completed for the analyst on the Preliminary Traffic Signal Warrant Analysis Form, as can be seen in Exhibit 12-18[1]

If the 85th percentile speed of major street traffic exceeds 40 mph in either an urban or rural area or when the intersection lies within the built-up area of an isolated community (typically non-MPO) having a population of less than 10,000, reduce the target volume for the warrants to 70 percent of the normal requirements, as shown in the preliminary traffic signal warrant analysis sheet in Exhibit 12-19.

[1] Note that the value of 8,850 calculated in the analysis example is the same as the value on the worksheet for this scenario.

Exhibit 12-19 Preliminary Traffic Signal Warrant Analysis Form

Oregon Department of Transportation **Transportation Development Branch** **Transportation Planning Analysis Unit**					
Preliminary Traffic Signal Warrant Analysis[1]					
Major Street:			**Minor Street:**		
Project:			**City/County:**		
Year:			**Alternative:**		
Preliminary Signal Warrant Volumes					
Number of Approach Lanes		ADT on Major Street Approaching From Both Directions		ADT on Minor Street, Highest Approaching Volume	
Major Street	Minor Street	Percent of Standard Warrants		Percent of Standard Warrants	
		100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8,850	6,200	2,650	1,850
2 or more	1	10,600	7,400	2,650	1,850
2 or more	2 or more	10,600	7,400	3,550	2,500
1	2 or more	8,850	6,200	3,550	2,500
Case B: Interruption of Continuous Traffic					
1	1	13,300	9,300	1,350	950
2 or more	1	15,900	11,100	1,350	950
2 or more	2 or more	15,900	11,100	1,750	1,250
1	2 or more	13,300	9,300	1,750	1,250
5.65% of the above ADT volumes is equal to the MUTCD vehicles per hour (vph)					
	100 percent of standard warrants				
	70 percent of standard warrants[2]				
Preliminary Signal Warrant Calculation					
	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major				
	Minor				
Case B	Major				
	Minor				
Analyst and Date:			Reviewer and Date:		

[1] Meeting preliminary signal warrants does not guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

[2] Used due to 85th percentile speed in excess of 40 mph or isolated community with population of less than 10,000.

Determining the number of approach lanes and determining the approach volumes to use in the warrant analysis requires knowledge of the involved intersection. A spreadsheet calculator is available on the Technical Tools page that streamlines these calculations including the right turn discounts.

1. Major Street (Higher Volume Street)

   - Include only the through and through/left turn lanes in the number of approach lanes.
   - The major street number of approach lanes is directional, so a left-through in one direction and a through right (like at a ramp terminal) is considered as only a 1-lane major street, not a 2-lane major street.
   - An exclusive left turn lane and a through lane in one direction would be considered as a 2-lane major street, even if the other direction had only one lane.
   - An exclusive right turn lane is not counted in the number of directional approach lanes. An exclusive right turn lane and a through lane in one direction, and one lane in the other direction, would be considered as a 1-lane major street.
   - For the ADT, count total volume approaching from both directions, including all turn movements.

2. Minor Street (Lower Volume Street)

   - Include only the through, through/turn and left turn lanes in the number of approach lanes. However, in cases of where a minor street approach is just a right turn lane, code this as a lane in the worksheet. The right turn discount is applied normally as described below.
   - For the ADT, count the highest approaching volume (one direction only, do not include the ADT approaching from both directions) including some or none of the right turn volume as discussed in the following scenarios and examples:
     - **Scenario # 1 – Shared Left-Through-Right Lane**: Some of the right turns are included in the minor street approach ADT if the right turn demand is greater than 85% of the capacity of the shared lane. Use unsignalized capacity analysis to calculate the capacity of the shared lane. The right turn discount is 85% of the shared lane capacity (85% of the capacity is used because once the v/c exceeds 0.85, drivers suffer longer delay and begin to take unsafe gaps). Subtract the right-turn discount from the total right turn volume to determine the number of right turns in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

Example 12-4 Right Turn Discount for Shared Left/Through/Right Lane

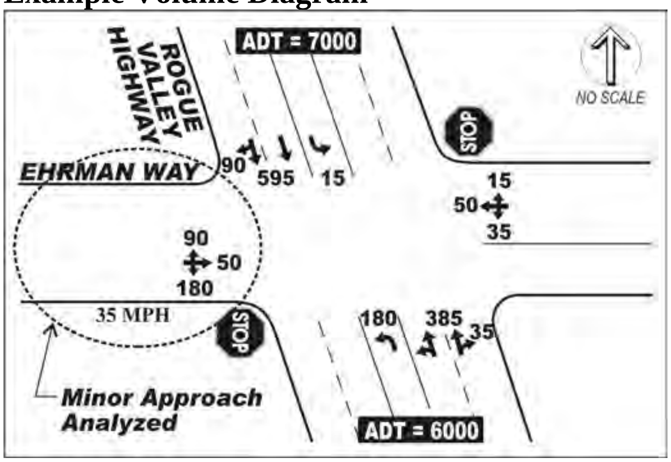
Example Application: Right Turn Discounts (Only for the minor road.)

The diagram below shows a typical unsignalized intersection, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The

85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound shared lane capacity is 120 vph. The right-turn discount is 85% of the shared lane capacity, 120 x 0.85 = 102 right turns. The number of right turns included in the warrant would be 180 – 102 = 78.
- Determine the minor approach ADT. The minor street approach peak hour volume used in the warrant is 90+50+78 = 218. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (218 / 0.10) = 2,180.

Example Volume Diagram



The figure below shows the Preliminary Signal Warrant Analysis for this example. The preliminary signal warrant is not met because the Minor Street ADT is less than the warrant volume in Condition A and the Major Street ADT is less than the warrant volume in Condition B.

Warrant Analysis of Minor Approach #1 Example Conditions

Oregon Department of Transportation
Transportation Development Branch
Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	Rogue Valley Highway	**Minor Street:**	Ehrman Way
Project:	Ehrman Way	**City/County:**	Medford
Year:	1995	**Alternative:**	Single Ln Minor Appr L/T/R

Preliminary Signal Warrant Volumes

Number of Approach lanes		ADT on major street approaching from both directions		ADT on minor street, highest approaching volume	
Major Street	Minor Street	Percent of standard warrants		Percent of standard warrants	
		100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8850	6200	2650	1850
2 or more	1	10600	7400	2650	1850
2 or more	2 or more	10600	7400	3550	2500
1	2 or more	8850	6200	3550	2500
Case B: Interruption of Continuous Traffic					
1	1	13300	9300	1350	950
2 or more	1	15900	11100	1350	950
2 or more	2 or more	15900	11100	1750	1250
1	2 or more	13300	9300	1750	1250
X	100 percent of standard warrants				
	70 percent of standard warrants[2]				

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2	10600	13000	N
	Minor	1	2650	2180	
Case B	Major	2	15900	13000	N
	Minor	1	1350	2180	
Analyst and Date:			Reviewer and Date:		

[1] Meeting preliminary signal warrants does **not** guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

- **Scenario # 2 – Exclusive Right-Turn Lane**: Some of the right turns are included in the approach ADT if the right turn lane demand is greater than 85% of the capacity of the right turn lane. Use unsignalized capacity analysis to calculate the capacity of the right turn lane. The right turn discount is 85% of the right turn lane capacity. Subtract the right turn discount from the total right turning volume to determine the number of right turns that will be included in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

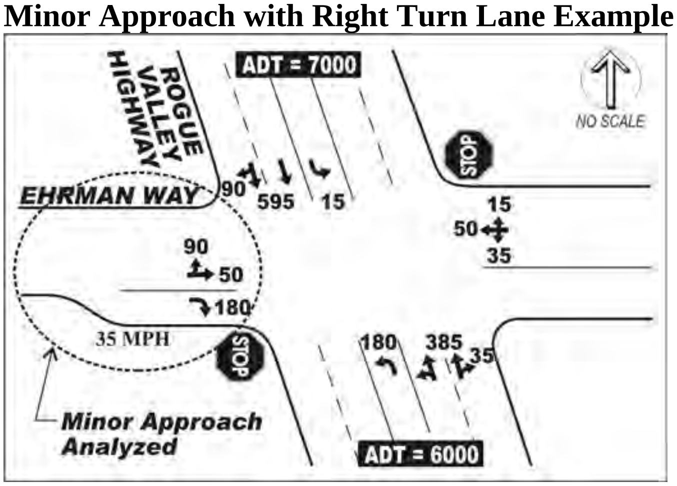
Example 12-5 Right Turn Discount for Exclusive Right Lane

The diagram below shows a typical unsignalized intersection with a separate right turn lane on the eastbound approach, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The 85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound right turn lane capacity is 639 vph. The right turn discount is 85% of the shared lane capacity, 0.85 x 639 = 543 right turns. The number of right turns included in the warrant is 180-543 = -363 = 0. If the number is less than or equal to zero, do not include any right turns in the warrant. The EB right turn lane is not included in the number of approach lanes.
- Determine the minor approach ADT. The minor approach peak hour volume used in the warrant is 90+50+0 = 140. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (140/0.10) = 1,400.

The form below shows the Preliminary Signal Warrant Analysis for this example. The preliminary signal warrant is not met since the Minor Street ADT is less than the warrant volume in Condition A and the Major Street ADT is less than the warrant volume in Condition B.

Minor Approach with Right Turn Lane Example



Warrant Analysis of Minor Approach #1 Example Conditions

Oregon Department of Transportation
Transportation Development Branch
Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	Rogue Valley Highway	**Minor Street:**	Ehrman Way
Project:	Ehrman Way	**City/County:**	Medford
Year:	1995	**Alternative:**	2 Lane Minor Approach L/T, F

Preliminary Signal Warrant Volumes

Number of Approach lanes		ADT on major street approaching from both directions		ADT on minor street, highest approaching volume	
		Percent of standard warrants		Percent of standard warrants	
Major Street	Minor Street	100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8850	6200	2650	1850
2 or more	1	10600	7400	2650	1850
2 or more	2 or more	10600	7400	3550	2500
1	2 or more	8850	6200	3550	2500
Case B: Interruption of Continuous Traffic					
1	1	13300	9300	1350	950
2 or more	1	15900	11100	1350	950
2 or more	2 or more	15900	11100	1750	1250
1	2 or more	13300	9300	1750	1250
X	100 percent of standard warrants				
	70 percent of standard warrants[2]				

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2	10600	13000	N
	Minor	1	2650	1400	
Case B	Major	2	15900	13000	N
	Minor	1	1350	1400	

Analyst and Date:	Reviewer and Date:

[1] Meeting preliminary signal warrants does **not** guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

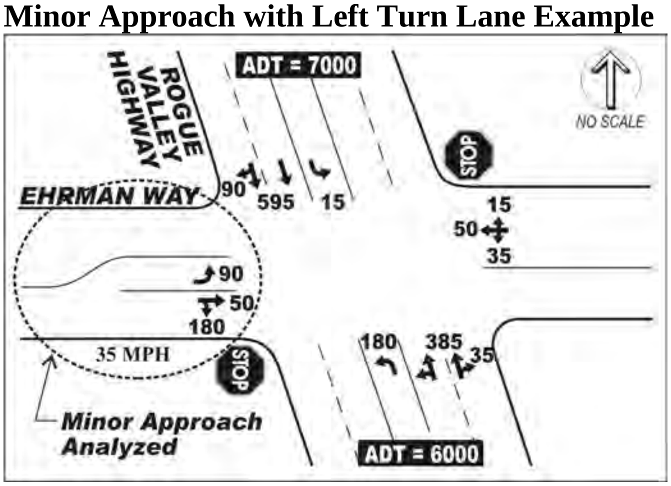
- **Scenario # 3 – Shared Through-Right Lane**: Some of the right turns are included in the approach ADT if the right turn demand is greater than 85% of the capacity of the shared through-right lane. Use unsignalized capacity analysis to calculate the capacity of the through-right shared lane. The right turn discount is 85 % of the shared lane capacity. Subtract the right turn discount from the total right turn volume to determine the number of right turns in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

Example 12-6 Right Turn Discount for Shared Through/Right Lane

The diagram below shows a typical unsignalized intersection with a shared through-right lane on the eastbound approach, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The 85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound shared lane capacity is 277 vph. The right turn discount is 85% of the shared lane capacity, 0.85 x 277 = 235 right turns. The number of right turns included in the warrant is 180 – 235= -55 = 0. If the number is less than or equal to zero, do not include any right turns in the warrant.
- Determine the minor approach ADT. The minor approach peak hour volume used in the warrant is 90+50+0= 140. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (140 / 0.10) = 1,400.
- The form below shows the Preliminary Signal Warrant Analysis for this example. The preliminary signal warrant is not met since the Minor Street ADT is less than the warrant volume in Condition A and the Major/Minor Street ADT's are both less than the warrant volumes in Condition B.

Minor Approach with Left Turn Lane Example



Warrant Analysis of Minor Approach #1 Example Conditions

Oregon Department of Transportation

Transportation Development Branch

Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	Rogue Valley Highway	**Minor Street:**	Ehrman Way
Project:	Ehrman Way	**City/County:**	Medford
Year:	1995	**Alternative:**	2 Lane Minor Approach L, T/R

Preliminary Signal Warrant Volumes

Number of Approach lanes		ADT on major street approaching from both directions		ADT on minor street, highest approaching volume	
Major Street	Minor Street	Percent of standard warrants 100	70	Percent of standard warrants 100	70
Case A: Minimum Vehicular Traffic					
1	1	8850	6200	2650	1850
2 or more	1	10600	7400	2650	1850
2 or more	2 or more	10600	7400	3550	2500
1	2 or more	8850	6200	3550	2500
Case B: Interruption of Continuous Traffic					
1	1	13300	9300	1350	950
2 or more	1	15900	11100	1350	950
2 or more	2 or more	15900	11100	1750	1250
1	2 or more	13300	9300	1750	1250
X	100 percent of standard warrants				
	70 percent of standard warrants[2]				

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2	10600	13000	N
	Minor	2	3550	1400	
Case B	Major	2	15900	13000	N
	Minor	2	1750	1400	

Analyst and Date:	Reviewer and Date:

[1] Meeting preliminary signal warrants does **not** guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

- **Scenario # 4 – Double Right-Turn Lane**: Include all of the right turning volume in the approach ADT if a double right turn lane is required. If such is the case, the number of approach lanes for warrant analysis is 2 or more.

The above information is meant to serve as general guidelines only. Engineering judgment may be required when one or both streets are one way, the intersection is not a typical four-legged design or the highest volume is associated with a turn movement. Engineering judgment must be the deciding factor in preliminary warrant analysis.

12.4.2 MUTCD Signal Warrants

PSWs are used to project signalization needs. MUTCD warrants are limited to 3 years or less and are for actual approval of installation of a traffic signal. This requires an engineering study including an evaluation of the full set of 9 MUTCD signal warrants. Provisions for this evaluation are found in the ODOT Traffic Signal Policy and Guidelines, OAR 734-020-0400 thru 734-020-0500 and ODOT Traffic Manual.

12.5 Estimating Vehicle Queue Lengths

Vehicle queues can have a significant effect on highway safety and operation. Queues that exceed the provided storage at turn lanes can block the adjacent through lanes creating a temporary reduction in capacity as well as an unexpected obstruction in the travel lane that could result in a crash. In through lanes long queues can block access to turn lanes, driveways and minor street approaches, in addition to spilling back into upstream intersections. Under these conditions there are significant losses in capacity that can quickly spread to other upstream intersections and adjacent streets. There can also be a higher potential for crashes as drivers turning onto or off of the highway are required to pass through gaps in the queue that provide limited visibility and other drivers incurring long delays become more aggressive. Therefore, the estimation of vehicle queue lengths is an important traffic analysis procedure that should be included in most operational and safety projects.

Estimates of queue lengths should be based on the anticipated arrival patterns, duration of interruptions and the ability of the intersection to recover from momentary heavy arrival rates. The average queue length and the 95th percentile queue length should be shown in the report. The 95th percentile queue length shall be used for design purposes. A queue blockage or spillback condition is considered a problem when the duration exceeds 5 percent of the peak hour. The average vehicle length, including buffer space between vehicles, to be used in analysis shall be 25-feet, unless a local study indicates otherwise, with all queue length calculations rounded up to the next 25-foot increment. Queue lengths subject to over-capacity conditions can only be adequately assessed with simulation software. The 25-foot average does not apply to microsimulation, where vehicle lengths differ by vehicle type. Refer to Chapter 15 for microsimulation guidance.

STOP *The minimum storage length for urban or rural left turn lanes at unsignalized intersections on state highways is 100 feet. Left Turn Lane layouts/dimensions are available in HDM Section 500 and Traffic Line Manual (TLM) Section 310.*

12.5.1 Two-Way Stop Control Intersection Queuing

TPAU Models

At unsignalized intersections, the movements of interest are often the major street left turns and all minor street movements. The most common methodologies used for estimating queue lengths for these movements include the Highway Capacity Software (HCS)[2] and the Two-Minute Rule.

[2] *Highway Capacity Software*, McTrans, University of Florida, Gainesville, Florida.

TPAU has conducted studies on modeling queue lengths at two-way stop controlled (TWSC) intersections[3]. The studies checked the relative performance of the two-minute rule and the HCM 2000 method. One of the conclusions was that the two-minute rule was overestimating and the HCM methodology was underestimating the queue lengths. In addition, existing methods were not found to be accurately predicting queue lengths for more than 50 percent of the cases.

Poisson regression models were developed to improve the queue length estimations. Model validation shows that the refined models are predicting queue lengths better than other methods. The HCM methodology was found to consistently underestimate the queue length. A Two-Way Stop Queue Length Calculator is available on the Planning Section website under Tools. Exhibit 12-19 summarizes the developed models, and applicable ranges of input data for each model type. When the range of independent variables exceeds the limiting value, use queue length models with caution.

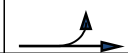
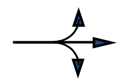
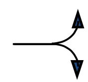
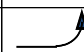
Exhibit 12-20 TPAU Two-way Stop Controlled Intersection Queue Length Models

Lane Group	Queue Length Model Equation[1]
MJL[2]	Queue Length = $e^{(0.3925+0.0059*VOL+0.00104*CONVOL+0.49*Signal-0.81*LT)}$
MNLTR[3]	Queue Length = $e^{(-0.7844+0.01636*VOL+0.0006*CONVOL-0.0000043*VOL*CONVOL)}$
MNLR[4]	Queue Length = $e^{(-0.6319+0.0173*VOL+0.00066*CONVOL-0.000007913*VOL*CONVOL)}$
MNL[5]	Queue Length = 0.95+ 0.014*VOL +0.00074*CONVOL+3.01*(VOL/CONVOL)
MNR[6]	Queue Length = 0.865+ 0.0000534*VOL*CONVOL +0.2372*(VOL/CONVOL)

[1] Use this method with caution if volumes fall outside the variable ranges shown below:
[2] MJL VOL = 0 to 300 vph; CONVOL = 0 to 2,000 vph; SIGNAL = 0 or 1; LT = 0 or 1
[3] MNLTR VOL = 0 to 300 vph; CONVOL = 0 to 3,000 vph
[4] MNLR VOL = 0 to 300 vph; CONVOL = 0 to 3,000 vph
[5] MNL VOL = 0 to 300 vph; CONVOL = 0 to 2,000 vph
[6] MNR VOL = 0 to 250 vph; CONVOL = 0 to 1,500 vph

[3] Development of Queue Length Models at Two-way STOP Controlled Intersection: A Surrogate Method (accessed on January 23, 2014)

Where:

VOL	Traffic volume on the subject approach in vehicles per hour	
CONVOL	Conflicting traffic volume in vehicles per hour	
SIGNAL	Presence of an upstream signal within ¼ mile of an intersection, applicable for major left turn only, 1 if there is a signal, otherwise 0	
LT	Presence of a separate left turn lane, applicable for major left turn only (1 if there is an exclusive left turn lane/median left turn lane/ two-way left turn lane, otherwise 0)	
MJL	Major street left turn approach	 or 
MNLTR	Minor street shared left-through-right approach	
MNLR	Minor street shared left-right approach	
MNL	Minor street exclusive left turn lane	
MNR	Minor street exclusive right turn lane	

As the HCM method was found to consistently underestimate queue lengths, and two-minute rule consistently overestimates queue lengths, neither method should be used for two-way stop control queue length estimation. Either simulation or the models in Exhibit 12-19 may be used. Example 12-7 and Example 12-8 outline the step-by-step process of queue length estimation using developed models. The queues in this methodology represent the maximum queues for the peak 15-minute period which are an acceptable approximation of the 95th percentile queue length.

This procedure estimates the number of vehicles in queue. This number is multiplied by the appropriate average vehicle storage length obtained from Exhibit 12-20 to determine the queue length.

Exhibit 12-21 Storage Length Adjustments for Trucks

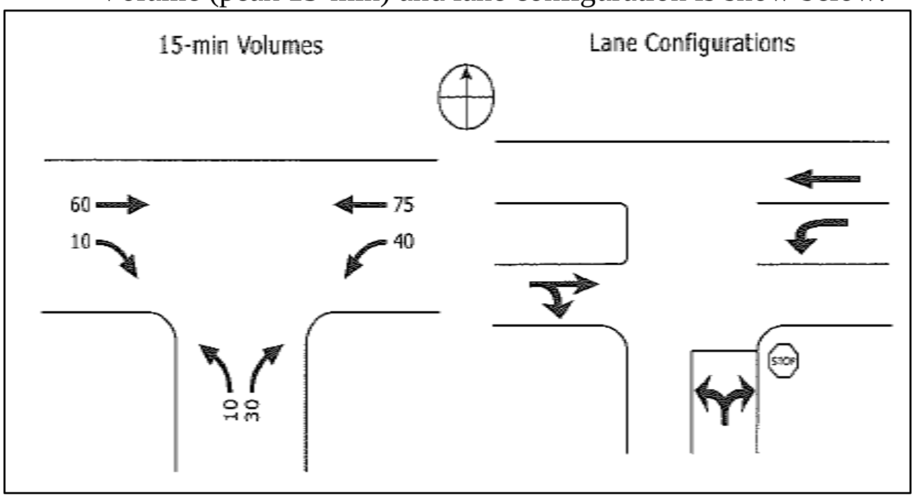
Percent Trucks in Turning Volume	Average Vehicle Storage Length
< 2%	25 ft
5%	27 ft
10%	29 ft

Example 12-7 Queue Length Estimation at a Three-legged Stop-controlled Intersection

This example demonstrates the application of the TPAU queue length estimation models at a three- legged Stop- controlled intersection.
(Source: Example Problem 1 of HCM 6 Chapter 32)

<u>Data</u>

- Volume (peak 15-min) and lane configuration is show below:



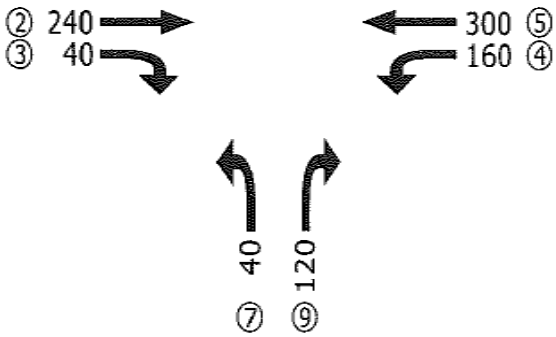
- Level grade on all approaches;
- Percent of heavy vehicles on all approaches is 10 percent;
- No flared approaches;
- No upstream signal;
- No pedestrians;
- Length of analysis is 1 hour (This example in HCM 6 uses 0.25 h);

Step1: Choose Lane Groups to Apply the Queue Length Models

- MNLR: North bound (minor approach)
- MJL: West bound TWLT lane

Steps 2 and 3: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities According HCM 6, Chapter 20 Methodology

Peak 15-min volume is multiplied by 4 to get the flow rate. Movement numbers are circled.



Step 4: Compute Conflicting Flow Rates (CONVOL) as per HCM 6 Equations 20-4 through 20-29

<u>WB MJL</u>

$V_{c,MJL} = V_{c,4}$

$V_{c,4} = V_2 + V_3$

$V_{c,4} = 240+40 = 280$ veh/h

<u>NB MNLR</u>

$V_{c,MNLR} = v_{c,7} + v_{c,9}$

$V_{c,7} = v_2 + 0.5v_3 + 2v_4 + v_5$

$V_{c,7} = 240 + 0.5(40) + 2(160) + 300 = 880$ veh/h

$V_{c,9} = v_2 + 0.5v_3$

$V_{c,9} = 240 + 0.5(40) = 260$ veh/h

$V_{c,MNLR} = 880+260 = 1140$ veh/h

Step 5: Compute Queue Lengths using Models

<u>WB MJL</u>

VOL = 160 veh/h is within the range (0, 300]

CONVOL = 280 veh/h is within the range (0, 2000]

SIGNAL = 0 ; LT = 1

$QL = e^{(0.3925 + 0.0059 * VOL + 0.00104 * CONVOL + 0.49 * Signal - 0.81 * LT)}$

$QL = e^{(0.3925 + 0.0059 * 160 + 0.00104 * 280 + 0.49 * 0 - 0.81 * 1)}$

QL = 2.3 ≈ 3 vehicles

<u>NB MNLR</u>

VOL = 160 veh/h is within the range (0, 300]

CONVOL = 1140 veh/h is within the range (0, 3000]

$QL = e^{(-0.6319+0.0173*VOL+0.00066*CONVOL-0.000007913* VOL* CONVOL)}$

$QL = e^{(-0.6319 + 0.0173*160 + 0.00066*1140 -0.000007913* 160* 1140)}$

QL = 4.2 ≈ 5 vehicles

Summary
Maximum QL for WB LT = 3 Vehicles
Maximum QL for NB approach = 5 Vehicles
Estimates from other queue length models are presented below:

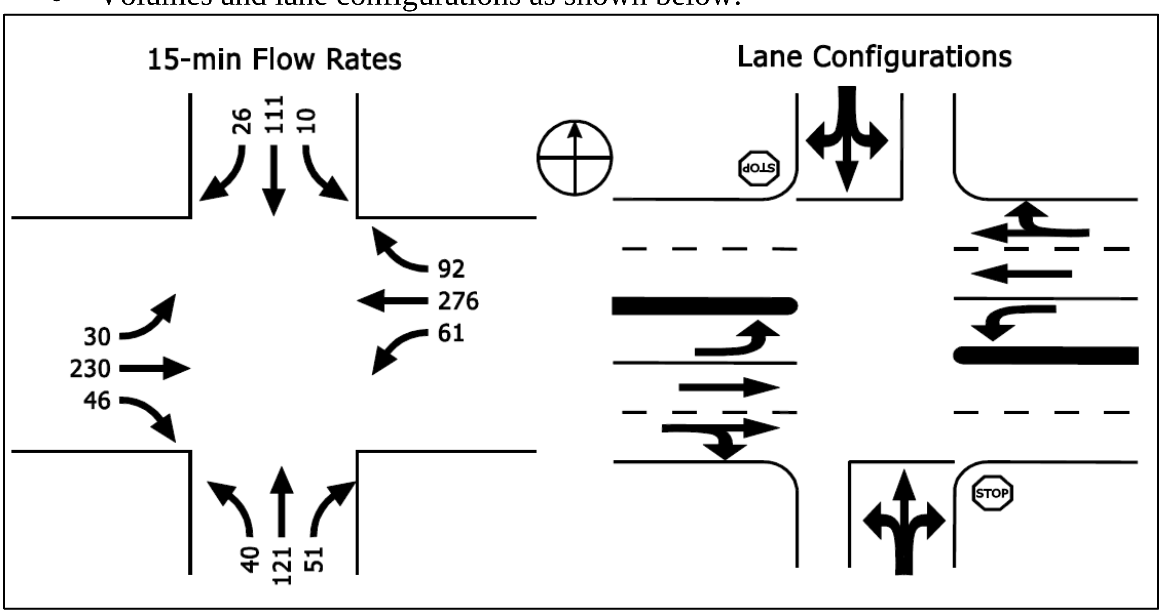
Method	Queue Length for WB LT (veh)	Queue Length for NB (veh)
HCM	2	1
Two-minute Rule	10	10
QL Model	3	5

Based on the percentage of trucks in the traffic stream, queue lengths (number of queued vehicles) from models are converted to feet using Exhibit 12-20.
From the above heavy vehicle percentage conversion table, the queue lengths for:
WB LT = 3 x 29 ft. = 87 ft. ≈ 100 ft.
NB approach = 5 x 29 ft. =145 ft. ≈ 150 ft.

Example 12-8 Queue Length Estimation at a Four-legged Two-Way Stop-controlled Intersection

This example demonstrates the application of the TPAU queue length estimation models at a four-legged two-way STOP controlled intersection.
(Source: Example Problem 3 of 2010 HCM Chapter 32, Page 32-7)
Data

- Volumes and lane configurations as shown below:



- Major street with two lanes in each direction, minor street with one lane on each approach that flares with storage for one vehicle in the flare area, and median storage for two vehicles at one time available for minor-street through and left-turn movements;
- Level grade on all approaches;

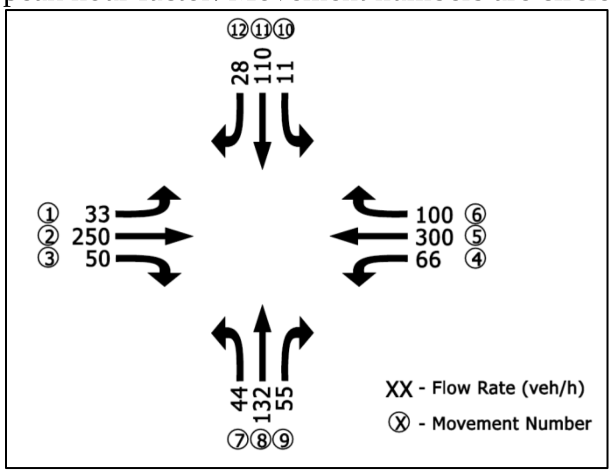
- Percent heavy vehicles on all approaches = 10%;
- Peak hour factor on all approaches = 0.92;
- Length of analysis period = 1.0 h.

Step1: Choose Lane Groups to Apply the Queue Length Models

- MNLTR – NB and SB
- MJL – EB and WB

Steps 2 and 3: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities According HCM 6, Chapter 20 Methodology

For each movement, peak hour flow rate is obtained by dividing the hourly volume by the peak hour factor. Movement numbers are circled.



Step 4: Compute Conflicting Flow Rates (CONVOL) as per HCM 6 Equations 19-4 through 19-29

<u>EB MJL</u>

$v_{c,1} = v_5 + v_6 + v_{16}$ = 300 + 100 + 0 = 400 veh/h

<u>WB MJL</u>

$v_{c,4} = v_2 + v_3 + v_{15}$ = 250 + 50 + 0 = 300 veh/h

<u>NB MNLTR</u>

$v_{c,NB} = v_{c,7} + v_{c,8} + v_{c,9}$

$v_{c,7} = v_{c,I,7} + v_{c,II,7}$ (two-stage gap acceptance)

$v_{c,I,7} = 2(v_1 + v_{1u}) + v_2 + 0.5v_3 + v_{15}$

= 2(33+0) + 250 + 0.5(50) + 0 = 341 veh/h

$v_{c,II,7} = 2(v_4 + v_{4u}) + 0.5v_5 + 0.5v_{11} + v_{13}$

= 2(66+0) + 0.5(300) + 0.5(110) + 0 = 337 veh/h

$v_{c,7} = v_{c,I,7} + v_{c,II,7} =$ 341 + 337 = 678 veh/h

$v_{c,8} = v_{c,I,8} + v_{c,II,8}$ (two-stage gap acceptance)

$v_{c,I,8} = 2(v_1 + v_{1u}) + v_2 + 0.5v_3 + v_{15}$

= 2(33+0) + 250 + 0.5(50) + 0 = 341 veh/h

$v_{c,II,8} = 2(v_4 + v_{4u}) + v_5 + v_6 + v_{16}$

= 2(66+0) + 300 + 100 + 0 = 532 veh/h

$v_{c,8} = v_{c,I,8} + v_{c,II,8} =$ 341 + 532 = 873 veh/h

$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{4U} + v_{14} + v_{15}$

$= 0.5(250) + 0.5(50) + 0 + 0 + 0 = 150$ veh/h
$v_{c,NB} = v_{c,7} + v_{c,8} + v_{c,9} = 678 + 873 + 150 = 1701$ veh/h
<u>SB MNLTR</u>
$v_{c,SB} = v_{c,10} + v_{c,11} + v_{c,12}$
$v_{c,10} = v_{c,I,10} + v_{c,II,10}$ (two-stage gap acceptance)
$v_{c,I,10} = 2(v_4 + v_{4u}) + v_5 + 0.5v_6 + v_{16}$
$= 2(66+0) + 300 + 0.5(100) + 0 = 482$ veh/h
$v_{c,II,10} = 2(v_1 + v_{1u}) + 0.5v_2 + 0.5v_8 + v_{14}$
$= 2(33+0) + 0.5(250) + 0.5(132) + 0 = 257$ veh/h
$v_{c,10} = v_{c,I,10} + v_{c,II,10} = 482 + 257 = 739$ veh/h
$v_{c,11} = v_{c,I,11} + v_{c,II,11}$ (two-stage gap acceptance)
$v_{c,I,11} = 2(v_4 + v_{4u}) + v_5 + 0.5v_6 + v_{16}$
$= 2(66+0) + 300 + 0.5(100) + 0 = 482$ veh/h
$v_{c,II,11} = 2(v_1 + v_{1u}) + v_2 + v_3 + v_{15}$
$= 2(33+0) + 250 + 50 + 0 = 366$ veh/h
$v_{c,11} = v_{c,I,11} + v_{c,II,11} = 482 + 366 = 848$ veh/h
$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{1U} + v_{13} + v_{16}$
$= 0.5(300) + 0.5(100) + 0 + 0 + 0 = 200$ veh/h
$v_{c,SB} = v_{c,10} + v_{c,11} + v_{c,12} = 739 + 848 + 200 = 1787$ veh/h

Step 5: Compute Queue Lengths using Models
<u>EB MJL</u>
VOL = 33 veh/h is within the range (0, 300]
CONVOL = 400 veh/h is within the range (0, 2000]
SIGNAL = 0 ; LT = 1
$QL = e^{(0.3925 + 0.0059 * VOL + 0.00104 * CONVOL + 0.49 * Signal - 0.81 * LT)}$
$QL = e^{(0.3925 + 0.0059 * 33 + 0.00104 * 400 + 0.49 * 0 - 0.81 * 1)}$
$QL = 1.2 \approx 2$ veh

<u>WB MJL</u>
VOL = 66 veh/h is within the range (0, 300]
CONVOL = 300 veh/h is within the range (0, 2000]
SIGNAL = 0 ; LT = 1
$QL = e^{(0.3925 + 0.0059 * VOL + 0.00104 * CONVOL + 0.49 * Signal - 0.81 * LT)}$
$QL = e^{(0.3925 + 0.0059 * 66 + 0.00104 * 300 + 0.49 * 0 - 0.81 * 1)}$
$QL = 1.3 \approx 2$ veh

<u>NB MNLTR</u>
VOL = 231 veh/h is within the range (0, 300]
CONVOL = 1701 veh/h is within the range (0, 3000]
$QL = e^{(-0.7844 + 0.01636 * VOL + 0.0006 * CONVOL - 0.0000043 * VOL * CONVOL)}$
$QL = e^{(-0.7844 + 0.01636 * 231 + 0.0006 * 1701 - 0.0000043 * 231 * 1701)}$
$QL = 10.2 \approx 11$ veh

SB MNLTR
VOL = 149 veh/h is within the range (0, 300]
CONVOL = 1787 veh/h is within the range (0, 3000]
$QL = e^{(-0.7844 + 0.01636 * VOL + 0.0006 * CONVOL - 0.0000043 * VOL * CONVOL)}$
$QL = e^{(-0.7844 + 0.01636 * 149 + 0.0006 * 1787 - 0.0000043 * 149 * 1787)}$
QL = 4.8 ≈ 5 veh

Summary
Maximum QL for
EB LT = 2 veh
WB LT = 2 veh
NB approach = 11 Veh
SB approach = 5 Veh
Estimates from other queue length models are presented below:

Method	EB LT (Veh)	WB LT (Veh)	NB (Veh)	SB (Veh)
HCM	0	0	6	3
Two-minute Rule	2	4	15	10
QL Model	2	2	11	5

Based on the percentage of trucks in the traffic stream, queue lengths (number of queued vehicles) from models are converted to feet using Exhibit 12-20.
From the above heavy vehicle percentage conversion table, the queue lengths for:
EB LT = 2 x 29ft. = 58 ft. ≈ 75 ft.
WB LT ≈ 75 ft.
NB approach = 11 x 29 ft. = 319 ft. ≈ 325 ft.
SB approach = 5 x 29ft. = 145 ft. ≈ 150 ft.

Simulation

If simulation is being performed as part of the analysis, queue lengths should be taken from the simulation results. If simulation is not being done, it should be considered especially if the v/c ratios are approaching 0.90. If the effort to do a simulation analysis is not desired, the TPAU queue length estimation models should be used. Refer to Chapter 15 for simulation procedures.

Two-Minute Rule

The Two-Minute Rule is a rule of thumb methodology that shall only be used for sketch planning level analysis or for lane groups not addressed in the TPAU method. This method estimates queue lengths for major street left turns and minor street movements by using the queue that would result from a two-minute stoppage of the turning demand volume. This method does not consider the magnitudes and impacts of the conflicting flows on the size of the queue. The calculation of the 95^{th} percentile queue using the two-minute rule methodology shall use the following equation:

S = (v) (t) (L)

where:
S = the 95th percentile queue storage length (feet)
v = the average left-turn volume arriving in a 2-minute interval
t = a variable representing the ability to store all vehicles; usually 1.75 to 2.0 (See Exhibit 12-21)
L = average length of the vehicles being stored and the gap between vehicles; 25 ft. for cars. This value can be increased where a significant number of trucks are present in the turning volume using the same relationship between average vehicle storage length and percent trucks in turning volumes shown for the signalized movement rule of thumb method discussed earlier in this chapter.

Exhibit 12-22 Selection of "t" Values

Minimum "t" Value	Percentile
2.0	98 %
1.85	95 %
1.75	90 %
1.0	50 %

Appendix 12A/13A – Software and Settings for Intersection Analysis

References

(i) Robinson, Bruce W., et al. *Roundabouts: An Informational Guide*. No. FHWA-RD-00-067. 2000.

13 SIGNALIZED INTERSECTION ANALYSIS

13.1 Purpose

This chapter presents commonly used signalized intersection deterministic analysis procedures and identifies specific methodologies and input parameters to be used on ODOT projects. Simulation procedures are covered in APM Chapter 15. Software settings are covered in Appendix 12/13. Topics covered include:

- Turn Lanes at Signalized Intersections
- Auxiliary Through Lanes
- Signalized Intersection Capacity Analysis
- Signal Progression Analysis
- Estimating Queue Lengths at Signalized Intersections

> *The scope of this chapter is limited to auto mode analysis at signalized intersections. A complete evaluation of signalized intersections requires a broader evaluation including of non-auto modes. Refer to APM Chapter 10 for modal considerations such as for left and right turn lanes, and to Chapter 14 for multimodal analysis procedures such as MMLOS. The need for other evaluations such as per the Traffic Manual and HDM should be coordinated with Region Roadway/Traffic or Traffic Engineering Section.*

13.2 Criteria for Turn Lanes at Signalized Intersections

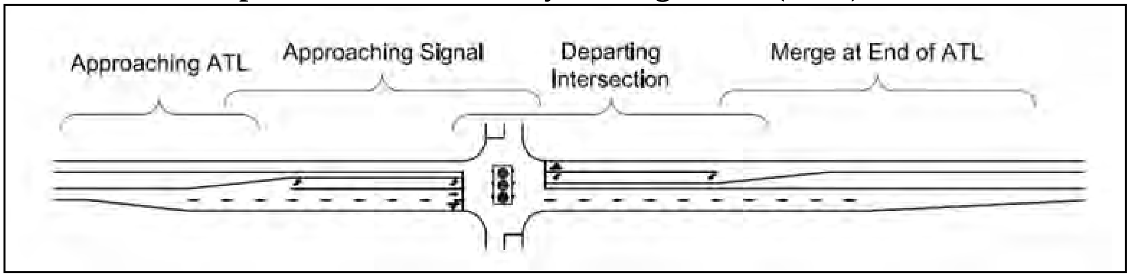
Turn lanes at signalized intersections are determined differently than at unsignalized intersections. At signalized intersections a left turn lane is always desirable, while a right turn lane is generally determined based on signal capacity needs. At signalized intersections, installation of turn lanes must be consistent with the requirements in ODOT's Traffic Signal Policy and Guidelines and the Traffic Manual and approval must be received.

13.3 Auxiliary Through Lanes at Signalized Intersections

The following procedure is intended for the analysis of existing ATLs only. Installation of an Auxiliary Through Lane (ATL) on a state highway is generally not allowed. ODOT has reviewed NCHRP Report 707 Guidelines on the Use of Auxiliary Through Lanes at Signalized Intersections. While the document provides discussion about the use of auxiliary through lanes and creates a potential process to follow when considering installing an auxiliary through lane, it was found the research and analysis was neither comprehensive nor definitive enough to fully support the recommendations. Therefore, the installation of an auxiliary through lane on the state highway system will require approval from the State Traffic Engineer and will be considered on a case-bv-case basis.

An Auxiliary Through Lane (ATL) is a limited length through lane added midblock upstream and downstream of a signalized intersection (Exhibit 13- 1). Configurations different than shown in the exhibit, including when accesses are present, are not considered ATLs and are add/drop lane areas instead, which are not covered in this section. Typically, the ATL form has been used as a way to meet an operational standard with future street widening deferred to a later date. ATLs are more commonly found on local rather than state facilities.

Exhibit 13- 1 Components of an Auxiliary Through Lane (ATL)



13.3.1 ATL Issues

There are several issues regarding ATLs and transit, access points, pedestrians and bicycles and other conditions. Overall, ATLs are discouraged and in some cases should be reconfigured. A few of the issues are identified below:

- Access points within ATLs can be both safety and operational concerns.
- Pedestrian crossing distance and time is longer at an ATL, which can lead to longer exposure, cycle times and increased delay.
- Transit stop locations can be a problem within ATLs. Without a transit pull-out, the presence of a bus will reduce ATL utilization.
- Some ATLs may be used as a passing lane, causing a safety concern in the speed differential between the two lanes during congested hours.

13.3.2 ATL Analysis

As noted above, in the review of NCHRP Report 707, ODOT does not fully support recommendations regarding the NCHRP 707 (1) procedure for estimating the lane utilization, taper length, or prescribed length of an ATL.

For analysis of an existing ATL, the lane utilization should be measured in the field if possible. If the lane utilization cannot be measured, assume a lane utilization of 15% for a shared ATL with one continuous through lane, 12% if two continuous through lanes exist. Add 3% for an exclusive right turn lane.

To analyze the adequacy of the taper or ATL length, such as for performing microsimulation, the analyst needs to work with the designer to determine what lengths should be used. This may be an iterative process where the analyst runs the analysis with several different lengths to determine the impact of length on the analysis results.

13.4 Signalized Intersection Analysis

Signalized intersection control can generally be classified into three categories; pre-timed, semi-actuated and fully actuated operations. A pre-timed signal has the cycle length, phases, green times and change phases all preset to be constant for every cycle. A semi-actuated signal operates by designating a “main street” that is served until actuation from the “side street” occurs. Under this type of operation, the cycle length and green times may vary based on vehicle demand. ODOT has effectively upgraded all formerly semi-actuated intersections to fully actuated. A fully actuated signal allows detection on all legs and phases of the intersection and cycle lengths and green times are determined based on the demand for each movement.

In addition to the type of signal operating, each signalized intersection has characteristics associated with it related to how the timing of a signal is allocated over a cycle. These characteristics relate to phases, intervals, change intervals, green time, lost time, yellow and all-red clearance times and effective green time. All these characteristics can be part of signalized operations and can affect the overall intersection operations. For more information on characteristics of signals and signal operations analysis, refer to Chapter 19 of the HCM.

13.4.1 Saturation Flow Rates

As previously discussed in Chapter 3, saturation flow rates are critical components in the analysis of signalized intersection capacity and can be defined as the flow in vehicles per hour that can be accommodated by a lane group assuming that the green phase is displayed 100 percent of the time. Saturation flow rates can be measured in the field or calculated by applying adjustment factors to a default “ideal” saturation flow rate. For more information regarding the calculation and application of saturation flow rates, refer to Chapter 3.

Chapter 31 of the HCM 7th Edition provides adjusted saturation flow rates for through movements, along with saturation flow adjustment factors for protected and permitted left turns, that reflect the presence of connected and automated vehicles (CAVs) in the traffic stream. CAVs offer the potential to increase the saturation flow rate by being able to cooperatively form platoons that have shorter headways between platooned vehicles than human-driven vehicles can achieve safely. These shorter headways allow more vehicles to enter an intersection per hour of green time, increasing the capacity of through and protected left movements. In addition, they can result in longer gaps in opposing traffic that can be used by permitted left-turn movements. Both effects can result in higher movement capacities, particularly at higher percentages (>60–80%) of CAVs in the traffic stream. Appendix 6B provides guidance on estimating saturation flow rates for use in longer-range planning analyses testing the potential effects of CAVs on signalized intersection and arterial capacity.

As of 2022, no vehicles were available commercially that met the definition of a CAV for the purposes of the capacity adjustments provided for signalized intersection analyses in the HCM (i.e., a vehicle with an operating cooperative adaptive cruise control system that is capable of communicating with other vehicles and driving without human intervention in any situation). The saturation flow rate adjustments presented in Appendix 6B are intended for use only in longer-range planning analyses. That appendix also provides guidance on estimating the percentage of CAVs in the traffic stream in a future year and example problems.

Because CAVs are not yet commercially available, saturation flow rate adjustments for CAVs should not be made in near-term analyses such as traffic impact studies.

13.4.2 Right Turn on Red (RTOR)

Oregon law permits a right-turn movement by a vehicle facing a circular red or a red arrow indication after stopping and yielding to pedestrians and any conflicting vehicles, unless posted otherwise. For future conditions, an engineering study should be performed to evaluate appropriate traffic control options such as RTOR prohibition for safety reasons – contact Region Traffic for guidance. Warrants for turn prohibitions are found in OAR 734-020-0020. Additional guidance is found in MUTCD Section 2B.54. Region Traffic Engineer/Manager approval is required for No Turn On Red signs. The remainder of this section assumes that it has been determined that RTOR will not be prohibited.

For existing conditions, the HCM advises that counts may be used to obtain the RTOR volume, which is then subtracted from the total right turn volume in the analysis. However, it is often not practical to obtain RTOR counts.

For future conditions or where RTOR counts are not obtained, the HCM does not provide a methodology to estimate right turn on red (RTOR) volume. The following options for analysis can be considered.

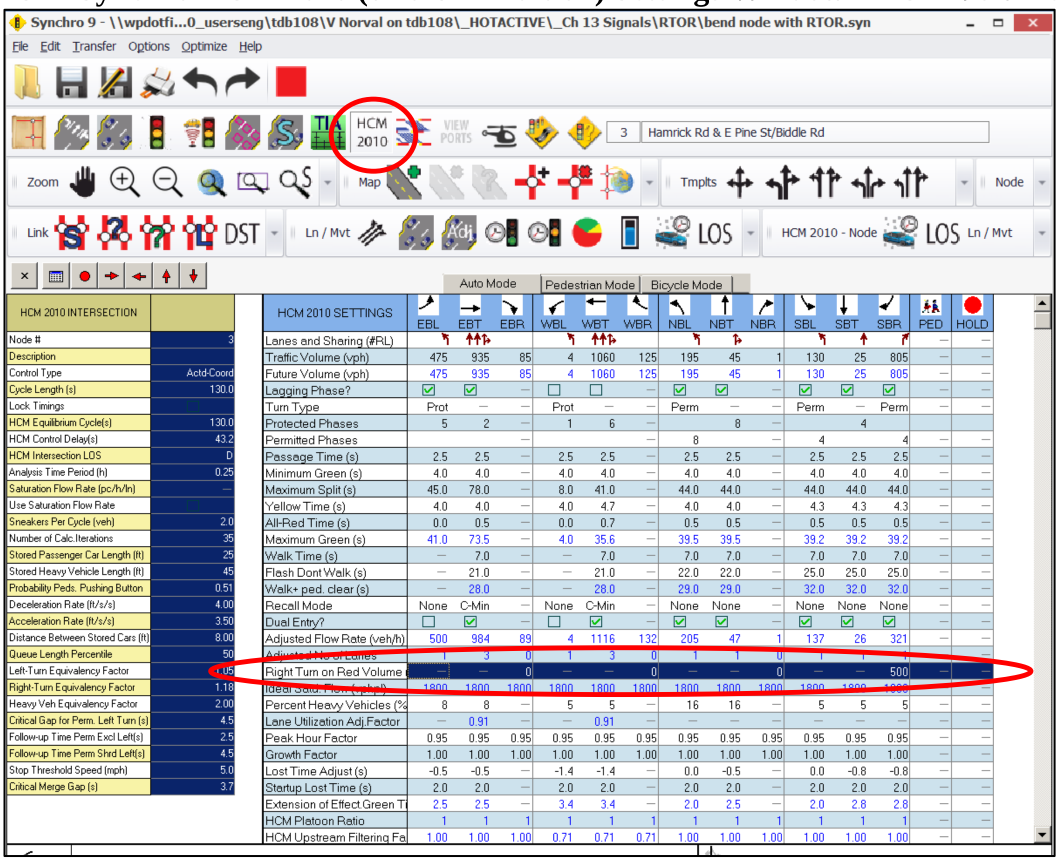
1. No reduction for RTOR – The HCM recommends not applying a reduction for RTOR for future conditions. This provides a conservative result. If v/c ratio and queuing are not an issue, no further RTOR analysis may be deemed necessary; however for simulation

RTOR should be accounted for. The operational benefit of RTOR is a function of the volume of right turns, volume of conflicting traffic and signal timing/phasing.

2. Synchro –RTOR can be enabled by checking the RTOR box in the Lane settings window. In this method, a saturated flow rate for RTOR is calculated. The right turn on red saturated flow rate (sRTOR) is the potential volume if the signal was red 100% of the time. In order to reflect RTOR in the HCM (2010 or later) report, a RTOR volume must be entered in the HCM 2010/6th/7th settings window as shown in Exhibit 13- 2 below. The Synchro estimated RTOR volume (vRTOR) can be obtained from this equation:

   vRTOR = sRTOR * r/C, where r/C is the red to cycle ratio

Exhibit 13- 2 Synchro HCM 2010 (or 6th/7th Edition) Settings Window RTOR Volume



3. SIDRA – Check the Turn on Red Checkbox to identify approaches where RTOR is allowed. SIDRA will then internally calculate and apply the RTOR volume reduction similar to Synchro.

4. Vistro – In Vistro the analyst can select the approaches where RTOR is allowed either using a global setting or by approach. However, Vistro does not calculate the RTOR volume so it must be input manually. First, assume no RTOR and determine if there is a v/c ratio or queuing problem. If no v/c ratio or queuing problem is found, no further analysis is necessary. If a v/c ratio or queuing problem is found assuming no RTOR, options for estimating RTOR in Vistro include using Synchro to obtain the RTOR volume, or one of the following steps.

5. Planning level method for shared through/right lanes - can be used for estimating RTOR volume in Vistro (2). This method addresses only vehicle conflicts with the RTOR movement. It does not address pedestrian conflicts or bicycles in the traffic stream. A significant volume of pedestrians may warrant posting of no right turn on red signage.

   The method estimates RTOR volume using the following model (3).

$$N_{RTOR} = \min(X_r, 1.0) \times \left(\frac{1-p}{p}\right) \times \frac{3600}{C}$$

   where

   N_{RTOR} = expected number of RTORs for the analysis period (veh)
   Xr = demand volume-to-capacity ratio for the shared lane subject approach
   p = proportion of through vehicles shared lane for the analysis period (veh/h)
   C = average cycle length (s) during the analysis period.

   The RTOR volume is deducted from the total right turn volume.

6. Planning method for exclusive right turn lanes – Assumes 50% of right turn volumes turn right on red, unless high pedestrian traffic or sight distance constraints are present, in which case assume 30% RTOR (4).

7. Wisconsin method for exclusive right turn lanes – Applies a reduction factor to the total right turn volume as follows[1]:
   - Single Right-Turn Lanes at Intersections: 0.62
   - Single Right-Turn Lanes at Interchanges: 0.34
   - Dual Right-Turn Lanes (Intersections and Interchanges): 0.70

Example 13- 1 Planning Level RTOR Method for Shared Through/Right Lane

A two-lane approach has one through lane and one shared through/right lane. The through volume is 760 vph and the right turn volume is 250 vph. The v/c ratio for the shared lane group is 0.97. The cycle length is 100 sec.

Xr = 0.97

Total lane group volume = 760 + 250 = 1010 vph

Assuming balanced lane volumes, the volume in each lane is 1010/2 = 505 vph

Shared lane through volume = 505 – 250 = 255 vph

p = proportion of through vehicles in shared lane = 255/505 = 0.50

[1] https://wisconsindot.gov/dtsdManuals/traffic-ops/manuals-and-standards/teops/16-15.pdf

$$N_{RTOR} = \min(0.97, 1.0) \times \left(\frac{1-0.50}{0.50}\right) \times \frac{3600}{100} = 35 \text{ vph}$$

Therefore, for this example the RTOR volume can be estimated as 35 vph.

13.4.3 Critical Movement Analysis

The critical movement analysis method is a planning-level tool to estimate capacity of a signalized intersection with existing or forecasted volumes. It is for estimation only; not to report final v/c ratios or compare to mobility targets. The analysis requires intersection approach volumes, number of lanes, and lane assignments per approach.

Each movement pair in conflict (e.g. westbound left and eastbound through) are added for a total volume. Identify the highest total (or critical movement pair) for each roadway. If available, use lane utilization for duplicate lane assignments on an approach. If lane utilization data does not exist, then use an even distribution. The critical movement pairs for each roadway are summed and compared with the thresholds shown in .

Exhibit 13- 3 Intersection Performance Assessment by Critical Volume

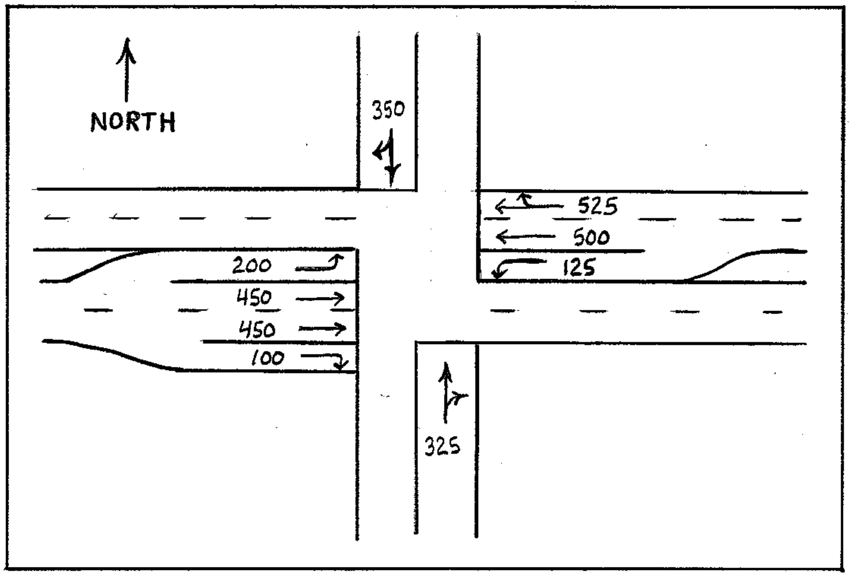
Sum of Critical Volumes (Vehicles/Hour/Lane)	Performance
0 to 1,200	Under Capacity
1,201 to 1,400	Near Capacity
1,401 and Above	Over Capacity

Critical movement analysis only estimates an intersection's capacity. It does not estimate vehicle delay, level of service or vehicle queue lengths.

Example 13- 2 Critical Movement Analysis

The Critical Movement figure shows the signalized intersection of a five-lane highway with a two-lane cross street. For this intersection, conduct critical movement analysis.

Critical Movement Analysis Example



Solution:

For the east-west roadway, the conflict pairs include:

- 200 (EB LT) + 525 (WB TH/RT) = 725
- 200 (EB LT) + 500 (WB TH) = 700
- 125 (WB LT) + 450 (highest EB TH) = 575
- 125 (WB LT) + 100 (EB RT) = 225

The highest conflict pair is EB LT and WB TH/RT. Therefore, the critical movement volume for the east-west roadway is 725 vehicles.

For the north-south roadway, the conflict pairs include:

- 350 (SB TH/RT) = 350
- 325 (NB TH/RT) = 325

For these approaches there are no conflicting movements, thus the highest total approach volume is the north-south critical movement, 350 vehicles. The sum of the critical movement volumes for the intersection:

725 (east-west) + 350 (north-south) = 1,075

Compared to the thresholds shown in , this intersection is estimated to operate under capacity.

13.4.4 Critical Intersection v/c Ratio

For signalized intersections, the reported v/c ratio is based on the critical intersection v/c ratio, not the movement or approach v/c ratio. The OHP refers to the intersection v/c ratio (another name for the critical v/c ratio) in Action 1F.1. However, many software programs (e.g., Synchro, SIDRA, etc.) just show the highest approach or movement v/c rather than the needed intersection or critical v/c ratio and require a separate calculation. The critical intersection v/c ratio is also known as Xc in the HCM. It involves summing the flow ratios of the critical movements. This value is not generally affected by the approach green times (except in cases with shared left turns). See the HCM equation below (note that the critical intersection equation from any edition of the HCM is acceptable).

Critical Intersection Volume to Capacity Ratio (for signalized intersections)

$$X_C = \left(\frac{C}{C-L}\right)\sum_{i\in ci} y_{c,i}$$

With

$$L = \sum_{i\in ci} l_{t,i}$$

Where:

X_C = critical intersection volume to capacity ratio
C = cycle length (sec)
$y_{c,i}$ = critical flow ratio for phase $i = \frac{v_i}{(Ns_i)}$
$L_{t,i}$ = phase i lost time $= l_{1,i} + l_{2,i}$ (sec)
ci = set of critical phases on the critical path
L = cycle lost time (sec)
v_i = lane group flow rate for phase i
N = number of lanes for lane group i
s_i = lane group saturation flow rate for phase i

It is important to highlight that ***lane group*** flow rates (v_i) and saturation flow rates (s_i) used for the critical intersection v/c calculation (not movement groups). This may be somewhat confusing as these two designations are very similar. In general, a separate ***lane group*** is created for:

a. each lane (or group of adjacent lanes) that serve one movement, and
b. each lane served by two or more movements.

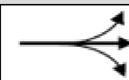
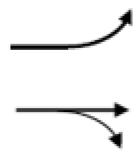
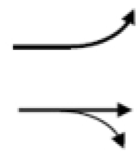
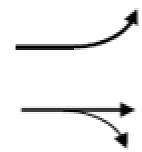
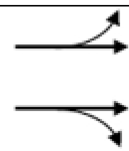
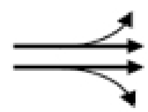
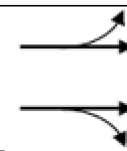
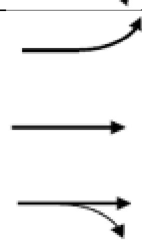
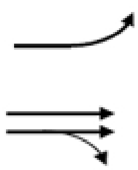
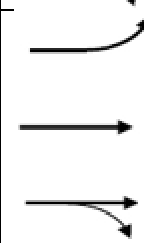
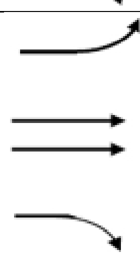
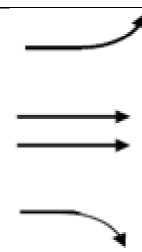
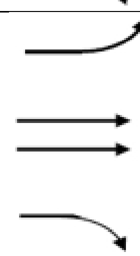
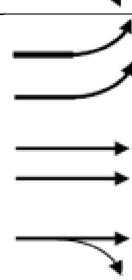
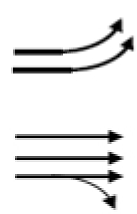
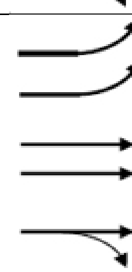
Keep in mind that ultimately lane groups are what is used in the calculation. Movement groups can be considered as a support to the lane group methodology. For example, when there is a shared lane and the through and turn movements must be identified separately (e.g., in the case of permitted turns).

In general, a separate ***movement group*** is created for:

a. each turn movement with one or more exclusive lanes, and
b. the through movement (including any turn movements that share a lane with the through movement).

The difference between the two groups occurs when a shared lane is on an approach with two or more lanes. Some guidance will be provided here but also reference HCM 7 Chapter 19 for detailed discussion. Exhibit 13- 4 shown below provides examples of a variety of typical movement groups and lane groups.

Exhibit 13- 4 Typical Lane Groups

Number of Lanes	**Movements by Lanes**		**Movement Groups (MG)**		**Lane Groups (LG)**	
1	Left, through, and right:		MG 1:		LG 1:	
2	Exclusive left: Through and right:		MG 1: MG 2:		LG 1: LG 2:	
2	Left and through: Through and right:		MG 1:		LG 1: LG 2:	
3	Exclusive left: Through: Through and right:		MG 1: MG 2:		LG 1: LG2: LG 3:	
4	Exclusive Left: Through: Through: Exclusive right:		MG 1: MG 2: MG 3:		LG 1: LG2: LG 3:	
5	Exclusive left: Exclusive Left: Through: Through: Through and right:		MG 1: MG 2:		LG 1: LG2: LG 3:	

Notice that a lane group may include one or more lanes. Use the following rules to determine the lane groups for an intersection approach:

- An exclusive left-turn lane or lanes should be designated as a separate lane group. The same is true of an exclusive right-turn lane.
- Any shared lane should be designated as a separate lane group.
- Any lanes that are not exclusive turn lanes or shared lanes should be combined into one lane group.

A movement group may contain one or more lanes and/or more than one lane moving in more than one direction. Use the following rules to determine the movement groups for an intersection approach:

- A turn movement is served by one or more exclusive lanes, and no shared lanes should be designated as a movement group.
- Any lanes not assigned to a group by the previous rule should be combined into one movement group.

The lane group concept is important to help identify the appropriate flow rate and saturation flow rate from the software reports to use in the flow ratio calculations. If the approach has no shared lanes or has only one lane, the lane group and movement group will both have the same flow rates. An analyst will need to pull the flow rate for each lane and then appropriately combine them for each individual lane group. It is these lane group values that will then be used to calculate the flow ratios used in the critical intersection v/c calculation.

Numerous issues have been noted with the identification of intersection critical movements. Generally, there has been an over-reliance on the critical movement(s) identified by the Synchro and SIDRA reports. The key to identifying critical movements is to use phasing and flow ratios. It is critical to methodically sketch out the identified phasing. Skipping steps and moving directly to an Excel spreadsheet will lead to problems. Once the critical movements have been identified using the highest v/s flow ratios dependent upon the phasing type, the critical intersection v/c ratio can be calculated using the HCM equation.

Phasing and Flow Ratios

The flow ratio calculation depends on summing the lane group flow ratio for each phase.

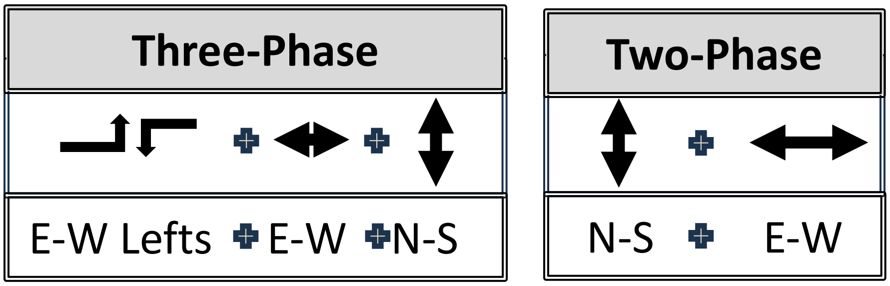
$$Flow\ Ratio = \frac{v}{s}$$

Where:

$$v = \text{volume}$$
$$s = \text{saturated flow}$$

There will be a v/s term for each phase in the total calculation. For example, if there are two phases, then there will be two v/s components; if there are three phases, there will be three v/s components, etc. This is represented graphically in Exhibit 13- 5 below.

Exhibit 13- 5 Number of Phases and Number of Flow Ratios



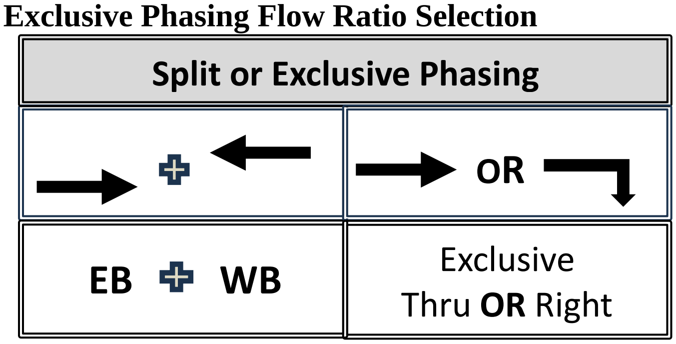
Calculating the v/s ratio for all lane groups will help indicate critical movements/pairs and controlling movements (i.e. exclusive through vs exclusive right) and will be needed in future analysis. Generally, for the critical v/c calculation the highest lane-group v/s will be used for each phase and for each barrier pair the highest combined phase v/s in ring one or ring two will be used. However, each phasing type has a nuanced approach which will be described below.

*It is important to review and sketch out the phase rotation. Also, calculate the v/s for all **lane groups**. Phase rotation may be available on timing sheets. If not, ask Region Traffic or a local contact. Actual phase rotation may not be clearly apparent in the field.*

Split or Exclusive Phasing

For split or exclusive phasing each approach should be treated separately. In this scenario to calculate the critical pair, check whether the through or the right turn movement controls by noting which one has the highest v/s ratio. Either the through or the right will be used in the calculation as depicted in Exhibit 13- 6.

Exhibit 13- 6 Split or Exclusive Phasing Flow Ratio Selection



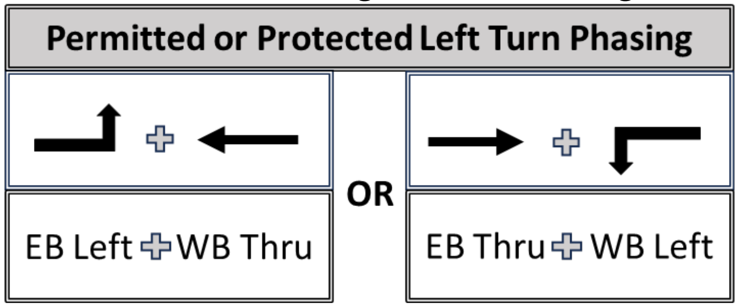
Permitted or Protected Left Turn Phasing

Left turn phasing may be either permitted or protected. For permitted or protected left turn phasing, the phase pair[2] (left and through) with the highest v/s should be used as shown in Exhibit 13- 7. Protected left turn phasing may be in lead-lag (common when one left turn

[2] *Phase pairs are within the same ring and barrier that cannot run concurrently (e.g EB left turn and WB thru)*

movement is significantly higher than the other and is used to optimize timings), lead-lead (most common), or lag-lag patterns.

Exhibit 13- 7 Permitted or Protected Lead-Lag Left Turn Phasing Flow Ratio Selection



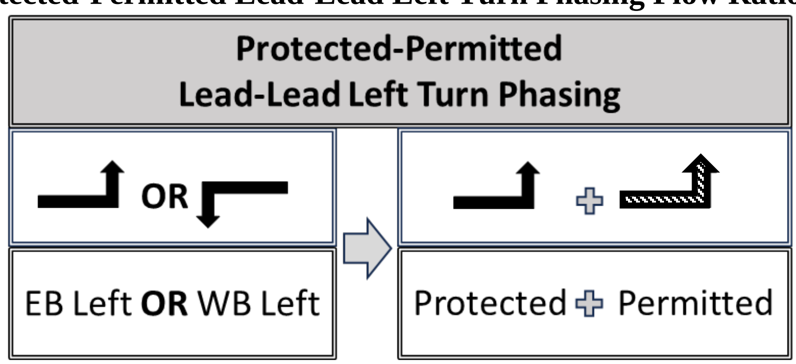
When the left turn phasing is protected, it is important to identify where lead-lag phasing is in use as the automatic Synchro/SIDRA reports may flag multiple left or through movements. Generally, these extra movements are not likely used in the overall v/c calculation.

Protected-Permitted Lead-Lead or Lag-Lag Left Turn Phasing

A more complex situation arises when there is protected-permitted left turn (P+PLT) phasing. In this case the calculation will also depend on the lead-lead/lag-lag or lead-lag configurations. For this configuration, four rules define possible critical paths through a phase sequence. These rules utilize Exhibit 13- 6 through Exhibit 13- 9 and are summarized in Exhibit 13- 10.

When there is P+PLT phasing with a lead-lead or lag-lag configuration, the v/s of protected lefts and permitted lefts in a direction should be summed and compared as show in Exhibit 13-8 below. Drawing a ring and barrier diagram to illustrate leading or lagging protected left and its protected left, see Ring 1 or 2 of the first Barrier in Example 13- 3. Note that there is only one unit of lost time for this direction.

Exhibit 13- 8 Protected-Permitted Lead-Lead Left Turn Phasing Flow Ratio Selection



When there is P+PLT phasing with a lead-lag configuration, the v/s needs to be summed from the leading and lagging left and the highest, controlling, permitted left as shown in Exhibit 13- 9. First, sum the v/s from both protected left leading and lagging phases. Second, identify which through phase has the highest permitted left turn v/s ratio. Lastly, add the permitted left v/s that corresponds to the identified highest v/s through phase (i.e. moves in the same direction) from the second step to the protected leading and lagging v/s sum from the first step. Make sure that the through phase v/s is not accidentally used. It is important to note that there are two units of lost time used for this direction (from the leading and lagging left turn portions).

Exhibit 13- 9 Protected-Permitted Lead-Lag Left Turn Phasing Flow Ratio Selection

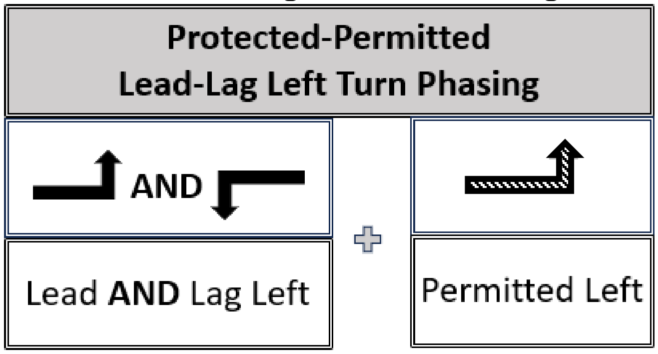


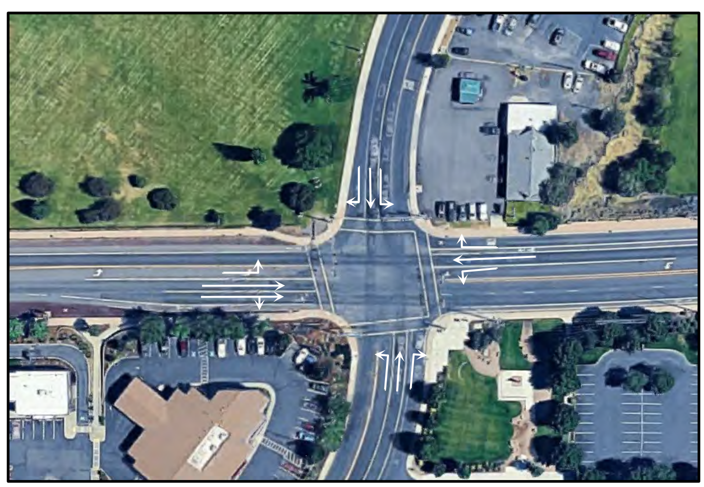
Exhibit 13- 10 summarizes what phase combinations and their applicable flow rates should be considered when selecting a critical path. The phase combination with the highest flow ratio are the critical phases, see Example 13- 3.

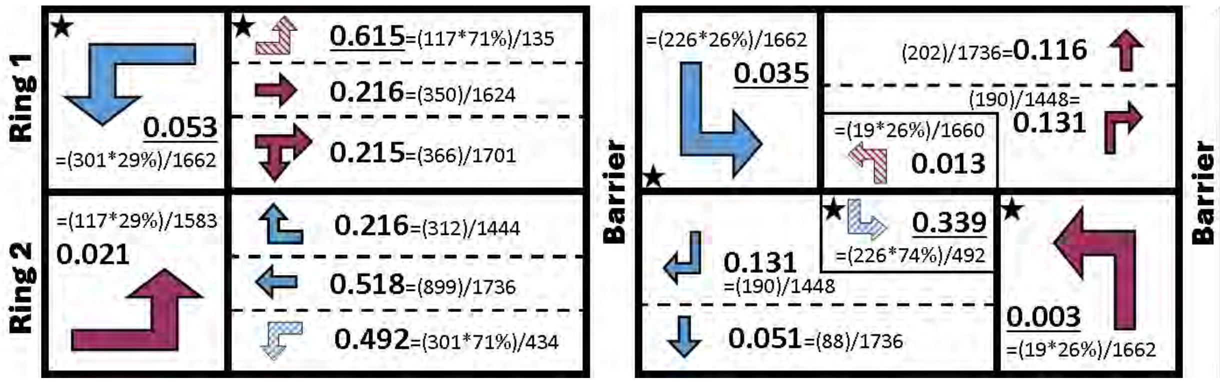
Exhibit 13- 10 Summary for Critical Path Selection

Rule	Applicable Scenarios	Phases for Flow Ratio Calculation	Exhibit/s
1	All	Phases associated in Ring 1 phase sequence	Exhibit 13- 6 and Exhibit 13- 7
2	All	Phases associated with Ring 2 phase sequence	
3	P+PLT (Lead/Lead or Lag/Lag)	Protected Left (Max Total Left Turn A/B) + Permitted Left (Max Total Left Turn A/B)	Exhibit 13- 8
4	P+PLT Lefts (Lead/Lag)	Protected Left (A) + Max Permitted Left (A/B) + Protected Left (B)	Exhibit 13- 9

Example 13- 3 Determining Critical Phases and Cycle Flow Ratio

This example evaluates the urban 4-way signalized intersection of OR216 and SW Rimrock Way in Redmond, OR that is pictured to the right. Immediately north of this intersection is a high school. Three of the four approaches have an exclusive left turn, through, and right turn lane. The west approach has an exclusive left turn and through lane, and a shared through-right lane. Both the E-W and N-S phase pairs have protected-permitted lefts, lead-lead and lead-lag respectively.





★=*Critical Phase*

The phase diagram above represents this intersection and includes each phase's flow ratio and its calculation. Flow ratios for each phase are calculated by dividing a lane group's volume by saturation flow of the analyzed period, typically peak hour. Note that the P+PLT volumes for each direction are proportions of a total left turn volume in a direction. For this example, the proportions are based on "green time" for the movement as show in the table below.

WB Left Movement	Total Vol.	Green Time	Total Time (sec)	LT Ratio	Adj. Volume	Sat. Flow	Flow Ratio
Protected	301	20 sec	70 *(=20+50)*	~29% *(=20/70)*	87 *(=301*29%)*	1662	0.053 *(=87/1662)*
Permitted		50 sec		~71% *(=50/70)*	214 *(=301*71%)*	434	0.492 *(=214/434)*

Next, phase pair flow ratios are summed according to the four rules in Exhibit 13- 10 as tabled below.

Rule #	1st Barrier: E-W	2nd Barrier: N-S
1 (Ring 1)	**<u>0.668</u> = 0.053+0.615** *> (0.053+ 0.216)* *> (0.053+0.215)*	**0.166 = 0.035+0.131** > (0.035+*0.116)*
2 (Ring 2)	**0.539 = 0.021+0.518** *> (0.021+0.492)* *> (0.021+0.216)*	**0.134 = 0.131+0.003** *> (0.051+0.003)*
3 P+PLT (Lead/Lead or Lag/Lag)	**0.636 = (0.021+0.615)** > (0.053+0.492)	n/a
4 P+PLT (Lead/Lag)	n/a	**<u>0.377</u> = (0.035+0.339+0.003)** > (0.035+0.013+0.003

The maximum flow ratio for each barrier is identified (underlined in the table above). The phases associated with the maximum flows are critical phases.

Finally, the cycle flow ratio is the summation critical phases flow ratios (**1.045** = 0.668+0.377)

Calculation Methods

There is a critical intersection v/c calculation tool available on the Technical Tools webpage under the Signalized Intersections dropdown box to assist with Synchro and SIDRA-based analyses. The critical intersection v/c can also be calculated with available intersection analysis tools such as Vistro, Synchro and SIDRA.

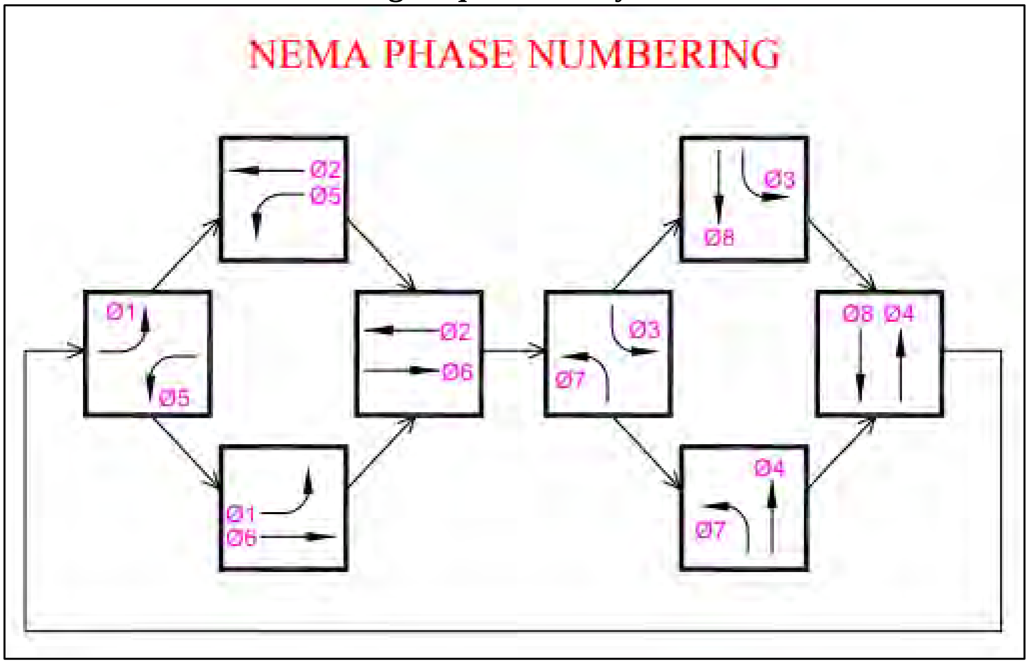
Vistro

The critical intersection v/c ratio is calculated automatically and reported out. Not requiring separate calculation of this value is one of the advantages of using Vistro, particularly where several intersections and/or alternatives are involved. Example 13- 4 illustrates the critical v/c calculation in Vistro.

Synchro

The critical intersection v/c ratio is not provided and must be post-processed. To accomplish this, the most up to date HCM edition report should be used. The HCM 2010 and later methodologies in Synchro expect strict application of NEMA (National Electrical Manufacturers Association) phasing (see Exhibit 13- 11). NEMA phasing requires each phase to have a unique phase number, even if they run at the same time. Odd numbers are used to designate the protected left turn phases while even numbers are used to designate the through phases. A maximum of four left turn phases and four through phases can be designated using NEMA phasing. Numbers 1 and 5 are used to designate the main street left turn phases while Numbers 2 and 6 are used to designate the main street through phases. Phases 1 and 6 are located on the same approach while Phases 2 and 5 are located on the opposite approach. Numbers 3 and 7 are used to designate the cross street left turn phases while Numbers 4 and 8 are used to designate the cross-street through phases. Phases 3 and 8 are located on the same approach while Phases 4 and 7 are located on the opposite approach. Note that for split phasing the through movement phase number needs to be used to allow the Synchro reports to print out.

Exhibit 13- 11 NEMA Phasing Required for Synchro



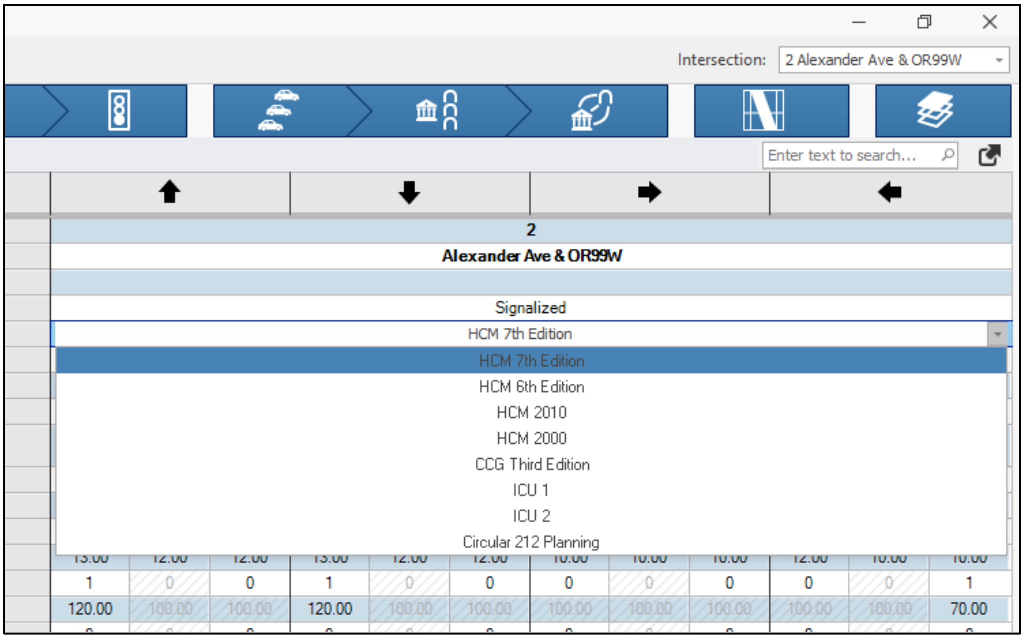
Depending on the signal phasing and the intersection geometry, it may be necessary to create a "workaround". This is described in detail below. Then according to the signal type, using the guidance provided in the discussion above the critical flow ratios can be calculated manually and the critical movements may be identified. The critical intersection v/c ratio can then be calculated using the HCM equation. The procedure to post-process Synchro HCM 2010 and later output is illustrated in Example 13- 5 and Example 13- 6.

<u>SIDRA</u>

The critical intersection v/c ratio is not provided and must be post-processed. Flow ratios can be calculated using report outputs and critical movements identified. The signal type guidance provided in the discussion above will need to be used to identify critical flow ratios. Then the critical v/c can be calculated using the HCM equation. The procedure to post-process critical intersection v/c ratio from SIDRA output is illustrated in Example 13- 7.

Example 13- 4 Calculating Critical Intersections v/c Ratio in Vistro

As stated above, Vistro directly provides the signalized intersection critical v/c ratio. Note that several different analysis methods may be chosen and HCM 7th Edition should be used as it is the most current. On the north and south legs of the intersection lead-lag left turn phasing is coded in this example (northbound lead; southbound lag) for consistency with the Synchro and Sidra examples. However, for optimization purposes both orientations should be checked (southbound lead; northbound lag) because one may perform better than the other.



After the intersection geometry, volume, signalization, and other pertinent data have been entered into Vistro, the v/c can be displayed by clicking on the “Show Intersection Traffic Conditions” button on the left side of the screen. The figure below shows the delay, LOS, and critical v/c for the example intersection.

Synchro is the only tool that requires two examples to explain the critical v/c calculation. As stated previously, the calculation should be performed using output from the most current HCM Report to be applying the most up-to-date methodology. However, the Synchro HCM 2010 and later methodologies have some limitations in that fully compliant NEMA phasing is required. In practice this means it does not support custom phasing schemes or a protected-permitted left turn from a shared lane; a left turn bay is required. In some project analyses there will be intersections requiring analysis that do not match these criteria. In this situation a "workaround" will be required to calculate the critical v/c. Example 13- 5 will analyze the intersection with geometry consistent with the Vistro and SIDRA examples, uses split phasing, and incorporates the necessary "workaround." Example 13- 6 adds a left turn bay and demonstrates a more straightforward use of the HCM (2010 and later) Report.

Example 13- 5 Calculating Critical Intersection v/c Ratio in Synchro with NEMA Phasing Adjustments

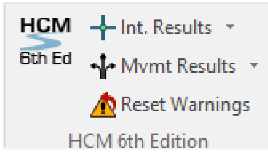
This example assumes that the signalized intersection has protected left turn lead-lag signal phasing on the north and south approaches and split phasing on the east and west approaches. However, it is recommended to only use the HCM 6th /7th Report to identify the lane group flow rates and the saturation flow rates and this report expects NEMA phasing and does not tolerate split-phasing left turns. NEMA phasing requires a unique phase number for each of the four left turn and four through movements.

If the phasing shown below for the east and westbound directions below are used, the *"HCM 6th (or 7th) Edition methodology expects strict NEMA phasing"* error message will be displayed when viewing the HCM 6th /7th Edition Report.

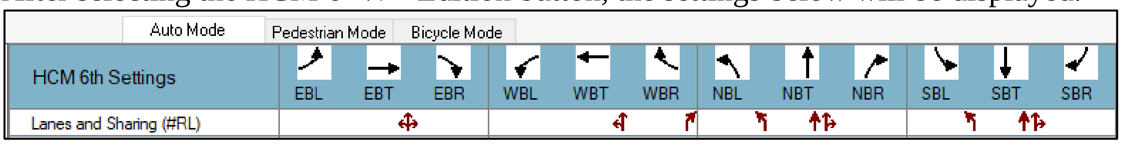
TIMING SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)												
Traffic Volume (vph)	10	2	2	25	1	65	3	1275	60	140	1625	5
Future Volume (vph)	10	2	2	25	1	65	3	1275	60	140	1625	5
Turn Type	Split	—	—	Split	—	Prot	Prot	—	—	Prot	—	—
Protected Phases	3	3	—	4	4	4	1	6	—	5	2	—
Permitted Phases			—						—			—
Permitted Flashing Yellow	—	—	—	—	—	—	—	—	—	—	—	—
Detector Phases	3	3	—	4	4	4	1	6	—	5	2	—
Switch Phase	0	0	—	0	0	0	0	0	—	0	0	—
Leading Detector (ft)	—	78	—	—	78	78	78	183	—	78	183	—
Trailing Detector (ft)	—	2	—	—	2	2	2	177	—	2	177	—
Minimum Initial (s)	3.0	3.0	—	5.0	5.0	5.0	3.0	10.0	—	3.0	10.0	—
Minimum Split (s)	32.5	32.5	—	33.5	33.5	33.5	13.0	27.0	—	13.0	26.0	—
Total Split (s)	32.5	32.5	—	33.5	33.5	33.5	13.0	37.0	—	13.0	37.0	—
Yellow Time (s)	4.0	4.0	—	4.0	4.0	4.0	3.5	4.0	—	3.5	4.0	—
All-Red Time (s)	0.5	0.5	—	0.5	0.5	0.5	0.5	1.0	—	0.5	1.0	—
Lost Time Adjust (s)	—	0.0	—	—	0.0	0.0	0.0	0.0	—	0.0	0.0	—
Lagging Phase?	☑	☑	—	☐	☐	☐	☐	☐	—	☑	☑	—
Allow Lead/Lag Optimize?	☑	☑	—	☑	☑	☑	☑	☑	—	☑	☑	—

The following steps will adjust the existing phasing to NEMA phasing:

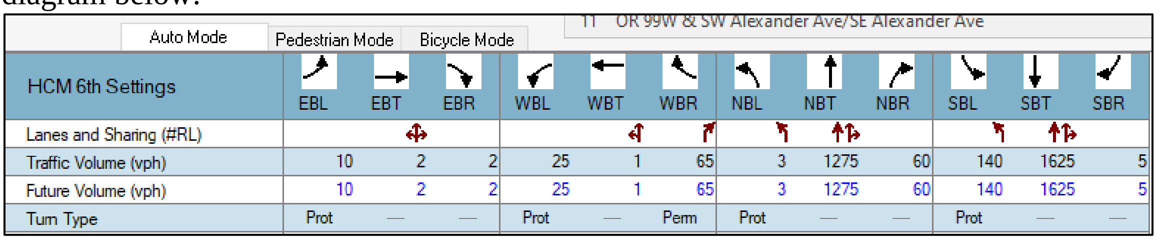
1. From the map view, click on the desired intersection and select the HCM 6th /7th Edition Settings button on the Home tab.



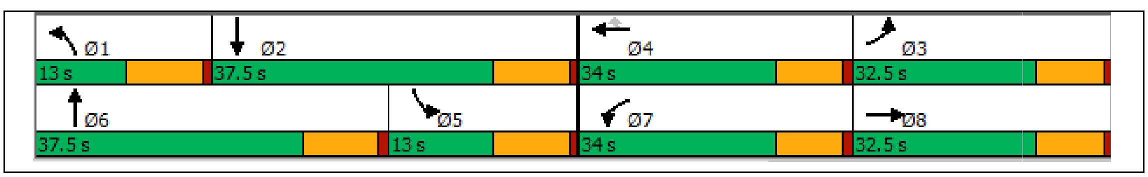
2. After selecting the HCM 6th /7th Edition button, the settings below will be displayed.



3. In the field, the example intersection has a shared eastbound left-through-right and a shared westbound through-left. This does not meet the NEMA phasing requirements. The solution is to assign a unique phase to all left and through movements even if they run concurrently. So, the amended eastbound and westbound phasing will look like the diagram below:



HCM 6th Settings	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)												
Traffic Volume (vph)	10	2	2	25	1	65	3	1275	60	140	1625	5
Future Volume (vph)	10	2	2	25	1	65	3	1275	60	140	1625	5
Turn Type	Prot	—	—	Prot	—	Perm	Prot	—	—	Prot	—	—



Note that in some instances there is a much more straightforward fix to meet the NEMA phasing requirement (although there will be times that custom phasing schemes will require the workaround described). In this specific situation a quicker fix is to change the label on Phase 3 to Phase 8 to align with the phasing shown in Exhibit 13- 11. For split phasing use the through movement phase number to allow the Synchro reports to print out. ***The results produced are exactly the same.***

It is important to note that the v/s calculation is based on "Lane Group" values and not exclusive "Lane" values. This becomes an important distinction when there are multiple lane groups in a movement. This occurs when shared through-turn lanes are present. This is seen in the Synchro report shown below for the northbound through and the southbound through lane group movements (circled in red).

The through movement includes two lane groups:

1. Through-right lane
2. Through lane

In the case of multiple lane groups in a movement, the “Group Volume” in vehicles per hour (“Grp Volume(v), veh/h” row in report) should be used for the v/s calculation. The higher lane group v/s for the movement between the through and the through-right should be used for the calculation. This is where the Synchro report provides the required lane group value.

HCM 6th Signalized Intersection Summary
11: OR 99W & SW Alexander Ave/SE Alexander Ave 08/24/2023

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	10	2	2	25	1	65	3	1275	60	140	1625	5
Future Volume (veh/h)	10	2	2	25	1	65	3	1275	60	140	1625	5
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.94	1.00		1.00	1.00		0.99	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1900	1900	1900	1900	1900	1900	1976	1826	1841	1945	1856	1900
Adj Flow Rate, veh/h	24 +	5 +	5	35 +	1	92	3	1401	66	156	1806	6
Peak Hour Factor	0.41	0.41	0.41	0.71	0.71	0.71	0.91	0.91	0.91	0.90	0.90	0.90
Percent Heavy Veh, %	0	0	0	0	0	0	0	5	4	2	3	0
Cap, veh/h	0	67	67	0	151	127	21	2038	96	217	2569	9
Arrive On Green	0.00	0.08	0.08	0.00	0.08	0.08	0.01	0.59	0.58	0.12	0.69	0.69
Sat Flow, veh/h	0	844	844	0	1900	1605	1882	3459	163	1853	3696	12
Grp Volume(v), veh/h	0	0	10	0	1	92	3	738	729	156	906	906
Grp Sat Flow(s),veh/h/ln	0	0	1688	0	1900	1605	1882	1826	1795	1853	1856	1853
Q Serve(g_s), s	0.0	0.0	0.3	0.0	0.0	3.1	0.1	15.6	15.7	4.5	16.3	16.3
Cycle Q Clear(g_c), s	0.0	0.0	0.3	0.0	0.0	3.1	0.1	15.6	15.7	4.5	16.3	16.3
Prop In Lane	0.00		0.50	0.00		1.00	1.00		0.09	1.00		0.01
Lane Grp Cap(c), veh/h	0	0	134	0	151	127	21	1076	1058	217	1289	1288
V/C Ratio(X)	0.00	0.00	0.07	0.00	0.01	0.72	0.14	0.69	0.69	0.72	0.70	0.70
Avail Cap(c_a), veh/h	0	0	859	0	1662	1404	302	1076	1058	298	1289	1288
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.00	0.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	0.0	0.0	23.9	0.0	23.7	25.2	27.4	7.9	8.0	23.8	5.1	5.1
Incr Delay (d2), s/veh	0.0	0.0	0.2	0.0	0.0	5.6	2.2	3.6	3.7	4.1	3.2	3.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.0	0.0	0.1	0.0	0.0	1.3	0.0	5.1	5.1	2.1	4.0	4.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	0.0	0.0	24.1	0.0	23.8	30.8	29.6	11.5	11.6	28.0	8.3	8.3
LnGrp LOS	A	A	C	A	C	C	C	B	B	C	A	A
Approach Vol, veh/h		10			93			1470			1968	
Approach Delay, s/veh		24.1			30.7			11.6			9.9	
Approach LOS		C			C			B			A	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	4.6	42.9	0.0	8.4	10.6	37.0	0.0	8.4
Change Period (Y+Rc), s	4.5	4.5	4.0	4.0	4.5	4.5	4.0	4.0
Max Green Setting (Gmax), s	8.5	32.5	9.0	49.0	8.5	32.5	29.5	28.5
Max Q Clear Time (g_c+I1), s	2.1	18.3	0.0	5.1	6.5	17.7	0.0	2.3
Green Ext Time (p_c), s	0.0	13.6	0.0	0.4	0.1	13.1	0.0	0.0

Intersection Summary		
HCM 6th Ctrl Delay		11.2
HCM 6th LOS		B

Notes
User approved changes to right turn type.

Next, the flow ratios need to be calculated for all movements. The guidance at the beginning of this section should be used according to the phasing type to determine the correct combination of highest flow ratios. In this example there is protected lead-lag left turn phasing on the northbound and southbound approach (follow Exhibit 13- 7). Also recall that because there are four phases, four v/s flow ratios will be used. As described previously, the movements with the highest v/s flow ratio should be used; either

1. northbound-left or southbound left, and
2. northbound through or southbound through, or
3. northbound thought-right or southbound through-right.

In the northbound direction, the northbound shared through-right lane group has the higher v/s over the through lane group. In the southbound direction, the southbound shared through-right lane group has the higher v/s over the southbound through lane group. The table below can be created to help with this determination using the values shown in the Synchro report above.

Road Name	Approach	Lane Group	Lane Group Flow (vph)	Saturation Flow Rate (vph)	v/s Flow Ratio	Use?
OR 99W	NB	Left	3	1882	0.002	x
		Through	738	1826	0.404	
		Shared Thru-Right	729	1795	0.406	
OR 99W	SB	Left	156	1853	0.084	
		Through	906	1856	0.488	
		Shared Thru-Right	906	1853	0.489	x

For the through movements the two-lane groups in each direction were compared to identify the higher v/s (through or shared through-right). In this example the shared through-right lane group was higher in both directions. Then the higher of these two values was used in the calculation as shown in bold below.

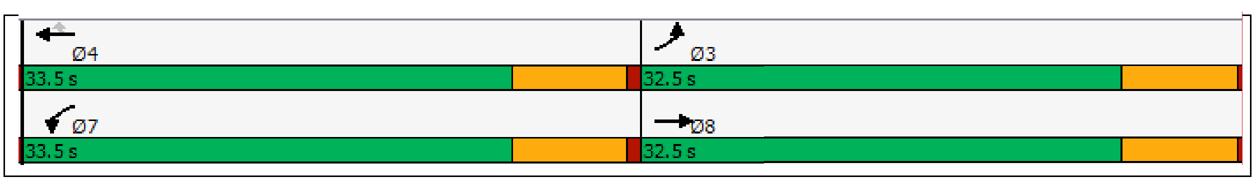
For protected lead-lag phasing from Exhibit 13- 7. Only Rules 1 and 2 of Exhibit 13- 10 are applicable and should be compared.

Rule 1) NBL + SBTR = 0.002 + 0.489 = 0.491
Rule 2) SBL + NBTR = 0.084 + 0.406 = 0.490

The NBL+ SBTR path flow ratio should be used in the calculation (0.491>0.490).

Next the eastbound and westbound movements need to be evaluated. Here there is split phasing. Following the guidance presented above, there are three steps:

1. Each through approach is separate (eastbound + westbound).
2. Check for controlling movements between through and right lane groups.



3. Flow ratios (v/s) are calculated for each movement group by dividing the adjusted flow rate by the saturated flow rate as depicted in the table below.

Road Name	Approach	Lane Group	Lane Group Flow (vph)	Saturation Flow Rate (vph)	v/s Flow Ratio	Use?
Alexander	EB	Shared Left-Thru-Right	34	1688	0.020	x
Alexander	WB	Shared Left-Thru	36	1900	0.019	
		Right	92	1605	0.057	x

There is only one flow ratio for the eastbound movement, so by default, this one is used. For the westbound movement the highest flow ratio between the through and the right movements is chosen which is for the right turn. (0.057>0.019)

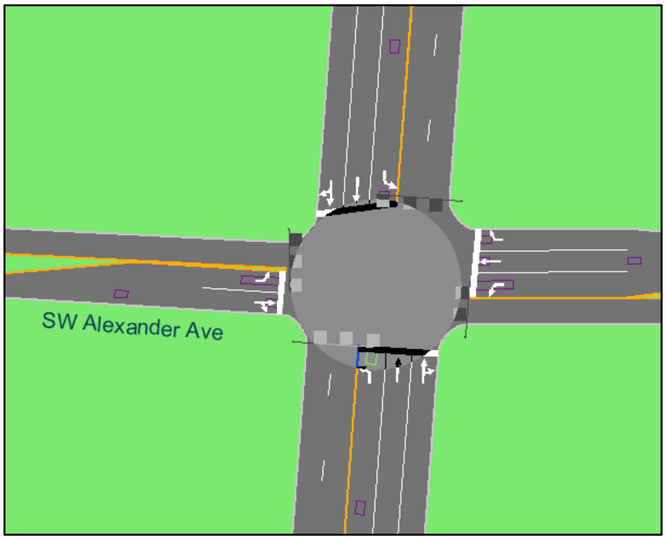
The sum of all critical movement flow ratios is then calculated: 0.002 + 0.489 + 0.020 + 0.057 = 0.568

Cycle length = 116 sec
Lost time per phase = 4 sec
Total Lost time = 16 sec

The critical intersection v/c ratio is then calculated using the HCM equation:
Xc = Sum of critical flow ratios * C/(C-L) = 0.568 * 116/ (116-16) = **0.66**

Example 13- 6 Calculating Critical Intersection v/c Ratio in Synchro

This example uses the same intersection as the previous example but adds eastbound left and westbound left turn bays as shown in the diagram below. Therefore, no modifications to the phasing are necessary to meet the NEMA phasing requirements unlike Example 13- 5.

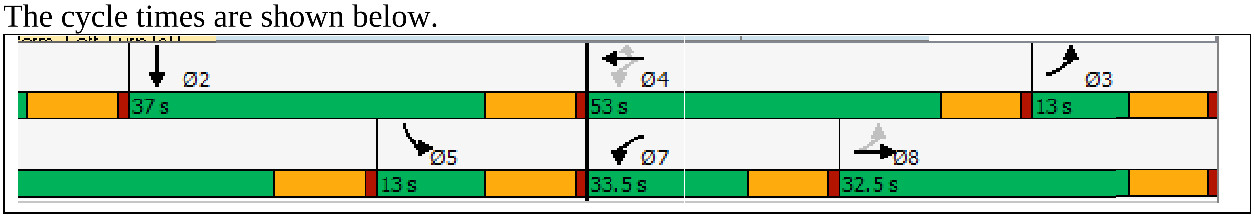


The eastbound and westbound protected-permitted left turn phasing can be coded directly as shown below.

Auto Mode | Pedestrian Mode | Bicycle Mode

HCM 6th Settings	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)												
Traffic Volume (vph)	10	2	2	25	1	65	3	1275	60	140	1625	5
Future Volume (vph)	10	2	2	25	1	65	3	1275	60	140	1625	5
Turn Type	pm+pt	—	—	pm+pt	—	Perm	Prot	—	—	Prot	—	—
Protected Phases	3	8	—	7	4		1	6	—	5	2	—
Permitted Phases	8		—	4		4			—			—
Lagging Phase?	☑	☑	—	☐	☐	☐	☐	☐	—	☑	☑	—

The cycle times are shown below.



Next the HCM 6th /7th Edition Report should be created to determine the lane group flow and the saturation flow rate values which are necessary for the critical v/c calculation.

HCM 6th Signalized Intersection Summary
11: OR 99W & SW Alexander Ave/SE Alexander Ave 08/29/2023

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	10	2	2	25	1	65	3	1275	60	140	1625	5
Future Volume (veh/h)	10	2	2	25	1	65	3	1275	60	140	1625	5
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	0.89		0.94	1.00		1.00	1.00		0.99	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1900	1900	1900	1900	1900	1900	1976	1826	1841	1945	1856	1900
Adj Flow Rate, veh/h	24	5 +	5	35	1	92	3	1401 +	66	156	1806 +	6
Peak Hour Factor	0.41	0.41	0.41	0.71	0.71	0.71	0.91	0.91	0.91	0.90	0.90	0.90
Percent Heavy Veh, %	0	0	0	0	0	0	0	5	4	2	3	0
Cap, veh/h	131	59	59	145	147	124	16	2068	97	203	2584	9
Arrive On Green	0.03	0.07	0.07	0.03	0.08	0.08	0.01	0.60	0.59	0.11	0.70	0.69
Sat Flow, veh/h	1810	840	[illegible]	1810	1900	1604	1882	[illegible]	[illegible]	1853	[illegible]	[illegible]
Grp Volume(v), veh/h	24	0	10	35	1	92	3	738	729	156	906	906
Grp Sat Flow(s),veh/h/ln	1810	0	1681	1810	1900	1604	1882	1826	1795	1853	1856	1853
Q Serve(g_s), s	0.0	0.0	0.5	1.6	0.0	4.7	0.1	23.0	23.2	6.9	24.3	24.3
Cycle Q Clear(g_c), s	0.0	0.0	0.5	1.6	0.0	4.7	0.1	23.0	23.2	6.9	24.3	24.3
Prop In Lane	1.00		0.50	1.00		1.00	1.00		0.09	1.00		0.01
Lane Grp Cap(c), veh/h	131	0	117	145	147	124	16	1092	1073	203	1297	1296
V/C Ratio(X)	0.18	0.00	0.09	0.24	0.01	0.74	0.19	0.68	0.68	0.77	0.70	0.70
Avail Cap(c_a), veh/h	278	0	587	278	664	560	201	1092	1073	241	1297	1296
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	40.1	0.0	36.8	38.6	36.0	38.1	41.6	11.5	11.5	36.6	7.5	7.5
Incr Delay (d2), s/veh	0.5	0.0	0.3	0.6	0.0	6.2	4.3	3.4	3.5	10.8	3.1	3.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.5	0.0	0.2	0.7	0.0	2.1	0.1	8.9	8.8	3.6	8.2	8.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	40.6	0.0	37.1	39.2	36.0	44.4	45.9	14.8	15.0	47.4	10.6	10.6
LnGrp LOS	D	A	D	D	D	D	D	B	B	D	B	B
Approach Vol, veh/h		34			128			1470			1968	
Approach Delay, s/veh		39.6			42.9			15.0			13.5	
Approach LOS		D			D			B			B	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	4.7	63.1	6.2	10.6	13.3	54.5	6.8	9.9
Change Period (Y+Rc), s	4.5	4.5	4.0	4.0	4.5	4.5	4.0	4.0
Max Green Setting (Gmax), s	8.5	52.0	9.0	29.5	10.5	50.0	9.0	29.5
Max Q Clear Time (g_c+I1), s	2.1	26.3	2.0	6.7	8.9	25.2	3.6	2.5
Green Ext Time (p_c), s	0.0	24.0	0.0	0.3	0.1	20.9	0.0	0.0

Intersection Summary		
HCM 6th Ctrl Delay		15.4
HCM 6th LOS		B

Notes

User approved changes to right turn type.

Follow the guidance above to walk through the phases and determine the critical movements. The northbound/southbound pair have protected lefts in a lead-lag phasing arrangement. Because there are two phases in the north and south directions there will be two flow ratios used in the calculation which is the highest combination of the northbound left with the southbound through or the southbound left and the northbound through.

Flow ratios (v/s) are calculated for each movement group by dividing the group flow rate by the saturated flow rate as depicted in the table below. The northbound left and southbound left are protected lead-lag phasing and Exhibit 13- 7 should be used as a guide. Recalling from the guidance above that when there is protected lead-lag phasing the higher combination of the northbound left and the southbound through or the southbound left and the northbound through v/s ratio should be chosen. However, in this example, the northbound and southbound through movements each consist of a through and a through-right lane group. Each lane group must be considered and then the highest v/s chosen. In this case, the shared through right lane groups in both directions have a higher v/s over the through lane group.

Road Name	Approach	Lane Group	Lane Group Flow (vph)	Saturation Flow Rate (vph)	v/s Flow Ratio	Use?
OR 99W	NB	Left	3	1882	0.002	x
		Through	738	1826	0.404	
		Shared Thru-Right	729	1795	0.406	
OR 99W	SB	Left	156	1853	0.084	
		Through	906	1856	0.488	
		Shared Thru-Right	906	1853	0.489	x

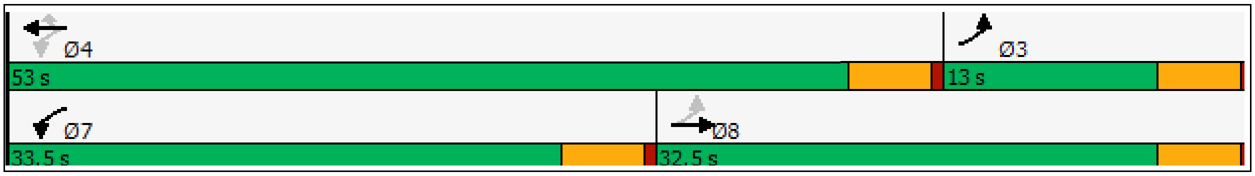
For protected lead-lag phasing, reference Exhibit 13- 7. Only Rules 1 and 2 of Exhibit 13- 10 are applicable and should be compared.

Rule 1) NBL + SBTR = 0.002 + 0.489 = 0.491
Rule 2) SBL + NBTR = 0.084 + 0.406 = 0.490

The NBL+ SBTR path flow ratio should be used in the calculation (0.491>0.490).

Next consider the eastbound and westbound movements that have lead-lag protected-permitted left phasing. Follow the guidance presented above, then there are three additional steps to evaluate Exhibit 13- 10 Rule 4:

i. Identify highest v/s flow ratio of permitted lefts.
ii. Sum the flow ratios of the protected leading and lagging lefts, and the permitted left previously identified, see Exhibit 13- 9.
    a. Note: HCM 2000 Report is needed to isolate saturation flow rates for the protected and permitted lefts.



Using the phasing diagram shown above, the westbound protected left is leading, and the eastbound protected left is lagging. There is not an explicit way to split out the protected and the permitted portions of the left turn volumes. Therefore, the analyst will need to create a proportion based on the cycle time as this is the best proxy available. The phase diagram can be used to create a table such as the one below. A ratio is created for each movement by dividing the protected or permitted portion of the time by the total phase time. This ratio can then be applied to the adjusted flow rate (from the HCM 6^{th} /7^{th} Edition Report) to split the volume into the protected and the permitted portions.

	Left Turn Time (sec)			**Left Turn Ratios**		**Left Turn Flows (vph)**		
Movement	**Total Phase Time**	**Protected**	**Permitted**	**Protected**	**Permitted**	**Total Left**	**Protected**	**Permitted**
Eastbound	32.5	13	19.5	0.40	0.60	24	**10**	**14**
Westbound	53	33.5	19.5	0.63	0.37	35	**22**	**13**

The other element of the flow ratio calculation is the Saturated Flow Rate. The HCM 6^{th} /7^{th} Edition Report does not provide this separately for the protected and the permitted portions. The HCM 2000 Report will need to be created to get these values which can be seen boxed in red below (*note that this is the only value that the HCM 2000 Report should be used for*).

With the HCM reports, the elements necessary for the critical intersection v/c calculation have been created and can be seen in the next table. The four rules associated with Exhibit 13- 10 will then be considered as follows:

- Rule 1 A) WB thru and EB protected left,
- Rule 1 B) WB right and EB protected left, and
- Rule 2) WB protected left and EB shared through-right movements.
- *Rule 3) n/a*
- Rule 4 A) EB protected left, WB protected left, and EB permitted left
- Rule 4 B) EB protected left, WB protected left, and WB permitted left

HCM Signalized Intersection Capacity Analysis
11: OR 99W & SW Alexander Ave/SE Alexander Ave 08/31/2023

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	10	2	2	25	1	65	3	1275	60	140	1625	5
Future Volume (vph)	10	2	2	25	1	65	3	1275	60	140	1625	5
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	10	10	10	12	10	10	13	12	12	13	12	12
Total Lost time (s)	4.0	4.0		4.0	4.0	4.0	4.0	4.0		4.0	4.0	
Lane Util. Factor	1.00	1.00		1.00	1.00	1.00	1.00	*1.00		1.00	*1.00	
Frpb, ped/bikes	1.00	0.99		1.00	1.00	0.99	1.00	1.00		1.00	1.00	
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Frt	1.00	0.93		1.00	1.00	0.85	1.00	0.99		1.00	1.00	
Flt Protected	0.95	1.00		0.95	1.00	1.00	0.95	1.00		0.95	1.00	
Satd. Flow (prot)	1685	1621		1805	1773	1488	1865	3591		1829	3687	
Flt Permitted	1.00	1.00		1.00	1.00	1.00	0.95	1.00		0.95	1.00	
Satd. Flow (perm)	1773	1621		1900	1773	1488	1865	3591		1829	3687	
Peak-hour factor, PHF	0.41	0.41	0.41	0.71	0.71	0.71	0.91	0.91	0.91	0.90	0.90	0.90
Adj. Flow (vph)	24	5	5	35	1	92	3	1401	66	156	1806	6
RTOR Reduction (vph)	0	5	0	0	0	86	0	2	0	0	0	0
Lane Group Flow (vph)	24	5	0	35	1	6	3	1465	0	156	1812	0
Confl. Peds. (#/hr)			15			1			7			12
Heavy Vehicles (%)	0%	0%	0%	0%	0%	0%	0%	5%	4%	2%	3%	0%
Turn Type	pm+pt	NA		pm+pt	NA	Perm	Prot	NA		Prot	NA	
Protected Phases	3	8		7	4		1	6		5	2	
Permitted Phases	8			4		4						
Actuated Green, G (s)	3.3	3.3		4.6	4.6	4.6	1.0	35.2		12.4	46.6	
Effective Green, g (s)	3.3	3.3		4.6	4.6	4.6	1.5	35.7		12.9	47.1	
Actuated g/C Ratio	0.05	0.05		0.07	0.07	0.07	0.02	0.51		0.18	0.67	
Clearance Time (s)	4.0	4.0		4.0	4.0	4.0	4.5	4.5		4.5	4.5	
Vehicle Extension (s)	2.5	3.0		2.5	2.5	2.5	2.5	4.6		2.5	4.6	
Lane Grp Cap (vph)	82	75		123	115	96	39	1815		334	2459	
v/s Ratio Prot	0.01	0.00		0.01	0.00		0.00	c0.41		0.09	c0.49	
v/s Ratio Perm	c0.01			c0.01		0.00						
v/c Ratio	0.29	0.07		0.28	0.01	0.06	0.08	0.81		0.47	0.74	
Uniform Delay, d1	31.0	32.2		31.5	30.9	31.0	33.9	14.6		25.8	7.7	
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Incremental Delay, d2	1.4	0.4		0.9	0.0	0.2	0.6	4.0		0.8	2.0	
Delay (s)	32.4	32.6		32.4	30.9	31.2	34.5	18.5		26.5	9.7	
Level of Service	C	C		C	C	C	C	B		C	A	
Approach Delay (s)		32.5			31.5			18.6			11.0	
Approach LOS		C			C			B			B	

Intersection Summary			
HCM 2000 Control Delay	15.0	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.77		
Actuated Cycle Length (s)	70.6	Sum of lost time (s)	16.0
Intersection Capacity Utilization	72.0%	ICU Level of Service	C
Analysis Period (min)	15		
c Critical Lane Group			

Road Name	Approach	Lane Group	Lane Group Flow (vph)	Saturation Flow Rate (vph)	v/s Flow Ratio	Use?
Alexander	EB	Permitted Left	14	1773	0.008	
		Protected Left	10	1685	0.006	x
		Shared Thru-Right	10	1681	0.006	
Alexander	WB	Permitted Left	13	1900	0.007	
		Protected Left	22	1805	0.012	
		Through	1	1900	0.001	
		Right	92	1604	0.057	x

Flow ratios are calculated for each movement lane group by dividing the adjusted flow rate by the saturated flow rate:

EB Protected Left = 10/1681 = 0.006
WB Protected Left = 22/1805 = 0.012
EB Permitted Left = 14/1773 = 0.008

The flow ratio combinations are then compared to determine the critical phases:

Rule 1 A) 0.001+0.006 = 0.007
Rule 1 B) 0.057+0.006 = **0.063**
Rule 2) 0.012+0.006 = 0.018
Rule 3) n/a
Rule 4 A) 0.006+0.012+0.008 = 0.026
Rule 4 B) 0.006+0.012+0.007 = 0.025

The scenario with westbound right and eastbound protected-left (Rule 1 B) has the highest flow ratio and should be used.

Next, sum of all critical phases flow ratios: 0.002+0.489+0.006 +0.057 = **0.554**

Cycle length = 116 sec
Lost time per phase = 4 sec
Total Lost time = 16 sec

Note that although five flow ratios are used in the calculation, there are still only four phases. So, the total lost time is 16 seconds.

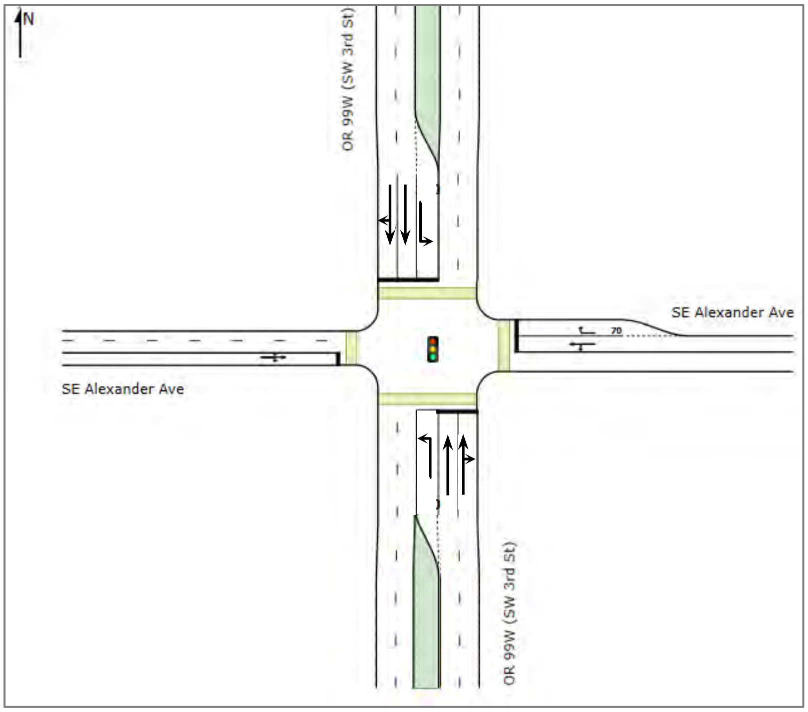
The critical intersection v/c ratio is then calculated using the HCM equation:
Xc = Sum of critical flow ratios * C/(C-L) = 0.517 * 116/ (116-16) = <u>**0.64**</u>

SIDRA is known to create a critical intersection v/c ratio that is more of an outlier than the other tools. The analyst should be aware of this and understand why this is the case.

1. SIDRA is known to use several extensions and additions, such as the throttling of volumes upstream and downstream from the intersection if capacities are exceeded. This will impact the saturated flow rates by reducing them which will cause a higher v/c, but it will also reduce the volume reaching the intersection which will cause a lower v/c. Depending on which effect creates a larger reduction will determine which direction the v/c shifts.
2. SIDRA is correctly “seeing” the existing capacity issues adjacent to the intersection which would limit traffic reaching it. This provides a more realistic result for congested conditions.
3. SIDRA is unique. The User Guide provides the following information: “SIDRA INTERSECTION is compatible with the Highway Capacity Manual. However, unlike other software packages, the HCM Setup in SIDRA INTERSECTION does not claim to be a simple replication of the HCM procedures. Instead, SIDRA INTERSECTION offers various extensions on the capabilities HCM offers.”

Example 13- 7 Calculating Critical Intersection v/c Ratio in SIDRA Intersection

This example shows the calculations for the intersection v/c ratio for a four-leg four-phase intersection in SIDRA. The phase rotation is given as protected lead-lag left turn signal phasing on the north and south approaches and permitted split phasing on the east and west approaches. The cycle time is 116 seconds with four seconds of lost time assumed for each phase (for consistency with the other examples). SIDRA has a different optimization routine than the other software and frequently uses internally computed practical cycle time, which was 137 seconds here (vs. user defined cycle time). The signalized intersection layout is shown below.



Calculate the critical flow ratios for each phase using output from the from the “Lane Flow and Capacity Information” report found withing “Detailed Output”. Note that Lane 1 is the left turn lane and Lane 2 is the through-right lane. Earlier versions of SIDRA gave the flow ratios directly, but Version 9.1 will require that they be calculated separately. Flow ratios are calculated by dividing the total arriving flow by the saturation flow accounting for lane blockage. Note that while this example does not have any lane blockage effects, the values in the two saturation flow columns can be substantially different when there are lane blockages.

LANE FLOW AND CAPACITY INFORMATION

Lane No.	Total Arv Flow veh/h	Lane Width ft	Adj. Basic tcu/h	Saturation Flow Rate: W/O Lane Blockage 1st veh/h	W/O Lane Blockage 2nd veh/h	With Lane Blockage 1st veh/h	With Lane Blockage 2nd veh/h	End Cap veh/h	Tot Cap veh/h	Deg. Satn x	Lane Util %
South: OR 99W (SW 3rd St)											
1	3	13.0	1976	1951		1951		53	712	0.005	100
2	737	12.0	1900	1803<		1803<		0	658	1.119	100
3	730	12.0	1900	1780	357	1780	357	0	652	1.119	100
East: SE Alexander Ave											
1	37	12.0	1900	1813		1813		0	132	0.277	100
2	92	10.0	1900	1610	1610	1610	1610	0	1446	0.063	100
North: OR 99W (SW 3rd St)											
1	156	13.0	1976	1473		1473		52	527	0.295	100
2	850	12.0	1900	1632<		1632<		0	596	1.428	100
3	961	12.0	1900	1843	21	1843	21	0	673	1.428	100
West: SE Alexander Ave											
1	34	10.0	1900	599	1481	599	1481	77	102	0.336	100

< Reduced saturation flow due to a short lane effect
Delay and stops experienced by drivers upstream of the short lane entry have been accounted for.

Basic Saturation Flow in this table is adjusted for area type factor, lane width, approach grade, parking manoeuvres and number of buses stopping.
Saturation flow scale (Demand & Sensitivity dialog) applies if specified.

Saturation Flow rates Without (W/O) Lane Blockage are used for signal timing purposes when the signal timing option to exclude downstream lane blockage effects is selected in Network analysis.

The "Saturation Flows" report may also be used. The Saturation Flow Rate in the column furthest to the right is the appropriate value to use as it includes both downstream lane blockage and short lane effects. The values are identical to the "Saturation Flow Rate" report values so either may be used.

SATURATION FLOWS

Site: [OR 99W at SE Alexander (Site Folder: General)]

Output produced by SIDRA INTERSECTION Version: 9.1.1.200

OR 99W at SE Alexander
SB Approach dist is from flashing beacon
NB Approach dist is from Viewmont
Site Category: (None)
Signals - Actuated Isolated Cycle Time = 137 seconds (Site Practical Cycle Time)
This Site is not connected to the Network.

Lane Saturation Flow Rates	Basic Satn Flow [1] tcu/h	CTORS Satn Flow [3]] veh/h	Green Period	Other Model Elements [4] veh/h	Lane Block. [5] veh/h	Short Lane [6] veh/h
South: OR 99W (SW 3rd St)						
Lane 1	1900	1882	1	1951	1951	1951
Lane 2	1900	1810	1	1810	1810	1803[B]
Lane 3	1900	1782	1	1780	1780	1780
			2	357	357	357
East: SE Alexander Ave						
Lane 1	1900	1813	1	1813	1813	1813
Lane 2	1900	1610	1	1610	1610	1610
			2	1610	1610	1610
North: OR 99W (SW 3rd St)						
Lane 1	1900	1845	1	1473	1473	1473
Lane 2	1900	1845	1	1845	1845	1632[B]
Lane 3	1900	1843	1	1843	1843	1843
			2	21	21	21
West: SE Alexander Ave						
Lane 1	1900	1790	1	599	599	599
			2	1481	1481	1481

The guidance provided at the beginning of this section indicates that there will be two v/s flow ratio elements used for the north-south directions. For protected lead-lag phasing reference Exhibit 13- 7. Only Rules 1 and 2 of Exhibit 13- 10 are applicable and should be compared. The values boxed in red in the detailed report shown above can be used to calculate all the flow ratios for the applicable lane groups and create a table like the one shown below.

Road Name	Approach	Lane Group	Lane Group Flow (vph)	Saturation Flow Rate (vph)	v/s Flow Ratio	Use?
OR 99W	NB	Left	3	1951	0.002	x
		Through	737	1803	0.409	
		Shared Thru-Right	730	1780	0.411	
OR 99W	SB	Left	156	1473	0.106	
		Through	850	1632	0.521	
		Thru-Right	961	1843	0.521*	x

** Note that the southbound v/s appear to be the same, but the through-right is slightly higher when taken out to four decimals as a tie-breaker (0.5214 vs 0.5208).*

Rule 1) NBL + SBTR = 0.002 + 0.521 = 0.523
Rule 2) SBL + NBTR = 0.106 + 0.411 = 0.517

The NB v/s flow ratio should be used in the calculation = **0.523**

For the eastbound-westbound phase, there are two saturation flow rates given for the different green periods (this is a reflection of a varying saturation flow rate caused by turning vehicles or overflow effects from the adjacent turn lane), as detailed in the" Lane Flow and Capacity Information" report shown above. For the calculation, typically the larger saturation flow rate is used. Again, using the "Lane Flow and Capacity Information" provided in the Detailed Report shown above the table below can be created.

Road Name	Approach	Lane Group	Lane Group Flow (vph)	Saturation Flow Rate (vph)	v/s Flow Ratio	Use?
Alexander	EB	Shared Left-Thru-Right	34	1481	0.023	x
Alexander	WB	Shared Left-Thru	37	1813	0.020	
		Right	92	1610	0.057	x

Because this is a split phase, only Exhibit 13- 10 Rule 1 is applicable, and the critical flow ratio can then be calculated:

i. WBR v/s flow ratio = highest of the shared through-right or exclusive right lane group which is determined to be the WBR (0.057>0.020);
ii. These are then summed to obtain the critical pair flow ratio = 0.023 + 0.057 = **0.080**

The critical v/c ratio (X_c) is then calculated by dividing the cycle length by the cycle length minus the lost time per cycle and then multiplying this times the sum of the critical flow ratios for each phase. As stated previously, there are 4 seconds of lost time per phase and a 116 second cycle time was used (not the Site Practical Cycle Time provided by SIDRA). This is a four-phase intersection, so 16 seconds of total lost time is used.

X_c = [C / (C-L)] * (N-S + E-W) = [116s / (116s-16s)] * (0.523 + 0.080) = **0.70**

Important Software Notes

- Comparison of tools may lead to disservice as each tool has different strengths and internal methodologies.
    - Synchro is best used on signalized arterials with standard intersection configurations where progression is important.
    - SIDRA is best used when congestion is evident or where intersections are non-standard or affected by additional modes (i.e. bike signals, bus lanes, etc.).
    - Vistro is best used for comparing multiple scenarios across simpler intersection configurations and where the greatest efficiency is required.

The resulting intersection v/c's are all generally consistent, but Synchro tends to be lower, SIDRA higher, and Vistro in the middle. The overall context of the study area, the questions to be answered, and the desired analysis need should drive the tool choice. Results will be relative within a single tool and may not be consistent when compared to other tools, so the analyst must understand the differences between them.

- SIDRA will always tend to be more of an outlier as it has numerous HCM extensions and a more rigorous congestion methodology, but under congested conditions it may provide results more in line with actual operations. This is because it accounts for decreased traffic flow in congested and saturated conditions. It should be thought of more in the "spirit of" the HCM rather than a strict replication but still retains basic compatibility.

- Vistro is the only one of the three tools that directly provides the signalized intersection critical v/c ratio with no post-processing required, but it does have simpler inputs and is not as flexible as Synchro or SIDRA, so this is a trade-off.
- Synchro expects strict NEMA phasing which may require careful “workarounds”.
- Protected + permitted left turn phasing is a topic of ongoing discussion regarding best practices as current HCM reporting does not include all the necessary outputs. When using Synchro, extra steps are required when calculating v/c ratios in this scenario. However, Vistro calculates the v/c ratio internally and doesn’t require post-processing, negating this issue.

13.4.5 Analysis Procedures Regarding Signal Timing

Capacity analysis of signalized intersections should be performed in accordance with the methods and default parameters contained in this manual. ODOT has established the following criteria for traffic impact studies regarding the timing chosen for the capacity analysis of signalized intersections. ODOT reserves the right to reject any operational improvements that in its judgment would compromise the safety and efficiency of the facility.

Phase Splits

Thirteen seconds is the lowest total split that should be used including yellow and all-red time. Clear documentation of the selected maximum splits for each phase must be provided in the analysis. The total side street splits should not be greater than the highway splits. Except in cases where the analyst is directed otherwise by ODOT staff, the splits are considered optimized when they yield the lowest overall intersection v/c ratio. This optimization should be done for each capacity analysis.

Non-Coordinated Signals

Cycle lengths and phase splits should be optimized to meet an ideal level of service, queuing and/or volume to capacity ratio for a non-coordinated traffic signal intersection. If simulation is going to be needed, existing signal timing will be necessary for the calibration process. For a new signal, the cycle length for the analysis should not exceed 60 seconds for a two-phased traffic signal, 90 seconds for a three-phased traffic signal (e.g., protected highway left turns, and permissive side streets left turns) or 120 seconds for a four or more phased traffic signal. The signal cycle length should cover the pedestrian clearance time for all crosswalks. For information on pedestrian crossings, see ODOT Traffic Signal Policy and Guidelines.

Signals in Coordinated Signal System

At the start of a project, ODOT staff will determine whether the analysts should use the existing signal timings for all analysis scenarios or develop optimized timings for the coordinated system. The existing timings may need to be used to calibrate a simulation model. If the existing timings are to be used in the analysis, Region traffic shall provide timing files, timing sheets or Synchro files of the existing settings. If optimized timings are to be developed, those settings are subject to approval by ODOT, and those conditions become the baseline for all comparisons.

The following settings should be optimized for each analysis scenario when the analyst is asked to use optimum coordination settings.

- Cycle Length
- Phase Length (Splits)
- Phase Sequence (Lead/Lag Left Turns)
- Intersection Offsets

The optimum settings must meet the criteria established in OAR 734-020-0480 as it relates to progression analysis while also attempting to find the lowest v/c ratio for each intersection. This OAR only applies when modifications are proposed to a signal which would affect the settings of the coordination plans. Examples of these modifications are changes in cycle length, decreased green time for mainline, additional phases, longer crosswalks and intersection relocation. For specific software setting requirements refer to Appendix 12/13.

<u>Adaptive Signal Timing</u>

In non-adaptive/responsive control, signalized intersections operate off a set of timing plans that are programmed into the signal controllers. Adaptive Signal Timing (AST) technology allows the signal controller to continuously vary the signal timing (green time or splits) based on detection of real time traffic flows. AST is normally installed as a system with multiple signals in coordination and focuses on progression. AST can better progress traffic when the signal system is under-saturated as compared to set timing plans. When these signals operate under fully saturated or oversaturated conditions the timing can be more consistent since the splits are maxed out. There are different types of AST platforms that are currently available and installed on ODOT facilities and those of local jurisdictions (i.e. SCOOT, SCATS, RHODES, OPAC, Insync, Synchro Green, etc.).

Traditional capacity analysis methods based on the Highway Capacity Manual (e.g. Synchro, SimTraffic, HCS) analyze signalized intersections assuming a set timing scheme and do not model AST behavior. Multiple analysis methods are possible. The simplest method to analyze AST is to assume all intersections are actuated and coordinated, and to optimize the signal timing even for Existing conditions.

Other possible methods include

- Run different scenarios over a full range of cycle lengths and splits and take the average of the results.
- Some adaptive signal controller data can be input directly into Vissim.
- Use Vissim’s custom adaptive signal timing

Future Signals

For future signals, left turns should be assumed to have the appropriate phasing (i.e., permitted, protected-permitted or protected only) according to the criteria for left turn treatment contained in the current ODOT Traffic Signal Policy and Guidelines. The Region Traffic Section and the Traffic Section should be consulted any time a new signal is proposed. It should always be considered that while new traffic signals provide a benefit to some users, the capacity of the mainline is typically cut in half by new signal installations and improper or unjustified signals can increase the frequency of rear-end collisions, delays, disobedience of signal indications and the use of less adequate routes.

Signal Timing Sheets

If it is desired to closely match the current traffic operations, the timing parameters installed in the signal controller need to be used in the analysis. The field timing parameters are recorded on the signal timing sheets located in the signal cabinet. Signal timing sheets should be obtained from the Region Traffic office as they generally have the most recent copies from the signal cabinet. Signal timing changes frequently, so the analyst should make sure to have the most recent version. For the analyst, not all the included sheets are necessary, but it is important that all the needed sheets are obtained. The following shows the important sheets (shown in Exhibits 13-13 through Exhibit 13- 19), Sheets 2, 3, 6, 7 and 8. Sheets 4 and 5 are required if multiple timing plans exist) and what to look for on each sheet. The example signal timing sheet used to illustrate this section is the intersection of US 97 (Bend Parkway) and Pinebrook Boulevard in Bend.

<u>Sheet 2 – Phase Rotation Diagram</u>

The phase rotation diagram shows how the signal operates through its cycle. This diagram is needed so the signal is entered correctly into Synchro or another program. For complicated phasing, the diagram is an invaluable source. Exhibit 13- 13 shows a phase rotation diagram for US 97 and Pinebrook Boulevard, which is a two-phase signal. Many timing sheets, especially the electronic ones, are missing the phase rotation diagram. Contact the appropriate Region Traffic section to obtain.

Exhibit 13- 12 Signal Timing Sheet 2

SHEET 2

Date sheet in effect:

Date sheet voided:

Location: Hwy 97 @ Pinebrook

Table Numbers refer to Trafficview & Translink

TABLE 3

Clock, EV and Misc. (C + Key)		
Function	Key	
Year	0	Clock Location C + 3 uses Call / Active Display Sunday = 1
Month	1	
Date	2	
Day of Week	3	
Hour	4	
Minute	5	
Second	6	
1/10 Second	7	
		Phase Number: 1 2 3 4 5 6 7 8
	8	
Handicap Ped	E	

Phase Rotation Diagram

TABLE 3

Preemption Data (E+Key)			
Function	Key	Parameter	Timing
EVA	0	Delay	0
	1	Minimum	1
EVB	2	Delay	0
	3	Minimum	1
EVC	4	Delay	0
	5	Minimum	1
EVD	6	Delay	0
	7	Minimum	1
Overlaps	8	Red Revert	5.0
Railroad	9	Delay	
	A	Minimum	

		Phase Number							
		1	2	3	4	5	6	7	8
RR Clear Ph	B								
RR Permit	C								
RR OL Permit	D								
Nema Hold Ph	E								
	F								

TABLE 6 (also see sheet 6)

Miscellaneous (D + Code)			
Function	Code	Value	Notes
Floating Ped	2E		0 = Off 1 = On (Ph. 7 & 8 Not permitted)
ID Number	2F	061	Range 0 to 253 (1)
Coordination Ped Recalls	3E	1	0 = Recall 1 = No Recall
Rest in WALK	3F		0 = Off 1 = ON
Advance Warning End of Green	4E		Extend time for green after sign turns on (2) (5)
Advance Warning	4F		Delay time for sign after yellow (2) (5)
			red / d flash
			1 = Flash / Flash Red
NEMA Inputs	66		Non zero value reassigns C1 inputs. (3)
Bus Delay	6D		Delay time before preemption (4)
Bus Timer #1	6E		Extension of max green for phases 2 & 6 (Free operation)
Bus Timer #3	6F		Force off time for Ph 4 & 8 (only in Free operation)
JHK Protocol	76		0 = no 0.1 = yes
JHK Area No. & 1st digit local	7D		Area No. 0 - 7 and Local 001 - 510 (1)
EV minimum timed Start / end of call	7E		0 = at start of call 1 = at end of call
EV On Indicators	7F		0 = Off, 1 = Flash, 5 = solid indication (5)

Notes

(1) JHK ID no. is formed by Area no. (0 to 7) and 3 digit Local no. (001-510). Left most digits entered as x.x in location 7D and rightmost as xx in location 2F

(2) See Sheet 6, Location B+0+E

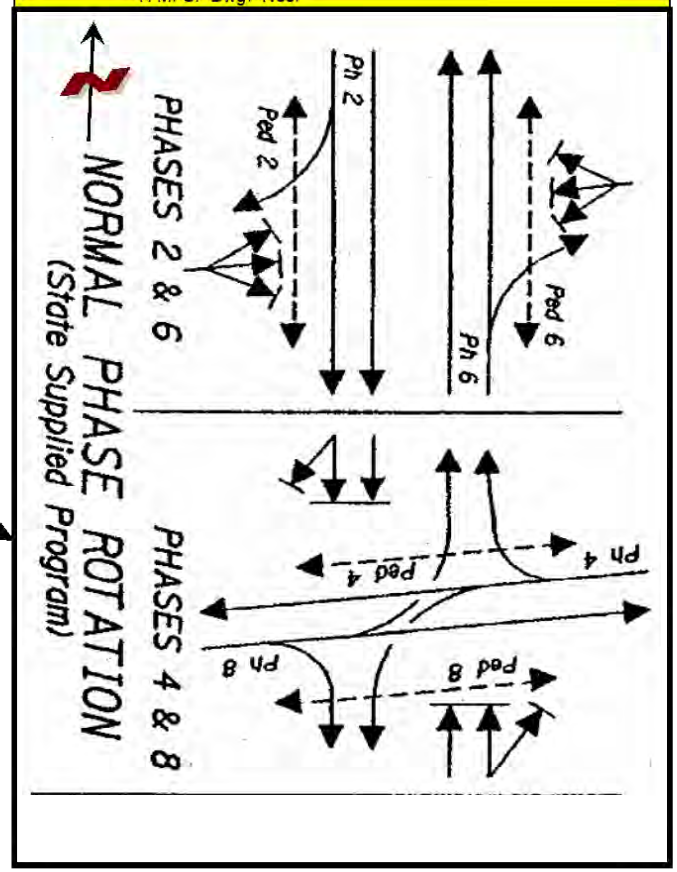
(3) C1 pins 54, 63, 64, 75, 76, and 77. See sheet 6, Location B+0+D.

(4) Entering 25.5 in this location is the only way of disabling bus preempt.

(5) Ped yellow outputs, C1-35, 36, 37, and 38 are used by Rt. Turn Overlaps, EV on indicators, TOD/DOW programmable outputs, Fiber Optic sign for RR flash yellow clearance, and Advance Warning sign operation.

Phase Rotation Diagram

T. M. S. Dwg. Nos:



Sheet 3 – Table 1 Phase Functions

Table 1 (Exhibit 13- 14) shows the basic phasing properties and Exhibit 13- 14 shows the pedestrian timings and the advanced actuated phasing properties needed for signalized analysis and simulation programs. Vehicle Recall (Key =0) shows what phases will appear for at least a minimum amount of time in each cycle the signal would return to if there is no demand on the side street. Permitted Phase (Key=4) shows what phases are present at this intersection. Overlap A-D (Key A-D) shows what phases operate together on each of the overlap outputs on the controller. If there are no checked boxes in this section, then there are no overlapping phases, but there may be signal heads displaying outputs from two phases such as the common vertical five-section right-turn signal head.

Sheet 3 – Table 1 Phase Timing

For non-coordinated signals, the cycle length and phase splits can be determined from the Phase Timing portion of Table 1. If multiple timing plans exist, then they will be listed on Sheet 4 and/or Sheet 5. The only values that are needed to determine splits and cycle lengths from this portion of Table 1 are the maximum greens (Key = ph + 0), max 2 greens (Key = ph +1), yellow time (Key = ph + C) and all-red time or red clear (Key = ph + D).

The cycle length of actuated signals will vary from cycle to cycle depending on the vehicle demand. Synchro's phase splits include yellow and all-red, which is different from the maximum green on the timing sheet. Synchro also forces the maximum greens to add up perfectly to the cycle length. Therefore, the maximum cycle length needs to be proportionally adjusted down to match with Synchro's cycle length (the cycle length that is entered into the program). The maximum cycle length can be determined by summing the maximum greens (or max 2 greens if those are used in the analysis hour) and the yellow/all-red for each phase. The max green values on Sheet 3 are just that, i.e., maximum green times. The total maximum split used in Synchro will be the sum of the max green (or max 2 green), yellow and all-red. To convert the Sheet 3 timing into Synchro-compatible timing, the following is done.

1. Add up the Synchro cycle lengths from Sheet 3 by summing the maximum greens.
2. Add the yellow time and all-red time to the cycle length calculated in Step 1 to obtain the maximum cycle length.
3. The Synchro phase lengths are calculated by dividing the green + yellow + all-red time for a phase by the maximum cycle length. This ratio is then multiplied by the Step 1 Synchro cycle length.
4. Repeat for each phase.

The sum of the Synchro phases should add up to the Step 1 cycle length.

Exhibit 13- 13 Signal Timing Sheet 3 – Basic Phase Settings

Da... Hwy 97 @ Pinebrook

Vehicle Recall, Permitted Phases & Overlaps

SHEET 3

TABLE 1 Page 0

Phase Functions (0+Key)									
Function	Key	Phase Number *							
		1	2	3	4	5	6	7	8
Veh Recall	0		X				X		
Ped Recall	1								
Red Lock	2								
Yellow Lock	3								
Permit Phase	4		X		X		X		X
Ped Phases	5		X		X		X		X
Lead Phases	6	X		X		X		X	
Double Entry	7				X				X
Sequential	8								
Start Green	9		X				X		
OLA=	A								
OLB=	B								
OLC=	C								
OLD=	D								
Exclusive	E								
Sim Gap	F		X				X		

TABLE 2 Page 0

Miscellaneous (C+F+Key)			
Function	Key	Value	
Page ID	0	0	Keys 8 through F use Call/Active Display
	1		
	2		
	3		
OLA Red	4		
OLB Red	5		
OLC Red	6		
OLD Red	7		

		Phase Number							
		1	2	3	4	5	6	7	8
RT OLE	8								
RT OLF	9								
Red Rest	A								
Max Recall	B								
Flash Green	C								
	D								
Advance WALK	E								
Restrictive Ph	F								

* Shown on Call/Active Display

TABLE 1 Page 0

Phase Timing (Ph. No. + Key)									
Interval	Key		Southbound Hwy 97		Westbound Pinebrook		Northbound Hwy 97		Eastbound Pinebrook
		Phase Number							
		1	2	3	4	5	6	7	8
Max Green	0		50		30		50		30
Max2 / HFDW	1		40		35		40		35
Walk	2		5		5		5		5
Flashing DW	3		21		21		22		25
Max Initial	4		20		5		20		[illegible]
Min Green	5		10		5		10		[illegible]
TBR	6		10		5		10		[illegible]
TTR	7		20		5		20		[illegible]
Observe Gap	8								
Passage	9		5.2		3.5		5.2		[illegible]
Min Gap	A		3.2		1.0		3.2		[illegible]
Add per Act	B		1.5				1.5		[illegible]
Yellow	C		4.0		4.0		4.0		4.0
Red Clear	D		1.0				1.0		
Red Revert	E		5.0		5.0		5.0		5.0
Walk 2	F								

To observe timing for an individual phase :
Enter C + A + F for Ring A (Phase 1-4) or enter C + B + F for Ring B (Phase 5-8)

Page I.D. 0

TABLE 2 Page 0

Miscellaneous (9+Key)				
Parameter		Key	Value	Notes
Short Pwr Dn		0		Clock Correction Speed up 1 - 9 Slow down 11 - 19
Long Power Dn		1		
Preemption Delay Types	EVA	2		Preemption Delay Types: Hold 1 Latch 2 Both 3 Neither 0
	EVB	3		
	EVC	4		
	EVD	5		
	RR	6		
OLD	Green	E		
	Yellow	F		

Maximum Green and Max 2 Green Times

Sheet 8 indicates when each is in effect.

Keyboard Entries when not in Free Display

- A Advance
- B Back
- C Clear Display
- D Column Advance
- E Enter and Advance
- F Free Display

Phase Conditions as shown on Free Display

- 00 Initial Entry
- 02 WALK
- 03 Flashing DW
- 05 Min Green
- 08 Rest
- 09 Passage
- 0B Added Initial
- 0C Yellow
- 0D Red C
- 0E Red R
- 11 Gap Out
- 12 Force Off
- 14 Max Out
- 15 Red Revert Timed out

Yellow and All-red Time

Phase Data Copy

C + x + C + y + D

- x From Phase (x cannot be 3 or 8)
- y To Phase(s) - up to 3

Exhibit 13- 14 Signal Timing Sheet 3 - Advanced Phase Settings

Date sheet in effect:

Date sheet voided:

Location: Hwy 97 @ Pinebrook

TABLE 1 Page 0

Phase Functions (0+Key)									
Function	Key	Phase Number * 1	2	3	4	5	6	7	8
Veh Recall	0		X				X		
Ped Recall	1								
Red Lock	2								
Yellow Lock	3								
Permit Phase	4		X		X		X		X
Ped Phases	5		X		X		X		X
Lead Phases	6	X		X		X		X	
Double Entry	7				X				X
Sequential	8								
Start Green	9		X				X		
OLA=	A								
OLB=	B								
OLC=	C								
OLD=	D								
Exclusive	E								
Sim Gap	F		X				X		

TABLE 2 Page 0

Miscellaneous (C+F+Key)		
Function	Key	Value
Page ID	0	0
	1	
	2	
OL		
OL		
OL		
OL		

Keys 8 through F

		1	2	3	4	5	6	7	8
RT OLE	8								
RT OLF	9								
Red Rest	A								
Max Recall	B								
Flash Green	C								
	D								
Advance WALK	E								
Restrictive Ph	F								

* Shown on Call/Active Display

SHEET 3

Actuated Phasing Settings for Timing Plans and Simulation

TABLE 1 Page 0

Phase Timing (Ph. No. + Key)									
Interval	Key	1	2 Southbound Hwy 97	3	4 Westbound Pinebrook	5	6 Northbound Hwy 97	7	8 Eastbound
Max Green	0		50		30		50		30
Max2 / HFDW	1		40		35		40		35
Walk	2		5		5		5		5
Flashing DW	3		21		21		22		25
Max Initial	4		20		5		20		5
Min Green	5		10		5		10		5
TBR	6		10		5		10		5
TTR	7		20		5		20		5
Observe Gap	8								
Passage	9		5.2		3.5		5.2		3.5
Min Gap	A		3.2		1.0		3.2		1.0
Add per Act	B		1.5				1.5		
	C		4.0		4.0		4.0		4.0
	D		1.0				1.0		
	E		5.0		5.0		5.0		5.0
	F								

Walk and Flashing Don't Walk times

To observe timing for an individual phase:
Enter C + A + F for Ring A (Phase 1-4) or
enter C + B + F for Ring B (Phase 5-8)

Page I.D. 0

Phase Conditions as shown on Free Display

00	Initial Entry	0C	Yellow
02	WALK	0D	Red Clear
03	Flashing DW	0E	Red Revert
05	Min Green	11	Gap Out
08	Rest	12	Force Off
09	Passage	14	Max Out
0B	Added Initial	15	Red Revert Timed out

TABLE 2 Page 0

		Key	Value	Notes
Short Pwr Dn		0		Clock Correction Speed up 1 - 9 Slow down 11 - 19
Long Power Dn		1		
Preemption Delay Types	EVA	2		Preemption Delay Types: Hold 1 Latch 2 Both 3 Neither 0
	EVB	3		
	EVC	4		
	EVD	5		
	RR	6		
Ped Inhibit		7		Usually should be 0
OLA	Green	8		Overlap Yellow Time should always be specified
	Yellow	9		
OLB	Green	A		
	Yellow	B		
OLC	Green	C		
	Yellow	D		
OLD	Green	E		
	Yellow	F		

Keyboard Entries when not in Free Display

A	Advance	D	Column Advance
B	Back	E	Enter and Advance
C	Clear Display	F	Free Display

Reinitialization
D + 1 + F + 1 + E
(Use **only** when in flash)

Phase Data Copy
C + x + C + y + D
x From Phase (x cannot be 3 or 8)
y To Phase(s) - up to 3

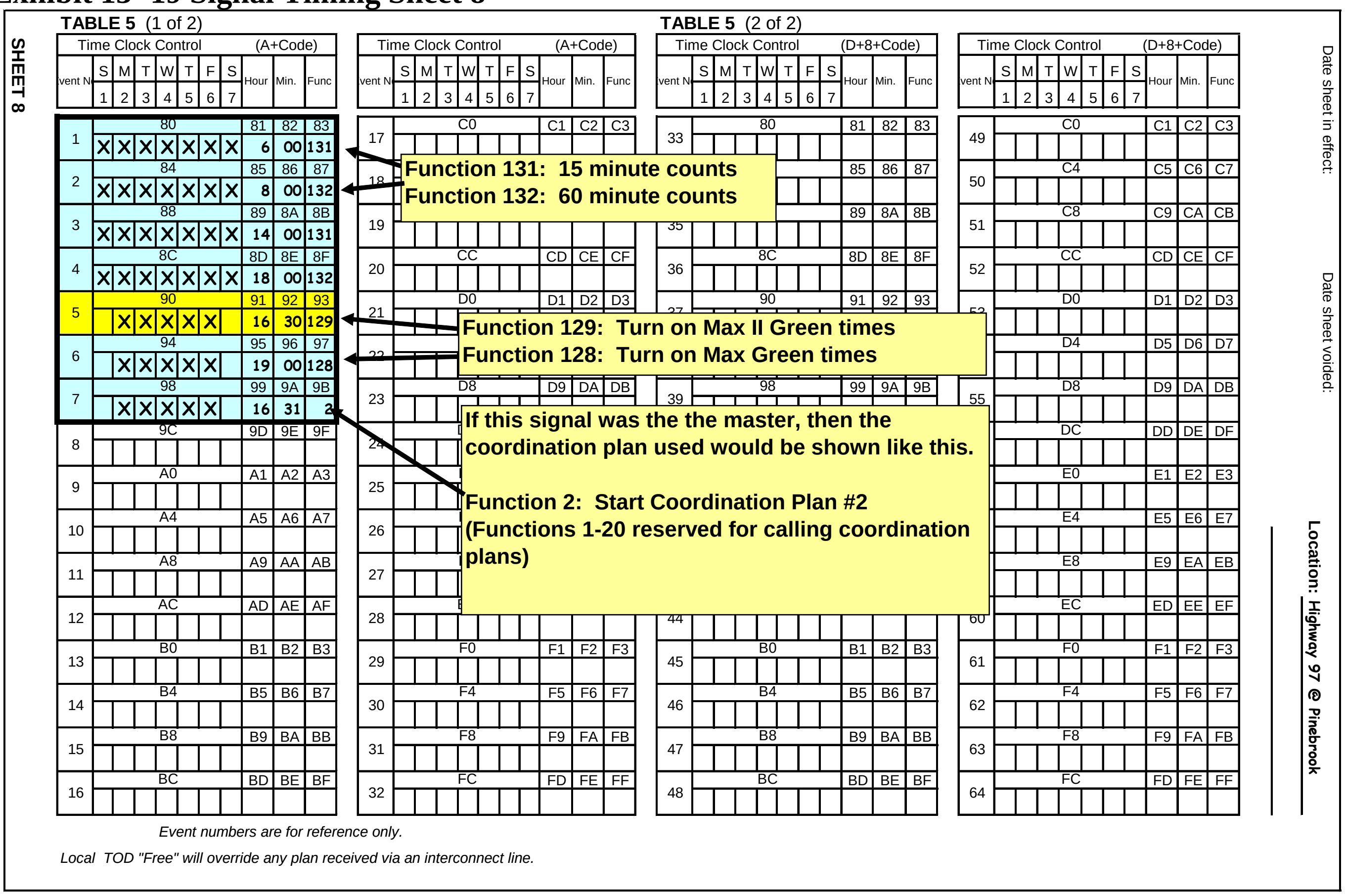
Example 13- 8 Signal Phase Splits

Example values for Sheet 3 are ():

- Vehicle Recall = Phases 2 and 6 (US 97)
- Permitted Phases = 2, 4, 6 and 8. From the phase rotation diagram in it is seen that Phase 2 and 6 on US 97 go together and Phase 4 and 8 on Pinebrook go together.
- Overlaps = No overlapping phases

If this signal was not coordinated (it isn't) then the maximum cycle length would be the maximum greens plus the yellow times plus the all-red times. In checking Sheet 8,

Exhibit 13- 19 Signal Timing Sheet 8



it is found that the max 2 green time is in effect starting at 4:30 PM, so the max 2 green time will be used to calculate the cycle length.

Maximum Cycle length = Max 2 green for Phase 2 and 6 + Max 2 green for Phase 4 and 8 + yellow x 2 phases + all-red x 1 phase = 40 + 35 + (4 x 2) + 1= 84 seconds.

Synchro phase split conversion:

1. Synchro Cycle length = 40 + 35 = 75 s
2. Maximum cycle length = 75 + 4(2) +1 = 84 s
3. Synchro Phase 2&6 = ((40 + 4 + 1) / 84) x 75 = 40 s
4. Synchro Phase 4&8 = ((35 + 4) / 84) x 75 = 35 s
5. Check = 40 +35 = 75 s = Step 1 cycle length

In the above example the differences in the phase splits are small, resulting in Synchro splits that are the same as the timing sheet splits. The splits are different if the maximum greens were used instead of the max 2 greens, as shown below.

1. Synchro Cycle length = 50 + 30 = 80 s
2. Maximum cycle length = 80 + 4(2) +1 = 89 s
3. Synchro Phase 2&6 = ((50 + 4 + 1) / 89) x 80 = 49 s
4. Synchro Phase 4&8 = ((30 + 4) / 89) x 80 = 31 s
5. Check = 49 +31 = 80 s = Step 1 cycle length

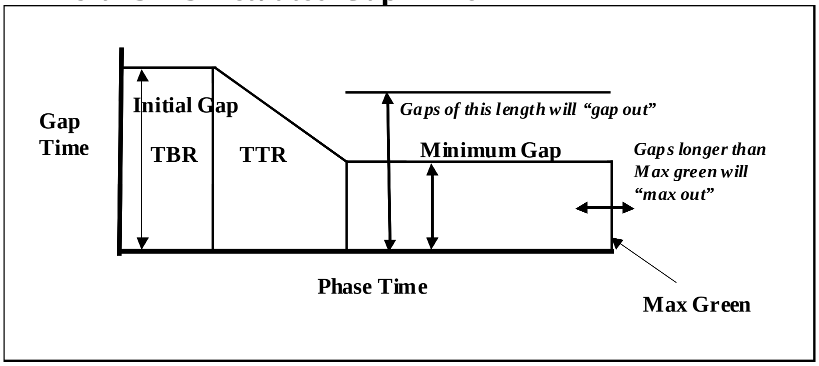
For most new actuated signals, additional settings need to be pulled from Table 1. Pedestrian settings can have a large impact on signal operation and the resulting intersection v/c especially if there are many pedestrian calls per hour on an approach. For creating a calibrated simulation, the actual pedestrian timing should be used as shown in Table 1 (Key= ph + 2 and Key = ph + 3) If the timing is not known, the ODOT standard walk time is 7.0 seconds with the curb-to-curb flashing don't walk time based on a 4.0 ft/s walk time.

Table 1 also covers the actuated signal phasing parameters that are needed for creating timing plans and calibrated simulations. These five parameters are:

- **Minimum Green** (Key= ph + 5) - Minimum green time that a signal indication will occur once the phase is served.
- **Time Before Reduce** (TBR) (Key= ph + 6) – Time elapsed before gap time is reduced
- **Time To Reduce** (TTR) (Key = ph + 7) - Time elapsed during gap time reduction to minimum.
- **Passage** (Key = ph +9) – This is the time that a phase is initially extended after a call is placed on a vehicle approach. Also known as initial gap.
- **Minimum Gap** (Key = ph + A) – Gap time after reduction until end of phase.

Exhibit 13- 16 shows the progression of the gap time from when a green indication starts at the initial gap in the TBR period down to the minimum gap time. During the TTR period, the initial gap time is reduced to the minimum gap time as specified on the timing sheet. If during the minimum gap time, the minimum gap is exceeded, then the signal will turn yellow (also known as a "gap out"). If vehicles keep approaching, the passage time will extend the green time to the maximum green time and then turn yellow (also known as a "max out"). Having a signal gap out is preferable, as dilemma vehicles (vehicles that either quickly accelerate or decelerate under yellow) can occur under max out conditions.

Exhibit 13- 15 Actuated Gap Time



<u>Sheet 6 – Table 6 Operation</u>

Table 6 indicates whether the signal is ever coordinated over the course of a day or week. If Mode (Key = B+0+4) is a non-zero value, then the intersection is coordinated. The intersection may or may not be in coordination during the analysis periods. The actual times that coordination plans are in effect are entered on Sheet 8 of the local controller or on Table 5 of the On-Street Master Controller. Exhibit 13- 16 shows that the example intersection is coordinated but is not the master.

Exhibit 13- 16 Signal Timing Sheet 6

Date sheet in effect:

Date sheet voided:

Location: Hwy 97 @ Pinebrook

SHEET 6

TABLE 6 (Also see sheet 2)

Operation (B + 0 + Key)		
Key	Parameter	Value
0	Present Plan	
1	Time of Day Plan	
2	Hardwire Plan	
3	MODEM Plan	
4	Mode (0 - 4 see right)	3
5	Master (0 - 4 see right)	0
6	Master Cycle Clock	
7	Local Cycle Clock	
8	Local Timer	
9		
A		
B		

Mode:
0 = Free
1 = TBC
2 = Hardwire
3 = Modem
4 = TM System

Master:
0 = Off
1 = Modem Master
2 = Hardwire Master
3 = 1 + 2
4 = TM Master

		Phase Number							
Key		1	2	3	4	5	6	7	8
C									
D	NEMA CNA								
E	Adv. Warn.								
F	MRI Phases		X		X		X		X

OSM ?	Y	N
		X

OSM Location

Powers Rd.

Coordination Mode and Master Type

Function Code Index						
Function		Time Clock On	Time Clock Off	Manual On	Manual Off	Notes
Outputs	A	71	81			*Phase 2 ped yellow (C1-35) (1)*
Outputs	B	72	82			*Phase 6 ped yellow (C1-36) (1)*
Outputs	C	73	83			*Phase 4 ped yellow (C1-37) (1)*
Outputs	D	74	84			*Phase 8 ped yellow (C1-38) (1)*
TOD Red Rest		25	24			*See Sheet 10 at B + C + D to set phases*
TOD Max Recall		27	26			*See Sheet 10 at B + A + E to set phases*
TOD Ped Recall		29	28			*See Sheet 10 at B + B + E to set phases*
WALK 2		55	54			*Use WALK 2 times set on Sheets 3, 4, 5*
Plan No.		1 - 18		1 - 18	0	*Sets operation to coordination plans on Sheet 7*
Free		20		20		*Sets operation to fully actuated*
Flash		19 or 33	32	19 or 33	0	*Sets operation to flash*
Max 2		129	128	129	0	*Use Max 2 times set on Sheets 3, 4, 5*
Det. Count 15		131	130			*Log Detector Counts - 15 min. intervals*
Det. Count 60		132	130			*Log Detector Counts - 60 min. intervals*
[illegible]		[illegible]				*Clear Detector Count Log*
[illegible]		[illegible]	[illegible]			*Enable Detector Diagnostics and log*
[illegible]		[illegible]	[illegible]			*Enable Detector Diagnostics without log*
Clear Det Diag.		138				*Clear Detector Diagnostic Log*
Send Real Time		199		199		*Modem master only*
Time Transfer		100		100		*Implements Page 0*
Time Transfer		101		101		*Implements Page 1*
Time Transfer		102		102		*Implements Page 2*
Page Copy				93	---	*Copies Page 0 data to Pages 1 & 2 Make sure Page 0 is the active Page*
Burn EEPROM				94	---	*Places active timing data into backup timing (Use reinitialization to place backup into active)*
Print Out				96	0	*Conect printer to C2 connector*

Note
(1) These C1 pins are used for other functions. See note (5) on Sheet 2.

Manual (D + 1 + E)	

TABLE 13 (Also see Sheet 10)

Miscellaneous (E + F + Key)		
Function	Key	Time
Railroad Max 2	0	
Ped Permissive Plan 1	1	
Ped Permissive Plan 2	2	
Ped Permissive Plan 3	3	
Ped Permissive Plan 4	4	
Ped Permissive Plan 5	5	
Ped Permissive Plan 6	6	
Ped Permissive Plan 7	7	
Ped Permissive Plan 8	8	
Ped Permissive Plan 9	9	
Number of Long Powerouts	A	
Number of Short Powerouts	B	
Failed Detector Number	C	
Max 2 On	D	
No Daylight Savings	E	
Revision Level	F	

TABLE 10 (Also see Sheet 12)

Sample Detectors (0 = off, 1 = on) (A + 3 + 9)

Sampling detectors are assigned using extended input codes on Sheet 11

For Protected / Permissive Left Turns

Left Turn Type (0, 1, or 2) (A + 3 + A)

0 = Off

1 = Left turn places call on cross street

2 = Left turn is omitted until cross street is serviced

Note: This feature works only with leading left turn phases 1, 3, 5, or 7. It is used to prohibit a green arrow from immediately following a green ball.

<u>Sheet 7 – Table 7 Coordination Timing</u>

If a signal operates in coordinated mode, then the timing shows up in Table 7. Timing values such as lead-lag settings on Sheet 7 override the values on Sheet 3. A signal controller will not exceed the max greens from Sheet 3 nor the force-offs (when the phase is forced “off” by the clock) on Sheet 7. The cycle length shown on Sheet 7 can be directly entered into Synchro. Using the force-offs the actual phase splits can be calculated. These values can also be directly entered into Synchro.

Exhibit 13- 17 shows Table 7 for the example. In this case, Plan 2 with the 80 second cycle length is in operation during the afternoon peak. Read down the column. At 0 seconds Phases 2 and 6 are forced off. At 35 seconds Phases 4 and 8 are forced “off.” Phases 2 and 6 operate from 35 seconds around to 0 seconds on the clock (80 – 35 = 45 seconds). In this case Phases 2 and 6 are 45 seconds and Phases 4 and 8 are 35 seconds. Note how this is would be different if this intersection was not coordinated, as shown under Sheet 3.

Exhibit 13- 17 Signal Timing Sheet 7

Date sheet in effect: Date sheet voided:

Location: Hwy 97 @ Pinebrook

TABLE 7 (1 of 2)

Hardwire Conversion	Dial	1			2			3		
	Offset	1	2	3	1	2	3	1	2	3

Parameter	Key	Coordination Timing (B + Plan No. + Key)								
		1	2	3	4	5	6	7	8	9
Cycle Length	0	70	80							
Forceoffs for Phase Indicated by Key No.	1									
	2	0	0							
	3									
	4	31	35							
	5									
	6	0	0							
	7									
	8	31	35							
Offset	9	45	48							
Permissive	A	2	2							
Max. Dwell	B	30	35							

Plan Number

Coordination Timing Plans

Plan #2 is used in the example.

Sheet 8 of the master controller shows when each plan is in effect.

Plan			1	2	3	4	5	6	7	8
1	C	Lead Phases	X		X		X		X	
	D	Coord. Phases		X				X		
	E	Perm. 2 Ph.								
	F	Min. Recall								
2	C	Lead Phases	X		X		X		X	
	D	Coord. Phases		X				X		
	E	Perm. 2 Ph.								
	F	Min. Recall								
3	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
4	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
5	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
6	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
8	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
9	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								

TABLE 7 (2 of 2)

Parameter	Key 2	Coordination Timing (B + D + Key 1 + Key 2)								
Plan Number		10	11	12	13	14	15	16	17	18
Key 1		7	8	9	A	B	C	D	E	F
Cycle Length	0									
Forceoffs for Phase Indicated by Key No.	1									
	2									
	3									
	4									
	5									
	6									
	7									
	8									
Offset	9									
Permissive	A									
Max. Dwell	B									

Plan			1	2	3	4	5	6	7	8
10	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
11	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
12	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
13	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
14	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
15	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
16	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
17	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
18	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								

SHEET 7

Sheet 8 – Table 5 Time Clock Control

Table 5 shows the times that various timing plans and max greens are in effect for a particular intersection. In the absence of timing sheets from an on-street master controller (noted as "OSM" on the front of the timing sheet), the analyst will have to contact Region Traffic to verify which timing plan on Sheet 7 is in effect during the desired analysis period. Generally, during the PM peak plan #2 is in effect. The master controller would indicate in Table 5 which coordination plan shown on Sheet 7 would be operating at any given time. The function codes in the right-hand column in Table 5 can tell the analyst what maximum green applies. Code 128 is for the maximum green while Code 129 is for the max 2 green. Codes 100, 101 and 102 apply to Page 0, 1, 2 (on Sheets 3, 4 or 5) respectively, so the analyst can determine what phase timing is in effect. Codes 131 and 132 are just to tell the controller to count the traffic volume data in 15-minute intervals or 60-minute intervals, respectively.

Exhibit 13- 19 shows the timing plans in effect for the example intersection. The controller for this intersection is coordinated but is not the master. If this signal was not coordinated, Code 129 would be indicated starting at 4:30 PM, in which case the max 2 green would be used for calculating the cycle length and phase splits.

If this controller was the master controller, an event would be listed showing when each plan went into effect. Event 7 has been added to the table to illustrate this.

Exhibit 13- 18 Signal Timing Sheet 8

SHEET 8

TABLE 5 (1 of 2)

Time Clock Control (A+Code)

Event No.	Day Code	S 1	M 2	T 3	W 4	T 5	F 6	S 7	Hour	Min.	Func
1	80								81	82	83
		X	X	X	X	X	X	X	6	00	131
2	84								85	86	87
		X	X	X	X	X	X	X	8	00	132
3	88								89	8A	8B
		X	X	X	X	X	X	X	14	00	131
4	8C								8D	8E	8F
		X	X	X	X	X	X	X	18	00	132
5	90								91	92	93
			X	X	X	X	X		16	30	129
6	94								95	96	97
			X	X	X	X	X		19	00	128
7	98								99	9A	9B
			X	X	X	X	X		16	31	2
8	9C								9D	9E	9F
9	A0								A1	A2	A3
10	A4								A5	A6	A7
11	A8								A9	AA	AB
12	AC								AD	AE	AF
13	B0								B1	B2	B3
14	B4								B5	B6	B7
15	B8								B9	BA	BB
16	BC								BD	BE	BF

Time Clock Control (A+Code)

Event No.	Day Code	S 1	M 2	T 3	W 4	T 5	F 6	S 7	Hour	Min.	Func
17	C0								C1	C2	C3
18											
19											
20	CC								CD	CE	CF
21	D0								D1	D2	D3
22											
23	D8								D9	DA	DB
24											
25											
26											
27											
28											
29	F0								F1	F2	F3
30	F4								F5	F6	F7
31	F8								F9	FA	FB
32	FC								FD	FE	FF

TABLE 5 (2 of 2)

Time Clock Control (D+8+Code)

Event No.	Day Code	S 1	M 2	T 3	W 4	T 5	F 6	S 7	Hour	Min.	Func
33	80								81	82	83
34									85	86	87
35									89	8A	8B
36	8C								8D	8E	8F
37	90								91	92	93
38											
39	98								99	9A	9B
40											
41											
42											
43											
44											
45	B0								B1	B2	B3
46	B4								B5	B6	B7
47	B8								B9	BA	BB
48	BC								BD	BE	BF

Time Clock Control (D+8+Code)

Event No.	Day Code	S 1	M 2	T 3	W 4	T 5	F 6	S 7	Hour	Min.	Func
49	C0								C1	C2	C3
50	C4								C5	C6	C7
51	C8								C9	CA	CB
52	CC								CD	CE	CF
53	D0								D1	D2	D3
54	D4								D5	D6	D7
55	D8								D9	DA	DB
56	DC								DD	DE	DF
57	E0								E1	E2	E3
58	E4								E5	E6	E7
59	E8								E9	EA	EB
60	EC								ED	EE	EF
61	F0								F1	F2	F3
62	F4								F5	F6	F7
63	F8								F9	FA	FB
64	FC								FD	FE	FF

Function 131: 15 minute counts
Function 132: 60 minute counts

Function 129: Turn on Max II Green times
Function 128: Turn on Max Green times

If this signal was the the master, then the coordination plan used would be shown like this.

Function 2: Start Coordination Plan #2
(Functions 1-20 reserved for calling coordination plans)

Event numbers are for reference only.

Local TOD "Free" will override any plan received via an interconnect line.

Date sheet in effect:

Date sheet voided:

Location: Highway 97 @ Pinebrook

13.4.6 Progression Analysis

This section pertains to planning analyses as provided for traffic signal engineering investigations, corridor studies and other planning efforts. Oregon Administrative Rule (OAR) 734-020-0480 stipulates that a progression analysis is required for the approval of new or revised traffic signal systems if the proposed location is within ½ mile of an existing or possible future traffic signal. The roadway segment analyzed, to the extent possible, shall include all traffic signals in the existing or future traffic signal system. The purpose of a planning progression analysis is to ensure that a new signal or revised traffic signal will function acceptably with other nearby signals.

At the start of a project, ODOT traffic operations staff will determine whether the analyst should use the existing signal timings for all analysis scenarios or develop optimized timings for the coordinated system. If the existing timings are to be used in the analysis, Region traffic shall provide timing files, timing sheets or Synchro files of the existing settings. If optimized timings are to be developed, those settings are subject to approval by ODOT and those conditions become the baseline for all comparisons. The following settings should be optimized for each analysis scenario when the analyst is asked to use optimum coordination settings:

- Cycle Length;
- Side Street Phase Lengths (Splits);
- Phase Sequence (Lead/Lag Left Turns);
- Intersection Offsets; and
- Link Speed or Progression Speed

The optimum settings must meet the criteria established in OAR 734-020-0480 as it relates to progression analysis while also attempting to find the lowest intersection v/c ratio and minimizing queue lengths. This OAR only applies when modifications are proposed to a signal which would affect the settings of the coordination plans. Examples of these modifications are changes in cycle length, decreased green time for mainline, additional phases, longer crosswalks and intersection relocation.

Requirements for Signal Progression Analysis

For planning analysis, the following requirements must be met:

- Demonstrate acceptable existing and future traffic signal system operation during commute peak hours
- Provide for a progressed traffic band speed within 5 mph of the existing posted speed for both directions of travel during the off-peak periods and within 10 mph of the existing posted speed during peak periods. Approval by the State Traffic Engineer or designated representative shall be required where speeds deviate more than the above.
- Demonstrate sufficient vehicle storage is available at all locations within the traffic signal system without encroaching on the functional boundaries of adjacent

lanes and signalized intersections. The functional boundary of an intersection shall be determined using procedures specified by the ODOT Access Management Unit.

- Provide a common cycle length with adequate pedestrian crossing times at all signalized intersections.

The analysis must demonstrate that the additional or revised signal still allows the signal system to have a progression bandwidth as large as that required or as presently exists, for through traffic on the state highway at the most critical intersection within the roadway segment. The most critical intersection is the intersection carrying the highest through volume per lane on the state highway. Unless directed otherwise by ODOT traffic signal operations staff, the analysis should use optimized timing settings. The carrying capacity of the progression bandwidth should be estimated with the following equation:

$$\text{Bandwidth Capacity (veh/cycle)} = \frac{(\text{Bandwidth(sec)} - 4) \times (\text{Adj. Sat. Flow Rate})}{3600}$$

This capacity should be compared with the average platoon size expected to arrive at the most critical intersection for both directions of travel. The average platoon size may be found by the following simplified calculation.

$$\text{Average Platoon Size} = \frac{C * V}{3600}$$

where:

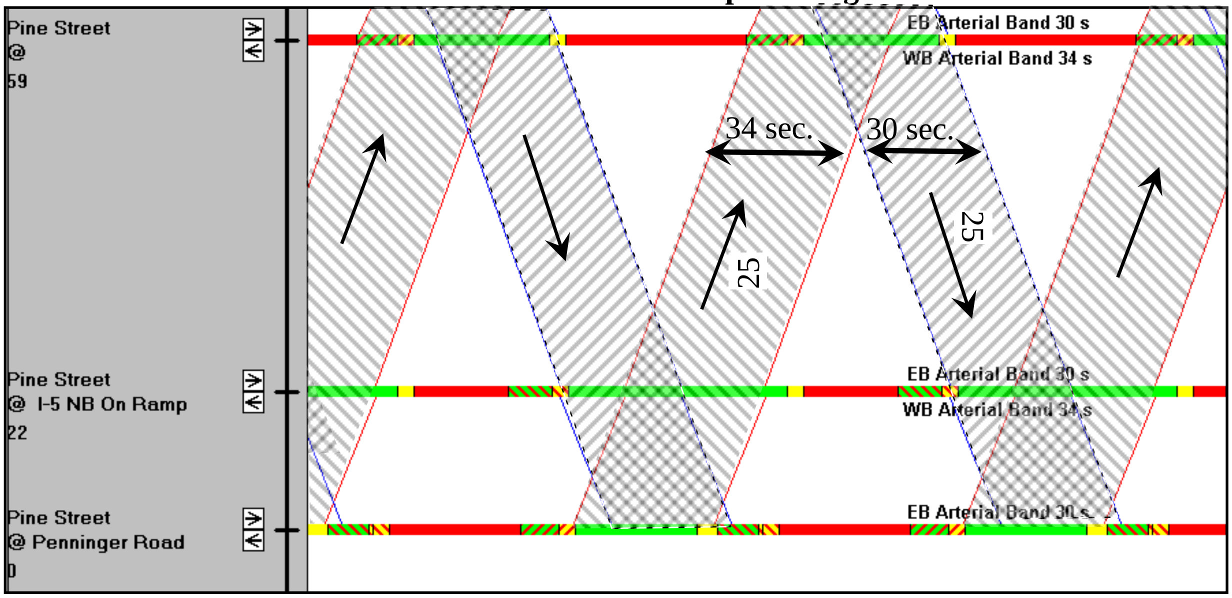
C = cycle length
V = volume (adjusted for PHF)

Complete time-space diagrams are required for each of the analysis scenarios, including the existing coordinated system. They should indicate the offsets, phasing and split times for each of the signals in the system. If using Synchro, the bandwidth shall be reported for the maximum green times or the 90th percentile arrival rates. The reported bandwidth may include green and yellow clearance times. An example time-space diagram is shown in Exhibit 13- 19.

If the analysis shows that the proposed signal will not meet the requirements of OAR 734-020-480, other alternatives should be evaluated. These may include:

- Moving the new/revised intersection;
- Reducing phases on one or more signals;
- Providing additional lanes to reduce side street green or increase mainline capacity
- Decrease side street demands through TDM measures or construction of alternative routes.

Exhibit 13- 19 Illustration of Bandwidths on a Time-Space Diagram



To implement the requirements of OAR 734-020-480, analysts may use the coordinated system software program of their choice (see Section 13.5). Hand calculations and time-space diagrams are also acceptable. Refer to Appendix 13A for settings for each of these tools.

Microsimulation programs such as SimTraffic, CORSIM and Vissim do not produce signal progression timing. They can model signal progression timing as an input. SimTraffic automatically models progression timing developed in Synchro. Refer to Chapter 15 for simulation guidance.

13.5 Estimating Queue Lengths for Signalized Intersections

For signalized movements, queue length estimates are most often recommended to be calculated using traffic analysis software. The use of software in estimating vehicle queue lengths can often be conducted simultaneously with capacity analysis, which can make it a very convenient method. There are many different software programs available that provide queue length estimates. However, caution should be used in selecting one as results may vary significantly between programs. As an example, the HCS has been found to produce consistently poor queue length estimates as compared to field measurements and should not be used for this purpose.

The minimum storage length for urban or rural left turn lanes at signalized intersections on state highways is 100 feet. Left Turn Lane layouts/dimensions are available in HDM Section 500 and Traffic Line Manual (TLM) Section 310.

Whether queue lengths have been calculated through manual methods or computer software, as a general rule-of-thumb the installation of signalized turn lanes with more than 350-feet of storage should be reconsidered through discussions with Region Traffic. In some cases, it may be preferable to install dual turn lanes with shorter storage bays.

For the estimation of queues at intersections belonging to a coordinated signal system, over-capacity conditions and areas where queue spill-back may be a problem, it is recommended that simulation software be used to report the 95th percentile queues. Refer to Chapter 15 for further information.

However, manual methods are also available that can offer acceptable estimates without requiring access to a computer. In either case, engineering judgment should be used to discern whether the results obtained are reasonable.

13.5.1 Manual Methods

Manual methods offer a practical means of estimating queue lengths with little equipment or data required. While they can produce reasonable results, unless otherwise noted, they are generally recommended for planning-level analysis, with the use of specialized software preferred for design purposes.

13.5.2 Left Turn Movement Queue Estimation Technique

A “rule of thumb” equation[3] that can be used to manually estimate queue lengths for single lane left turn movements is shown below.

Storage Length = (Volume/Number of Cycles Per Hour) x (t) x (25-feet)

Where “t” is a variable, the value of which is selected based on the minimum acceptable likelihood that the storage length will be adequate to store the longest expected queue. Suggested values are listed in Exhibit 13- 20. Typically, transportation analysis uses the 95th percentile queue.

Exhibit 13- 20 Selection of "t" Values

Minimum "t" Value	Percentile
2.0	98 %
1.85	95 %
1.75	90 %
1.0	50 %

[3] *Discussion Paper No. 10: Left-Turn Bays*, Transportation Research Institute, Oregon State University, 1996, p. 17.

It should also be noted that the value of 25-feet used in the equation represents the average storage length required for a passenger car. If a significant number of trucks are present in the turning volumes, the average storage length per vehicle should be increased, as shown in Exhibit 13- 21. This adjustment is only for manual methods; software packages may require a different adjustment.

Exhibit 13- 21 Storage Length Adjustments for Trucks

Percent Trucks in Turning Volume	Average Vehicle Storage Length
< 2%	25 ft
5%	27 ft
10%	29 ft

While the rule of thumb equation is intended for use in estimating vehicle queue lengths for single lane left turn movements, the vehicle queue lengths for double left turn lanes can be estimated by dividing the results of this method by 1.8. This value represents the assumption that queued vehicles will not be evenly distributed between the turn lanes.

13.5.3 Right Turn Movement Queue Estimation Techniques

A similar rule of thumb equation, sometimes referred to as the "red time" formula[4], is also available for signalized single lane right turn queue estimates. It is represented by the following equation.

Storage Length = (1-G/C) (V) (K) (25-feet) / (Number of Cycles Per Hour) (N_L)

where:

G = Green time provided for the right turn movement
C = cycle length
V = right turning volume
K = random arrival factor
N_L = number of right turn lanes

A value of 2 should be used for the random arrival factor (K) where right-turn-on-red is prohibited. Where right-turn-on-red is allowed, a value of 1.5 should be used.

As with the equation for left turn queue estimates, the value of 25-feet used in the equation represents the average storage length required for a passenger car. If a significant number of trucks are present in the turning volumes, the average storage length per vehicle should be increased in the same manner recommended for the left turn queue estimate using .

[4]Koepke, F. J., Levinson, H. S., *Access Management Guidelines for Activity Centers*, NCHRP Report 348, TRB, Washington, D.C., 1992, p. 99.

Another, less accurate, method for manually estimating vehicle queue lengths is using the assumption that “V” vehicles per hour per lane entering a signalized lane with a cycle length of 90 seconds will produce a “V”-foot-long queue per lane. For example, if the volume turning left from a dual left turn lane is 400 vehicles per hour, a ballpark queue length estimate would be 400/2 = 200 feet per lane.

13.6 Available Analysis Tools

A few of the computer software programs capable of performing operational, progression, and queuing analysis of signalized intersections include:

Synchro is a software application for optimizing traffic signal timing and performing capacity analysis. The software optimizes splits, offsets and cycle lengths for individual intersections, an arterial or a complete network. Synchro performs capacity analysis using current HCM methods. Synchro provides detailed time space diagrams that can show vehicle paths or bandwidths. Synchro can be used for creating data files for SimTraffic and other third-party traffic software packages. The software supports the Universal Traffic Data Format (UTDF) for exchanging data with signal controller systems and other software packages. Synchro is used in conjunction with SimTraffic for microsimulation analysis (refer to Chapter 15).

Vistro is a software application for optimizing traffic signal timing and performing capacity analysis. The software optimizes splits, offsets, and cycle lengths for individual intersections, an arterial or a complete network. Vistro performs capacity analysis using current HCM methods. Has embedded graphics to create customized reports including volume figures. Can create HCM 7th edition critical intersection v/c ratios without extra calculations or use of HCM 2000. Works well for multiple scenarios for a single intersection in the same file, such as all-way stop, two-way stop, roundabout, and signalized intersection. Can be used as a starting point to create a Vissim simulation network, or to detail a network from Visum. Refer to Appendix 8B PTV network setup guide. Good for lot of scenario management. Vistro is used in conjunction with Vissim for microsimulation analysis (refer to Chapter 15).

SIDRA is a software application for optimizing traffic signal timing and performing capacity analysis. The software optimizes splits, offsets, and cycle lengths for individual intersections, an arterial or a complete network. SIDRA performs capacity analysis using current HCM methods and offers enhancements through extensions. SIDRA will also reduce lane capacities in a network based on oversaturated upstream or downstream segments. Full flexibility to handle non-standard intersections easily (e.g. three-way stops), multiple modes (e.g. bicycles, streetcars) and related facilities (e.g. bus lane).

[Appendix 12A/13A – Software and Settings for Intersection Analysis]

References

1 Nevers, B., H. Steyn, Y. Mereszczak, Z. Clark, N. Rouphail, J. Hummer, B. Schroeder, Z. Bugg, J. Bonneson, and D. Rhodes. *NCHRP Report 707: Guidelines on the Use of Auxiliary Through Lanes at Signalized Intersections.* Transportation Research Board of the National Academies, Washington, D.C., 2011.

2 Thomas Creasey, F & Stamatiadis, Nick & Viele, Kert. (2011). Right-Turn-on-Red Volume Estimation and Incremental Capacity Models for Shared Lanes at Signalized Intersections. Transportation Research Record: Journal of the Transportation Research Board. 2257. 31-39. 10.3141/2257-04.

3 Right-Turn-on-Red Volume Estimation and Incremental Capacity Models for Shared Lanes at Signalized Intersections, F. Creasey, Nikiforos Stamatiadis, Kert Viele, Transportation Research Record: Journal of the Transportation Research Board Dec 2011, Vol. 2257, pp. 31-39

4 *Right Turn on Red Study Minnesota*, Finkelstein, Jonah et al, Spack Consulting, 2017

14 MULTIMODAL ANALYSIS

14.1 Purpose

To truly quantify the operation of a roadway segment, all the modes that use it need to be analyzed. This includes pedestrians, bicycles, transit in addition to automobiles and trucks. This chapter will eventually cover a range of different multimodal analysis types and modal considerations that will apply to plans and projects of all detail levels.

14.2 Multimodal Analysis Methodologies

The current generation of multimodal analysis methodologies are generally a perception-based rating system of the safety, comfort, and convenience of transportation facilities from the perspective of the user, whether a motorist, bicyclist, pedestrian or transit rider. The range of methodologies presented in this chapter is meant to be complementary, not competitive, and the methodologies have been tested for compatibility. There are many types of multimodal analysis methodologies available; however, not all are suitable for all applications. The overall context of the plan or project and the resulting scope of work will control the ultimate methodological choice. Some methods require very specific data which may not typically be collected in a high-level study such as a transportation system plan. Some methods are too simple and will not be able to answer the questions posed in the design of a modernization project.

Applicability of multimodal analysis methods by project type is illustrated in Exhibit 14-1. As the application increases in level of detail, more specific questions can be addressed, but the analysis will require more data and resources. Regardless of method applied, it is important to include some sort of multimodal analysis on all analysis efforts.

Exhibit 14-1 Multimodal Analysis Tool Applications[1]

		Increasing Detail		
		Qualitative Multimodal Assessment (QMA)	**Level of Traffic Stress (LTS)[2]**	**Multimodal Level of Service (MMLOS)**
Increasing Project Complexity	Regional Transportation Plan (RTP)	❍	●	
	Transportation System Plan (TSP)	●	●	
	Facility Plan/Interchange Area Management Plan (IAMP)	❍	❍	●
	Project Development		❍	●
	Development Review		❍	●

[1]Solid circles represent the preferred methodology. Outlined circles represent where methodology can also be used.
[2]Use of LTS for project development and development review should be limited to a screening-based analysis to quickly identify existing and future needs

Any project or plan could use any single level or multiple levels of multimodal analysis, but certain levels of analysis are more suited to a particular application. For example, Level of Traffic Stress (LTS) could be used at a system level to identify key locations, which then can be analyzed further using Multimodal Level of Service (MMLOS).

The primary tool for Regional Transportation Plans (RTP) is LTS as this methodology can be easily adapted to use travel demand model inputs or can be generalized enough to apply to a whole region without requiring too much data and effort. The Qualitative Multimodal Assessment (QMA) can be used to fill in other modes that are not covered by LTS. These methods require limited data, most of which can be obtained from existing inventories, aerial photography, or from “windshield” field surveys. These methods will be able to identify areas of concern whether in system connectivity (LTS) or in operations (Qualitative Multimodal Assessment).

Transportation System Plans (TSP) have enough detail in the inventory and analysis to provide for adequate QMA and/or LTS analyses. Each mode should only be analyzed using one methodology, even if more than one methodology is being used in the effort.

More detailed planning efforts such as facility plans, and Interchange Area Management Plans (IAMP) typically will use MMLOS-based methods as there is a need for more objective results especially in comparisons of alternatives. This level usually will have a higher amount of detailed data available which is consistent with the smaller analysis segments and more specific detail required. Most elements could be obtained without doing a detailed field inventory, provided that unobstructed, high-quality aerials that can be used for the basis of measurements are available. These data levels will make comparison easier across concepts and time periods with less subjectivity than with QMA. LTS and/or QMA can still be used if a plan will be relatively standalone. Plans that need to be consistent with future potential project development efforts especially with environmental assessments or environmental impact statements should use MMLOS-based methods for alternatives and limit LTS/QMA to screening analysis.

Project development requires the highest amount of data as objective design-level decisions need to be supported. The MMLOS methods are the most rigorous and commensurate with the typical available data. LTS can also be used as an initial screening measure to identify areas with existing or future needs. Analysis with the MMLOS segment and intersection methodologies even with appropriate ODOT defaults will take more effort and have a greater chance of needing additional specific field inventory data.

Assessing multimodal impacts in development review will typically involve use of LTS to quickly identify existing/future needs or development impacts and then using MMLOS techniques to identify mitigation scenarios. The urban context will need to be considered as the more urban an area is, even a standard zone change (i.e. residential to commercial) may require more detail. Transportation Planning Rule (TPR) -0060 analysis for a plan amendment can likely rely on more use of LTS (however, transit is only available at the MMLOS level) Transportation Impact Analyses (TIA) would likely need to primarily use MMLOS techniques in order to capture the specific scenario details.

While the designation of Multimodal Mixed Use (MMA) areas are based solely on safety concerns, once the designation is in place, non-automobile multimodal impacts can still be analyzed. Depending on the level of effort desired for a plan/project/TIA that involves a MMA, the multimodal analysis could use any of the methodologies.

14.3 Qualitative Multimodal Assessment

The Qualitative Multimodal Assessment (QMA) methodology is based on work done by David Evans and Associates and generally uses the principles of the full 2010 and later Highway Capacity Manual (HCM) MMLOS but was modified to stay consistent as much as possible with the more objective methods presented later in this chapter. This methodology uses the roadway characteristics and applies a context-based subjective “Excellent/Good/Fair/Poor” rating. This method is best applied when comparing different alternatives side-by-side to each other but can also be used with a single scenario to compare the proposed improvement to existing conditions and to applicable standards. For example, a six-foot sidewalk is standard in a residential area and would be rated

Good (or Excellent if it had a buffer). Ratings can be "averaged" to obtain one for every mode, or they can be shown for every element if more detail is desired e.g. in a technical appendix. Factors can be documented for each rating in tabular form such as shown in Exhibit 14-2. This will allow for easy reference and consistent application. This method is most appropriate when one or more of the following conditions apply:

- The subject roadway does not easily divide into segments with uniform characteristics between intersections.
- The subject roadway has rural/suburban characteristics with infrequent or no signal control, where the MMLOS methodology is not applicable.
- Insufficient data are available to complete a MMLOS analysis
- Future alternatives may not have enough detail to properly quantify roadway characteristics required by other methodologies.

> *If a roadway has limited facilities because they are provided on parallel roadways (i.e. bike boulevards), consideration should be given to also applying the methodology to that parallel facility. This way the complete picture of the multimodal facilities offered along a corridor can be shown.*

Exhibit 14-2 Sample Factor Documentation

Table 3: Transit Qualitative Multimodal Assessment Methodology

Category	Excellent	Good	Fair	Poor
Frequency and On-Time Reliability	<15-minute headways	15 to 30-minute headways	30 to 60-minute headways	60+ minute headways
Schedule Speed/ Travel Times	<20% slower than driving	20% to 40% slower than driving	40% to 60% slower than driving	>60% slower than driving
Transit Stop Amenities	Shelter with bench and sign	Bench with sign	Sign with waiting area	No sign and/or no waiting area
Connecting Pedestrian/Bicycle Network	Wide shoulders or bike lanes and sidewalks with frequent crossing	Standard shoulders or bike lanes and sidewalks with crossings	Substandard shoulders or bike lanes and sidewalks with no crossing	No shoulders, bike lanes, or sidewalks and no crossings

The full HCM MMLOS is most applicable to urban roadways with uniform segments broken up by signalized intersections. The MMLOS only evaluates segments bracketed by signalized intersections, but the qualitative assessment can be done at all types of traffic control (e.g. roundabouts). Many communities do not have any signals or have too few signals to make the full HCM MMLOS method usable. This methodology allows for a multimodal analysis at a reasonable cost without requiring intensive data gathering. For most planning efforts, design details are not generally available until later, within phases such as refinement plans or project development, so it can be difficult to properly create the MMLOS inputs. All the elements below should be considered for each mode. However, not all the elements below will be contextually applicable in every community (i.e. volumes not sufficient for traffic signals or all-way stop control) so deviations should occur as necessary but need to be documented.

14.3.1 Pedestrian

On segments, the following factors are considered:

- **Outside travel lane width:** Wider travel lanes are rated better than narrower travel lanes because of the larger buffer space between vehicles and pedestrians.
- **Bicycle lane/shoulder width:** The addition of bicycle lanes or shoulders creates greater separation between vehicles and pedestrian traffic and acts as a buffer. Wider or more separated (e.g. buffered or separated bike lanes) facilities are rated better than narrow or non-existent facilities.
- **Presence of buffers (landscaped or other):** Buffer presence that separates pedestrians from traffic results in an improved rating. Wider buffers are rated better than narrower or non-existent ones.
- **Sidewalk/path presence:** The presence of sidewalks or paths will rate higher versus shoulders or no facilities at all. Wider sidewalks/paths rate better than narrower or non-existent ones.
- **Lighting:** The presence of lighting, whether roadway or pedestrian-scale, is rated better than roadways without lighting.
- **Travel lanes and speed of motorized traffic:** Less travel lanes and lower vehicle speeds will rate higher than more lanes and higher speeds.

At intersections, the following factors are considered:

- **Traffic control:** Intersections with a traffic signal or all-way stop control, or with marked crosswalks are rated better than locations with only two-way stop control or locations without marked crosswalks.
- **Crossing width:** Fewer turn or through travel lanes to be crossed is rated better than more turn/though lanes because the exposure to traffic and potential conflicts are less.
- **Median islands:** The presence of a median island is rated better than no islands as two-stage crossings significantly improve the associated safety and ease when using a crossing.

14.3.2 Bicycle

On segments, the following factors are considered:

- **Preferred Bicycle facility type:** Bicycle facilities with greater separation from vehicles rate higher than shared or less separated facilities. Wider bicycle facilities rate better than narrower or non-existent ones. Ideally, arterials (7000+ AADT) have separated bicycle lanes with vertical barriers, buffered bike lanes, or shared/multi-use paths); low-speed collectors (1500-7000 AADT) have buffered or standard bike lanes; and local streets have shared facilities. This will vary by location, context, and size of the community. For more information, please refer to Section 306 in the Highway Design Manual.
- **Shoulder presence/width:** Shoulders serve bicyclists in the absence of marked bike lanes, and wider shoulders rate higher than narrower or non-existent ones.
- **Outside travel lane width:** Wider travel lanes are rated better than narrower travel lanes on higher volume/speed roadways (i.e. arterials) because of the larger buffer space between vehicles and bicyclists. On low volume/speed urban streets, narrower lanes are better than wider lanes for better shared lane utilization when sharrow markings are used, since the bicyclist is more likely to take the center of the lane rather than potentially being squeezed to the side.
- **Grade:** Level roadways/shallow grades are rated better than roadways with steep grades.
- **Pavement condition:** Poor pavement condition or obstacles (such as sewer grates, skewed railroad crossings, or in-street trackage) affect bicycling so better pavement condition and lack of obstacles will rate better than poor condition and many obstacles.
- **Obstructions:** Shoulders/bike lanes free of debris and other temporary obstacles such as construction barricades are rated higher than ones that are usually littered with gravel, glass, or frequently blocked.
- **On-street parking:** No parking or low parking utilization is rated better than high utilization and turnover rates because of potential conflicts with bicycles. Back-in parking is rated better than front-in parking. Parallel parking is rated better if it includes a buffer from the bike lane.
- **Travel lanes and speed of motorized traffic:** Less travel lanes and lower vehicle speeds will rate higher than more lanes and higher speeds.

At intersections, the following factors are considered:

- **Traffic control:** Intersections with a traffic signal or all-way stop control with crosswalks are rated better than locations with only two-way stop control or locations without crosswalks. Intersections with bike signals are rated the highest.
- **Crossing width:** Fewer turn or through travel lanes to be crossed is rated better than more turn/though lanes because the exposure to traffic and potential conflicts are less.

14.3.3 Transit

The following factors are considered for transit:

- **Frequency and on-time reliability:** More frequent service and higher on-time schedule reliability are better than less frequent service and less reliable schedules.
- **Schedule speed/travel times:** Faster average peak hour schedule speeds and travel times are rated better than slower speeds and longer travel times.
- **Transit stop amenities:** The presence of shelters, benches, and lighting is rated better than stops with limited or no amenities. High-rated stops should have adequate boarding/maneuvering areas.
- **Connecting pedestrian/bike network:** Stops connected to a network of paths or sidewalk-equipped streets with improved crossings are better than those with no pedestrian facilities.

14.3.4 Auto

The following factors are considered for the auto mode:

- **Volumes/queues:** Lower observed volumes and queues are rated higher than higher volumes/queues on mainline and side-street intersection approaches. The number of lanes and functional class can be used as a surrogate to actual volumes if they are not readily available at this stage.
- **Safety:** Roadway conditions that provide for a decreased chance of crashes such as having illumination, longer intersection/driveway spacing, lower speeds, turn lanes and greater separation between fixed objects are better than conditions that may promote more crashes. The values of the seven criteria below can be aggregated to obtain a single value for safety if desired.
    - Lighting: Roadways with lighting are rated better than ones without.
    - Driveway density: Lower driveway density is rated better than higher driveway density
    - Intersection spacing: Longer intersection spacing distances are rated higher than shorter intersection spacing.
    - Speed: Lower speeds are rated higher than higher speeds
    - Fixed objects: Roadways with fewer fixed objects (trees, signs, barriers, etc.) close to the roadway (less than 25 feet) are rated higher than ones with more.
    - Median/traffic separators: Presence of a median and/or traffic separators are rated higher than segments without.
    - Turn Lanes: Intersection/driveway approaches with turn lanes are rated higher than approaches without turn lanes.

Example 14-1 Qualitative Multimodal Application

This example is based on work by David Evans and Associates on the OR99 Corridor Plan but has been simplified and modified from the original analysis to illustrate the methodology.

The study area on OR99 in Talent, Oregon south of Medford is approximately one mile in length with a single traffic signal at Rapp Road. South of Rapp Road the area becomes increasingly less dense and suburban/rural to the southern city limits. There are limited bicycle, pedestrian, and transit facilities. OR99 is currently a four-lane undivided section, so a five-lane and a three-lane scenario was developed for analyzing potential future project alternatives. Conditions along the OR 99 corridor (limited signalization, limited data, and difficult to subdivide into homogenous segments) support the use of the QMA methodology to assess the multimodal aspects of existing and future scenarios. The table at the end of the example summarizes the analysis results.

Pedestrian & Bicycle Facilities - Existing Conditions
No existing separate pedestrian or bicycle facilities are in the corridor except the Bear Creek Greenway Trail located to the east of OR99 but not adjacent to the highway in this location. Pedestrians must walk on the shoulder and bicycles must share the right lane with vehicles, so the pedestrian and bicycle facilities are rated poor throughout.

Pedestrian & Bicycle Facilities - Future Scenarios
Both future scenarios would add a sidewalk or path to each side of the highway and would include a buffer on at least one side of the highway and bike lanes on both sides. The segments were rated as good for these conditions. The less travel lanes in the three-lane scenario rated higher than the five-lane scenario as it creates a better environment for bicycles and pedestrians. At intersections, the three-lane scenario was rated better than the five-lane scenario because there would be fewer travel lanes for a pedestrian or a bicyclist to cross.

Transit Facilities – Existing and Future
Conditions are not expected to change in any substantial way from existing conditions. While connectivity to stops would increase, frequency, reliability, speed and travel time will be unchanged, therefore positive change will not be enough to change the grade overall.

Auto Facilities – Existing and Future
The assessment reflects the volumes and the safety evaluation. Analysis of existing conditions and both future scenarios resulted in relatively lower volumes with shorter queues on side street approaches. The low volumes minimize conflicts between through and turning vehicles, so the safety conditions are relatively close for all scenarios.

Segment/ *Intersection*	**Mode**			
	Pedestrian	**Bicycle**	**Transit**	**Auto**
Existing Conditions – (Four Lanes)				
Rapp Rd to Arnos Rd	Poor	Poor	Fair	Good
OR99 at Arnos Rd	Poor	Poor	Fair	Good
Arnos Rd to Creel Rd	Poor	Poor	Fair	Good
OR 99 at Creel Rd	Poor	Poor	Fair	Good
Scenario 1 - Five lanes				
Rapp Rd to Arnos Rd	Good	Fair	Fair	Good
OR99 at Arnos Rd	Fair	Fair	Fair	Good
Arnos Rd to Creel Rd	Good	Fair	Fair	Good
OR 99 at Creel Rd	Fair	Fair	Fair	Good
Scenario 2 – Three lanes				
Rapp Rd to Arnos Rd	Good	Good	Fair	Good
OR99 at Arnos Rd	Good	Good	Fair	Good
Arnos Rd to Creel Rd	Good	Good	Fair	Good
OR 99 at Creel Rd	Good	Good	Fair	Good

14.4 Bicycle Level of Traffic Stress

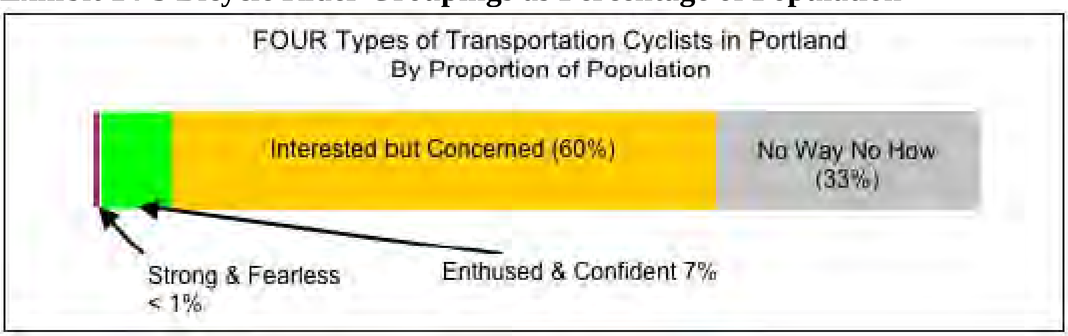
The Bicycle Level of Traffic Stress (BLTS) methodology breaks road segments into four classifications for measuring the effects of traffic-based stress on bicycle riders. The original methodology can be obtained from the paper, "*Low Stress Bicycling and Network Connectivity*", Mineta Transportation Institute, Report 11-19, May 2012. The version of the methodology described in this section has been modified from the original to correct inconsistencies in the tables, allow for additional intersection and bicycle features, and allow for more flexibility and engineering judgment in practice. Support for left turn lanes, one-way streets, roundabouts, buffered and separated bike lanes, and shared lane markings have been added. A methodology for high-speed rural applications has been added since the original was for primarily urban areas. More detailed information on changes is provided in the specific topic areas.

This measure of traffic stress quantifies the perceived safety issue of being in close proximity to vehicles whether on a spacing distance or speed basis. The methodology does not include require explicit consideration of traffic volumes as the proximity stress is present regardless of how much traffic happens to be occurring at that time. For example, a bicyclist travelling on a higher-speed arterial in the early morning hours without any bike lanes will still be having traffic (even though volumes are low) passing by closely and at high speeds. This bicyclist will experience higher stress than one riding in a buffered bike lane under the same conditions because the proximity to traffic is greater. An analogy to this would be as a pedestrian, having sidewalks with landscaped buffers is much more pleasant to walk on than curb-tight sidewalks right next to moving traffic.

This methodology allows a quick assessment of system connectivity without going into the data requirements (i.e. traffic volumes) and calculations of the HCM Bicycle Multimodal Level-of-Service (MMLOS) method. BLTS is well suited for high-level plans such as corridor and transportation system plans (TSP). This method can also be used in detailed refinement-level plans and projects as a screening or flagging tool. Most of the data should be available as part of TSP inventories and/or supplemented with aerial photos. Depending on the community, TSP inventories may be limited to collector and arterial streets. Field inventory may still be needed to verify elements or when vegetation or other obstructions obscure views. Traffic counts/daily volumes are not required except for higher-speed rural applications. The methodology is designed for urban application but can also be used for rural locations. The methodology is visual based so the results can be easily communicated from the engineer to other agency and local government staff and the general public.

The tendencies of the general population to choose the bicycle as a mode and make route choices can be broken into four overall groupings based on City of Portland, [Oregon] surveys (Exhibit 14-3). While the percentages may change in different cities and rural/suburban areas, the groupings are still applicable.

Exhibit 14-3 Bicycle Rider Groupings as Percentage of Population



Source: "*Four Types of Transportation Cyclists in Portland*" by Roger Geller (2006)

The smallest group, "Strong and Fearless" represents people who will travel by bike under any condition and on any roadway. A second group, the "Enthused and Confident" represents advanced cyclists who travel on most roadways but avoid high volume and speed conditions. Over half of the population falls into the largest group, "Interested but Concerned" who would ride if roadway conditions were perceived to be safe enough. The last group, representing around a third of potential riders, is "No Way No How", who will not ride under any circumstances. More information on this methodology can be obtained from "*Four Types of Transportation Cyclists in Portland*" by Roger Geller (2006) and "*Four Types of Cyclists? Examining a Typology to Better Understand Bicycling Behavior and Potential*" (2012) Jennifer Dill and Nathan Winslow McNeil.

The Bicycle Level of Traffic Stress methodology adopted the above groupings, as the perception of user comfort being impacted by the proximity of vehicular traffic is one of the major decisions on whether one chooses this mode of travel. Further separation generally means less stress for users. The smallest group “Strong and Fearless” (avid cyclists and/or commuters) will travel most routes under any conditions, weather, light level, etc. and will tolerate the highest stress levels. On the other end, the “Interested but Concerned” group (casual or inexperienced riders) has little stress tolerance and will only accept the routes with the greatest perceived safety (separation). The research further breaks the largest “Interested but Concerned” group into adult and children riders where children require more safety awareness than adults along roadways and at intersections. Lastly, the “No Way No How” group was not included since the methodology concentrates on the current or potential bicycle-riding population.

Different trip purposes could have multiple ranges of acceptable stress levels for the same person. Someone making a work-based trip will likely have a greater stress tolerance than if they were riding merely for recreation. Going for a bike ride might mean a low stress tolerance for some riders, but they might accept a much higher stress level if they are on their way to work. Familiarity with the route, costs associated with driving and parking a car daily near a worksite, available bicycle infrastructure, vehicle availability/ownership, and other factors can influence someone’s maximum acceptable level of traffic stress.

The overall rider groupings are translated into four levels of traffic stress (LTS) classifications.

- LTS 1 – Represents little traffic stress and requires less attention, so is suitable for all cyclists. This includes children that are trained to safely cross intersections (around 10 yrs. old/5th grade) alone and supervising riding parents of younger children. Generally, the age of 10 is the earliest age that children can adequately understand traffic and make safe decisions which is also the reason that many youth bike safety programs target this age level. Traffic speeds are low and there is no more than one lane in each direction. Intersections are easily crossed by children and adults. Typical locations include residential local streets and separated bike paths/cycle tracks.

- LTS 2 – Represents little traffic stress but requires more attention than young children would be expected to deal with, so is suitable for teen and adult cyclists with adequate bike handling skills. Traffic speeds are slightly higher, but speed differentials are still low, and roadways can be up to three lanes wide for both directions. Intersections are not difficult to cross for most teenagers and adults. Typical locations include collector-level streets with bike lanes or a central business district.

- LTS 3 – Represents moderate stress and is suitable for most observant adult cyclists. Traffic speeds are moderate but can be on roadways up to five lanes wide in both directions. Intersections are still perceived to be safe by most adults. Typical locations include low-speed arterials with bike lanes or moderate speed non-multilane roadways.

- LTS 4 – Represents high stress and suitable for experienced and skilled cyclists. Traffic speeds are moderate to high and can be on roadways from two to over five lanes wide for both directions. Intersections can be complex, wide, and or high volume/speed that can be perceived as unsafe by adults and are difficult to cross. Typical locations include high-speed or multilane roadways with narrow or no bike lanes.

14.4.1 Additional Rider Factors

The general Bicycle Level of Stress methodology does not include all the other comfort factors that may be important to bicycle riders that should be taken into consideration in application such as grades and surface conditions. Section 14.4.11 contains optional comfort measures that can be introduced as needed to add these details. Sometimes, systematic deviations are required to properly capture the overall context of a community or are relevant to a particular project area. For example, there have been cases, where the BLTS ends up being the same on all facilities as all roadways are 25 mph and two lanes, but there are noticeable differences between the subject roadways. New tables and BLTS adjustments can be created; however, as some of these can be subjective, adequate documentation needs to be provided outlining the reasons for the deviations. Where possible, these deviations should be explained in the methodology and assumptions memorandum before analysis begins but may require a separate memorandum if issues come up during the analysis.

Congested conditions can also be considered if they add difficulty to getting gaps in traffic to get into a right or left turn lane for instance. Roadway locations with either a documented (reported total bike crashes including any injury or fatal ones) or a perceived (near misses, known unreported crashes) crash history should be flagged for reference. Roadways where biking is prohibited, such as along certain segment of urban freeways designated in OAR 734-020-0045, should also be noted.[1]

14.4.2 BLTS Targets

A target level of traffic stress for the bikeway system may be identified to maximize the bicycle mode share with the available resources. A BLTS 2 is often used as the target as it will typically appeal to the majority of the potential bike-riding population and

[1] Shoulders should be available for pedestrians to access the nearest exit during mechanical incidents or after collisions, but it is not preferred to accommodate bicycle or pedestrian travel on shoulders on urban limited access facilities. Instead, pedestrian and bicycle travel should be accommodated on a parallel multi-use path, separated bikeway, or parallel streets. Limited access highway shoulders should only be used as a primary pedestrian and bicycle accommodation in low volume rural areas and/or where physical constraints and sparse surrounding network make a parallel route infeasible.

maximize the available bicycle mode share. Other BLTS levels may also be used as targets depending on a jurisdiction's needs and maturity of the available bike network.

When evaluating networks near schools (within ¼ mile), the desirable level of traffic stress is BLTS 1 since BLTS 1 is targeted at 10-yr olds (5th grade) or parents of younger children. Elementary school-age children should be able to travel between homes and schools without having to cross arterial streets (LTS 3 and 4). Ideally, elementary schools and their related attendance boundaries should be placed to allow at least a few BLTS 1 routes. Middle and high school placement may not allow only BLTS 1 routes, but routes should be no more than BLTS 2 since older students can use these without difficulty.

14.4.3 BLTS Criteria

The traffic stress criteria in the BLTS methodology are applied from lookup tables in three categories: segments, intersection approaches, and intersection crossings. Depending on the community context and the detail level desired, segments can be block-by block or be between higher functionally classified roadways (arterials or collectors). Segments are typically considered to be two-way but there are areas where conditions are not the same on each side of the street (i.e. parking only on one side). Both directions can be reported separately, or the worst direction reported. One-way streets might have different conditions on either side, so either both sides or the worst condition should be reported. The overall methodology can usually be simplified based on the general consistency of facility types, as certain elements (i.e. no turn lanes, no bike lanes, limited speeds, etc.) may not exist in a particular community.

The methodology uses the worst overall BLTS value for each overall segment. For example, if a segment has a BLTS 2 but there is an intersection approach at the end of the segment at BLTS 4, then the whole segment is coded BLTS 4. The same applies for entire routes which are typically reported in a single direction between two points of interest and can contain many segments and intersections. It is likely that the BLTS will be different (i.e. right turn lane vs. left turn lane) in the two directions, so both directions should be reported. One poor crossing at BLTS 4 will render a route unacceptable to most people even though the rest of the route is at BLTS 2.

14.4.4 BLTS Segment Criteria

The BLTS segment criteria are broken into three classes: physically separated paths and lanes, standard bike lanes, and without bike lanes (mixed traffic). The physically separated paths include bike paths and separated bike lanes which may be separated from motor vehicles by landscaped buffers, curbs, bollards, bioswales, on-street parking, or other vertical delineators. Physically-separated bike paths and lanes (assuming full bike standards) are generally classified as BLTS 1 regardless of the number of lanes or speed on a segment. Note that separated bike lanes may be combined with buffered or standard bike lanes on separate sides of a street (either one or two-way).

Marked bike lanes have different criteria depending on whether they are adjacent to a parking lane, as shown in Exhibit 14-4 and Exhibit 14-5. These exhibits are formatted differently from the original methodology to fix inconsistencies with roadways with bike lanes having higher stress levels than roadways without bike lanes. In addition, slight changes were made so bike lane width makes a difference in the lower stress levels. Buffered bike lanes have been added to Exhibit 14-5 to account for their increased positive separation effects. Existing bike lanes without a useable width of at least 4’ (caused by striped too-narrow widths, drainage grates, poor curb-gutter/pavement interfaces, etc.) should be recorded as mixed traffic instead. Bike lanes less than 4’ do not provide adequate separation from motor vehicles.

The criteria are based on through lanes per direction, the sum of the width of the bike and parking lanes, speed limit or prevailing speed, and any bike lane blockage (in commercial areas from driveways, loading zones, stopped buses, or parking maneuvers). The methodology uses the worst overall BLTS value for each overall segment: the dimension with the worst level of stress governs. For example, a roadway with one lane per direction, 25 mph, but has frequent bike lane blockages will be at BLTS 3 which overrides the BLTS 1 values of the other components. The segment length default for urban areas would typically be on a block-by-block basis but could be defined on a larger scale if desired. A trade-off for longer segment lengths will be a loss of detail which could make it harder to determine the controlling worst condition (e.g. a missing section may not have the same influence in a longer segment versus the default length).

Exhibit 14-4 BLTS Criteria for Segment with Bike Lane and Adjacent Parking Lane

Prevailing or Posted Speed	1 Lane per direction			≥2 lanes per direction	
	≥ 15’ bike lane + parking	14’ – 14.5’ bike lane + parking	≤ 13’ bike lane + parking or Frequent blockage[1]	≥ 15’ bike lane + parking	≤ 14.5’ bike lane + parking or Frequent blockage[1]
≤25 mph	BLTS 1	BLTS 2	BLTS 3	BLTS 2	BLTS 3
30 mph	BLTS 1	BLTS 2	BLTS 3	BLTS 2	BLTS 3
35 mph	BLTS 2	BLTS 3	BLTS 3	BLTS 3	BLTS 3
≥40 mph	BLTS 2	BLTS 4	BLTS 4	BLTS 3	BLTS 4

[1]Typically occurs in urban areas (i.e. delivery trucks, parking maneuvers, stopped buses).

Exhibit 14-5 BLTS Criteria for Segment with Bike Lane, no Adjacent Parking Lane

Prevailing or Posted Speed	1 Lane per direction				≥2 lanes per direction	
	≥ 7’ (Buffered bike lane)	5.5’ – 7’ Bike lane	≤ 5.5’ Bike lane	Frequent bike lane blockage[1]	≥ 7’ (Buffered bike lane)	<7’ bike lane or frequent blockage[1]
≤30 mph	BLTS 1	BLTS 1	BLTS 2	BLTS 3	BLTS 1	BLTS 3
35 mph	BLTS 2	BLTS 3	BLTS 3	BLTS 3	BLTS 2	BLTS 3
≥40 mph	BLTS 3	BLTS 4	BLTS 4	BLTS 4	BLTS 3	BLTS 4

[1]Typically occurs in urban areas (i.e. delivery trucks, parking maneuvers, stopped buses).

Mixed traffic conditions are roadways without any bike markings (including widened shoulders not marked as bike lanes), or existing bike lanes with useable width < 4'. Designated bike boulevards or "sharrow" markings present also fall under mixed traffic conditions. Markings and signs give bicyclists more perceived safety and warn drivers about bicycles potentially being in the roadway, which tends to lower overall speeds. Mixed traffic segment criteria for urban/suburban sections are based on the speed limit or the prevailing speed if different, and the number of lanes by direction, and the two-way average daily traffic (ADT) or functional class if ADT is not available as shown in Exhibit 14-6 and Exhibit 14-7[2].

While not a main focus of this method, biking is allowed on Interstate highways and freeways in certain urban areas. However, even with wider shoulders, due to high vehicle and heavy truck volumes as well as high-speed conflicts at ramps, the level of traffic stress should be coded as BLTS 4 for these sections. Certain freeway sections in Portland and Medford prohibit bicycles as shown in Oregon Administrative Rule OAR 734-020-0045, so these sections should show the BLTS as not applicable ("N/A").

Exhibit 14-6 Criteria for Urban/Suburban Mixed Traffic Segment – 30 mph or less

Number of Lanes	ADT (vpd)[1]	Functional Class	Posted or Prevailing Speed (mph)		
			≤20	25	30
Unmarked Centerline	≤750	Local	BLTS 1	BLTS 1	BLTS 2
	750 - ≤1,500	Local /Collector	BLTS 1	BLTS 1	BLTS 2
	1,500 - ≤3,000	Collector	BLTS 2	BLTS 2	BLTS 2
	>3,000	Arterial	BLTS 2	BLTS 3	BLTS 3
1 through lane per direction	≤750	Local	BLTS 1	BLTS 1	BLTS 2
	750 - ≤1,500	Local /Collector	BLTS 2	BLTS 2	BLTS 2
	1,500 - ≤3,000	Collector	BLTS 2	BLTS 3	BLTS 3
	>3,000	Arterial	BLTS 3	BLTS 3	BLTS 3
2 through lanes per direction	≤8,000	Arterial	BLTS 3	BLTS 3	BLTS 3
	>8,000	Arterial	BLTS 3	BLTS 3	BLTS 4
3+ though lanes per direction	Any ADT	Arterial	BLTS 3	BLTS 3	BLTS 4

[1]ADT is both directions for two-way streets. For one-way streets use 1.5*ADT.

[2] Furth, P., Level of Traffic Stress Criteria for Road Segments, Version 2.0, Northeastern University, June 2017.

Exhibit 14-7 BLTS Criteria for Urban/Suburban Mixed Traffic Segment – 35 mph or more

Number of Lanes	ADT (vpd)[1]	Functional Class	Posted or Prevailing Speed (mph)		
			35	40	>45
Unmarked Centerline	≤750	Local	BLTS 2	BLTS 3	BLTS 3
	750 - ≤1,500	Local /Collector	BLTS 3	BLTS 3	BLTS 4
	1,500 - ≤3,000	Collector	BLTS 3	BLTS 4	BLTS 4
	>3,000	Arterial	BLTS 3	BLTS 4	BLTS 4
1 through lane per direction	≤750	Local	BLTS 2	BLTS 3	BLTS 3
	750 - ≤1,500	Local /Collector	BLTS 3	BLTS 3	BLTS 4
	1,500 - ≤3,000	Collector	BLTS 3	BLTS 4	BLTS 4
	>3,000	Arterial	BLTS 3	BLTS 4	BLTS 4
2 through lanes per direction	≤8,000	Arterial	BLTS 3	BLTS 4	BLTS 4
	>8,000	Arterial	BLTS 4	BLTS 4	BLTS 4
3+ though lanes per direction	Any ADT	Arterial	BLTS 4	BLTS 4	BLTS 4

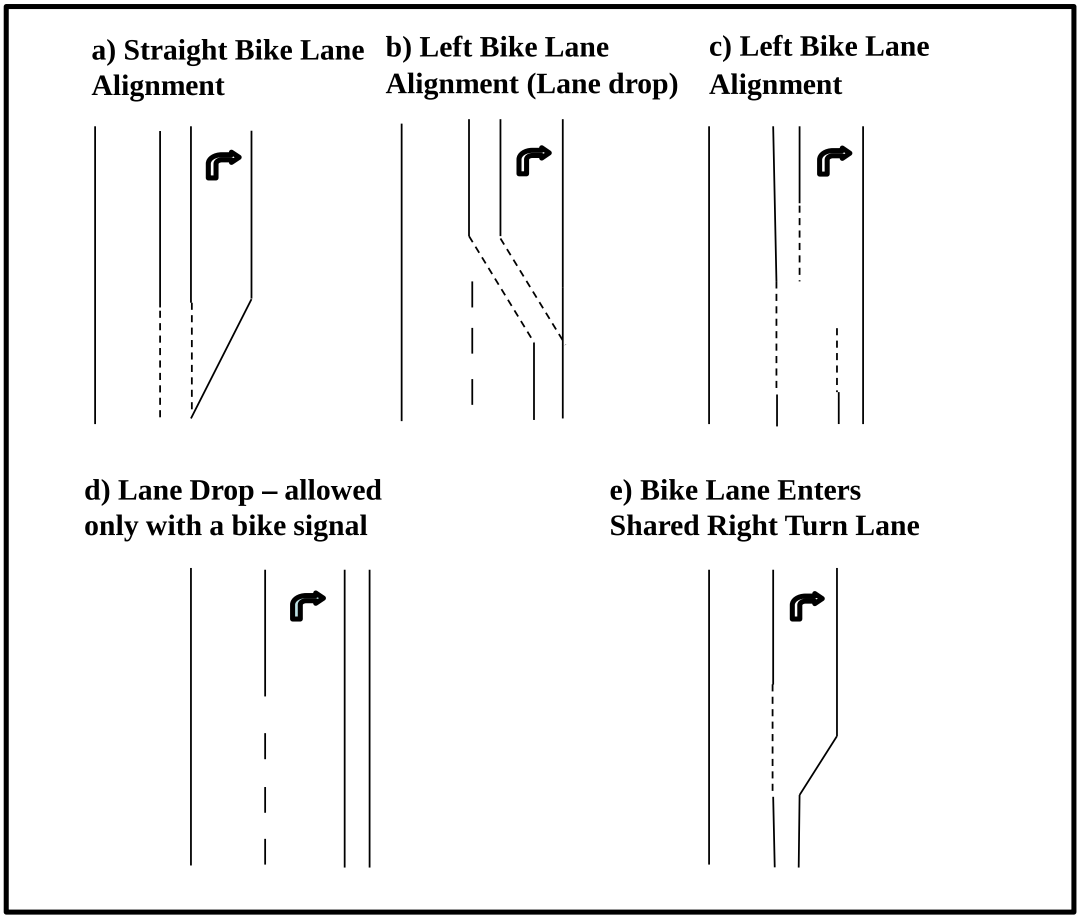
[1]ADT is both directions for two-way streets. For one-way streets use 1.5*ADT.

14.4.5 BLTS Intersection Approach Criteria

Intersection approach criteria are based on the presence and type of right or left turn (vehicular) lanes. If there are no turn lanes on an approach, then this portion of the methodology is skipped.

ODOT Bicycle & Pedestrian Design Guide standards have the right turn lane to the right of the bike lane, so the bike lane continues straight and requires vehicles to move into the turn lane and yield to bicyclists across a marked dashed bike lane in advance of the intersection (see Exhibit 14-8a). Locations where the through travel lane becomes a right turn lane (lane drop) may have a more stressful design where the bike lane shifts to the left while the travel lane continues straight (Exhibit 14-8b or c). Exhibit 14-8b shows an older marking style while Exhibit 14-8c is the current version. In this case, the bike lane cannot be to the right of a right-turn lane unless controlled by a separate bicycle signal (see Exhibit 14-8d), as the through bicycle lane would directly conflict with the right turn lane with the potential for many “right-hook” type crashes. Other intersection designs may have the bike lane end where the right turn lane begins (e.g. T-intersections, roundabouts) and re-appear on the other side of the intersection (Exhibit 14-8e).

Exhibit 14-8 Right Turn Lane Types



The right turn BLTS criteria are based on whether the bike lane stays straight or shifts to the left, turn lane length and turning speed. The longer the turn lane, the longer a bicyclist will have traffic on both sides in close proximity if continuing straight or mixing with traffic if turning right. When the bike lane stays straight, turn lanes of 150' or less (100' is typical for most urban applications) and low turning speed (15 mph is a common for most residential and commercial areas) will have a BLTS 2 as seen in Exhibit 14-9. Longer turn lanes, higher turning speeds or at skewed intersections will result in a BLTS 3 rating. Turn lanes more than 300' can create a "sandwich" effect on the bicyclist especially under higher volume and speed conditions resulting in BLTS 4. Dual shared or exclusive right turn lanes are typically in very high-volume locations which add additional stress and are BLTS 4.

A roadway with no marked bike lanes and a right turn lane will be a high stress location unless the right turn lane is short and rarely used. This condition will also occur if a bike lane is dropped ahead of an intersection. If the turn lane is short (less than 100' including taper) then there is no impact on the BLTS. If speeds are 20 mph or less and sharrow markings are present in the shared turn lane then the BLTS can be reduced by one level. Approaches with bike signals (as would be the case as shown in Exhibit 14-8d) can be BLTS 1 unless there is excessive delay, high non-compliance, or other issues present.

If a separated bike lane uses a protected intersection design where the bike lane is separated by the same distance or bends out further away from the travel lane, then this is BLTS 1. Approaches that "bend-in" the bike lane to be adjacent to the travel lane have more vehicle proximity stress and are BLTS 2. Approaches that require the separated bike lane to have a leftward lateral shift in which vehicles also cross are similar to Exhibit 14-8b and ones that terminate the bike lane into a shared "mixing zone" with right turning traffic are similar to Exhibit 14-8e.

Exhibit 14-9 BLTS Right Turn Lane Criteria[1]

Right-turn lane configuration	Right-turn lane length[2] (ft)	Bike Lane Approach Alignment	Vehicle Turning Speed (mph)[3]	BLTS
Exhibit 14-7a	≤ 150	Straight	≤ 15	BLTS 2
Exhibit 14-7a	>150 to 500' maximum	Straight	≤ 20	BLTS 3
Exhibit 14-7b or c	<150	Shift to Left	≤ 15	BLTS 3
Exhibit 14-7d	N/A	N/A	N/A	BLTS 1
Exhibit 14-7e	≤ 75	Straight	≤ 15	BLTS 2
Exhibit 14-7e	>75' to 150' maximum	Straight	≤ 15	BLTS 3

[1]Use BLTS 4 for any lengths, speeds, or configurations (e.g. dual right turns or Exhibit 14-8d) not shown in the table.
[2]For the purposes of this methodology, the right turn lane length includes the length of the taper.
[3]This is vehicle speed at the corner, not the speed crossing the bike lane. Corner radius can also be used as a proxy for turning speeds.

The original Mineta methodology did not consider the impact of left turn lanes. Left turn lane criteria are based on logical breaks in stress levels with the following considerations. Left turn lanes are more stressful than right turn lanes. Left turns require the cyclist to yield and merge into traffic like a vehicle and occupy the through and/or the left turn lane. The more through lanes a cyclist must cross to reach the left turn lane the higher the stress level, especially in higher speed locations, as both longitudinal and lateral mixing with traffic are increased, as shown in Exhibit 14-10.

Shared through-left lanes where a bike lane is present can act like mixed traffic conditions as the rider only has to move into the adjacent lane from the bike lane. Similarly, roadways with no bicycle lanes also act like mixed traffic conditions as the rider may already be in the shared left-through lane or just needs to move laterally into a left turn lane. Separate left turn lanes require the rider to occupy a through lane for some distance (to allow for signaling intentions to following vehicles). Dual left turn lanes (either shared or exclusive) indicate high-volume locations which add additional stress above and beyond the speed and necessary lateral movements and should be BLTS 4.

If bicyclists typically use a lower stress two-stage left turn maneuver using the crosswalks or facilitated with special bike box or left turn queue box markings, then the BLTS is controlled by the appropriate intersection crossing criteria in Exhibit 14-11 and Exhibit 14-12 for each of the crossed roadways instead of the left turn criteria shown in Exhibit 14-10. Low-speed signalized intersections that are set up for bicyclists to make two-stage left turns with regular and left-turn queue bike boxes can be BLTS 1.

> *For rating routes, only include the effect of the left turn lane if the route requires a left turn and typically uses the vehicle lane versus a two-stage movement. For through and right turn movements, include the effect of the right turn criteria.*

Exhibit 14-10 BLTS Left Turn Lane Criteria[1]

Prevailing Speed or Speed Limit (mph)	No lane crossed[2]	1 lane crossed	2+ lanes crossed
≤25	BLTS 2	BLTS 3	BLTS 4
30	BLTS 3	BLTS 4	BLTS 4
≥ 35	BLTS 4	BLTS 4	BLTS 4

[1]Use BLTS 4 for any shared/exclusive dual left turn lane configuration.
[2]For shared through left lanes or where mixed traffic conditions occur (no bike lanes present)

14.4.6 BLTS Intersection Crossing Criteria

Unsignalized Intersections

Unsignalized intersection crossings can act as a barrier to bicyclists especially with a high number of lanes or higher speeds. The crossing can be an impediment to travel if the bicyclist has to cross five or more lanes at any speed or has to cross a 35 mph (or greater) four-lane street. The criteria for unsignalized intersection crossings include consideration of the presence of a median of sufficient width to provide for a two-stage crossing. Pedestrian/bicycle over/underpasses would be considered as separate facilities and are BLTS 1.

Where there is no median refuge, the BLTS depends on the speed and two-way average daily traffic (or functional class if ADT is not available), as seen in Exhibit 14-11.

Exhibit 14-11 BLTS Criteria for Unsignalized Intersection Crossing without a Median Refuge[1]

Prevailing Speed or Speed Limit (mph)	Total Through/Turn Lanes Crossed (Both Directions)[2]					
	≤ 3 Lanes			4 -5 Lanes		≥ 6 Lanes
	Functional Class/ADT (vpd)					
	Local	Collector	Arterial	Arterial		Arterial
	≤ 1,200	1,200 - ≤3,000	>3,000	≤ 8,000	>8,000	Any ADT
≤ 25	BLTS 1	BLTS 1	BLTS 2	BLTS 3	BLTS 4	BLTS 4
30		BLTS 1	BLTS 3	BLTS 3	BLTS 4	BLTS 4
35		BLTS 2	BLTS 3	BLTS 4	BLTS 4	BLTS 4
≥ 40		BLTS 3	BLTS 4	BLTS 4	BLTS 4	BLTS 4

[1]For street being crossed.
[2]For one-way streets use Exhibit 14-12.

To accommodate one-way streets, the intersection crossing with a median refuge criteria was changed to lanes per direction versus total lanes crossed. One-way streets carry higher volumes than two-way streets of the same number of lanes and thus can have greater stress levels applied to them. Use Exhibit 14-12 for one-way street applications.

Exhibit 14-12 has the maximum number of lanes a bicyclist encounters on each side of a median refuge. The presence of a median refuge generally implies a roadway with a substantial amount of traffic volume, so specific ADT values are not included in the exhibit, adding a median refuge of at least six feet in width (10 feet for BLTS 1 eligibility) will decrease the BLTS versus when a refuge is not present. The presence of turn lanes is also accounted for as they add conflict points and vehicle paths to the awareness needs.

Exhibit 14-12 BLTS Criteria for Unsignalized Intersection Crossing with a Median Refuge[1]

Prevailing Speed or Speed Limit (mph)	Maximum Through/Turn Lanes Crossed per Direction			
	1 Lane	2 Lanes	3 Lanes	4+ Lanes
≤ 25	BLTS 1[2]	BLTS 2[2]	BLTS 2	BLTS 3
30	BLTS 1[2]	BLTS 2	BLTS 3	BLTS 3
35	BLTS 2	BLTS 3	BLTS 4	BLTS 4
≥ 40	BLTS 3	BLTS 4	BLTS 4	BLTS 4

[1]For street being crossed.
[2]Refuge should be at least 10 feet to accommodate a wide range of bicyclists (i.e. bicycle with a trailer) for BLTS 1, otherwise BLTS=2 for refuges 6 to <10 feet.

Since crossings are not part of a link, the BLTS to cross the major street is applied to the minor street. If the crossing BLTS is greater than the minor street link BLTS, the crossing BLTS applies (controls) to that link.

Roundabouts

Calculation of the traffic stress at roundabouts will determine if bicyclists are expected to use a shared sidewalk that surrounds it or if they will ride in the vehicular lanes under mixed traffic conditions[3]. For a sidewalk to serve as a potential bicycling path, all the following criteria must be met. If both routes are possible, evaluate both options and use the lower stress option as the controlling BLTS.

For bicyclists to be expected to use a shared sidewalk, it must meet the following criteria:

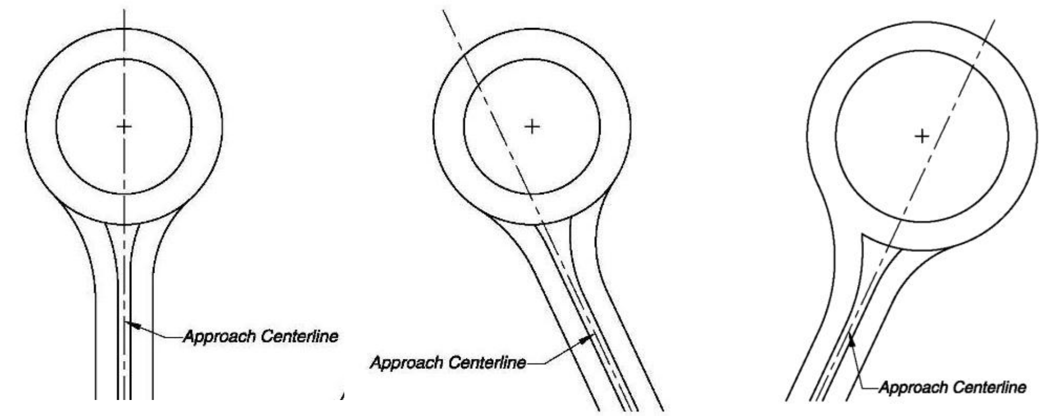
- Minimum six-foot clear width (allows slow-speed passing assuming no obstructions that would prevent a cyclist from riding close to the edge of the walkway).
- Offset from edge of circulating roadway to path crossing should be no more than 30’ to minimize out-of-direction travel.
- Path geometry should have no turns greater than 90 degrees and allow a cyclist to see (without looking behind them) if it is safe to cross within 10’ from the crossing (allows for minimum 5 mph travel speed).
- Separate bike ramps need to be provided to transition riders between the sidewalk and street (or bike lane) on entry and exit legs in a reasonably direct manner and provide a safe re-entry. If a single ramp is intended to provide both bicycle and pedestrian traffic, then it needs to be wider than the standard (8-10’) pedestrian curb ramp.

A separate 8’+ path or sidewalk surrounding a roundabout will be normally BLTS 1 for the segments between the leg crossings. Narrower 6’ sidewalks are BLTS 2 as it is more difficult to overtake pedestrians or bicyclists traveling in the same direction or allow opposing traffic to pass. Each of the roundabout leg crossings will need to be evaluated as that will be the source of traffic stress for bicyclists using the sidewalk crossing as shown in Exhibit 14-14. All the individual leg crossing BLTS’s are compared, and the highest one will be used to represent the roundabout BLTS.

Leg crossings will generally control over the mixed traffic condition (check in Exhibit 14-6) except in cases where tangential legs are used, or higher volumes are present (See Exhibit 14-13b or c). Tangential legs occur when the approach centerline does not go through the roundabout center. These have little deflection in the vehicle path and results in much higher speeds through the pedestrian crossing. Most roundabouts should have non-tangential approaches as shown in Exhibit 14-13a. The highest BLTS from Exhibit 14-14 or Exhibit 14-16 will be used in this case.

[3] Furth, P. Level of Traffic Stress (LTS) Criteria for Roundabouts, Northeastern University, March 2014.

Exhibit 14-13 Roundabout Approach Geometry



gure from NCHRP Report 672, Roundabouts: An Informational Guide, 2nd Edition, Transportation Research Board, 2010.

Exhibit 14-14 Roundabout Leg Crossing BLTS

Entry/Exit Type	Non-Tangential[1]	Tangential[1]
Single entry lane	BLTS 1	BLTS 2
Single exit lane	BLTS 1	BLTS 3
Dual entry lane	BLTS 1	BLTS 3
Dual exit lane	BLTS 3	BLTS 4

[1]An exit/entry lane is tangential if a driver does not have to turn to the right when entering or exiting. This is a non-standard design.

If there is no adequate alterative path or sidewalk to use, then the bicyclist will need to use the vehicle lane under mixed traffic conditions through the roundabout. The BLTS is computed for these cases in Exhibit 14-6. Dual-lane roundabouts will always be BLTS 4 as these always have at least one multilane exit which sets up a potentially hazardous conflict between circulating bicyclists and exiting traffic from the inside lane. Partial two-lane roundabouts are considered to have two circulating lanes for the purposes of this methodology as stress level is controlled by the worst condition.

Right-turn bypass lanes outside of the roundabout (Exhibit 14-15a) should be considered as mixed traffic conditions as in Exhibit 14-6 or possibly Exhibit 14-7 for higher speed movements. Right-turn bypass lanes within the roundabout (Exhibit 14-15b) would be considered as right turn lanes as shown in Exhibit 14-9.

Exhibit 14-15 Roundabout Bypass Lane Types

a) Bypass Lane Outside of Roundabout b) Bypass Lane Inside of Roundabout

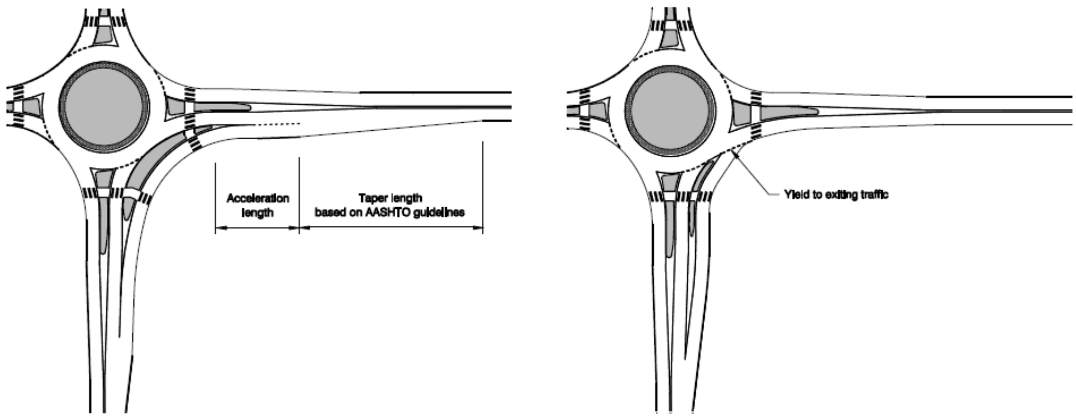


Figure from NCHRP Report 672, Roundabouts: An Informational Guide, 2nd Edition, Transportation Research Board, 2010.

Exhibit 14-16 Roundabout Circulating BLTS

Number of circulating lanes	Total Entry Leg ADT (vpd)	LTS
1	≤ 4,000	BLTS 1
1	4,000 - ≤ 6,000	BLTS 2
1	>6,000	BLTS 3
2+ (Partial or full)	Any	BLTS 4

Signalized Intersections

As signalized crossings usually do not create a barrier as the signal provides a protected way across, BLTS 1 is generally assumed for the crossing movements. Note that in the presence of turn lanes, this criterion generally will not control. Pretimed intersections and isolated or coordinated activated signalized intersections with working specific bicycle detection loops/ other activation technology will preserve the BLTS 1 rating. Signalized intersections do pose risks for right-turn “hook” crashes, however, especially where right turn lanes are not present.

At certain locations, bicyclists may have difficulty triggering the signal detection (where no specific bicycle detection is in place) or an intersection may not have the proper striping, ramps, and push button accommodations for bicyclists. These crossing locations force the bicyclist to use the crosswalk like a pedestrian instead and should be BLTS 2. Permissive left and right turns with or without turn lanes introduce conflicts as bicyclists are harder to see versus oncoming vehicles, so these will increase the crossing to BLTS 2. There may be other areas where engineering judgment is required in assigning stress levels higher than BLTS 1 at signalized intersections such as the combination of shorter phase length and longer crossing distances or complex geometry (e.g. more than four legs, high skews) which could push potentially to BLTS 3. The reasons for higher signalized BLTS levels should be documented. The presence of bike signals or regular and left-turn bike boxes may be a mitigating factor in higher-risk areas thus keeping the BLTS at 1.

14.4.7 BLTS Application Example

Example 14-2 Level of Traffic Stress

This example illustrates the use of BLTS for the central section of the City of Burns in eastern Oregon in Harney County. This covers the signalized junction of US20 and OR78 in downtown Burns as well as including surrounding residential areas. Data were quickly obtained by using available state highway inventory data and views from commercial aerial photos.

Segment BLTS:
Most roadway sections are two lanes and 25 mph except for US20 west of the OR78 junction which has four lanes. No bike lanes are in the example area, so all roadways are considered mixed traffic, use Exhibit 14-5. BLTS 3 for the two-lane major roadways (US20 and OR78) is based on the average daily traffic (3,300 for US20 and 2,600 for OR78), BLTS is 3 for the four-lane section of US20, and BLTS is 1 for the local streets (no marked centerlines).

Approach BLTS:
On the southbound and westbound approaches to the US20/OR78 junction, the right turn lanes are both a full block long with an adjacent shared-through left lane. These right turn lanes will create a high stress level for a bicyclist as it forces them to mix directly with right turning traffic if they wish to turn right or mix with through traffic in the southbound shared through-left lane if they wish to continue southbound. Because these are greater than 150 feet and do not have an adjacent bike lane, these are both coded BLTS 4, see Exhibit 14-8. The adjacent shared left-through lanes on both approaches would have a BLTS of 2, but the BLTS 4 right turn lane arrangement supersedes it.

On the northbound approach, there is a short 50' right turn lane with an adjacent shared – left lane. The right turn lane is short, so there is no additional impact on the BLTS. The adjacent shared though-left lane would be BLTS 2, as no lane would need to be crossed since an approaching bicyclist will end up in this lane if they are not turning right. Here, the approach BLTS 2 will override the segment BLTS 1 value.

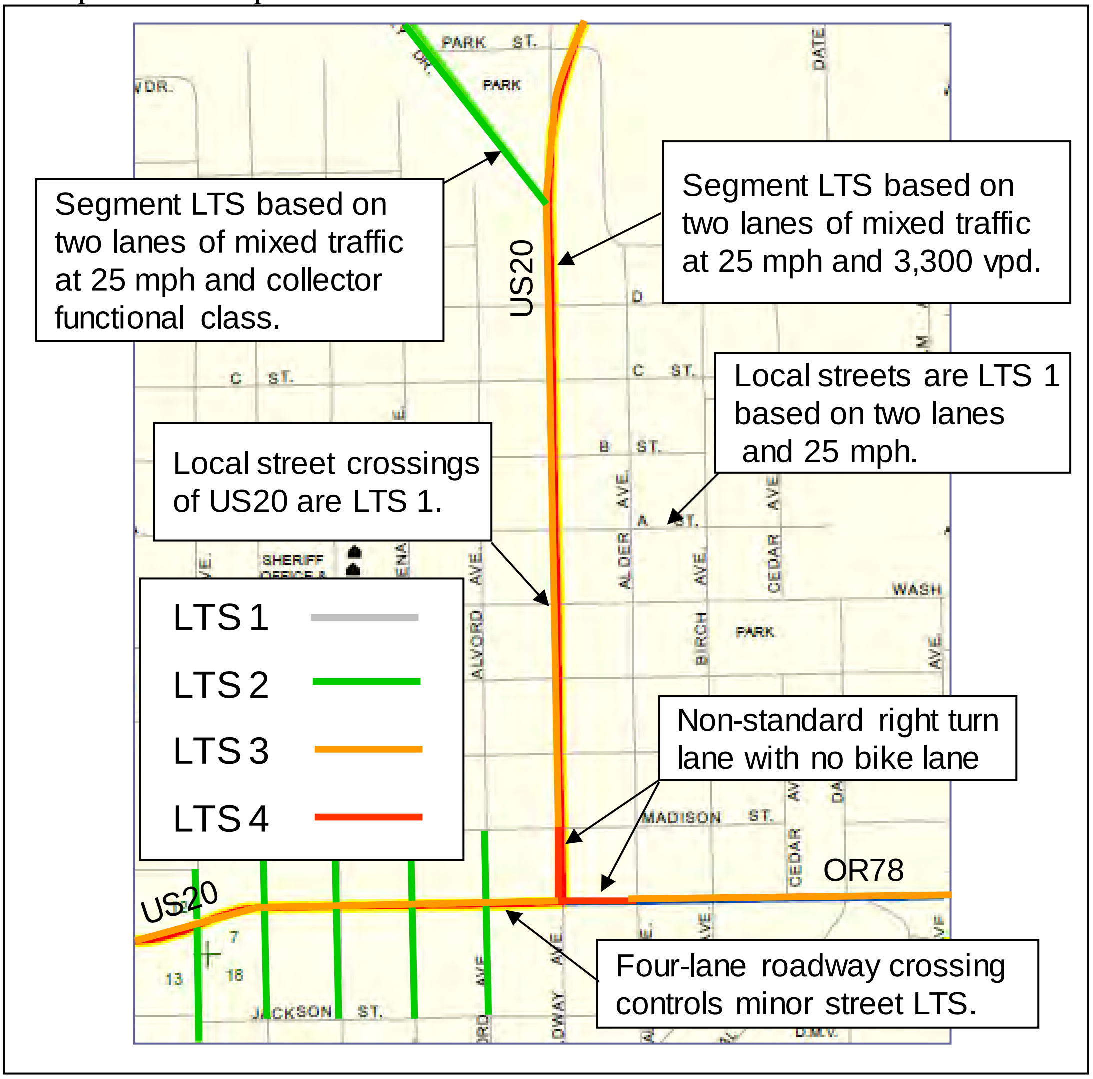
The eastbound approach has a left-lane drop lane where the left turn lane is a full block long with an adjacent through-right lane. Since mixed traffic conditions exist, the bicyclist would just move into the left turn lane. As seen in Exhibit 14-5, with ADT of 2600 and 25 mph speed, this would be BLTS 2, but the segment BLTS 3 level would still control.

Crossing BLTS:
The signalized intersection of US20 & OR78 is BLTS 1. However, this is overridden on the southbound and westbound approaches by the BLTS 4 for the approaches; on the eastbound side by the BLTS 3 four lane sections; and on the northbound side for the BLTS 2 approach. On the four-lane portion of US20, the local street crossings are increased to BLTS 2 which affects the coding of local street segments that are adjacent to US20.

Summary:
Since the highest BLTS controls, using the US20/OR78 intersection as an example, the approach criteria is greater than the segment or crossing criteria, except for the eastbound approach where it matches the BLTS 3 value. The resulting BLTS at the intersection can be seen in the figure below.

Most of the roadway system in the example is BLTS 1 or 2. The long right turn lanes coupled with the absence of bike lanes on US20 and OR78 approaching the highway junction convert a potential route using these roadways into BLTS 4, which most bicyclists will avoid. While the various parts of the city are generally well connected with BLTS 1 or BLTS 2 networks, it is easy to see the disconnect created along the primary arterials by the intersection of US 20 and OR 78.

Example BLTS Map



14.4.8 Rural Applications

The rural roadway environment is substantially different from the urban. High speeds are assumed, and vehicular operation is unpredictable as drivers are not expecting bicycles in or near the travel lanes. Paved shoulders when provided can narrow in guardrail sections and through bridges and tunnels creating higher potential conflict areas especially where volumes are higher. Gravel, cinders, and other debris is common in the shoulders which may limit their use and force the bicyclist to use the travel lanes. Given the geometric and operational differences, the rural bicyclist must be more aware and is likely higher stress tolerant than their urban counterpart. The BLTS rural nomenclature uses a “R” prefix to help set off the environmental difference, as an urban BLTS 2 is not the same experience as a rural BLTS R2 for the reasons above.

Rural <45 mph

While the original methodology was designed only for urban applications, it can also be used for rural roadways that have posted or operating speeds less than 45 mph. Rural roadways with speeds less than 45 mph tend to be one or two-lane local, undeveloped roadways that:

1) connect rural communities,
2) exist in parks or other recreational areas or
3) provide a connection to a tourist destination.

These are typically low volume and have no or little paved shoulder width. Sight distances are likely to be lower (sharper vertical and horizontal curves) because of the lower road design standards used. BLTS will be primarily based on speed in these cases. Use the regular BLTS mixed traffic criteria shown in Exhibit 14-6 and Exhibit 14-7 for these roadways and Exhibit 14-11 through Exhibit 14-16 for intersections. Approach criteria will probably not be applicable because low volume roadways generally do not have turning lanes. Add the "R" suffix to designate a rural roadway to the BLTS values in the tables. Segment lengths should be defined between significant intersections/locations at a minimum.

Rural ≥45 mph

Application of the BLTS methodology to the typical higher-speed rural environment requires considering paved shoulder widths and volumes. Daily bi-directional (combined) volumes are necessary for this method. This can be AADT for initial scoping-level assessments, but most analyses should take seasonal adjustment into account especially for coastal and summer recreational routes as most bike travel occurs in the summer months. The normal BLTS methodology tops out at 40 mph, while most typical state and county rural roadways are posted at 45 - 55 mph; some eastern Oregon state highways have higher speed limits. Segment lengths would be ideally based on paved shoulder width and AADT volume bin changes in Exhibit 14-17 or between significant intersections at a minimum.

Interstate highways are a special case and while they typically have shoulders of 10' or more, the posted speed limits can be up to 70 mph, and high traffic and truck volumes are a normal occurrence making the overall environment not very inviting to cyclists. Rural Interstate highways are coded as BLTS R4. Where no parallel routes exist, (e.g. certain sections of I5 in southern Oregon and I84 in eastern Oregon) there is a need to create separated facilities instead of attempting to widen shoulders as that would likely have limited likelihood to increase use.

A large portion of bicycle-vehicle crashes occur when a vehicle attempts to overtake a bicyclist on a roadway with no or little available paved shoulder width. The wider the shoulder the less likely a bicyclist will be in the same path as vehicles. The occurrence of bike crashes is highest on higher volume rural facilities with little or no paved shoulders, poorly placed rumble strips, or deteriorated shoulder pavement conditions.

Narrow or no shoulders and higher volumes (increased overtaking conflicts) will increase the stress level at any speed. Paved shoulders less than four feet in width are not considered rideable (even when clear of debris or obstacles) because there is effectively no opportunity to move out of the travel lane while riding on these segments. This is a high stress environment to ride in unless volumes are very low (<400 vehicles per day). Even with low volumes, a bicyclist is generally unable to move into the shoulder or could encourage unsafe/illegal close passing behavior.

Shoulders between four and six feet in width do allow riding in the shoulder continuously when clear of debris. The clear riding space may be constrained by debris or obstacles, but there is some space to navigate around occasional obstacles without leaving the shoulder. Where there is continuous debris, it is probably still physically possible to move out of the travel lane when vehicles approach. The shoulder may also pinch down for short segments (e.g. bridges), but there should generally be a wide enough shoulder approaching these pinch points that a bicyclist could wait in the shoulder for a gap in traffic before proceeding.

Shoulder width greater than six feet should generally provide room to ride and navigate around debris without leaving the shoulder, although it might require the bicyclist to get in closer proximity to the vehicular traffic. Shoulders greater than six feet can accommodate ODOT's common placement of rumble strips adjacent to the fog line, while still providing at least four feet of shoulder on the outside of the rumble strips. While rumble strips encroach on the clear rideable space, they also discourage cars from encroaching into the shoulder, which can improve bicyclist comfort.

Long-term debris in the shoulder (like leftover cinder from winter maintenance), limiting or preventing shoulder use, should be coded as no shoulder. However the true shoulder width (and resulting BLTS) should also be retained for documentation. Unless an adjacent separated multi-use path/bike lane is provided (BLTS R1), most rural roadways do not have bike lanes and bicyclists will depend on paved shoulders. Exhibit 14-17 shows the BLTS for typical rural conditions for higher speed rural roadways.

Exhibit 14-17 BLTS Rural Segment Criteria with posted speeds 45 mph or greater[1,2,3]

Daily Volume (vpd)	Paved Shoulder Width		
	0 – <4 ft	**4 - <6 ft**	**≥6 ft**
<400	BLTS R2	BLTS R2	BLTS R2
400 - 1500	BLTS R3	BLTS R2	BLTS R2
1500 - 7000[4]	BLTS R4	BLTS R3	BLTS R2
> 7000	BLTS R4	BLTS R4	BLTS R3

[1] Based on Figure 900-4 from the Highway Design Manual, 2025.
[2]Adequate stopping sight distances on curves and grades assumed. A high frequency of sharper curves and short vertical transitions can increase the stress level especially on roadways with less than 6' shoulders. Engineering judgment may be needed to determine what impact this will have on the BLTS level on a particular segment.
[3]Segments with flashing warning beacons announcing presence of bicyclists (typically done on narrower long bridges or tunnels) may, depending on judgment, reduce the BLTS by one, minimum BLTS R2.
[4]Over 1500 AADT, the Oregon Bicycle and Pedestrian Design Guide indicates the need for shoulders.

Rural high speed intersection crossing stress levels will be typically based on approach volumes and number of lanes (Exhibit 14-18). Since the rural environment is more unpredictable (higher speeds and motorists are less likely to be aware of or anticipate bicyclists) than the urban environment for cyclists, the minimum BLTS is R2.

Exhibit 14-18 BLTS for Unsignalized Rural Intersection Crossing with posted speeds 45 mph or greater[1]

Daily Volume (vpd)	≤ 3 Lanes	4 -5 Lanes	≥ 6 Lanes
<400	BLTS R2		
400 - 1500	BLTS R2		
1500 - 7000	BLTS R2	BLTS R3	
> 7000	BLTS R3	BLTS R4	BLTS R4

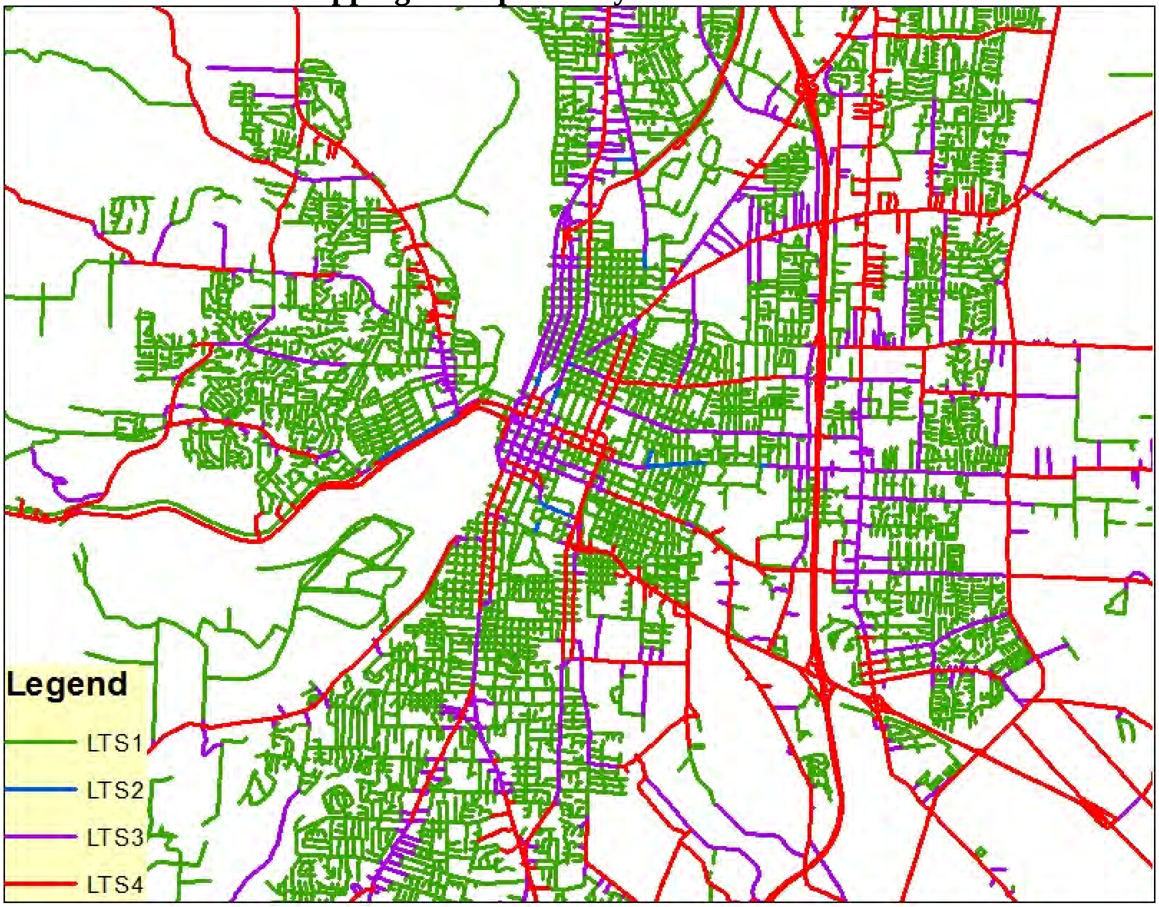
[1]For roadway being crossed.

For intersection approaches, the presence of left or right lanes will increase the BLTS at least by one level as they greatly increase the chance that vehicles will cut across the bicyclist's path or that the bicyclists will need to utilize these lanes to turn. Low volume roadways (less than 1500 ADT) are not likely to have turn lanes.

14.4.9 Route Connectivity using BLTS

The BLTS designations should be mapped on the system network. This can be facilitated with GIS or with a travel demand model if available. The objective of mapping is to identify locations with BLTS values exceeding a desired level that may then be targeted for improvements. Ideally, the displayed BLTS should be directional as it may differ on each side of a street. This will require some work with link offsets and layers to get this to show properly in GIS mapping software. Exhibit 14-19 shows an example of using BLTS showing the different stress levels. The high stress routes can easily be contrasted against the lower stress ones.

Exhibit 14-19 BLTS Mapping Example – City of Salem[1]

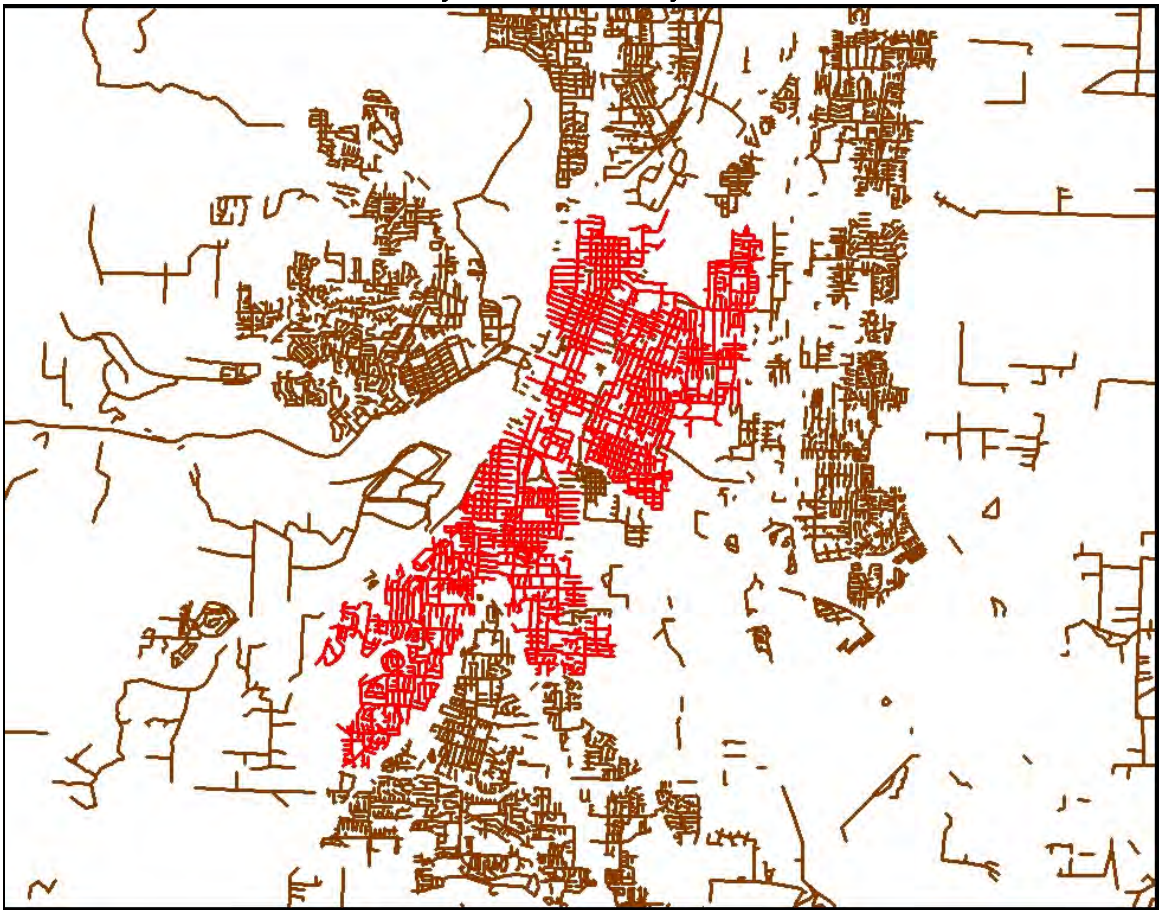


[1]Source: Haizhong Wong, Matthew Palm, Jonathan Mueller, Salem BLOS Application, OSU, 2012.

Another significant advantage of the BLTS methodology is that it allows the identification of connectivity “islands”, surrounded by higher BLTS streets/intersections and other natural and physical barriers (i.e. rivers and railroads). This allows for a true connectivity look versus just considering system gaps, as one high stress location may prevent many routes or connections between adjacent neighborhoods. Improvements can be prioritized by the amount of additional low stress routes or points connected, thereby enhancing the system in addition to just gap filling.

Exhibit 14-20 shows an example of mapping just the BLTS 1 and 2 routes. Barriers and high stress routes break the network into “islands” shown in brown and red. For emphasis in the original application, the downtown and surrounding system that can be reached via low stress routes is shown in red.

Exhibit 14-20 BLTS Connectivity "Islands" – City of Salem[1]



[1]Source: Haizhong Wong, Matthew Palm, Jonathan Mueller, Salem BLOS Application OSU, 2012.

14.4.10 Specific Routes and Out-of-Direction Travel

Instead of tracking an entire jurisdiction/area of individual segments and crossings, the BLTS mapping effort can also be applied based on routes between significant origins and destinations (i.e. neighborhoods to schools). Alternatively, this method may be used to help identify alternate (parallel) lower stress routes to help address a particular high stress location. It may be possible to attract potential cyclists by reducing the BLTS of key links. For example, if a BLTS 4 crossing is located along a route where all segments are at BLTS 2 or less, it may be a good candidate for adding a median crossing refuge because it would complete the route and make it more attractive to more riders.

For each identified alternative route (for example to bypass a steep hill or a high stress intersection) a check for the distance of out-of-direction travel should be made. For connectivity purposes, a route between two points should be low stress and without too much out-of-direction or extra travel distance. Too much extra travel time results in some riders choosing to travel the shorter, higher stress route, while other less stress-tolerant riders choose to not travel by bicycle at all, especially if they have a choice of modes.

According to the original research report[4], riders can typically tolerate up to 25% extra distance since the vast majority of trips are within 25% of the shortest-path available. However, most bicyclists choose trip paths that are only 10% longer than the shorter higher-stress routes, so 10% is a good target value. A 10% target represents a half-mile of extra travel on a five-mile trip. Short trips should not have detours of longer than approximately ¼ mile which represents about one and a half minutes of travel time at 10 mph[5]. In addition, the 25% maximum threshold for connectivity can also be used to predict route selection, to plan way-finding routes, or even analyze detour routes around a construction zone.

Routes can be assessed for acceptable out-of-direction travel if the either of the following relationships is true:

$L_k/L_4 \leq 1.25$; OR
$L_k - L_4 \leq 1{,}430$ feet (0.27 mile)

Where L_k = route distance at any given stress level, k.
L_4 = route distance using any links with stress levels up to and including BLTS 4 (but not including links where riding is prohibited).

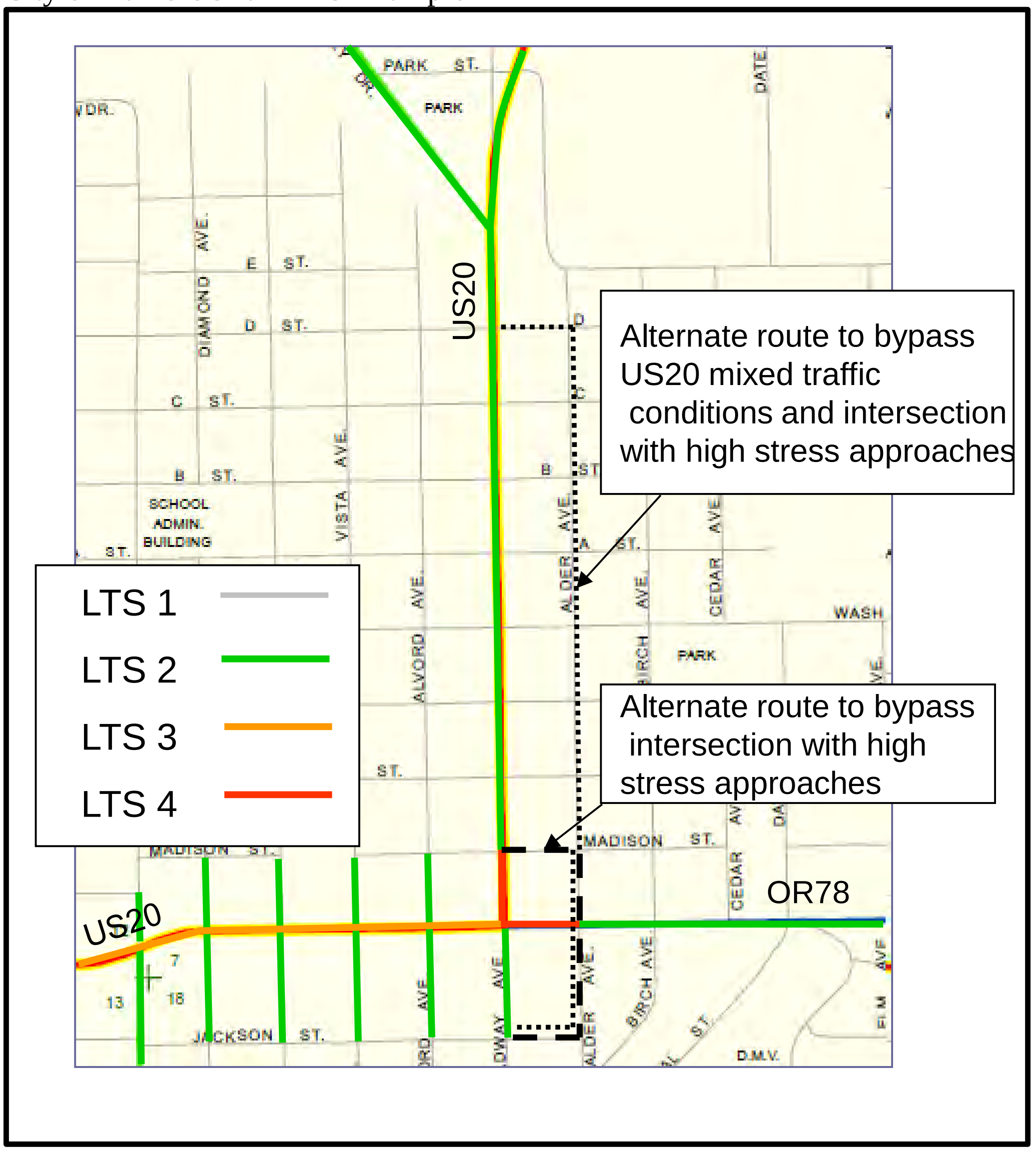
Note: Some routes with hills or many stops (or any of the previously mentioned additional considerations) may decrease desirability even though the criteria above are met.

[4] Low-Stress Bicycling and Network Connectivity, Mekuria, Furth, & Nixon, Mineta Transportation Institute, May 2012, pp14-15.
[5] Understanding and Measuring Bicycle Behavior: A Focus on Travel Time and Route Choice, Dill & Gliebe, Portland State University, December 2008.

Example 14-3 Alternate Route Out-of-Direction Travel

This example illustrates the impact of out-of-direction travel on alternate routing. Two routes, one short and one long, are shown bypassing a signalized intersection with high-stress approaches in the figure below. These are only two routes of the many that are available to use.

City of Burns US20 BLTS Example



For the (exaggerated) short route, the normal high-stress route through the intersection is 700 feet. Adding in the two extra blocks of travel (600' total) to cross on a lower-stress path creates a 1300-foot route. While the total length of extra travel distance is acceptable as 600 feet is less than 1,430 feet, the overall extra trip distance as a proportion of the total is not, as 1300 feet/700 feet = 1.86 or 86% extra distance. For this route, bicyclists are unlikely to take the alternate path. Higher stress-tolerant users will just deal with the

BLTS 4 section while less tolerant users will likely take another mode such as walking or driving.

For the longer route, the normal higher-stress route is 2,650 feet through the downtown section and across the intersection. Like with the short route, the extra distance is 600 feet for a total route distance of 3,250 feet. In this case, the extra distance has less of a proportional impact on the trip as 3,250 feet/2,650 feet is 1.23 or 23%. This is less than the 25% threshold so bicyclists may choose this route instead, especially if they are less-stress tolerant. This distance is still greater than the desirable 10% level so not all (especially higher-stress tolerant riders) will use this particular route. This path would need to be over twice as long to meet the 10% level with even just a couple extra blocks out of direction.

14.4.11 Additional Comfort Measures

Some rider factors are better classified as "level of comfort" than a stress-based measure as they do not directly affect the bicyclist's position on the bicycle facility or the general proximity to adjacent motor vehicle speeds and volumes. These comfort measures can be used as a "tiebreaker" in areas where the overall BLTS is generally the same to introduce nuance, where local input has indicated that it is not the same riding experience throughout that area. These measures could also be used to analyze off-street facilities such as multi-use and greenway paths as they are generally not subject to the motor vehicle spacing, speed, and volume stress factors (generally coded as BLTS 1).

These also can be used for additional detail that can be layered as considerations for the existing or future conditions or when identifying solutions or prioritizing improvements. These measures are optional to the overall method as they will increase the overall data collection needs but could be mostly done through evaluation of aerial mapping products and some "windshield" field observations. The measures are coded similarly to BLTS as they go from 1 to 4 with 1 being the best with an average condition on a segment controlling as these comfort levels will vary depending on the segment length. Longer segments may benefit from breaking into smaller pieces if conditions vary greatly or if it is difficult to establish a general average condition. None of the individual measures control over the others. While general guidance is provided below on categorizing comfort measures, these by their nature will be qualitative or judgment based. There should be explanation or documentation of any assumptions or conditions that are used to create these measures.

Surface Condition

One of the most important factors after vehicular volume, speed, and separation is surface condition. Poorly maintained pavement or debris (e.g. wet leaves, gravel) or even frequent ponding on rainy days can be a deterrent to cycling on a particular route especially for less confident riders. Poor surface conditions require more attention to the wearing surface rather than surrounding traffic by the bicyclist to maintain their position and speed. If conditions are such that they must move into the adjacent buffer or travel lane, this will create significant discomfort and potential hazards.

Drainage grates or gutter pan sections that are set lower than the pavement surface or if the grates lack transverse bars represent potential hazards for the bicyclist. Gutters can also present hazards if they are non-uniform in width and/or cross-slope, either due to settlement or by design, which could impede into the bike lane or shoulder.

Gutters and drainage grates can also collect debris such as leaves, pinecones, and leftover sand/cinders which can be a hazard if it builds up without regular maintenance sweeping. Wet leaves can be as slippery as ice and sand/cinders can also make riding difficult and even puncture tires. Built-up debris can also block inlets which can cause ponding during rain events which can extend across a bike facility or down a block for an extended distance. Ponding can also be caused by settlement or upheaval of the gutter or curb which could also be a debris collection location during drier periods. Debris and ponding both can obscure the view of the underlying pavement conditions which could mask additional hazards such as potholes.

Railroad/light-rail/streetcar tracks crossing, or along paved roadways also create potential hazards of dropping a wheel into the flangeway which could cause a fall if they are not crossed at or near 90 degrees. Manhole, meter, and valve covers in addition to pavement joints/patches can create slippery conditions when wet or can create unexpected bumps or riding hazards if pavement is raised or settled.

Pavement condition classification and pictures in Exhibit 14-21 and summarized in Exhibit 14-22 are based on the Good-Fair-Poor Pavement Condition Manual and the Pavement Distress Survey Manual from the ODOT Pavement Services Unit. The comfort levels can be used as-is for the evaluation of existing conditions. For future no-build or build conditions, the comfort level should be based on the pavement condition only as debris coverage and occurrence will be generally unknown, and seasonally variable unless there is knowledge of maintenance practices for the facility or local area. When the width of the facility also incorporates the width of a gutter pan, the condition of the gutter also needs to be considered.

Exhibit 14-21 Pavement & Debris Rating

S1 – New pavement which includes a very smooth surface with no cracking/roughness/patching or faulting. No debris in the bike facility or outside travel lane/shoulder (for mixed conditions). Bicycle facility-related striping should be clean, new, and complete. Transitions from the paved surface to the gutter pan should be smooth and with consistent cross-slopes.

S2 – Good pavement which includes smooth surfaces with minor hairline cracking/occasional patching evident but with smooth transitions and negligible differential settlement of curbs or gutters. Striping should be complete and well visible with little wear. Occasional debris in the bike facility or shoulder with no more than 25% coverage by area which can be avoided without moving into an adjacent buffer or travel lane no more than a couple times per city block or no more than 20 times in a mile in a rural segment.

.

S3 – Fair pavement which includes minor cracking, some patching/sealing/cold jointing evident or some raveling/spalling /rough areas. The cross-slope may be uneven with inconsistent transitions. Frequent transitions with differing surface wearing course changes (i.e. different pavement types or subtypes such as chip seal, open graded aggregate, etc.) or where the bicycle facility crosses over valve, meter or manhole covers may be evident. Striping is well-worn and in need of repainting and may be missing in small areas. Some debris in the bike facility or travel lane/shoulder with no more than 50% coverage by area which may require repositioning to the outer edge of the bikeway or occasionally into the adjacent buffer or travel lane more than a few times per segment (i.e. 3-4 instances per city block or 30-40 instances per rural mile). Evidence or observation of less than 50% coverage of the facility by runoff pounding during moderate or heavy rain events, due to inadequate drainage.

S4 – Poor pavement which includes potholes, large cracks (e.g. alligatoring), heavy raveling with loose aggregate, broken/lifted slabs, spalling, and generally has rough riding. There could be evidence of saw cuts or significant patching in the bicycle wheel path, overall uneven patching, noticeable differential settlement (e.g. resulting from tree roots), or poorly constructed joints. Bicycle facility striping is heavily worn or completely/partially missing. Frequent debris in the bike lane/shoulder with 75% or more covered by area requiring movement into the adjacent buffer or travel lane for most of the segment distance. Evidence or observation of more than half of the width of the facility covered by runoff pounding during moderate or heavy rain events, due to inadequate drainage.

Exhibit 14-22 Surface Condition Comfort Measure

Comfort	Surface Condition	Debris/Obstacles
S1	New	None
S2	Good	Occasional
S3	Fair	Some
S4	Poor	Frequent

Built Environment

The overall built environment can either enhance or degrade the overall biking experience. Higher use can increase driver expectations and improve driver behaviors around bicyclists while environments that are less conducive to riding such as in noisy heavy industrial areas with lots of large trucks or commercial areas with lots of driveway accesses and related conflicts can dampen the desire to travel on certain routes. Exhibit 14-23 shows typical built environment/ land uses and related bicycle comfort measures.

Exhibit 14-23 Built Environment Comfort Measure

Comfort	Overall Environment
BE1	Residential, central business districts, neighborhood commercial areas, parks/ public facilities, offices/office parks
BE2	Rural subdivisions, unincorporated communities, small-scale strip commercial, mixed employment areas
BE3	Light/medium industrial, auto-oriented (big-box) commercial
BE4	Heavy industrial, truck-oriented facilities, interchange areas

Grade

Grades can easily deter bicycle riders from certain routes or even using the bicycle mode if the alternative route has a higher level of discomfort than the rider can tolerate or has too much out-of-direction travel (see Section 14.4.9). Not all will want to walk their bike uphill assuming that there are adequate pedestrian facilities available or have access to an e-bike which can improve comfort to an acceptable level. Some areas will have all usable routes affected by grades of varying intensity, so the overall effect will be relative especially in terms of an individual’s fitness level. Long periods of shallower grades may have more of an effect on the rider than shorter steeper portions on a segment. Exhibit 14-24 shows the grade comfort measure and related descriptions.

Exhibit 14-24 Grade Comfort Measure

Comfort	Grade Description
G1	Level to slight grades, 0-3%, little to no exertion by average user needed over long periods, acceptable by all
G2	Moderate grade/rolling, 4-6%, some exertion by average user needed, will require downshifting to lower gears, will cause fatigue over long periods, e-bike assist optional
G3	Steep grade, 7- 9%, significant exertion needed by average user, likely requires lowest gears for most riders, e-bike assist helpful but likely required for beginning cyclists
G4	Very steep, 10%+,, extensive exertion required for any period of time, likely will require average user to walk bicycle, e-bike assist/or throttle-equipped use required

Illumination

Illumination or lighting levels (i.e. luminaire presence) can either improve or diminish the overall comfort of the riding experience. Adequately lit roadways and other bicycle facilities such as multi-use paths increase visibility and improve safety such as from motor vehicles on a roadway or from pedestrians if on a path. Illumination may also allow greater use of a facility into the evening or night hours. Exhibit 14-25 shows the illumination comfort measure and related condition descriptions.

Exhibit 14-25 Illumination Comfort Measure

Comfort	Illumination Condition
I1	Full coverage, no dark areas; luminaires on corners and midblock locations, or pedestrian-scale lighting present
I2	Limited coverage, some/occasional dark areas, luminaires on corners only
I3	Ambient/indirect coverage from adjacent buildings or parking lots, frequent dark areas, scattered/occasional luminaires
I4	No coverage, ambient light only, generally dark

Example 14-4 Comfort Measures

A small city as part of its transportation system plan, had been evaluated for BLTS. One east-west district highway corridor was identified to need a closer look. The BLTS was the same (BLTS 2) throughout the corridor, but residents didn’t feel that it was the same conditions both east and west of the main north-south highway given their experiences. The BLTS comfort measures were evaluated to see if any additional nuance could be added to the evaluation.

Surface Condition

Upon review of street-level imagery, the western segment has no bike facility or curbs, so shared lane use is present. The pavement where bikes would typically ride has is worn with some crack repairs, with some debris and ponding noted. The eastern segment is mostly curbed with no bike facilities but has some paved surface where a bicyclist could potentially travel outside of the travel lanes. The eastern segment surface condition has a rough surface with worn striping with numerous patching and crack repairs along with settling/longitudinal cracking on the edges and transitions with the wider paved areas. Significant debris is noted in the curbed areas and evidence of past ponding in the uncurbed sections. Using Exhibits 14-21 and 22, the western segment was determined to be most representative of fair conditions and coded as S3. The eastern segment has notably poorer pavement and surface conditions, so it was coded as S4.

Built Environment

Upon review of street-level and aerial imagery, the western segment is a mix of small commercial businesses, light industrial and some residences. The eastern segment is primarily residential. Using Exhibit 14-23, the western segment appears to have elements of the BE1 and BE3 classifications, however the smaller community context means that coding it at BE3 would likely be too intensive, so an average condition was set at BE2. The eastern segment was coded at BE1 since it is representing a residential zone.

Grade

Upon review of percent grade information, it was determined about half of the western segment had grades of 4-5 % with the rest at 0-3%. The eastern segment was all at 0-3%. Using Exhibit 14-24, the western segment was coded at G2 as there will be some exertion required in that section over what would be needed for the rest of the corridor (G1).

Illumination

Upon review of street-level imagery, luminaires were noted to be on corners only, with no additional ones in the middle of long blocks for both segments. This will mean that there will be illuminated intersections, but with periods of darkness only illuminated from ambient/indirect coverage from adjacent residences. Following Exhibit 14-25, both segments were coded I2.

Summary

Overall, the western segment has more challenging conditions for grades and overall driver expectancy in a more intensive built environment even though surface conditions were noted to be better than the eastern segment as shown in the table below. Bicyclists on the western segment would experience more overall discomfort than the eastern segment but may have to watch the pavement/surface conditions more for the eastern segment as they are worse.

Comfort Measure	West Segment	East Segment
Surface Condition	S3	S4
Built Environment	BE2	BE1
Grade	G2	G1
Illumination	I2	I2

14.4.12 Solutions to Decrease BLTS Level

There are several ways to lower stress levels and to achieve a desired BLTS level on a segment, approach, or crossing. For more detail on these solutions, please refer to the ODOT Traffic Manual and the ODOT Highway Design Manual. A few examples (not exhaustive):

- Adding bike lanes, preferably buffered, and low-speed bike boulevards.
- Creating segregated bike facilities such as separated bicycle lanes or multi-use/shared paths.
- Safety measures in design, such as couplets, medians, or pedestrian refuges. If four lanes of vehicular capacity are still needed, then investigate whether a couplet may achieve stress reductions.
- Increase width of outside lanes on roadways too narrow for striped bike lanes to create more buffer space and room for bicyclists.
- Paving/widening shoulders or removing parking.
- Reducing the number of lanes through a roadway reconfiguration.
- Install road markings and way-finding signs.
- Addition of flashing pedestrian activated beacons (PABs) or mid-block pedestrian hybrid beacons (PHBs) can improve higher-volume crossing locations.
- Removing or improving barriers, such as providing a safe grade-separated crossing over highways or railroads.
- Improving the pavement conditions on the shoulders of roadways.
- Adding two-stage left-turn bike boxes
- Adding bike signals to clarify bike movements.
- Reducing speeds, enforcement of speeds limit or education about speed.

14.5 Pedestrian Level of Traffic Stress

14.5.1 Purpose

The purpose of the Pedestrian Level of Traffic Stress (PLTS) is to create a high-level inventory and a walkability/connectivity performance rating of pedestrian facilities in a community without needing a significant amount of data. The Pedestrian Level of Traffic Stress methodology classifies roadway segments according to the level of pressure or strain experienced by pedestrians and other sidewalk users. Other users include non-motorized forms of transportation as well as motorized power chairs, scooters, and other wheeled mobility devices which are permitted and assumed to use pedestrian facilities[6]. The PLTS method would typically be used during the creation of a Regional Transportation Plan (RTP), or Transportation System Plan (TSP). It can also be used for screening in a facility plan or project (See Section 14.2 for more information on applications). This methodology is intended for use in urban areas. It can be used in rural conditions where pedestrian facilities exist, however the method will yield a high PLTS where there is higher speed traffic.

14.5.2 Methodology

PLTS was created to be a companion with the Bicycle Level of Traffic Stress (BLTS)[7]. Both methods group facilities into four different stress levels for segments, intersection approaches and intersection crossings. It is recommended that BLTS and PLTS be performed at the same time to evaluate the multimodal and intermodal deficiencies of an area. New techniques were developed to support the pedestrian segment method while the intersection crossings are adapted from the BLTS method, as those were based on a pedestrian's view of comfort and perceived safety. Like BLTS, the PLTS methodology does not require extensive data collection; much of the needed data is collected routinely and some of the PLTS data collection overlaps with BLTS.

Segment data:

- Sidewalk condition and width
- Buffer type and width
- Bike lane width
- Parking width
- Number of lanes and posted speed
- Illumination presence
- General land use

[6] A non-motorized form of transportation refers to vehicles that would not use the roadway to travel on a roadway.
[7] The BLTS methodology is based on the paper, *Low Stress Bicycling and Network Connectivity*, Mineta Transportation Institute, Report 11-19, May 2012 that was adapted by the Oregon Department of Transportation in 2014. This version can be found in the "Analysis Procedures Manual," Oregon Department of Transportation, Version 2, June 2015.

Crossing data:

- Functional class
- Number of lanes and posted speeds
- Roadway average daily traffic (ADT) [optional]
- Sidewalk ramps
- Median refuge & illumination presence
- Signalized general intersection features

For state highways, a good portion of the data needed are available in ODOT's databases including the on-line TransGIS application. Sidewalk condition and width, buffer presence, bike lane width, numbers of lanes, posted speeds, functional class, traffic volumes, and sidewalk ramps are available. Other jurisdictions may have existing TSP or public works inventories of some of these items. Use of Internet-based aerial imagery and street-level tools will capture any remaining widths or presence variables such as parking and buffer widths or intersection/mid-block crossing features. Sidewalk condition will likely require some sort of field inventory if it is not available from other sources. Volumes, if used, should be from existing sources, or already counted as part of the same study. Streets with similar characteristics with known volumes can be used as proxy for other streets in the study area. PLTS uses four levels of traffic stress with PLTS 1 being the lowest stress level:

- **PLTS 1-** Represents little to no traffic stress and requires little attention to the traffic situation. This is suitable for all users including children 10 years or younger, groups of people and people using a wheeled mobility device (WhMD[8]). The facility is a sidewalk or shared-use path with a buffer between the pedestrian and motor vehicle facility. Pedestrians feel safe and comfortable on the pedestrian facility. Motor vehicles are either far from the pedestrian facility and/or traveling at a low speed and volume. All users are willing to use this facility.
- **PLTS 2-** Represents little traffic stress but requires more attention to the traffic situation than of which young children may be capable. This would be suitable for children over 10, teens and adults. All users should be able to use the facility but, some factors may limit people using WhMDs. Sidewalk condition should be good with limited areas of fair condition. Roadways may have higher speeds and/or higher volumes. Most users are willing to use this facility.
- **PLTS 3-** Represents moderate stress and is suitable for adults. An able-bodied adult would feel uncomfortable but safe using this facility. This includes higher speed roadways with smaller buffers. Small areas in the facility may be impassable for a person using a WhMD and/or requires the user to travel on the

[8] A wheeled mobility device (WhMD) includes walkers, manual wheelchairs, power base chairs, and light weight scooters. Each of these devices requires the operator to maneuver and set the direction of travel. All these devices can be operated independently and do not require additional people to maneuver the device. The American with Disability Act (ADA) (1990) sets limits on the vertical change in a surface to 0.5 inches.

shoulder/bike lane/street. Some users are willing to use this facility.

- **PLTS 4-** Represents high traffic stress. Only able-bodied adults with limited route choices would use this facility. Traffic speeds are moderate to high with narrow or no pedestrian facilities provided. Typical locations include high speed, multilane roadways with narrow sidewalks and buffers. This also includes facilities with no sidewalk. This could include evident trails next to roads or ‘cut through’ trails. Only the most confident or trip-purpose driven users will use this facility.

It should be noted that the trip purpose and route options affect the level of stress a person is willing to experience. A person making a work-based trip is typically willing to experience a greater stress level than a person using the facility for recreation or exercise. Other elements including time of day, cost associated with other modes, ownership of vehicles, etc., influence the level of stress a person is willing to experience.

<u>Additional Pedestrian Considerations</u>

PLTS does not include some additional factors that may influence the overall level of traffic stress. These considerations may be somewhat subjective and may not be easily measured. These factors include, but are not limited to, steep grades, neighborhood crime/personal security, access density, crash history, and heavy bicycle use (on sidewalk or path). If desired, the methodology could be modified to include these factors. If one or more negative conditions apply to a roadway, the final score can be further downgraded with proper documentation. Additional notation should be included if the downgrade was based on subjective observations.

14.5.3 PLTS Targets

PLTS 2 is generally a reasonable minimum target for pedestrian routes. This level of accommodation will generally be acceptable to most users. Higher stress levels may be acceptable in limited areas depending on the land use, population types, and roadway classifications, but they will generally not be comfortable for most users. Each land use has specific needs for the pedestrian network and study areas should have multiple targets for the different areas.

Facilities within a quarter mile of schools, and routes heavily used by children should use a target of PLTS 1. This is because of the large number of children that may use the system with little or no adult supervision. The area around elementary schools should contain no PLTS 3 or 4 because of the associated safety concerns and the discouraging effect that such facilities have on walking rates. Pedestrian facilities near middle and high schools may include PLTS 2, since the students are in the older age group, but PLTS 1 routes are preferred.

Other land uses should also have a target of PLTS 1; these include downtown cores, medical facilities, areas near assisted living/retirement centers, and transit stops. Downtown cores, for example, should have wide sidewalks with street furniture. Roadways near medical facilities and residential retirement complexes should have sidewalks in good condition with adequate width.

Transit stops should have facilities that connect the passengers from the origin of their trip to the destination of their trip. The PLTS should be overlaid with the typical ¼ mile walking distance to transit for transit routes (or a roadway for a proposed route) to fully show where PLTS 1 is desired.

When setting targets, looking at the end user is vital. The land use that surrounds a corridor, pedestrian walking behavior, and local demographics will all influence the target PLTS for a corridor.

14.5.4 PLTS Criteria

PLTS measures are derived from the physical characteristics of the roadway segment and intersection crossing. Pedestrians will go either direction on a sidewalk. If there is not a sidewalk, pedestrians typically walk in the opposite direction of traffic and both sides of the roadway should be classified. The PLTS is broken into several different segment and crossing tables based on several physical characteristics of the corridor.

Variable Definitions: To complete the segment PLTS analysis, information on six different variables is used. The variable definitions are listed below:

Sidewalk[9] Width: The physical width of the solid smooth surface (typically poured concrete, but could be asphalt, brick, or concrete paver blocks) that pedestrians use. This does not include solid surfaces that contain vegetation, additional lighting, street furniture, parking meters, etc. If a sidewalk has frequent obstructions (posts, poles, mailboxes, and encroaching vegetation) that limit the usable width, use the narrower or effective width instead of the physical width.

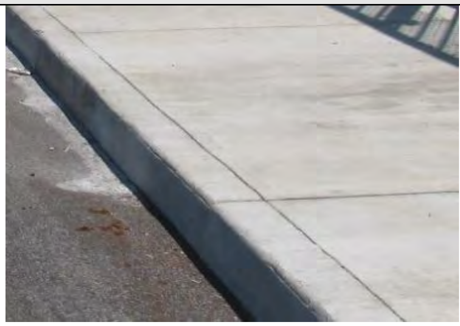
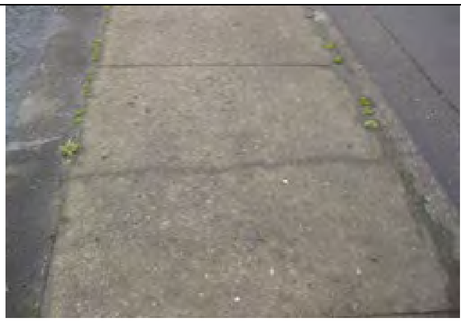
Sidewalk Condition: The sidewalk condition is a visual high-level classification process (see Exhibit 14-26). Sidewalk condition can vary within a block segment. Use the worst sidewalk condition, as a section of poor sidewalk can block some users from using the facility.

[9] Sidewalk refers to sidewalks, shared-use paths, and pedestrian paths. The methodology was designed to be used for sidewalks, but can apply to other pedestrian facilities.

The criteria and pictures for each category are based off the Good-Fair-Poor (GFP) Pavement Condition Rating Manual for Bicycle and Pedestrian Facilities and the Pavement Distress Survey Manual developed by ODOT's Pavement Services Unit. These values are also generally compatible with the sidewalk condition ranking in ODOT's TransGIS tool. For each corridor segment the general pavement condition should be considered. A sidewalk segment that contains a mix of different conditions should be rated using the worst condition. For example, a sidewalk that is smooth with only minor cracking, but has a very large fault caused by a tree root, would be considered in "Very Poor" condition. For a sidewalk to be considered in" Fair" condition, none of the properties can be" Poor" or "Very Poor" and at least one must be in the "Fair" category. For a sidewalk to be considered "Good"' all the criteria must be met, and it must be of relatively new construction. Additional examples are in Appendix B.

If obtaining data from ODOT's online FACS_STIP or TransGIS tools for use in a PLTS analysis, please be aware that there is no "Very Poor" equivalent currently. Analysts will need to field verify sidewalk sections marked as "Poor" to ensure that there are no "Very Poor" sections within them.

Exhibit 14-26 Sidewalk Condition Rating

Rating	Facility Properties	Example
Good	• No minor cracking • No patching or raveling and has a very smooth surface • No faulting • New construction	
Fair	• Minor cracking (generally hairline) • Minor patching and possibly some minor raveling evident. Surface is generally smooth • Minor faulting (less than ¼”)	
Poor	• Minor cracking in several locations • Rough areas present but not extensive • Faulting may be present but less than ½” (No major faulting)	
Very Poor	• Major cracking patterns • Rough conditions (major deterioration, raveling, loose aggregate, missing pavement, etc.) • Faulting greater than ½”	
No sidewalk	• No solid and smooth surface is present on the side of the roadway. Pedestrians use the travel lane, paved shoulder, or soil shoulder to travel along the roadway.	

<u>Physical Buffer Type</u>: The physical buffer is the distance from the outside edge of sidewalk to the edge of pavement or curb. The buffer type is categorized into five major groups. This area is also referred to as the furniture or planter zone.

No Buffer: The narrower sidewalk (<10 feet in width) is adjacent to the curb (curb tight). The facility may still include a bike lane and/or on street parking (see total buffering width distance).

Solid Surface: The buffer is a hard surface that can contain buffering elements such as lighting, street furniture, parking meters, and bicycle racks. If the buffer is wide enough, street trees can also be present which help improve the walking experience. The buffer still allows people to maneuver to the roadway edge without leaving the solid surface. The surface material can also change to indicate a buffer (i.e., stamped concrete, pavers). Purely decorative buffers usually do not have any "furniture elements" in them. A wide sidewalk (10+ feet) can also be itself a buffer even if there is no extra delineation.

Landscaped: the area between the edge of the sidewalk and the curb includes a soil area with low shrubs or vegetation. The vegetation does not create a wall or reduce pedestrian sight distance. These can also have a ditch, slope, or other topographical feature.

Landscaped with trees: The area between the edge of the sidewalk and the curb includes trees. Once the trees are mature, a canopy effect is created over the pedestrian facility and the edge of roadway. Trees are spaced for heathy growing and sight distance is not limited. This buffer type tends to be wider than a regular landscaped buffer and can have a ditch, slope, or other topographical feature included.

Vertical: A vertical buffer (i.e. retaining wall) elevates the pedestrian facility higher than the roadway surface. This typically contains an additional fence or pedestrian buffer facility.

Prevailing or Posted Speed: The prevailing (or average) speed is the recommended speed to be used in the methodology. Private probe speed data are a good source for prevailing speeds (See Chapter 3). If prevailing speed data are not available posted speed should be used.

Total Buffering Width: The total buffering width is the distance from the edge of the sidewalk to the edge of the travel lane. This includes but is not limited to:

- the physical buffer (above),
- on-street parking, if parking is not striped then assume the standard parking distances (six to eight feet) for the facility type
- Bicycle facility, and
- Shoulder

Total Number of Travel Lanes: The total number of travel lanes includes the total number of lanes on the segment. This includes the number of thru lanes for both directions, two-way left turn lanes (TWLTL), and continuous right turn lanes. For example, a five-lane roadway could have two thru lanes in each direction and one two-way left turn lane. Note: This category is different than used in the BLTS method because pedestrians can use either side of the roadway to go either direction or are not limited by one-way streets.

General Land Use

The general land use of an area with the corresponding building placement, amenities, and attractions/destinations affects the overall desired walkability of a segment. Areas that are more pedestrian-friendly typically have more destinations for walking trips, a higher pedestrian presence, and the corresponding expectation from a vehicle driver's perspective. Land use types are grouped by the likelihood for a high number of origins and/or destinations, likely pedestrian presence, perceived attractiveness and exposure, noise, heavy vehicle use, and directness.

Intersection variable definitions:

Functional Class – This is the local or state functional class assigned to a roadway. These are typically included in a Transportation System or Regional Transportation Plan document.

Average Daily Traffic – This is the total daily traffic in both directions. These can be obtained from ODOT's Transportation Volume Tables, local counting programs, calculated from traffic counts or estimated from shorter duration counts. See APM Chapters 3 and 5. If ADTs are not readily available, the methodology allows a mid-range value to substitute.

14.5.5 PLTS Classifications

The PLTS criteria are broken into two primary sections. Table-based criteria are applied separately for segments and intersection crossings. The follow sections outline the nine tables used to classify the PLTS for a roadway. The first four tables are the roadway segment criteria and the last five are for roadway intersections. The methodology uses the worst overall PLTS value for each segment and intersection crossing. The worst (highest) PLTS value of a series of segments and crossings will control a route. The segment length default for urban areas would typically be on a block-by-block basis but could be defined on a larger scale if desired. A trade-off for longer segment lengths will be a loss of detail which could make it harder to determine the controlling worst condition (e.g. a missing section may not have the same influence in a longer segment versus the default length).

14.5.6 PLTS Sidewalk Criteria

The condition and geometry of the sidewalk is the first criterion in the PLTS methodology. The criterion splits sidewalks into greater than five feet and less than five feet in width. The five-foot condition is based on federal and state design codes and recommendations. The federal standard for a sidewalk is five feet. In Oregon, the Oregon Bicycle and Pedestrian Design Guide (OBPDG) states that the standard pedestrian zone is six feet, and that five feet may be acceptable in some areas (local and residential streets). Short (<200') sections can have widths as narrow as four feet. While sidewalks along a state highway may need to be wider, sidewalks in central business districts of heavy used pedestrian areas may also need to be wider. Refer to the ODOT Highway Design Manual for more information.

Exhibit 14-27 uses the overall condition and the effective (useable) width of the sidewalk. The purpose is to rate which groups of users can safely and comfortably utilize a facility. A narrow (from obstructions or actual width) or low-quality sidewalk will not be passable for all user groups. The actual sidewalk width, especially if it is less than five feet, will impact the use by disabled people while effective width rates the comfort and flow of pedestrians along a sidewalk. A sidewalk needs to have at least six feet of space with no obstructions, like signs and poles, to be eligible for the effective width. The effective width is the simple average clear width of a sidewalk segment rather than following the more-detailed Highway Capacity Manual procedure.

Use the actual sidewalk width first in Exhibit 14-27 to see if the minimum actual width is present, then check the effective width if the sidewalk is at least six feet wide to determine the appropriate PLTS. For example, a seven-foot sidewalk in fair condition would be eligible for a PLTS 2 as the actual width is greater than five feet, but if the effective (or clear) width was judged to be at least six feet then PLTS 1 would be used. A PLTS 1 sidewalk must be accessible to all users, have six effective feet or wider path, and in good or fair condition.

However, in-sidewalk obstructions will impede disabled users if the clear width is less than five feet, so use the corresponding actual width rows to determine the PLTS. Given the same seven-foot fair condition sidewalk above, if the effective (clear) width was only three feet because of a building column then PLTS 4 would be used.

If a segment does not have illumination, consider increasing the PLTS up one level. The impact of darkness requires increased awareness for safety/security and especially if the sidewalk is in poor condition or is not present.

Exhibit 14-27 PLTS based on Sidewalk Conditions [1,3]

Actual/Effective Sidewalk Width (ft)[2]		**Sidewalk Condition**				
		Good	**Fair**	**Poor**	**Very Poor**	**No Sidewalk**
Actual	<4	PLTS 4	PLTS 4	PLTS 4	PLTS 4	PLTS 4
	≥4 to <5	PLTS 3	PLTS 3	PLTS 3	PLTS 4	PLTS 4
	≥5	PLTS 2	PLTS 2	PLTS 3	PLTS 4	PLTS 4
Effective	≥6[4]	PLTS 1	PLTS 1	PLTS 2	PLTS 3	PLTS 4

[1]Can include other facilities such as walkways and shared-use paths
[2]Effective width is the available/useable area for the pedestrian clear of obstructions. Does not include areas occupied by store fronts or curb side features.
[3]Consider increasing the PLTS one level higher (Max PLTS 4) for segments that do not have illumination. Darkness requires more awareness especially if sidewalk is in fair or worse condition.
[4]Effective width should be proportional to volume as higher volume sidewalks should be wider than the base six feet. Use a minimum PLTS 2 for higher volume sidewalks that are not proportional (include documentation).

14.5.7 PLTS Physical Buffer Type Criteria

The treatment of buffers is split into two parts: the physical buffer type and the total buffering width, which includes the physical buffer and any on-street areas outside the travel lanes (parking, bike lanes, and shoulders). The HDM and the OBPDG have standards and guidance pertaining to buffers. There are several advantages of having a buffer or furniture zone on a facility. The advantages include an increased pedestrian sense of security, sidewalks that stay level over driveways, and improved drainage. Exhibit 14-28 shows stress levels associated with varying buffer types.

Exhibit 14-28 PLTS based on Physical Buffer Type

Physical Buffer Type				
Buffer Type[1]	**Prevailing or Posted Speed**			
	≤25 MPH	**30 MPH**	**35 MPH**	**≥40 MPH**
No Buffer (curb tight)	PLTS 2	PLTS 3	PLTS 3	PLTS 4
Solid surface	PLTS 2[2]	PLTS 2	PLTS 2	PLTS 2
Landscaped	PLTS 1	PLTS 2	PLTS 2	PLTS 2
Landscaped with trees	PLTS 1	PLTS 1	PLTS 1	PLTS 2
Vertical				

[1]Combined buffers: If two or more of the buffer conditions apply, use the most appropriate, typically the lower stress level.
[2]If street furniture, street trees, lighting, planters, surface change, etc. are present then the PLTS can be lowered to PLTS 1.

14.5.8 PLTS Total Buffering Width Criteria

Exhibit 14-29 considers the stress associated with the total distance from the pedestrian to the vehicular traffic on one side of the roadway. The number of lanes is used to imply the level of the traffic volumes and functional classification of the roadway.

Exhibit 14-29 PLTS based on Total Buffering Width

Total Number of Travel Lanes (both directions)	**Total Buffering Width (ft)[1]**				
	<5	**≥5 to <10**	**≥10 to <15**	**≥15 to <25**	**≥25**
2	PLTS 2	PLTS 2	PLTS 1	PLTS 1	PLTS 1
3	PLTS 3	PLTS 2	PLTS 2	PLTS 1	PLTS 1
4 - 5	PLTS 4[2]	PLTS 3	PLTS 2	PLTS 1	PLTS 1
6	PLTS 4[2]	PLTS 4[2]	PLTS 3	PLTS 2	PLTS 2

[1]Total Buffering Width is the summation of the width of buffer, width of parking, width of shoulder and width of the bike lane on the side same side of the roadway as the pedestrian facility being evaluated.
[2]Sections with a substantial physical barrier/tall railing between the travel lanes and the walkway (like might be found on a bridge) can be lowered to PLTS 3.

14.5.9 PLTS General Land Use Criteria

The general land use can create an overall positive effect on walkability and use of certain facilities if destinations are frequent and convenient. Higher pedestrian use leads to a greater driver expectation and driving behaviors typically reflect such (i.e. more likely to yield). Conversely, land use can create a dampening effect to the point that it will not matter how well the facilities are laid out or constructed, the desire to walk on a segment is diminished if the facility goes through a perceived unattractive/unsecure/noisy/too-busy area. Areas that are more vehicle oriented have lower driver expectations for pedestrians so yielding behaviors are much less likely. Exhibit 14-30 groups typical land use types and the land use contexts used in the ODOT Highway Design Manual by PLTS level with more pedestrian-friendly walkable areas getting better PLTS levels.

If the PLTS analysis will be covering existing or future no-build conditions, then the General Land Use criteria should be included to fully show the impacts to the pedestrians. If alternatives are being analyzed, then this criterion **should not** be included if additional targets are not identified. This will avoid accidentally eliminating the benefits of a solution due to the overall land use not changing. However, this criterion can be included for large-scale alternatives/developments that do change the overall land use.

Impacts of this criterion can be mitigated by identifying additional desired target levels as mentioned in Section 14.5.3. For example, PLTS 3 could be identified as the target for commercial and industrial areas reflecting the surrounding land use while lower targets are in use in commercial and residential areas. Even PLTS 4 could be acceptable as a target for small areas that are difficult to mitigate like in interchanges and heavy industrial areas. The PLTS 4 target in this case should be noted that it is for land use, and not used to legitimize missing or substandard facilities.

Exhibit 14-30 PLTS based on General Land Use

PLTS	Overall Land Use	HDM Land Use Context
PLTS 1	Residential, central business districts (CBD), neighborhood commercial, parks and other public facilities, governmental buildings/plazas, offices/office parks	Traditional Downtown/CBD Urban Mix Residential Corridor
PLTS 2	Low density development, rural subdivisions, un-incorporated communities, strip commercial, mixed employment	Suburban Fringe Rural Community
PLTS 3	Light industrial, big-box/auto-oriented commercial	Commercial Corridor
PLTS 4	Heavy industrial, intermodal facilities, freeway interchanges	

14.5.10 PLTS Crossing Criteria

Unsignalized crossings at intersections or at mid-block can act as barriers to pedestrians, especially where there are a high number of lanes or higher speeds. The crossing can be an impediment to travel if the pedestrian must cross four or more lanes at any speed or has to cross a 35 mph (or greater) street. The criteria for unsignalized intersection crossings depend on the functional class of the roadway, average daily traffic, speed limit, number of lanes, and presence of a median of sufficient width to provide for a two-stage crossing. Average daily traffic (ADT) of the roadway being crossed can be optional if data are not available by using the footnoted columns in the following exhibits. Over or underpasses are considered as separate facilities and are PLTS 1.

For functionally classified local and collector streets use Exhibit 14-31 for crossing with and without a pedestrian median refuge. The vast majority of these roadways should be under the 5,000 ADT limit for the table, but if it is known that a facility has an abnormally high amount of traffic for its functional class (there also should be a count performed on this section; (See APM Chapter 3), it should be compared with Exhibit 14-34 or Exhibit 14-35. If a collector-level roadway has more than two lanes or is one-way, then Exhibit 14-34 or Exhibit 14-35 should be used.

Unsignalized crossings at intersections or mid-block on functionally classified minor/major/principal arterial roadway sections should use Exhibit 14-32 for crossings without pedestrian median refuges. Sections with pedestrian refuge islands or are one-way should use Exhibit 14-34 and Exhibit 14-35. If ADT is not available for a section (or not possible to be estimated), use the midrange columns (as per table footnote) in these exhibits to find an appropriate PLTS. Enhanced arterial crossings (with or without refuge islands) can use Exhibit 14-33 to lower the PLTS to a maximum two-level reduction or minimum PLTS 2.

When a crossing lacks “standard” modern ramps, the facility is limited to able-bodied users. A standard modern ramp will have a flatter grade, may have a level landing surface, and some sort of detectable surface for visually impaired pedestrians (usually an etched-in cross hatching). Current ADA-standard ramps have a thermoplastic “truncated dome” insert attached to the ramp surface, so these are relatively easy to spot. Older ramps with short and or steep grades (these almost never have any detectable surfaces) are considered equivalent to no ramp at all. Impaired users will either not use the facility or will be forced into an uncomfortable position by using the street via a nearby driveway. In these cases, the minimum PLTS is 3.

Pedestrian median refuges need to be at least six feet in width (10 feet for PLTS 1 eligibility) and have a raised concrete or vegetated island for protection. Crossings at roundabouts should use PLTS 1 for a single lane crossing of an entry or exit assuming that the splitter island is at least 10 feet wide, otherwise use PLTS 2. Two-lane exits and entries are PLTS 2.

Increase the PLTS by one level (to a maximum PLTS 4) if the intersection or mid-block crossing is not illuminated in Exhibits 14-31 through 14-35. Unlit crossings require more awareness by the pedestrian as they are harder for drivers to see and/or expect in darkness.

Exhibit 14-31 PLTS on Collector & Local Unsignalized Intersection Crossing [1, 2, 3, 4]

Prevailing Speed or Speed Limit (mph)	No Median Refuge		Median Refuge Present
	Total Lanes Crossed		Maximum One Through/Turn Lane Crossed per Direction
	1 Lane	2 Lanes	
≤ 25	PLTS 1	PLTS 1	PLTS 1[5]
30	PLTS 1	PLTS 2	PLTS 1
35	PLTS 2	PLTS 2	PLTS 2
≥ 40	PLTS 3	PLTS 3	PLTS 3

[1]For street being crossed.
[2]Minimum PLTS 3 when crossing lacks standard ramps.
[3]Use Exhibit 14-34 or Exhibit 14-35 for one-way streets, when ADT exceeds 5,000, or total number of lanes exceeds two.
[4]Street may be considered a one-lane road when no centerline is striped and when oncoming vehicles commonly yield to each other.
[5]Refuge should be at least 10 feet for PLTS 1, otherwise use PLTS 2 for refuges 6 to <10 feet.

Exhibit 14-32 PLTS on Arterial Unsignalized Intersection Crossing Without a Median Refuge [1, 2]

Prevailing Speed or Speed Limit (mph)	Total Lanes Crossed (Both Directions)[3]					
	2 Lanes			3 Lanes		
	<5,000 vpd	5,000-9,000 vpd[4]	>9,000 vpd	<8,000 vpd	8,000-12,000 vpd[4]	>12,000 vpd
≤ 25	PLTS 2	PLTS 2	PLTS 3	PLTS 3	PLTS 3	PLTS 4
30	PLTS 2	PLTS 3	PLTS 3	PLTS 3	PLTS 3	PLTS 4
35	PLTS 3	PLTS 3	PLTS 4	PLTS 3	PLTS 4	PLTS 4
≥ 40	PLTS 3	PLTS 4	PLTS 4	PLTS 4	PLTS 4	PLTS 4

[1]For street being crossed.
[2]Minimum PLTS 3 when crossing lacks standard ramps.
[3]For one-way streets, use Exhibit 14-34 and Exhibit 14-35. Use PLTS 4 for crossings of four or more lanes.
[4]Use these columns when ADT volumes are not available

Exhibit 14-33 PLTS Adjustments for Arterial Crosswalk Enhancements[1]

Treatment[2]	Adjustment	Treatment	Adjustment
Markings[3]	-0.5	In-street signs	-1.0
Roadside signage[3]	-0.5	Curb extensions	-0.5
Illumination	-0.5	Raised crosswalk	-1.0
PAB (e.g. RRFB)	-1.0	Standard 12” flashing beacon	-0.5

[1]2.0 Maximum reduction or PLTS 2. Not intended for application at roundabouts.
[2]Pedestrian hybrid beacons (PHB) are equivalent to signalized crossings.
[3]Not applicable for roadways with pedestrian median refuges as crosswalk markings and roadside signage assumed as part of the basic installation.

Exhibit 14-34 PLTS Arterial Unsignalized Intersection Crossing (1 to 2 lanes) with a Median Refuge [1, 2]

Prevailing Speed or Speed Limit (mph)	Maximum Through/Turn Lanes Crossed per Direction			
	1 Lane	2 Lanes		
	Any	<5,000 vpd	5,000-9,000 vpd[4]	>9,000 vpd
≤ 25	PLTS 1[3]	PLTS 1[3]	PLTS 2	PLTS 2
30	PLTS 2	PLTS 2	PLTS 2	PLTS 2
35	PLTS 2	PLTS 2	PLTS 2	PLTS 3
≥ 40	PLTS 3	PLTS 3	PLTS 3	PLTS 4

[1]For street being crossed.
[2]Minimum PLTS 3 when crossing lacks standard ramps.
[3]Refuge should be at least 10 feet for PLTS 1, otherwise use PLTS 2 for refuges 6 to <10 feet.
[4]Use these columns when ADT volumes are not available.

Exhibit 14-35 PLTS Arterial Unsignalized Intersection Crossing (3 or more lanes) with a Median Refuge [1, 2]

Prevailing Speed or Speed Limit (mph)	Maximum Through/Turn Lanes Crossed per Direction			
	3 Lanes			4+ Lanes
	<8,000 vpd	8,000-12,000 vpd[4]	>12,000 vpd	Any
≤ 25	PLTS 1[3]	PLTS 2	PLTS 3	PLTS 4
30	PLTS 2	PLTS 2	PLTS 3	PLTS 4
35	PLTS 3	PLTS 3	PLTS 4	PLTS 4
≥ 40	PLTS 4	PLTS 4	PLTS 4	PLTS 4

[1]For street being crossed.
[2]Minimum PLTS 3 when crossing lacks standard ramps.
[3]Refuge should be at least 10 feet for PLTS 1, otherwise use PLTS 2 for refuges 6 to <10 feet.
[4]Use these columns when ADT volumes are not available.

The PLTS to cross the major street is applied to the minor street in the direction of travel along the route. If the crossing PLTS has a higher stress level than the minor street segment PLTS, the crossing PLTS applies (controls) to that minor street segment.

Signalized crossings usually provide a protected way across the roadway and are typically rated at PLTS 1. These also include midblock crossings with regular or PHB-type signals. The PLTS will be higher in areas if the following are evident:

- Permissive left or right turns. Pedestrians will need to be more wary about the potential for increased conflicts, so PLTS 2 is typically given in these cases.
- Missing basic features such as lighting or countdown pedestrian signal heads will increase the PLTS to PLTS 2.

- Presence of complex elements will increase the PLTS to PLTS 3:
    - Multiple or narrow (less than six feet) refuge islands where a pedestrian is not shielded or could wait,
    - No standard ramps,
    - Excessive crossing distance (>72'): more than six total lanes or lane equivalents such as allowances for parking, bike facilities, or painted medians crossed at once,
    - Non-standard geometry (more than four legs, or highly skewed approaches),
    - Closed or limited crosswalks available; Free-flow or yield-controlled channelized right turns

If the distance between crossing opportunities (i.e. signalized or a low-stress unsignalized) is greater than approximately 0.10 mile, then the resulting out-of-direction travel incurred by a pedestrian may be too great. This may deter or impede travel along a segment if the desired route includes a major street crossing.

14.5.11 Results

Mapping the PLTS for a community is a typical result from the analysis and can be readily done using GIS. Ideally, the displayed PLTS should be directional as it may differ on each side of a street. This will require some work with link offsets and layers to get this to show properly in GIS mapping software. The map shows the gaps and barriers in the system which can be used to inform stakeholders when creating a list of prioritized projects. The maps can also be included in planning documents and used to help inventory the pedestrian facilities.

14.5.12 Solutions to Decrease PLTS Level

There are several ways reduce PLTS and reach the chosen target for a roadway. Several publications including the Oregon Bicycle and Pedestrian Design Guide, and the ODOT Traffic Manual. The ODOT Highway Design Manual includes design considerations for pedestrian facilities. A few examples of actions that can reduce PLTS:

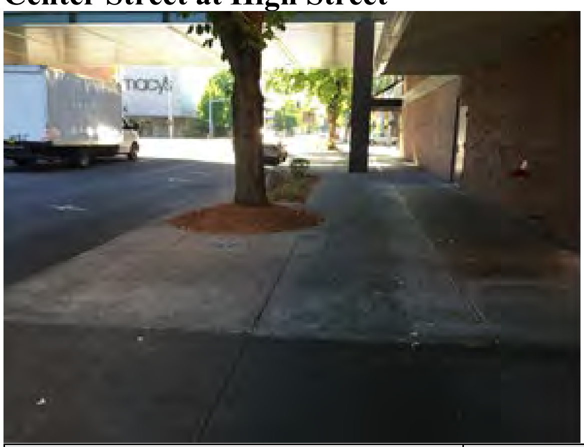
- Install pedestrian facilities, or expand facilities where pedestrian routes exist
- Create paved surfaces where there are trails or worn paths are evident
- Improve the condition of the sidewalk, including limiting vertical change and smoothing the surface
- Upgrade sidewalk ramps to current standards
- Infill gaps in sidewalk to create connectivity
- Redesign roadway to include wider or buffered sidewalks
- Create a multi-use path on a high-speed roadway
- Significantly change the roadway character and reduce speed limit

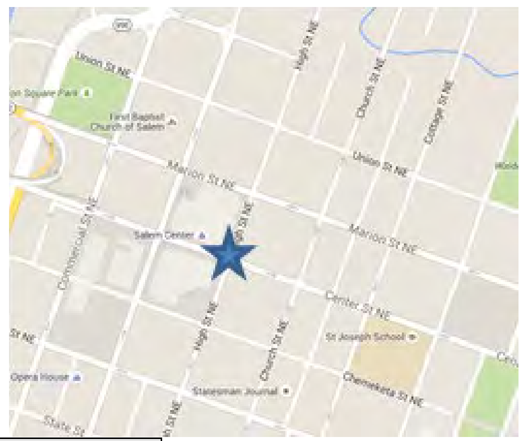
- Install additional crossing enhancements at unsignalized crossings (e.g. beacons, lighting, curb extensions)
- Remove barriers to connectivity
- Redesign buffer to include trees, large vegetation, and/or street furniture
- Land use changes over time to encourage more pedestrian-scale developments

Example 14-5 Pedestrian Level of Traffic Stress

The following are examples of corridor sections for each PLTS. All the examples are pedestrian facilities within the Salem city limits. The purpose of the example is to illustrate different PLTS levels.

Center Street at High Street





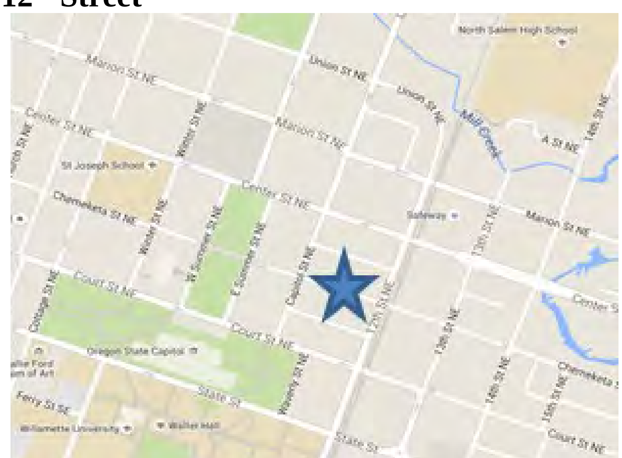
Street Name		**Center St at High St**
Sidewalk	Condition	Fair
	Width (ft)	6+
Buffer	Width (ft)	6
	Buffer Type	Solid Surface; street trees present
Bike Lane	Width (ft)	0
Parking	Width (ft)	8
Roadway	Number of Lanes	4
	Posted Speed (mph)	30
Land Use	Type	Central business district
Total Buffering Width (ft)		16

Center Street at High Street is located on a major roadway in downtown Salem. This segment is within the Salem Center Mall District with storefronts along the street. The segment contains a large 12-foot sidewalk with an effective width at least six feet and a solid surface buffer with street trees which leads to PLTS 1 ratings in the sidewalk and buffer type criteria. The total buffering width is just large enough to counteract the effect of the four-lane roadway, so the PLTS is 1. This location is within a central business district, so the general land use PLTS is 1. All of the categories are PLTS 1, so the overall PLTS is 1.

Street Name	**Center St at High St**	**Referring to:**
Sidewalk Condition	PLTS 1	Exhibit 14-21
Physical Buffer Type	PLTS 1	Exhibit 14-22
Total Buffering Width	PLTS 1	Exhibit 14-23
General Land Use	PLTS 1	Exhibit 14-24
Final PLTS	**PLTS 1**	

If a mid-block crossing of Center Street were to be analyzed, then the functional class of the roadway would need to be obtained. In this case, Center Street is an arterial. This is a one-way four-lane section, so ADT is not needed in the methodology. One-way sections need to use the tables for arterial streets with median refuges as the total lanes crossed are all in a single direction. The resulting PLTS would be 4 for a midblock crossing. This compares to the PLTS of 2 for the adjacent signalized intersections with permissive turns.

Chemeketa Street between Capitol Street and 12th Street



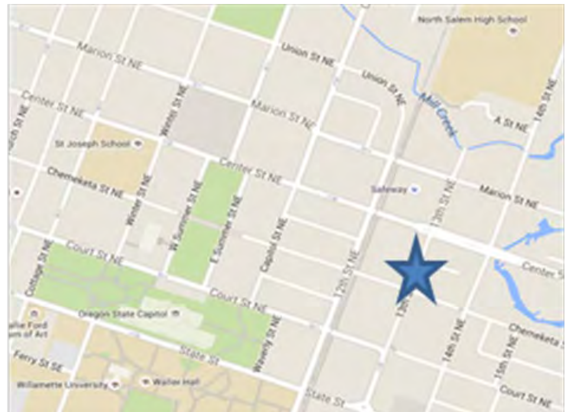
Street Name		Chemeketa St. between Capitol St & 12th St
Sidewalk	Condition	Good
	Width (ft)	5
Buffer	Width (ft)	10
	Buffer Type	Landscaped with trees
Bike Lane	Width (ft)	0
Parking	Width (ft)	15
Roadway	Number of Lanes	2
	Posted Speed (mph)	25
Land Use	Type	Office/Residential
Total Buffering Width (ft)		25

Chemeketa Street serves as a low volume street connecting 12th Street to parking areas around the Capitol mall area. The sidewalk condition is rated as good as it is of newer construction and has an actual width of five feet. This makes the facility a PLTS 2 under the sidewalk condition. The physical buffer type is landscaped with trees and the roadway has a 25-mph posted speed which makes the buffer PLTS 1. The total buffering width on this side of the roadway is 25 feet and there are two lanes on the roadway. This leads to the PLTS 1 for the total buffering width category. The general land use on this segment is offices and high density residential so the PLTS is 1. The sidewalk condition controls so the overall PLTS for this segment is 2.

Street Name	Chemeketa St. between Capitol & 12th St	Referring to:
Sidewalk Condition	PLTS 2	Exhibit 14-21
Physical Buffer Type	PLTS 1	Exhibit 14-22
Total Buffering Width	PLTS 1	Exhibit 14-23
General Land Use	PLTS 1	Exhibit 14-24
Final PLTS	**PLTS 2**	

If the adjacent intersection at 12th and Chemeketa were added to the segment, as would be done if a route was being investigated, the segment PLTS would not change. This signalized intersection has permissive left turns, but is free of complex elements, so the PLTS is 2, which is equal to the final segment PLTS.

13th Street at Chemeketa Street

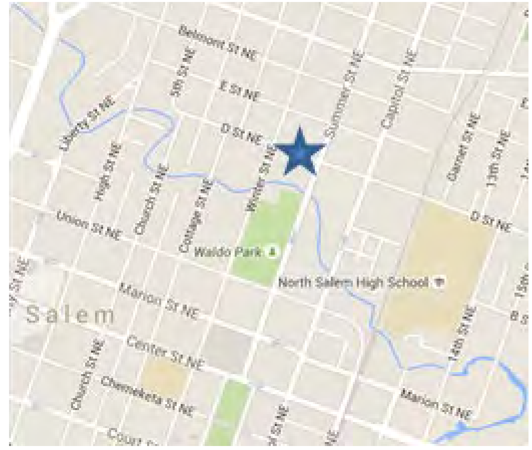


Street Name		13th St at Chemeketa St
Sidewalk	Condition	Good
	Width (ft)	5
Buffer	Width (ft)	4
	Buffer Type	Landscaped with trees
Bike Lane	Width (ft)	0
Parking	Width (ft)	0
Roadway	Number of Lanes	2
	Posted Speed (mph)	25
Land Use	Type	Office/Residential
Total Buffering Width (ft)		4

13th Street at Chemeketa Street is in the transition between downtown Salem and residential areas. With a sidewalk condition of good as it is of newer construction and a width of five feet the sidewalk condition PLTS is rated at 2. The buffer type is trees with a posted speed of 25 MPH which categories the facility at a PLTS 1. The total buffering width category is a PLTS 2. This is because the total buffering width is less than five feet and there are two travel lanes. This is in a mainly residential/office location, so the general land use PLTS is 1. The final PLTS for this facility is PLTS 2.

Street Name	13th St at Chemeketa St	Referring to:
Sidewalk Condition	PLTS 2	Exhibit 14-21
Physical Buffer Type	PLTS 1	Exhibit 14-22
Total Buffering Width	PLTS 2	Exhibit 14-23
General Land Use	PLTS 1	Exhibit 14-24
Final PLTS	**PLTS 2**	

D Street between Summer Street and Capitol Street



Street Name		**D St between Summer St & Capitol St**
Sidewalk	Condition	Fair
	Width (ft)	5
Buffer	Width (ft)	0
	Buffer Type	n/a
Bike Lane	Width (ft)	0
Parking	Width (ft)	0
Roadway	Number of Lanes	2
	Posted Speed (mph)	30
Land Use	Type	Residential
Total Buffering Width (ft)		0

D Street between Summer Street and Capitol Street is located on the edge of downtown Salem in a residential area. The sidewalk is in fair condition. There is no buffer between the sidewalk and the roadway. This, combined with the posted speed of 30 mph, categorizes this facility at a PLTS 3 and is the controlling PLTS.

Street Name	**D St between Summer & Capitol St**	**Referring to:**
Sidewalk Condition	PLTS 2	Exhibit 14-21
Physical Buffer Type	PLTS 3	Exhibit 14-22
Total Buffering Width	PLTS 2	Exhibit 14-23
General Land Use	PLTS 1	Exhibit 14-24
Final PLTS	**PLTS 3**	

If a crossing of D Street was to be analyzed, then the following additional information would be gathered:

- Functional Class = Collector
- ADT = 1600 vehicles per day
- Median refuge = Not present

Since D Street is a collector, ADT is not needed other than as a check to see that it is under the 5000 veh/day limit (typically it can be assumed that collectors and lower are under the limit without needing an ADT count to verify). Since there is no pedestrian median refuge, both lanes are crossed at once on this 30-mph roadway which is a PLTS 1.

Chemeketa Street at 14th Street

Street Name		**Chemeketa St at 14th St**
Sidewalk	Condition	Very Poor
	Width (ft)	5
Buffer	Width (ft)	8
	Buffer Type	Landscaped with trees
Bike Lane	Width (ft)	0
Parking	Width (ft)	7
Roadway	Number of Lanes	2
	Posted Speed (mph)	25
Land Use	Type	Residential
Total Buffering Width (ft)		15

Chemeketa Street at 14th Street is an old residential street with poor sidewalk condition. The sidewalk condition is very poor with several areas of substantial uplift and large cracks. This leads to the PLTS rating of 4 for sidewalk condition as it will make it impassable for disabled pedestrians and even difficult in spots for non-impaired individuals. The posted speed is 25 mph, and the buffer is a treed planter zone, so the buffer type is rated as PLTS 1. The general land use is residential, so this is a PLTS 1. The total buffer width is 15 feet, and the number of travel lanes is 2 for the roadway and because of these attributes the total buffer distance PLTS is 2. The overall PLTS for this segment is PLTS 4.

Street Name	**Chemeketa St at 14th St**	**Referring to:**
Sidewalk Condition	PLTS 4	Exhibit 14-21
Physical Buffer Type	PLTS 1	Exhibit 14-22
Total Buffering Width	PLTS 2	Exhibit 14-23
General Land Use	PLTS 1	Exhibit 14-24
Final PLTS	**PLTS 4**	

12th Street between Marion Street and Center Street

Street Name		12th St at Center St
Sidewalk	Condition	Poor
	Width (ft)	3
Buffer	Width (ft)	0
	Buffer Type	N/A
Bike Lane	Width (ft)	0
Parking	Width (ft)	0
Roadway	Number of Lanes	4
	Posted Speed (mph)	30
Land Use	Type	Mixed employment
Total Buffering Width (ft)		0

The 12th Street corridor is a moderate speed and volume facility in a mixed commercial/office area. The sidewalks along the west side of the roadway are narrow at three feet and in poor condition. This leads to a PLTS of 4 for sidewalk condition. There is no buffer and speed of 30 mph on the roadway which leads to a PLTS 3 for the buffer type. The total buffer distance is zero feet, and the total number of travel lanes is four, which is a PLTS 4 in the total buffer distance category. The general land use is a mix between commercial uses, offices and large employee parking lots, so this would be generally PLTS 2. With one or more categories at PLTS 4, the segment of roadway is a PLTS 4.

Street Name	12th St at Center St	Referring to:
Sidewalk Condition	PLTS 4	Exhibit 14-21
Physical Buffer Type	PLTS 3	Exhibit 14-22
Total Buffering Width	PLTS 4	Exhibit 14-23
General Land Use	PLTS 2	Exhibit 14-24
Final PLTS	**PLTS 4**	

If the adjacent intersections at 12th/Center and 12th/Marion were added to the segment as would be done if a route was being investigated, neither intersection's PLTS would control the overall segment. Both signalized intersections have permissive turns but are free of complex elements and would have a PLTS of 2, but these are still lower than the PLTS 4 for the segment.

14.6 Multimodal Level of Service

The Level of Service (LOS)–based methods presented in this section are intended for use when a detailed analysis is desired such as in facility plans or projects when a no-build alternative is compared to one or more build alternatives. These methods are not meant for defining overall needs or making prioritization decisions, those types of applications should use the Qualitative Multimodal Assessment or Level of Traffic Stress methodologies instead (see sections 14.2 to 14.4).

The Auto mode is not included as analysis at this level of detail would typically be done at intersections with applications such as Synchro, Highway Capacity Software, or Vistro. Application of the methodologies is via Excel-based calculators available on the Transportation Development – Planning Technical Tools webpage.

The full MMLOS methods as published in the HCM (Urban Street Method) have issues with being overly data-intensive. The following ODOT modified versions of MMLOS analysis are to be used. Some of these are simplified versions of the HCM methods which eliminate the onerous parts of the calculations, while others provide more specific procedures, default values and/or tools. Exhibit 14-36 summarizes the ODOT MMLOS methods including the APM sections and calculators for each.

Exhibit 14-36 ODOT Multimodal Level of Service Methods in the APM

APM Section	Method/Facility Type	Description	Calculator
14.9	Segment Pedestrian LOS	PLOS based on a simplified re-estimation of the original video clip data used to create the HCM Pedestrian LOS using fewer variables	Simplified MMLOS Calculator
14.10	Segment Bicycle LOS	BLOS based on a simplified re-estimation of the original video clip data used to create the HCM Bicycle LOS using fewer variables	Simplified MMLOS Calculator
14.11	Separated Bicycle Lanes	BLOS of separated bicycle lanes. Augments the re-estimated HCM bicycle methodology	Separated/Buffered Bikeways Calculator
14.12	Buffered Bike Lanes	BLOS of buffered bicycle lanes. Augments the re-estimated HCM bicycle methodology	Separated/Buffered Bikeways Calculator
14.13	Shared-use Paths	BLOS and PLOS for paved shared-use (multi-use) paths. Full application of the HCM method with addition of computational engine.	Shared Path Calculator
14.14	Unsignalized Intersections (TBD- In Progress)	TBD	TBD
14.16.1	Pedestrian Signalized Intersection LOS	PLOS for pedestrian crossings at a signalized intersection.	Pedestrian and Bicycle Signalized Intersection MMLOS Calculator
14.16.2	Bicycle Signalized Intersection LOS	BLOS for bicyclist crossing at a signalized intersection.	Pedestrian and Bicycle Signalized Intersection MMLOS Calculator
14.17	Transit LOS	Segment Transit LOS for fixed-route transit vehicles operating in exclusive or mixed-use lane. May include buses, BRT, streetcars, or LRT operating in mixed mode street-running conditions. Based on the HCM Transit LOS method using default assumptions.	Simplified MMLOS Calculator

14.7 Re-estimated Pedestrian & Bicycle Link Level Methodology Application

The pedestrian and bicycle procedures in Sections 14.9 to 14.12 are re-estimated versions of the link-level full *Highway Capacity Manual* (*HCM*) *2010* Multimodal Level of Service (MMLOS) methodologies. The use of probabilistic methodologies with the original research data allowed the number of variables to be significantly reduced while maintaining or improving accuracy of the results. These simplified procedures will still produce a Level of Service (LOS) letter grade, will indicate the current “state of the system”, and can be done in a fraction of the time that the full MMLOS methodology requires”.

> *These methodologies only include link-level detail. There are several issues with the intersection-level LOS in the full HCM MMLOS method that create non-intuitive results when combined with the link-level LOS by obscuring or limiting changes (the full method is rather insensitive to change compared to the links-only portion). Non-HCM but consistent analysis procedures for capturing the LOS for unsignalized and signalized street crossings for the bicycle and pedestrian modes will be added at a later date.*

These procedures are intended for application on urban arterial (excluding freeways)/collector-classed roadways. Roadways are segmented to ensure demand, control, and geometry are relatively uniform within each segment. Caution should be exercised if applying these on functionally classified local streets as results may not be intuitive. The Qualitative Multimodal Assessment (QMA, see Section 14.3) should be used for applications in other areas such as unincorporated communities and rural areas.

14.8 <u>Link Level Pedestrian & Bicycle LOS Criteria</u>

LOS scoring threshold criteria for pedestrian and bicycle modes shown in Exhibit 14-37Se are based on the updated HCM values.

Exhibit 14-37 Pedestrian and Bicycle LOS Criteria

LOS	Pedestrian & Bicycle LOS Score
A	≤1.5
B	>1.5 – 2.5
C	>2.5 – 3.5
D	>3.5 – 4.5
E	>4.5 – 5.5
F	>5.5

Multimodal LOS scores are based on user perceptions (traveler satisfaction) and are graded from best (LOS A) to worst (LOS F). This kind of perception-based rating varied from the many test respondents (there is no one single definition of a multimodal LOS grade) and was eventually grouped into LOS ranges. The methodology results represent the probability that a user (or the population of users) will pick a given LOS (or a range of LOS). Better conditions will result in better LOS scores. For example, narrower slower streets will rate better than wider faster ones for pedestrian and bicycle modes. Presence of sufficient-width sidewalks and bike lanes will score better than streets without them. Since these methodologies are a prediction of the user perception of quality of service, the LOS results need to be evaluated in context with other planning considerations (e.g. available funding for improvements, land use context, etc.).

14.9 Segment Pedestrian LOS

14.9.1 Methodology Summary

This methodology is based on a re-estimation of the original video clip data used to create the HCM Pedestrian LOS (*National Cooperative Highway Research Program* (*NCHRP*) *Project 3-70* and *Report 616 Multimodal Level of Service Analysis for Urban Streets*). Details on the research and methodology approach can be found in the paper *Cumulative Logistic Regression Model for Pedestrian Level of Service Rating* by Ali, Cristei, and Flannery, George Mason University (undated). By re-estimating the model using the individual response surveys instead of averages of *NCHRP Project 3-70*'s data, the researchers were able to isolate the variables that most significantly impact the pedestrian LOS. This allowed a significant reduction in the number of independent variables needed while creating a better LOS estimate using probability-based ranges.

Of the seven sidewalk-related independent variables in the full HCM method (i.e. sidewalk width, buffer width, presence of barriers, etc.), the strongest variable influencing pedestrian comfort was sidewalk width. The major traffic-related independent variables (same direction traffic volumes, number of traffic lanes, and speed limit) were all found to have strong negative impacts on pedestrian comfort. All the variables used in the model have categorized ranges of data input, so it is only necessary to know on what side of a threshold a data item lands, rather than the actual absolute amount.
14.9.2

14.9.3 Data Needs and Definitions

The simplified methodology uses four variables to estimate Pedestrian LOS. The analysis is intended to be applied on road segments on a per direction basis like most HCM-based methods. The variables and their category values are shown below:

- Sidewalk (Actual) Width (0-5 ft or >5 ft)
- Directional Traffic Volume (0-500 vph, 500-1500 vph, or >1500 vph)
- Number of (Through) Traffic Lanes per direction (1, 2, 3, or 4)
- (Posted) Speed Limit (20-40 mph or >40 mph)

Segments are at least defined between major (signalized) intersections or where the threshold values change between categories. For example, a change from 30 to 35 mph would not be significant in this method, but a change from 40 to 45 mph would be as the value changes categories and a new segment would also need to be created. Similarly, if a street had no sidewalks (zero feet in width) but had a section of six-foot sidewalk in the middle, then the street section would be broken into three segments (two 0-5 ft width

sections and one >5 ft width section). However, if the sidewalk section was a substandard three-foot width, then one overall segment would suffice.
Sidewalk width is the actual width, not the effective or clear width. The methodology implicitly assumes the larger sidewalk widths may also include increased buffer space with physical barriers (bike racks, meters, trees, etc.). These buffers and barriers generally increase the overall comfort (assuming that any elements do not intrude into the walking space) for a pedestrian resulting in better LOS levels.

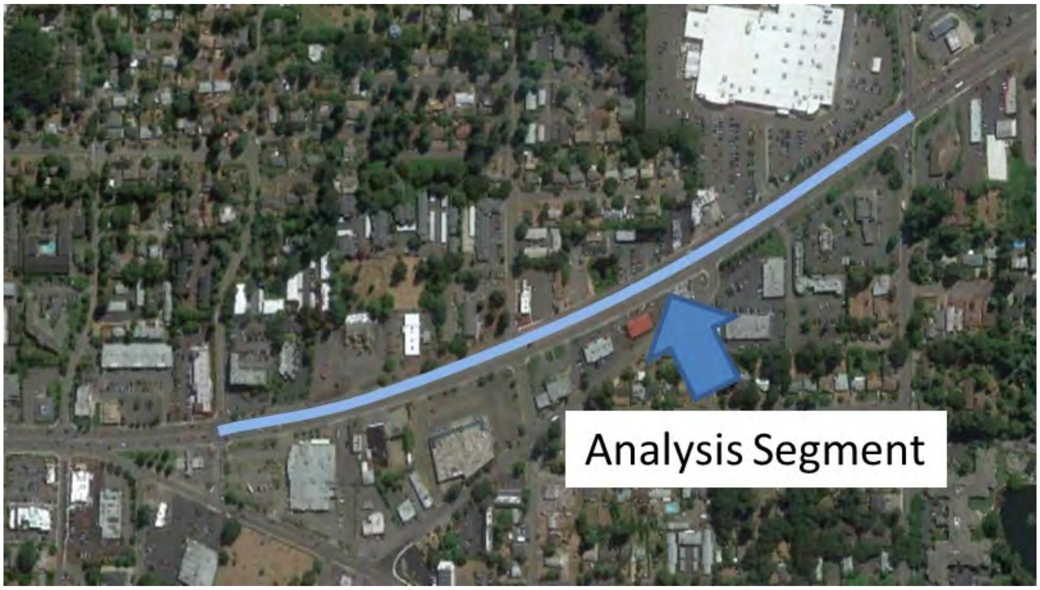
The Directional Traffic Volume is intended to be consistent with the analysis peak hour used for other analysis tasks, such as vehicle v/c ratio or LOS. It is possible that the final pedestrian LOS may be different between different peak hours such as AM and PM. Creation of the existing or future volumes should follow Chapter 5 or 6 using hourly counts or appropriate reductions from daily counts.

The number of lanes per direction considers the impact of through and shared through/turn lanes only. Ignore any center two-way left-turn lanes or exclusive turn lanes as this methodology is only for segments and not for crossings. There is no difference in the methodology between one-way and two-way segments except that all through lanes would be considered on a one-way segment for each side of the roadway (this is where the three and four lanes per direction will be mostly applied), instead of just half. The speed limit used to select the category should be the posted or statutory limit (i.e. 20 mph for downtown or 25 mph for residential areas, etc.).

The data values should be easily obtainable from inventories or aerial photographs. Results can be shown in tables or in a GIS-created map figure. A network-wide LOS could be estimated using a travel demand model with custom variables or expressions. The directional volumes, speeds, and lanes are common base variables. The sidewalk variable could be assumed, based on field data, or obtained from the model if it considered pedestrian trips in greater detail.

The values are entered into the Excel calculator to obtain the cumulative probabilities which are subtracted from one another to obtain the LOS probabilities. The highest probability is chosen as the most-likely LOS. Check to see if any probabilities fall within a (0.90 x the highest probability) range. If the next highest probability is within this range, assume that this is a LOS range (i.e. LOS E-F) with a total probability that is the sum of the individual probabilities.

Example 14-6 Pedestrian LOS



A segment of a five-lane suburban arterial is analyzed for the afternoon peak hour as part of a local transportation system plan. The roadway has a peak month ADT of 31,000 and has a 35-mph speed limit. Six-foot sidewalks and bike lanes exist on both sides of the street. The roadway traverses a commercial district so there are a substantial number of driveways on both sides. From count data at nearby intersections, there is 50/50 directional split (D-factor) and the percent of the daily traffic as part of the peak hour (K-factor) is 9%.

The ADT is converted into an approximate peak hour volume by multiplying the ADT by the segment K-factor. The peak hour volume is then converted into a directional volume by multiplying it by the directional split. (See Chapter 5 for more information on determining peak hour volumes).

Directional volume (vph) = ADT * K-Factor * D-factor = 31,000 * 0.09 * 0.50
= 1395 vph which falls into the 500-1500 vph category

From the existing conditions data given, sidewalks are greater than five feet, there are two lanes per direction, and the speed is between 20 and 40 mph.

These four pieces of data are entered into the calculator and the highest probable result is LOS C at 25.51%. Other close LOS possibilities indicated are LOS B at 21.48%, LOS D at 21.29% and LOS E at 20.12%. The 10% significance check value (90% of the LOS C value) is 22.96% which is greater than any of the other LOS probabilities, so the LOS C value is reported as the final result for the first direction. The calculator will always provide multiple segment probabilities for the purpose of helping the user decide which LOS to report, rather than reporting just the highest probability.

Final LOS Probabilities						Max	Probability	Final
F	E	D	C	B	A	Probability	90% Check	LOS
0.0832	0.2012	0.2129	0.2551	0.2148	0.0329	0.2551	0.2296	C

LOS C has the greatest chance of selected as the user-perceived value. This also can be thought over the percentage of the population that would view this as LOS C. Adding in other ranges; about 50% would view this section as acceptable as LOS A-C while 50% would view this section poorly as LOS D-F. LOS C is the approximate middle ground. A 50% poor probability is significant, so some improvement is likely necessary for this segment. The analysis is repeated for the second direction which will have the same answer in this case as the conditions are symmetrical.

14.10 <u>Segment Bicycle LOS</u>

14.10.1 Methodology Summary

This methodology is based on a re-estimation of the original video clip data used to create the HCM Bicycle LOS in *NCHRP Project 3-70* and *Report 616*. Details on the research and methodology approach can be found in the paper *Using Cumulative Logistic Regression Model for Evaluating Bicycle Facilities on Urban Arterials"* by Ali, Cristei, and Flannery, George Mason University (undated).

By re-estimating the model using the individual response surveys instead of averages of *NCHRP Project 3-70*'s data, the researchers were able to isolate the variables that most significantly impact the Bicycle LOS. This allowed a significant reduction in the number of independent variables needed while creating a better LOS estimate using probability-based ranges. In addition, the issue with the coefficients in the full MMLOS method, which generally prevent obtaining LOS A or B, has been eliminated.

Of the 13 independent variables in the full HCM method (e.g., volume, pavement condition, etc.) in *Project 3-70*, only four were found to be significant in the re-estimation. In addition, based on the validation in the research, it appears utilizing the other nine variables does not warrant the level of effort needed to obtain them. In other words, the time spent calculating pavement condition, heavy vehicles, percentage of on-street occupied parking, etc. does not enhance the ability to obtain an accurate LOS. This means, for many applications, there is no need to conduct more-detailed Bicycle LOS analysis.

The method in this section is only meant for analysis applications on roadway sections with shared-use vehicle lanes or regular bike lanes because of limitations in the original research dataset. Higher speed/volume roadways will most likely score poorly and indicate the need for some sort of separation of bicyclists from vehicular traffic. Following future sections will be applicable to other facility types. The LOS of this section would need to be compared with the generated LOS for other facility types to either establish operational ranges or a single LOS for an alternative (LOS with shared lane, LOS with bike lane, LOS with shared path, etc.). The Qualitative Assessment (see Section 14.3) may be the best choice if more factors are desired to be included in a facility plan analysis (See Section 14.2) without the limitations of the full MMLOS method. The simplified re-estimated methodology is also consistent with the Bicycle Level of Traffic Stress (see Section 14.4) if that method is used as a screening tool in a detailed refinement plan or project effort as poor LOS levels will result in poor segment stress levels and vice versa.

The major bicycle-related variables are presence or absence of a bike lane/usable paved shoulder and the number of unsignalized conflicts per mile, both of which are responsible for most of the variation in the LOS ratings from the response surveys and thus will have the biggest impact on the LOS results. For example, a section of roadway without any unsignalized driveway approaches could have a LOS D while the presence of driveways drops the section to a LOS F. The most significant vehicle-traffic related variables are the number of through traffic lanes and the posted speed limit, both of which have a negative impact on bicyclist comfort. All the variables used in the model have categorized ranges of data input, so it is only necessary to know on which side of a threshold is the particular data item, rather than the absolute value.

14.10.2 Data Needs and Definitions

The methodology uses four variables to estimate Bicycle LOS. The analysis is intended to be applied on road segments on a per direction basis like most HCM-based methods. Segments can be defined between major intersections or as desired. The variables and their category values are shown below:

- Number of Through Traffic Lanes per direction (1 or >1)
- Bike Lane or Paved Shoulder Present (Yes or No)
- (Posted) Speed Limit (<= 30 mph or >30 mph)
- Unsignalized Conflicts (Yes or No)

Like with the Pedestrian LOS, segments should be created at least between major intersections or when the variables change categories. For example, if the bike lane disappears along a roadway, then reappears later; a new segment is needed every time this happens. Short sections should be highlighted and documented especially if they are due to narrow bridges or other physical obstructions.

The number of lanes per direction is for through and shared through/turn lanes only. An exclusive turn lane could be considered if it extends the length of the segment. Ignore any center two-way left-turn lanes or exclusive turn lanes as this methodology is only for segments and not for crossings. A bike lane is assumed where there is a striped lane or where a useable paved shoulder that allows the bicyclist to be direct conflict with traffic exists. Mixed traffic conditions where there is no striped bike lane or shoulder stripe should be assumed to fall into the “No” category. The speed limit used to select the category should be the posted or statutory limit (i.e. 20 mph for downtown or 25 mph for residential areas, etc.).

The unsignalized conflicts account for the impact of any unsignalized intersections or driveways in the segment. All driveways (residential/commercial/industrial) should be accounted for as each creates potential conflict locations regardless of driveway volume.

The data values should be easily obtainable from inventories or aerial photographs. Results can be shown in tables or in a GIS-created map figure. A network-wide LOS could be estimated using a travel demand model with some use of custom variables or expressions. The directional speeds and lanes are common base variables. The unsignalized conflict variable would likely need to be defaulted to “yes” unless the facility segments were access–controlled and still legally allowed bikes. The bike lane variable could be assumed, based on field data, or obtained from the model if it considered bicycle trips in greater detail.

The values are entered into the calculator to obtain the cumulative probabilities which are subtracted from one another to obtain the LOS probabilities. The highest probability is chosen as the most-likely LOS. Check to see if any probabilities fall within a (0.90 x the highest probability) range. The calculator will highlight any probability that falls within 90% of the highest value. If the next highest probability is within this range, assume that this is a LOS range (i.e. LOS E-F) with a total probability that is the sum of the individual probabilities.

This modal methodology has the highest occurrence of LOS ranges. Probabilities that are greater than 10% apart can also be reported as a range. For example, LOS A is not possible as a reported final LOS with the most favorable parameters, but it does have a good likelihood of occurring. In this case, the final result could be reported as LOS B or LOS A-B depending on the engineer’s judgment of the overall context of that particular segment.

Example 14-7 Bicycle LOS

This example uses the same segment of roadway as described in the Pedestrian LOS section. A segment of a five-lane suburban arterial is analyzed for the afternoon peak hour as part of a local transportation system plan. The roadway has a 35-mph speed limit. Six-foot sidewalks and bike lanes exist on both sides of the street. The roadway traverses a commercial district so there are a substantial number of driveways on both sides.

From the given data, the number of traffic lanes per direction exceeds one, there is a bike lane present, the speed limit exceeds 30 mph and there are unsignalized intersection and driveway conflicts.

The four data elements are added into the calculator and the highest probable result is LOS F at 27.11% for the first direction. The 10% significance check (90% of LOS F) is

24.40%, which is about equal to the LOS E value at 24.41%. The calculator highlighted the highest probability and any that fall within 90% of that value. The overall reported result is a range and the probabilities added together which results in a 51% chance of a LOS E-F.

Final LOS Probabilities						Max	Probability	Final
F	E	D	C	B	A	Probability	90% Check	LOS
0.2711	0.2441	0.2081	0.1724	0.0832	0.0210	0.2711	0.2440	F

The calculator will always provide multiple probabilities for every segment for the purpose of helping the user decide which LOS to report rather than just reporting the highest probability. In this case, the “Final LOS” column would need to be manually overridden (as per the spreadsheet directions) to reflect a LOS E-F so it can be shown correctly on the summary output sheet. The same LOS E-F result is computed for the second direction as conditions are the same.

14.11 Separated Bicycle Lanes

The separated bike lane methodology augments the re-estimated HCM bicycle methodology of the previous section by adding new facility types. Low-stress tolerant users desire a greater degree of separation between them and the adjacent traffic stream. The standard bike lane or even a buffered bike lane does not offer the amount of separation needed especially for roadways with higher volume and/or speeds. Separated bicycle lanes (also known as cycle tracks or protected bike lanes) offer additional comfort (lower stress) to the bicyclist by creating a vertical delineation between the bicycle lane and the vehicle lanes and are a step up from a buffered bicycle lane. Vertical delineation can be simple as a line of posts (candlesticks), bollards, to large planters, to physically separating the bikeway with the vehicle parking strip. Exhibit 14-38 illustrates the differences in separation for the typical bicycle facilities from least to most.

The LOS is relatively poor for sections of standard bike lanes on higher speed and/or volume urban roadways, so adding a separated bike lane will allow for a better LOS. If a previously conducted Bicycle Level of Traffic Stress analysis indicated system needs, adding a separated bike lane on major routes can further enhance or help establish a low-stress network. For separated bicycle lanes to have the greatest benefit, they need to intersect other lower-stress bicycle facilities such as bike boulevards, streets with standard bike or buffered lanes, or even low-speed routes with sharrows rather than being an isolated facility. Routes with established substantial bicycle volumes or more direct routes that have limited use because of high-stress elements may be good candidates for separated bicycle lanes. Separated bicycle lanes appeal to the largest segment of current and potential bicyclists, so having them in certain high-connectivity corridors should help to increase the overall mode share along a route and increase the total amount of users.

The context of the corridor should be considered on whether a separated bike lane is the appropriate treatment. Not all roadways are suitable for separated bicycle lanes. Separated bicycle lanes have the greatest benefit on roadways with no or limited driveways and wider spaced intersections to maximize bicycle flow and minimize potential conflicts. Every intersection and driveway is a point of conflict and can introduce safety and operational issues especially when paired with adjacent parking. Parking between the travel lane and the separated bike lane can create sight distance issues. If sight distance is not maintained sufficiently (by prohibiting parking close to the intersection/driveway) then this may encourage vehicles to creep out and block the bike lane while waiting to turn. Higher volume and/or many driveways can substantially impede operations of bikes and increase the risk of collisions. The parking can also create visibility issues for drivers to see oncoming bicyclists (could be in both directions for a two-way bike lane) as they turn into a driveway and across the bike lane. If access management solutions to consolidate/minimize driveways are not possible, then a buffered bike lane may be more appropriate in a parking and /or driveway dense location.

A constrained right-of-way, and/or existing features (e.g. number of driveways or parking needs) may pose design challenges. This analysis should not be done in isolation as safety shall be evaluated whether the features associated with the separated bike lane treatment may affect bike users or another transportation mode’s safety. For example, more substantial separators such as bollards or large planters could create a fixed-object hazard for vehicles as they are close to the lane edge especially with higher speeds. The analyst needs to discuss the applicability of separated bicycle lanes with Region, Traffic-Roadway Section, or local jurisdiction roadway/bicycle-pedestrian staff (as appropriate) before pursuing a specific separated bike lane treatment.

14.11.1 Methodology Summary

This methodology is based on the paper, *A Level-of-Service Model for Protected Bike Lanes* by Foster, Monsere, Dill and Clifton, Portland State University, 2014. The research was based on recent video-clip data obtained in a similar manner as previous HCM research efforts. The methodology uses the same cumulative logistic model form as the re-estimated bike lane method (Section 14.7) so results will be consistent between the two. The methodology is limited to segments only. Intersection crossings will be covered in future sections.

The methodology does not cover roadways that have a substantial number of driveways and/or higher volume driveways as most of the research was based in central business districts or residential areas where high numbers of driveways or high-volume driveways or were uncommon. The most significant variables for estimating the performance of separated bicycle lanes are buffer type, direction of travel, and adjacent vehicle speed and daily volume. The resulting LOS scores are based on user perceptions of each video clip and are graded from best (LOS A) to worst (LOS F). The methodology results represent the probability that a user (or the population of users) will pick a given LOS (or a range of LOS). Better conditions will result in better LOS scores. An Excel-based calculator is available on ODOT’s Planning Section webpage.

14.11.2 Data Needs and Definitions

The methodology uses the following four variables to estimate the separated bike lane LOS. The analysis is intended to be applied on road segments on a per direction basis like most HCM-based methods. Segments with two-way separated bicycle lanes only need to be evaluated in one direction. The variables and their category values are shown below:

- Buffer type (posts, planters, parking strip, raised/parking)
- Direction of (bikeway) travel (one-way or two-way)
- Adjacent vehicle speed (25 – 35 mph)
- Average daily volume in both directions (9,000 – 30,000) vehicles per day

Segments are at least defined between major (signalized) intersections or where the threshold values change between categories. For example, a change from 25 to 30 mph or a change in vertical delineation type would be significant and a new segment would be needed.

Exhibit 14-38 Bicycle Facility Separation[10]

Exhibit 14-39 illustrates the different kinds of buffer types used in this methodology. Post-type (candlestick) buffers are the easiest to implement however they do not have the same sort of separation benefits as more physical barriers such as planters. The planter type buffer in the methodology is for substantially sized planters such as shown in Exhibit 14-39. Smaller planters are typically paired with posts so these should use the post-type buffers instead. The parked cars buffer type occurs when the parking is placed between the motor vehicle lanes and the bike lane. There should be enough buffered space, so open vehicle doors do not interfere with the bicyclist's path. Some designs have

[10] Separated Bike Lane Planning and Design Guide, FHWA, May 2015, p.14.

the separated bike lane raised slightly higher than the adjacent travel lanes but less than the adjacent sidewalk. If parking is provided along this type, then the parking is raised to the same level as the bike lane. This configuration is also known as a raised bike lane. Raised bike lanes were included in the study but not called out as their own buffer type as there were not enough separate sites. These should use the “raised/parking” buffer type as an equivalent.

Exhibit 14-39 Separated Bike Lane Buffer Types[11]

Posts

Parked cars

Planters

Raised

Most separated bicycle lanes are one-way in the direction of roadway travel but there are situations such as on a one-way street where a contra-flow bike lane is desired to limit out-of-direction travel. Separated bicycle lanes in these cases are typically two-way facilities. Two-way separated bicycle lanes require more considerations regarding intersections and driveways, so coordination with ODOT Region, Headquarters Traffic, or Roadway Section staff is necessary.

The methodology is limited to speeds between 25 and 35 mph and ADT values between 9,000 and 30,000 vehicles per day and thus will be limited in most cases to arterials in denser urban locations. Use caution if values extend outside of these limits. Higher volumes and speeds than the methodology limits will tend to make the LOS better than expected while lower volumes and speeds will make the LOS worse than expected. Lower speed and lower volume roadways will be more applicable to shared markings, a standard bike lane, or buffered bike lanes. Roadway applications with higher volumes or

[11] Images from Separated Bike Lane Planning and Design Guide, FHWA, May 2015, pp. 83-87.

speeds should gravitate toward total separation with a shared-use path (Section 14-13).

The values are entered into the calculator to obtain the cumulative probabilities which are subtracted from one another to obtain the LOS probabilities. The highest LOS probability is chosen as the most-likely LOS. The calculator will also flag any values within 10% of the highest probability. Check the LOS probabilities in the calculator for any highlighting. The highlighting will indicate the potential for a LOS range (i.e. LOS A-B) with a total probability that is the sum of the individual probabilities. Judgement based on the overall project context is required to decide to leave the LOS as calculated or override it to a lower/higher LOS or create a LOS range.

Example 14-8 Separated Bike Lane LOS

This example uses the same segment of roadway as described in the previous sections on Pedestrian and Bicycle LOS. The roadway has five lanes, a 35-mph speed limit and a peak month ADT of 31,000. The roadway currently has six-foot bike lanes on both sides. The previous Bicycle LOS analysis indicated a LOS E-F for the no-build conditions with just standard bike lanes. It was desired to improve the bicycle network in this area with the addition of a separated bikeway using a post-type of buffer.

From the given data, the buffer type is posts, the direction is one-way, the speed is 35 mph, and the ADT is 31,000. It was noted that the ADT was slightly outside of the top range (30,000) but judged close enough not to have too much LOS overestimation (at 30,000 ADT the highest probability is LOS B at 38.44%) or any non-intuitive results.

The four data elements are added into the calculator and the highest probable result is LOS B at 38.27%. The 10% significance check (90% of LOS B) also captures LOS A at 37.40% so a LOS range is possible. The calculator will always provide multiple probabilities for every segment for the purpose of helping the user decide which LOS to report rather than just reporting the highest probability. However, because of the chance of some LOS A overestimation with the high ADT, the LOS was left at LOS B instead of going to LOS A-B as seen below.

Final LOS Probabilities						Max	Probability	Final
A	B	C	D	E	F	Probability	90% Check	LOS
0.3740	0.3827	0.1757	0.0428	0.0160	0.0087	0.3827	0.3445	B

14.12 Buffered Bike Lanes

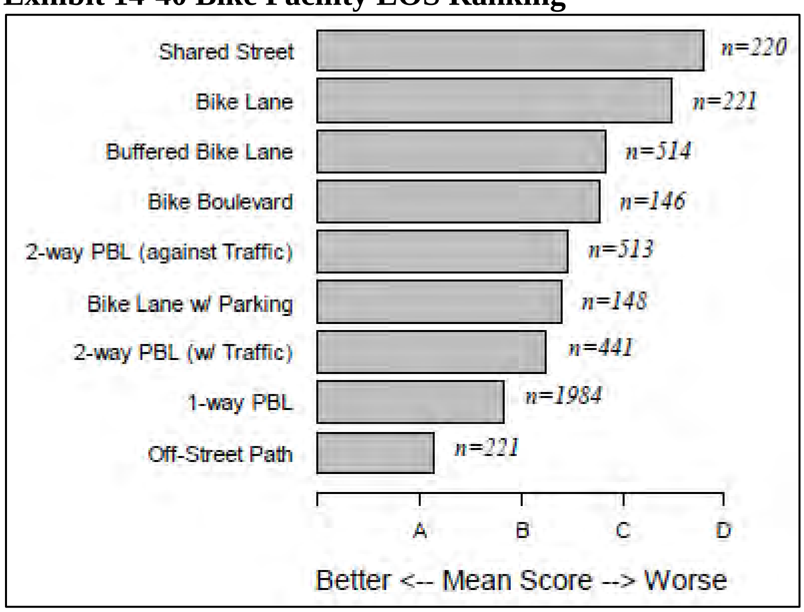
Buffered bike lanes offer additional comfort to the bicyclist by providing separation from the vehicle lanes using a striped and/or hatched buffer. Buffered bike lanes should be used when speeds exceed 25 mph or when volumes are more than 2,500 ADT and should be used up to speeds of 35 mph and/or 5,500 ADT[12]. Beyond these values, separated facilities should be used. This type of bike lane will allow a more acceptable LOS grade than a standard bike lane for higher volume and speed facilities, such as most urban state highways. Buffered bike lanes may also be a good compromise in areas with a substantial number of driveways that would make operations of a separated bikeway difficult or create a number of safety issue locations because of visibility/sight distance.

Methodology Summary

The methodology for the estimation of a buffered bike lane LOS is based on an extension of Exhibit 14-40. In the exhibit, the "n" is the observation sample size, and "PBL" is a protected (separated) bike lane. The LOS of a buffered bike lane falls in the LOS B-C range which is approximately halfway between the LOS of a standard bike lane at LOS C-D and a separated bike lane at LOS A-B. The estimated buffered bike lane LOS is obtained from averaging the LOS scores of both the bike lane and separated bikeway since both methods use the same cumulative logistic regression model form. This procedure is considered an interim method until better facility-specific methodologies are available. An Excel-based calculator is available on ODOT's Planning Section webpage which combines both methodologies for a quick but separate (i.e. the presence of unsignalized conflicts only applies to the standard bike lane methodology) comparison.

This estimated method is best applicable within the ranges of the separated bike lane methodology (speeds 25-35 mph & 9,000-30,000 AADT). Higher volumes and speeds (vice versa for low speeds/volumes) will tend to make the separated bike lane LOS better while the bike lane LOS becomes worse which will generally make the differences balance out between them.

Exhibit 14-40 Bike Facility LOS Ranking[13]



[12] Bicycle Facility Tier Identification Matrix, ODOT Blueprint for Urban Design, 2019, p 3-15.
[13] Foster, N., et.al, A Level-of-Service Model for Protected Bike Lanes, Fig.2, p. 7-8.

The standard and separated bike lane variables are entered into the calculator to obtain the cumulative probabilities which are subtracted from one another to obtain the LOS probabilities. The highest LOS probability is chosen as the most-likely LOS. The calculator will also flag any values within 10% of the highest probability. Check the LOS probabilities in the calculator for any highlighting. The highlighting will indicate the potential for a LOS range (i.e. LOS A-B) with a total probability that is the sum of the individual probabilities. Judgement based on the overall project context is required to decide to leave the LOS as calculated or override it to a lower/higher LOS or create a LOS range.

Once the base LOSs for the standard and separated bike lanes are calculated and optionally adjusted, the LOS grades are converted into scores which are averaged together and then reconverted into an estimated buffered bike lane LOS. Application is best for future scenarios as the LOS for a standard bike lane, buffered bike lane, and separated bike lane will all be shown for comparison but can also be used for an existing buffered bike lane (use posts buffer and a one-way bikeway).

Example 14-9 Buffered Bike Lane LOS

This example uses the same segment of roadway as described in the previous sections. The roadway has five lanes, a 35-mph speed limit and a peak month ADT of 31,000. The roadway currently has six-foot bike lanes on both sides and goes through a commercial area with a substantial amount of driveways. The previous Bicycle LOS analysis (Example 14-7) indicated a LOS E-F for the no-build conditions with just standard bike lanes. Example 14-8 calculated a LOS B for a separated bikeway. An additional alternative for a buffered bike lane was also needed to be analyzed as the higher number of driveways was thought by project staff to potentially create too much interference for good operation of the separated bike lane.

The additional bike lane data of greater than one travel lane in each direction and the presence of unsignalized conflicts are added into the calculator input tab supplementing the separated bike lane data.

The additional data elements are added into the calculator and by checking the bike lane results show a LOS F at 27.11%. The 10% significance check also captures the LOS E level at 24.41%. The final LOS is overridden in the bike lane LOS columns, so it will be reported correctly on the output sheet and in the overall buffered bike lane calculations.

The calculator averages the bike and separated bike lane results and provides an estimated buffered bike lane value at LOS C-D which would also be an improvement over a standard bike lane as seen below.

APM Example 14-8 | Protected Bikeway /Buffered Bike Lane MMLOS | 06/23/16
E.N. Gineer | Segment LOS Output Summary

Roadway	Dir	From-To	Prot. Bikeway LOS	Buffered Bike Lane Estimated LOS	Bike Lane LOS
Example Ave	S	1st St - 10th St	B	C-D	E-F

14.13 Shared-use Paths

14.13.1 Methodology Summary

This methodology is a full application of the Highway Capacity Manual (HCM) 2010 and later Chapter 23 method on paved shared-use (multi-use) paths. Use this methodology with caution on unpaved paths as the research only contained data from paved paths. A shared-use path is completely separated from roadway traffic for the use of non-motorized modes. Typically these paths have at least 35 feet of separation from an adjacent roadway, but they may be closer if the barrier between the path and roadway is substantial (e.g. soundwall, retaining wall) so that the effect of vehicular traffic is limited. Paths may also be on their own separate right-of-way, such as along a creek in a greenway. Paths with lesser buffers should be considered a protected bikeway or sidewalk and analyzed with the other methods in this chapter. This methodology is intended to work in concert with the other HCM-based "streamlined" and other segment and intersection methodologies in this chapter. More information on shared-use paths is available in Sections 800 and 900 of the Highway Design Manual.

The methodology considers the impacts on pedestrians and bicyclists by other path users mainly through the accounting of passing (overtaking) and meeting events considering volumes, speeds and densities. The path segment is divided into very small 0.01-mile pieces and impact of the approaching other users (pedestrians, adult and child bicyclists, runners and inline skaters) on a bicyclist is measured for each piece and then summed across the entire path segment. Impact on pedestrian users is handled more simply by considering flow rates of all path users and the relative difference between the average pedestrian and bicycle speeds. Both bicycle and pedestrian methods measure crowding on a path segment and how much interference there will be from passing, meeting, or being forced to wait to pass. The methodology is segment-based so segments should start and end at path junctions, intersections with roadways or where path width changes substantially.

Application of the shared-use path method needs to be done via an Excel-based calculator provided on the Transportation Development – Planning Technical Tools webpage as the math and statistical work required is too much for simple hand calculations. This methodology works equally well for analysis of existing or future paths. For planning applications (proposed paths or changes to existing ones) most of the inputs can be estimated. For actual project and detailed refinement planning efforts it is recommended that most of the inputs come from actual design/field values. For analysis of intersections of shared-use paths at unsignalized (future) or signalized roadway crossings, please refer to Sections 14.14 (future) through 14.16.

14.13.2 LOS Criteria

The applicable LOS criteria and descriptions for shared-use paths come from HCM Exhibits 23-4 and 23-5 which are combined into Exhibit 14-41 and Exhibit 14-421 below. The LOS criteria are mainly based on recreational users which considers the influence of child bicyclists, runners, skaters and walkers in addition to the bicyclist mode. More user conflicts (passings and meetings) will result in lower LOS grades. A poor LOS indicates less user satisfaction and could increase the probability of a potential route shift. Since a shared path will generally offer the lowest stress route, route shifting is unlikely unless there are nearby adjacent routes with no more than 10% extra out-of-direction travel distance and the path carries high amount of high-stress tolerant (commuter) bicyclists that could travel comfortably on an on-street bike facility. Poor LOS grades generally indicate that the path is too narrow for existing or projected users.

This method and LOS criteria assume that users stay on the path surface, especially while being passed. Frequent observations of side-stepping or use of the "shoulder" area may indicate a path that is too narrow and/or is reaching capacity regardless of the LOS results obtained. Also, the LOS criteria are based on user comfort and do not give any specific indication that the facility is compliant with the Americans with Disabilities Act (ADA) or other design standards.

Exhibit 14-41 Pedestrian and Bicycle LOS Criteria

LOS	Pedestrian Weighted Event Rate per hour	Bicycle LOS Score
A	<=38	>4.0
B	>38 - 60	>3.5 – 4.0
C	>60 - 103	>3.0 – 3.5
D	>103 - 144	>2.5 – 3.0
E	>144 - 180	>2.0 – 2.5
F	>180	<=2.0

Exhibit 14-42 Pedestrian and Bicycle LOS User Description

LOS	Pedestrian LOS Description	Bicycle LOS Description
A	Optimum conditions, bicycle conflicts rare	Optimum conditions, ample ability to absorb more riders
B	Good conditions, few bicycle conflicts	Good conditions, some ability to absorb more riders
C	Difficult to walk two abreast	Meets current demand, marginal ability to absorb more riders
D	Frequent bicycle conflicts	Many conflicts, some reduction in bicycle travel speed
E	Frequent and disruptive bicycle conflicts	Very crowded, significant reduction in bicycle travel speed
F	Significant conflicts, diminished experience	

14.13.3 Data Needs and Definitions

These are the inputs and definitions as used in the methodology and the available calculator.

Shared-use Path Volume (users per hour) – This is the total of the non-motorized mode users on the specific shared-use path segment in both directions. User modes include adult and child bicyclists, pedestrians, runners and (inline) skaters. This value may be estimated through demographics/available planning documents or from an actual count (required for design purposes) for existing conditions. Future conditions could be obtained from using historical bicycle/pedestrian count data or post-processed assignments/mode splits in a metropolitan area travel demand model. Volume changes of more than 10% should be broken into a new path segment.

Highest Directional Split – Expressed as a decimal, this is the highest total directional flow percentage (i.e. 0.57) within the hour of analysis on the segment. Path use may be dominated by commuter, recreational, or multi-purpose flows. If directional counts and resulting flows are not available, then reasonable defaults can be assumed (0.55) for commuter and (0.50) for recreational and multi-purpose (mixed) uses.

Peak Hour Factor – See APM Section 5.8 for the definition and example calculations. For this to be obtained explicitly in this method, counts with 15-minute breakdowns are required which may not be a common specification for bike/pedestrian counts. A default PHF can be assumed where detailed counts are not available (0.90 to 1.00) depending on whether this path segment is subject to peaking characteristics. Highly urban areas, biking/walking friendly areas, commuter corridors, or nearby employers with high non-motorized modal users may cause noticeable peaking on certain segments.

Segment Length (miles) – This is the length of a segment between path junctions, street crossings, any location where the volume changes by more than 10%, or path surface width changes.

Path Width (feet) – This is the width of the path surface. The minimum width is eight feet, and the maximum width is 20 feet as defined in this methodology. Paths less than eight feet wide become too narrow to function as a true multi-use path as passing becomes challenging (typically a user has to step to the side) and are really closer to a pedestrian footpath. The desirable width is 12 feet but can go as low as eight feet for pinch points with low volume and as much as 20 feet for high volume paths according to the Oregon Bicycle and Pedestrian Design Guide. A change of one foot or more should be broken into a new segment. For a given volume, changing of width on narrower paths will have more impact than on wider ones.

Centerline Presence – This indicates whether a path has a marked centerline. This is a Yes/No choice in the Input tab in the calculator tool. A marked centerline can constrain the maneuverability freedom of users and results in lower LOS scores.

User Mode Default Parameters – The calculator assumes defaults for mode split, average mode speed, mode speed standard deviation, and mode passing distance from the HCM. These can be completely updated if detailed information is available or at the very least, the mode split can be proportionally changed to reflect the typical path users or area demographics.

> *It is recommended that at least one count on an existing or for a future multiuse path be full featured (15-min breakdowns, directional, and by user class) so that the calculator can be customized for the specific application. Paths with significant commuter flows should be counted in the typical AM/PM peak periods. Paths that are recreational or are mixed use should have counts that cover the midday peak and/or weekend periods. It may be necessary to obtain a week-long count with daily and hourly volumes to determine when peak periods occur if this information is not available from sites with similar characteristics.*

Example 14-10 Pedestrian & Bicycle LOS

A section of paved shared-use path links two arterials along a creek-side greenway. The unbroken path segment is 1.2 miles long and is 12 feet wide with no marked centerlines. A recent volume count showed 100 users of all types in the peak hour of the facility.

The count had hourly breakdowns only and it was determined in the peak hour that the highest directional flow was 60%. A PHF was estimated at 0.95 as there was not much influence from uses that would cause higher spikes in the user volume. The default modal splits and other parameters were used.

The input data were entered into the calculator tool in the yellow-shaded boxes as shown below. Mode splits, speeds, and passing distances were left as defaults in the orange-shaded boxes.

Total shared-use path users per hour =			100			
Highest directional split (decimal)=			0.60			
Peak Hour Factor (PHF) =			0.95			
Segment Length (L) (mi) =			1.20			
Path Width (min 8 - max 20 ft) =			12.0			
Does path have a marked centerline? (Yes/No)			No			
	Mode Split	**Ave. Mode speed, ui**	**Mode Std. Dev.**	**Mode Passing**	**Flow rate , qi**	**Density**
	(decimal)	**(mph)**	**(mph)**	**Distance (ft)**	**users/h**	**users/mi**
Bike	0.55	12.8	3.4	100	35	2.71
Ped	0.20	3.4	0.6	60	13	3.72
Runner	0.10	6.5	1.2	70	6	0.97
Skater	0.10	10.1	2.7	100	6	0.63
Child bike	0.05	7.9	1.9	70	3	0.40
Yellow-shaded cells are user supplied data (Directional split and PHF can be defaulted)						
Orange-shaded cells are user-changeable defaults if better information is available						
Gray-shaded cells are calculations used as inputs to other tabs						

The calculator macro tool was run and the results obtained are shown below. Both the bicycle and pedestrian modes have a LOS B which is indicative of favorable conditions on this analysis segment.

Bike LOS Calculation				
Total Meetings per Min=			2.79	
Total Active Passings per Min=			0.81	
Total Weighted Events per Min			10.87	
Delayed Passings per Min =			0.08	
Bike LOS Score =		4.00	LOS =	B
Final LOS adjustment for low volume=				B
Pedestrian LOS Calculation				
Pedestrian Events/hr =			52	
Pedestrian LOS =			B	

14.14 Unsignalized Intersections (TBD- In Progress)

14.15 Pedestrian Crossing Treatments

Before deciding on specific treatments, project existing conditions and alternatives should be evaluated for adequate spacing of pedestrian crossings. Especially in urban areas, roadway geometry, volumes, and available pedestrian facilities may hinder access to the nearest improved crossing. Pedestrians generally will not walk more than about 10% to go out of their way to and back to use the nearest signalized crossing, but instead may not do a particular trip or risk crossing the roadway mid-block. Land use density and nearby pedestrian generators should be considered when deciding on a specific spacing. When possible, these target ranges should be met.

14.15.1 NCHRP 562 Application

One of the main products of the National Cooperative Highway Research Program (NCHRP) Report 562 was creation of a new spreadsheet tool that can be used as a guide to select or screen potential pedestrian crossing treatments for plans and projects. These treatments can range from signs and markings to full mid-block traffic signals. This tool uses relatively little data (i.e. speeds, volumes, widths) which is generally available to the typical TSP planning level all the way to the project development and TIA-level efforts. Selected treatments can also be analyzed with PLTS and MMLOS methods described earlier in this chapter.

Any crossing treatment should be reviewed by Region Traffic and/or the Traffic, Section for its appropriateness for the given location and plan/project type. These treatments may have additional requirements listed in the Highway Design Manual, the Bicycle Pedestrian Design Guide, the Manual of Uniform Traffic Control Devices, or the Traffic Manual.

The need for a crossing treatment should not be based solely on the results of the NCHRP 562 methodology. Other factors such as the distance to the nearest improved crossing, number of vulnerable users, and crash history should also be considered.

14.15.2 Treatment Categories

There are five main categories used in the tool as explained below. Most of these have additional information shown in the solutions section of Chapter 10. For planning level analysis, improvements should be identified using general categories rather than as specific treatments, in order to allow for flexibility in design.

- **No Treatment –** Occurs when the volumes of pedestrians are low (<20 pedestrians per hour at 35 mph or less or <14 pedestrians per hour at greater than 35 mph). Many locations in Oregon will fall into this category. When the tool indicates "no treatment" this means that no traffic control treatments such as signs, markings, or active pedestrian devices such as beacons are recommended. Traffic calming or pedestrian visibility-type measures can still be considered such as illumination, curb extensions, and/or median islands as appropriate. At certain locations such as transit stops and school crossings, it may be appropriate to provide a traffic control treatment even where there are few users in any given hour due to known continuous use throughout the day and presence of particularly vulnerable users. The analyst will need to review these results with the Region Traffic group to determine if the category should remain as "No Treatment" or be upgraded to the "Crosswalk" category.
- **Crosswalk –** Includes typical signing and crosswalk markings. This mainly applies to cases with sufficient pedestrian volume but low traffic volume.
- **Enhanced/Active –** Includes constant presence enhanced and active treatments. Examples of enhanced treatments are in-street signs, raised crosswalks, and curb extensions all of which are known to result in significantly increased stopping compliance. Active treatment examples are rapid-flashing and traditional flashing beacons that are pedestrian activated. This mainly apples to areas with sufficient pedestrians but higher traffic volumes or areas with high pedestrian volumes (>100 pedestrians per hour) and lower traffic volumes.
- **Red (Indication) –** Includes devices that show a "red" indication to vehicles such as pedestrian hybrid beacons (PHB) and mid-block traffic signals. These mainly apply to areas with roadway volumes (both directions) exceeding 1400 vph and peak pedestrian hour volumes less than 100 per hour.
- **Signal –** Includes conventional traffic signals at an intersection. This is a modification of the MUTCD pedestrian warrant. While bi-directional traffic volumes can be low as 400 vph, peak pedestrian volumes need to be at least 100 or more per hour. This category could also be applicable to the red indication devices if in a mid-block location. At certain locations where traffic signals would not be considered an appropriate measure (i.e. rural or an urban access-controlled expressway), this category can be used to indicate a need for a grade-separated pedestrian under or overcrossing.

14.15.3 Input Data

- Posted speed limit, 85th percentile speed, or statutory speed (i.e. 25 mph residential or 20 mph downtown)
- Population category of surrounding area (less or greater than 10,000)

The NCHRP 562 method does not include an allowance for just "marked crosswalks" when the population in less than 10,000. Treatment options are limited to no treatment, active/enhanced, red indication, and signal, so the method may overestimate the need for active/enhanced options. Adequate illumination, signs and continental crosswalk markings may be the expected outcome especially when volumes, speeds, or crossing distances are low/small.

- Bi-directional roadway vehicles per hour during the pedestrian peak hour. This can be either an existing condition or future projected volume. If a median/pedestrian refuge island of at least six feet wide is present, then each direction needs to be analyzed separately. When an island is present two spreadsheets will need to be worked up, one for each direction. Vehicle volume in Line 3a will always be the total of both directions, while the volume in Line 4e, will be each approach direction when an island is present or the total when an island is not present. The pedestrian peak hour is typically not the same hour as the vehicular peak hour. Typically it can occur around the lunchtime period (i.e. 12-1 PM) which will likely require longer duration traffic counts at higher pedestrian volume locations in a plan or project.

It can be helpful to analyze both the pedestrian peak hour and the vehicle peak hour especially when projecting future conditions. Treatments could be triggered with lower pedestrian volumes and higher vehicle volumes and vice versa.

- Pedestrian volume in the pedestrian peak hour – Sum existing condition or future projected pedestrian crossings from both directions at a mid-block location. If the analysis site is an intersection, sum both directions on both approaches (i.e. both east-west crosswalks north and south of the intersecting roadway). If separate pedestrian counts are done versus standard intersection classification counts, these should be at least 16-hours to capture multiple potential peaks (e.g. morning, noon, school, afternoon) and taken in good weather (spring/summer/fall) when pedestrians are mostly likely to be at the location. The choice of a weekday or weekend counts should be based on the surrounding land use and pedestrian destinations surrounding the crossing location. Future population growth rates should be considered to estimate future background pedestrian growth. It may be necessary to consider a larger area (approximately a ¼ mile radius) surrounding the crossing to help quantify or qualitatively explain the potential for induced growth above and beyond the background growth. Pedestrian warrant threshold reduction percentage –This parameter is used as part of the analysis for a full traffic signal. If the location experiences high pedestrian volumes (i.e. more than 100 peds/hr), then some consideration of the number of vulnerable pedestrians should be made. Otherwise, this parameter has no effect on the results and the default "no" can be retained.

- Pedestrian crossing distance from curb-to-curb (ft). If a median refuge is present, use the curb-to-refuge distance for each approach.
- Pedestrian walking speed – use the default walking speed of 3.5 ft/s unless a significant number of older or younger pedestrians or those with mobility challenges are expected, such as where schools or retirement or similar facilities are nearby, then use 3.0 ft/s. Pedestrian lost time (startup and clearance time) – use the default 3 seconds.
- Pedestrian delay (s) – This value is normally automatically calculated but a field-measured delay value can be entered to override the calculation.
- Expected motorist compliance – Generally, use the "low" value when speeds are 30 mph or higher or when pedestrians are crossing four or more lanes of traffic, due to low compliance rates in Oregon under those conditions.

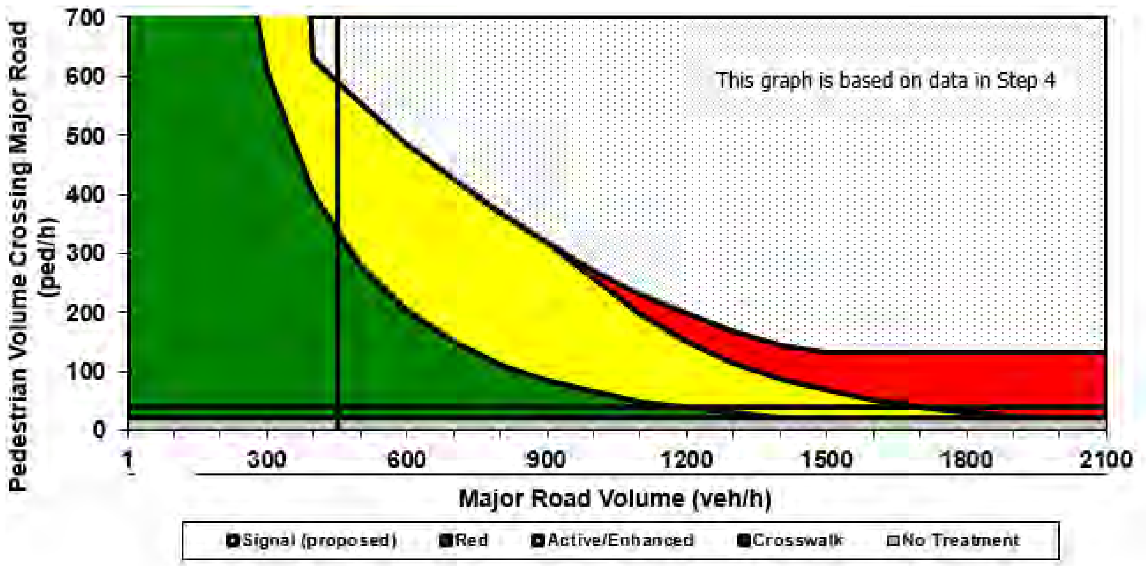
Example 14-11 Pedestrian Crossing Treatment Selection

A state highway in an urban area splits the commercial zone from the residential parts of the city. Residents report that it can be difficult to cross the roadway and it is too far out of direction to reach the nearest traffic signal. The roadway speed is 30 mph. A count was done, and the pedestrian peak hour was determined to be from 12-1 PM and about 40 pedestrians per hour cross at this location. The bi-directional roadway volume was determined to be 450 vph during this time at this location. The assumed pedestrian walking speed is 3.5 ft/s, with a 30 ft curb-to- curb distance. Pedestrian yielding rates remain low at this location regardless of the current pedestrians yielding law which may be due to relatively low visibility of the crossing. The data are entered into the tool which a recommends a marked crosswalk.

Improvements for this location are at least crosswalk signing, continental markings (if not already marked), and illumination.

Analyst and Site Information			
Analyst	E. N. Gineer	Major Street	Main Street
Analysis Date	March 29, 2018	Minor Street or Location	Elm Street
Data Collection Date	March 14, 2018	Peak Hour	12-1 PM

Step 1: Select worksheet:		
Posted or statutory speed limit (or 85th percentile speed) on the major street (mph)	*1a*	30
Is the population of the surrounding area <10,000? (enter ***YES*** or ***NO***)	*1b*	No
Step 2: Does the crossing meet minimum pedestrian volumes to be considered for a traffic control devi		
Peak-hour pedestrian volume (ped/h), V_p	*2a*	40
Result: Go to step 3.		
Step 3: Does the crossing meet the pedestrian warrant for a traffic signal?		
Major road volume, total of both approaches during peak hour (veh/h), V_{maj-s}	*3a*	450
[Calculated automatically] Preliminary (before min. threshold) peak hour pedestrian volume to meet warrant	*3b*	591
[Calculated automatically] Minimum required peak hour pedestrian volume to meet traffic signal warrant	*3c*	591
Is 15th percentile crossing speed of pedestrians less than 3.5 ft/s (1.1 m/s)? (enter ***YES*** or ***NO***)	*3d*	no
If 15th percentile crossing speed of pedestrians is less than 3.5 ft/s (1.1 m/s), then reduce *3c* by up to 50%. — % rate of reduction for *3c* (up to 50%	*3e*	
Reduced value or *3c*	*3f*	591
Result: The signal warrant is not met. Go to step 4.		
Step 4: Estimate pedestrian delay.		
Pedestrian crossing distance, curb to curb (ft), L	*4a*	30
Pedestrian walking speed (ft/s), S_p (suggested speed = 3.5 ft/s)	*4b*	3.5
Pedestrian start-up time and end clearance time (s), t_s (suggested start-up time = 3 sec)	*4c*	3
[Calculated automatically] Critical gap required for crossing pedestrian (s), t_c	*4d*	12
Major road volume, total both approaches OR approach being crossed if raised median island is present, during peak hour (veh/h), V_{maj-d}	*4e*	450
Major road flow rate (veh/s), v	*4f*	0.13
Average pedestrian delay (s/person), d_p	*4g*	15
Total pedestrian delay (h), D_p The value in 4h is the calculated estimated delay for all pedestrians crossing th	*4h*	0.2
major roadway without a crossing treatment (assumes 0% compliance). If the actual total pedestrian delay has been measured at the site, that value can be entered in 4i to replace the calculated value in 4h.	*4i*	
Step 5: Select treatment based up on total pedestrian delay and expected motorist compliance.		
Expected motorist compliance at pedestrian crossings in region: enter ***HIGH for High Compliance*** or ***LOW for Low Compliance***	*5a*	low
Treatment Category:	**CROSSWALK**	



This worksheet provides general recommendations on pedestrian crossing treatments to consider at unsignalized ntersections; in all cases, engineering judgment should be used in selecting a specific treatment for installation. This worksheet does not apply to school crossings. In addition to the results provided by this worksheet, users should consider whether a pedestrian treatment could present an increased safety risk to pedestrians, such as where there is poor sight distance, complex geometrics, or nearby traffic signals.

14.15.4 Pedestrian Crossing Safety

The need for a pedestrian crossing treatment is rooted on safety so that the safest crossing can be provided. The NCHRP 562 methods, explained previously, are generally based on volumes of pedestrians or vehicles and the crossing geometry. There can be many locations that fall into the “No Treatment” category for which a pedestrian crossing treatment may be considered because they have a noted crash history, have potential crash risks, or are near vulnerable populations (e.g. schools). Locations should also be screened for safety which can either confirm the NCHRP 562 treatments or suggest other possibilities. The publication, *“FHWA Guide for Improving Pedestrian Safety at Uncontrolled Crossing Locations”* contains a countermeasure methodology reproduced in Exhibit 14-43 and Exhibit 14-44. Exhibit 14-43 is a table of pedestrian crash countermeasures by geometry, speed and AADT. Not all the treatments indicated in a particular cell in Exhibit 14-43 should be considered at a single location.

Exhibit 14-43 Pedestrian Crash Countermeasures

Roadway Configuration	Posted Speed Limit and AADT								
	Vehicle AADT <9,000			Vehicle AADT 9,000–15,000			Vehicle AADT >15,000		
	≤30 mph	35 mph	≥40 mph	≤30 mph	35 mph	≥40 mph	≤30 mph	35 mph	≥40 mph
2 lanes (1 lane in each direction)	❶ 2 4 5 6	❶ 5 6 7 9	① 5 6 ❼ ❾	❶ 4 5 6	❶ 5 6 7 9	① 5 6 ❼ ❾	❶ 4 5 6 7 9	① 5 6 7 9	① 5 6 ❾
3 lanes with raised median (1 lane in each direction)	❶ 2 3 4 5	❶ ❸ 5 7 9	① ❸ 5 ❼ ❾	① 3 4 5 7 9	① ❸ 5 ❼ ❾	① ❸ 5 ❼ ❾	① ❸ 4 5 7 9	① ❸ 5 ❼ ❾	① ❸ 5 ❾
3 lanes w/o raised median (1 lane in each direction with a two-way left-turn lane)	❶ 2 3 4 5 6 7 9	❶ ❸ 5 6 7 9	① ❸ 5 6 ❾	① 3 4 5 6 7 9	① ❸ 5 6 ❼ ❾	① ❸ 5 6 ❾	① ❸ 4 5 6 7 9	① ❸ 5 6 ❾	① ❸ 5 6 ❾
4+ lanes with raised median (2 or more lanes in each direction)	❶ ❸ 5 7 8 9	❶ ❸ 5 7 8 9	① ❸ 5 8 ❾	① ❸ 5 7 8 9	① ❸ 5 ❼ 8 ❾	① ❸ 5 8 ❾	① ❸ 5 ❼ 8 ❾	① ❸ 5 8 ❾	① ❸ 5 8 ❾
4+ lanes w/o raised median (2 or more lanes in each direction)	❶ ❸ 5 6 7 8 9	① ❸ 5 ❻ 7 8 9	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ 7 8 9	① ❸ 5 ❻ ❼ 8 ❾	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ ❼ 8 ❾	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ 8 ❾

Given the set of conditions in a cell,

- # Signifies that the countermeasure is a candidate treatment at a marked uncontrolled crossing location.
- ● Signifies that the countermeasure should always be considered, but not mandated or required, based upon engineering judgment at a marked uncontrolled crossing location.
- ○ Signifies that crosswalk visibility enhancements should always occur in conjunction with other identified countermeasures.*

The absence of a number signifies that the countermeasure is generally not an appropriate treatment, but exceptions may be considered following engineering judgment.

1 High-visibility crosswalk markings, parking restrictions on crosswalk approach, adequate nighttime lighting levels, and crossing warning signs
2 Raised crosswalk
3 Advance Yield Here To (Stop Here For) Pedestrians sign and yield (stop) line
4 In-Street Pedestrian Crossing sign
5 Curb extension
6 Pedestrian refuge island
7 Rectangular Rapid-Flashing Beacon (RRFB)**
8 Road Diet
9 Pedestrian Hybrid Beacon (PHB)**

*Refer to Chapter 4, 'Using Table 1 and Table 2 to Select Countermeasures,' for more information about using multiple countermeasures.

**It should be noted that the PHB and RRFB are not both installed at the same crossing location.

This table was developed using information from: Zegeer, C.V., J.R. Stewart, H.H. Huang, P.A. Lagerwey, J. Feaganes, and B.J. Campbell. (2005). Safety effects of marked versus unmarked crosswalks at uncontrolled locations: Final report and recommended guidelines. FHWA, No. FHWA-HRT-04-100, Washington, D.C.; FHWA. Manual on Uniform Traffic Control Devices, 2009 Edition. (revised 2012). Chapter 4F, Pedestrian Hybrid Beacons. FHWA, Washington, D.C.; FHWA. Crash Modification Factors (CMF) Clearinghouse. http://www.cmfclearinghouse.org/; FHWA. Pedestrian Safety Guide and Countermeasure Selection System (PEDSAFE). http://www.pedbikesafe.org/PEDSAFE/; Zegeer, C., R. Srinivasan, B. Lan, D. Carter, S. Smith, C. Sundstrom, N.J. Thirsk, J. Zegeer, C. Lyon, E. Ferguson, and R. Van Houten. (2017). NCHRP Report 841: Development of Crash Modification Factors for Uncontrolled Pedestrian Crossing Treatments. Transportation Research Board, Washington, D.C.; Thomas, Thirsk, and Zegeer. (2016). NCHRP Synthesis 498: Application of Pedestrian Crossing Treatments for Streets and Highways. Transportation Research Board, Washington, D.C.; and personal interviews with selected pedestrian safety practitioners.

If a location has an identified crash history or has had recent crash analyses done for a plan or project this may indicate a safety issue. However, most pedestrian crashes are random, and a crash pattern may not be evident. There are a number of pedestrian crash risk factors that are associated with collisions and potential severe injuries:

- Excessive vehicle speed
- Inadequate visibility
- Failing to yield to pedestrians in crosswalks
- Insufficient separation from traffic

Exhibit 14-44 shows these safety risks and the associated countermeasures that address them. These are the same countermeasures shown in Exhibit 14-43 above. The crash reduction factors (CRF) from the original and supplemental ARTS CRF lists should be used as all but the parking restriction countermeasure are available. Appendix B of the FHWA guide does have a 30% CRF applied to a pedestrian crash basis for the parking countermeasure.

Exhibit 14-44 Safety Issues and Related Countermeasures

Pedestrian Crash Countermeasure for Uncontrolled Crossings	Safety Issue Addressed: Conflicts at crossing locations	Excessive vehicle speed	Inadequate conspicuity/ visibility	Drivers not yielding to pedestrians in crosswalks	Insufficient separation from traffic
Crosswalk visibility enhancement	🚶	🚶	🚶	🚶	🚶
High-visibility crosswalk markings*	🚶		🚶	🚶	
Parking restriction on crosswalk approach*	🚶		🚶	🚶	
Improved nighttime lighting*	🚶		🚶		
Advance Yield Here To (Stop Here For) Pedestrians sign and yield (stop) line*	🚶		🚶	🚶	🚶
In-Street Pedestrian Crossing sign*	🚶	🚶	🚶	🚶	
Curb extension*	🚶	🚶	🚶		🚶
Raised crosswalk	🚶	🚶	🚶	🚶	
Pedestrian refuge island	🚶	🚶	🚶		🚶
Pedestrian Hybrid Beacon	🚶	🚶	🚶	🚶	
Road Diet	🚶	🚶	🚶		🚶
Rectangular Rapid-Flashing Beacon	🚶		🚶	🚶	🚶

*These countermeasures make up the STEP countermeasure "crosswalk visibility enhancements." Multiple countermeasures may be implemented at a location as part of crosswalk visibility enhancements.

Example 14-12 Pedestrian Crossing Safety Countermeasures

This example uses the same basic data as in Example 14-11. This roadway has two lanes, 30 mph speed and has 450 vph (4500 vpd). This data is used in Exhibit 14-37 to determine the potential safety countermeasures. High visibility crosswalk markings, lighting, parking restrictions and warning signs are indicated as the top countermeasure to be considered. This is consistent with the NCHRP 562 results in the previous example. Exhibit 14-43 also indicates that in-street pedestrian crossing signs, curb extensions, and a refuge island also can be considered. Raised crosswalks were also in the exhibit, but these are not installed on state highway mainlines and would be dropped from consideration.

Roadway Configuration	Posted Speed Limit and AADT								
	Vehicle AADT <9,000			Vehicle AADT 9,000–15,000			Vehicle AADT >15,000		
	≤30 mph	35 mph	≥40 mph	≤30 mph	35 mph	≥40 mph	≤30 mph	35 mph	≥40 mph
2 lanes (1 lane in each direction)	❶ 2 4 5 6	❶ 5 6 7 9	① 5 6 ❼ ❾	❶ 4 5 6	❶ 5 6 7 9	① 5 6 ❼ ❾	❶ 4 5 6 7 9	① 5 6 7 9	① 5 6 ❾
3 lanes with raised median (1 lane in each direction)	❶ 2 3 4 5	❶ ❸ 5 7 9	① ❸ 5 ❼ ❾	① 3 4 5 7 9	① ❸ 5 ❼ ❾	① ❸ 5 ❼ ❾	① ❸ 4 5 7 9	① ❸ 5 ❼ ❾	① ❸ 5 ❾
3 lanes w/o raised median (1 lane in each direction with a two-way left-turn lane)	❶ 2 3 4 5 6 7 9	❶ ❸ 5 6 7 9	① ❸ 5 6 ❾	① 3 4 5 6 7 9	① ❸ 5 6 ❼ ❾	① ❸ 5 6 ❾	① ❸ 4 5 6 7 9	① ❸ 5 6 ❾	① ❸ 5 6 ❾
4+ lanes with raised median (2 or more lanes in each direction)	❶ ❸ 5 7 8 9	❶ ❸ 5 7 8 9	① ❸ 5 8 ❾	① ❸ 5 7 8 9	① ❸ 5 ❼ 8 ❾	① ❸ 5 8 ❾	① ❸ 5 ❼ 8 ❾	① ❸ 5 8 ❾	① ❸ 5 8 ❾
4+ lanes w/o raised median (2 or more lanes in each direction)	❶ ❸ 5 6 7 8 9	① ❸ 5 ❻ 7 8 9	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ 7 8 9	① ❸ 5 ❻ ❼ 8 ❾	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ ❼ 8 ❾	① ❸ 5 ❻ 8 ❾	① ❸ 5 ❻ 8 ❾

Given the set of conditions in a cell,

- \# Signifies that the countermeasure is a candidate treatment at a marked uncontrolled crossing location.
- ● Signifies that the countermeasure should always be considered, but not mandated or required, based upon engineering judgment at a marked uncontrolled crossing location.
- ○ Signifies that crosswalk visibility enhancements should always occur in conjunction with other identified countermeasures.*

The absence of a number signifies that the countermeasure is generally not an appropriate treatment, but exceptions may be considered following engineering judgment.

1 High-visibility crosswalk markings, parking restrictions on crosswalk approach, adequate nighttime lighting levels, and crossing warning signs
2 Raised crosswalk
3 Advance Yield Here To (Stop Here For) Pedestrians sign and yield (stop) line
4 In-Street Pedestrian Crossing sign
5 Curb extension
6 Pedestrian refuge island
7 Rectangular Rapid-Flashing Beacon (RRFB)**
8 Road Diet
9 Pedestrian Hybrid Beacon (PHB)**

*Refer to Chapter 4, 'Using Table 1 and Table 2 to Select Countermeasures,' for more information about using multiple countermeasures.

**It should be noted that the PHB and RRFB are not both installed at the same crossing location.

This table was developed using information from: Zegeer, C.V., J.R. Stewart, H.H. Huang, P.A. Lagerwey, J. Feaganes, and B.J. Campbell. (2005). Safety effects of marked versus unmarked crosswalks at uncontrolled locations: Final report and recommended guidelines. FHWA, No. FHWA-HRT-04-100, Washington, D.C.; FHWA. Manual on Uniform Traffic Control Devices, 2009 Edition. (revised 2012). Chapter 4F, Pedestrian Hybrid Beacons. FHWA, Washington, D.C.; FHWA. Crash Modification Factors (CMF) Clearinghouse. http://www.cmfclearinghouse.org/; FHWA. Pedestrian Safety Guide and Countermeasure Selection System (PEDSAFE). http://www.pedbikesafe.org/PEDSAFE/; Zegeer, C., R. Srinivasan, B. Lan, D. Carter, S. Smith, C. Sundstrom, N.J. Thirsk, J. Zegeer, C. Lyon, E. Ferguson, and R. Van Houten. (2017). NCHRP Report 841: Development of Crash Modification Factors for Uncontrolled Pedestrian Crossing Treatments. Transportation Research Board, Washington, D.C.; Thomas, Thirsk, and Zegeer. (2016). NCHRP Synthesis 498: Application of Pedestrian Crossing Treatments for Streets and Highways. Transportation Research Board, Washington, D.C.; and personal interviews with selected pedestrian safety practitioners.

14.16 Signalized Intersections Pedestrian and Bicycle Level of Service

This methodology is adapted from the publication, "*Pedestrian and Bicycle Level of Service, Methodology for Crossing at Signalized Intersections*", from the Charlotte, NC Department of Transportation. This intersection methodology is intended to complement the level of service (LOS) segment analysis methods presented in this chapter and should be used in tandem to completely analyze a pedestrian or a bicycle facility. This methodology is intended for project level and detailed planning studies where specific data are plentiful and available. System planning efforts should use the Level of Traffic Stress methodologies in Sections 14.4 and 14.5. The methodology is based on intersection elements that affect pedestrian and bicyclist safety and comfort using an expert judgment/index basis rather than research based as might be found in the Highway Capacity Manual. These intersection elements mainly include the impacts from reducing traffic conflicts, minimizing crossing distances, slowing down traffic speeds and raising user awareness. This method can be used without modification for signalized mid-block crossings (traffic signals not beacons). Traffic volumes are not explicitly used in this methodology in most areas but are implicitly included by using surrogates such as total crossing distance, number of lanes, speeds, and signal phasing.

This methodology is intended to be used with other intersection analysis data, tools and methods such as Highway Safety Manual crash analyses, intersection capacity analyses, pedestrian/bicycle volumes, etc. to have a complete picture of the overall operations. In addition, several design and operational-related features are not part of the methodology such as sight distance, illumination, pavement condition, signing, accessibility, and detection but should be considered as part of any project intersection or mid-block crossing design. Please coordinate with traffic engineering and roadway design staff in the appropriate ODOT Region office, the Traffic Section, or local jurisdiction regarding consideration of these features.

This methodology can be used to assess and improve pedestrian and bicycle user comfort for detailed planning and project development efforts by modifying design and operational features. It can also be used to select intersection features that meet the chosen pedestrian and bicycle LOS. The results can also be compared to traffic operations and other analyses given the objectives and priorities of the plan/project and the local jurisdiction. A companion spreadsheet LOS calculator was also developed and is available on ODOT's Planning and Technical Guidance's Technical Tools webpage.

It is assumed that any design and operational elements evaluated in this methodology follow applicable ODOT publications (or accepted local/national standards and guidelines for non-state intersections) such as the Highway Design Manual, the Bicycle & Pedestrian Design Guide, Traffic Manual, etc. This means that, for example, appropriate widths, distances, or adequate signal timing should be present as this methodology is primarily concerned with the presence/absence of intersection features.

This point-based methodology requires a number of design and operational elements that may be obtained from aerial/ground-level photography, field investigation, ODOT Region/Traffic Section/local jurisdiction staff for existing conditions. Project alternative designs/studies along with coordinating with appropriate staff should give the necessary information for future no-build or build conditions. These data elements are:

Pedestrian LOS

- Intersection lane configurations
- Median refuge presence and width
- Corner refuge island presence, number, and type (painted/curbed)
- Channelized right turn lane traffic control and design (high or low speed)
- Curb ramp design/condition
- Effective corner radius
- Total signal cycle length and number of phases
- Left and right turn signal phasing type
- Right/left-turn-on-red presence
- Pedestrian signal types and phasing
- Crosswalk markings

Bicycle LOS

- Intersection lane configurations
- Curb lane width
- Posted speed limit
- Approach and departure leg bicycle facility type
- Right turn lane/bike lane approach configuration
- Right turn lane volume
- Right turn lane length
- Buffer width at intersection
- Stop bar location
- Shared lane markings, conflict area paint, and turn box presence
- Left turn signal phasing
- Right-turn-on-red presence

Pedestrian and Bicycle LOS Criteria

The LOS scores are based on adding or subtracting points for the applicable physical and operational intersection elements on how well they perform based on the safety and comfort objectives for each approach. Higher point scores are equated with a better LOS. Points are summed from each element area: crossing distance, signal phasing and timing, pedestrian delay, corner radius, and crosswalk treatment. The subtotals from each area are summed into grand total for each intersection leg and compared with Exhibit 14-45.

Exhibit 14-45 Pedestrian and Bicycle LOS Criteria

LOS	Pedestrian and Bicycle Total Points	Interpretation
A	≥93	Conditions should be generally acceptable for users.
B	74 - 92	
C	55 - 73	Some issues exist that may make users uncomfortable.
D	37 - 54	
E	19 - 36	Significant issues exist that will make a majority feel uncomfortable. Likely that this intersection will deter users from using it completely or from certain paths.
F	≤18	

The individual intersection legs can be averaged to determine the LOS for the overall intersection. However, in some cases it may be best to report the leg LOS instead as the score for one exceptionally good or poor leg may be obscured in an intersection's average score. Since this methodology results in a reflection of the user's perception of safety and comfort, the LOS results need to be evaluated with other planning considerations (land use context, available funding etc.). The pedestrian and bicycle LOS calculated in this section will need to be weighed and prioritized alongside operational results for motor vehicles (v/c, LOS, etc.) based on the overall study area context, purpose, needs, goals, and objectives.

In some cases, the point assignments may need to be modified to better fit the specific context or allow for the consideration of more detailed elements or items not included in the methodology. Any deviations need to be part of a Methodology and Assumptions memorandum before any analysis work occurs or documented in other correspondence and agreed upon by Region/HQ Traffic and the Transportation Planning Analysis Unit.

14.16.1 Pedestrian Signalized Intersection LOS

The major issues for a pedestrian crossing at a signalized intersection are the total crossing distance (exposure to oncoming traffic) and potential conflicts with turning vehicles. Overall vehicle volume and speed also have some influence, but these can be mitigated by the presence of the traffic signal, physical intersection characteristics such as turning radii, and more restrictive signal phasing such as no right-turn-on-red. The factors included in the methodology focus on the physical crossing distances, intersection layout, and motor vehicle turning conflicts with the active pedestrian phase interval. Volumes and speeds are handled implicitly (i.e. assigned points drop more rapidly for a greater number of lanes crossed as wider roadways usually have higher volumes and speeds).

Crossing Distance

The roadway crossing distance is the primary component of the pedestrian methodology and receives the highest weight in determining the LOS. The more lanes a pedestrian has to cross, the lower the comfort and safety level are as they are exposed to cross-traffic for a greater distance. For example, two-and three lane signalized crossings are easy to cross and limit exposure as they are more “pedestrian-scaled” so this criterion does not change much at this level and decreases as the number of lanes increase.

Presence of (raised curb) median refuges break up the crossing distance into shorter lengths which can provide a positive LOS improvement especially for four-lane and wider roadways. For two-and three lane roadways, since exposure is limited, the impact of a median refuge is much less. For this criterion, note the presence of the median refuge but ignore pedestrian timing whether a crossing can be made all at once or requires multiple cycles. Extra crossing time is covered in the Pedestrian Delay criteria section.

The crossing distance is based on the number of lanes crossed to reach the other side of the roadway. Include turn, through, and channelized turn lanes in the total number of lanes in Exhibit 14-46. For example, if an intersection leg had a left turn lane, two through lanes, a channelized right turn lane with an island, and two opposing through lanes, this would be coded as a total of six lanes.

If bike lanes, shoulders, or parking are present, then the width of these should be considered as lane equivalents as these increase the overall exposure and motor vehicles could still cross the pedestrian path unexpectedly in these locations. For the purposes of the methodology, add a lane for each pair of bike lanes, shoulders, or parking that exists on a leg. Add two lanes where the outside lane widths are 20 feet or greater (i.e. which might occur where diagonal parking is present). Curb extensions shorten the overall crossing distance by removing the pavement width for parking so these will have a positive LOS impact if present (or added in an alternative.)

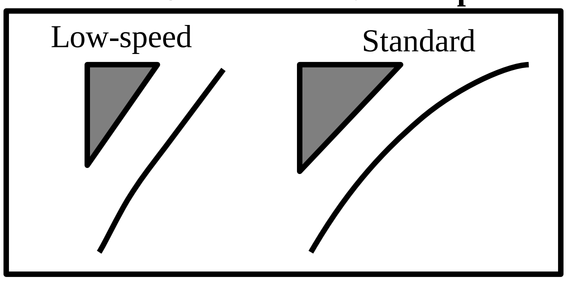
Exhibit 14-46 Crossing Distance

Total Lanes/Lane Equivalents Crossed[1]	**Points**		
	Median Refuge None or <4 ft	**Median Refuge 4 - <6 ft**	**Median Refuge ≥6 ft**
2	78	79	80
3	76	77	78
4	65	65	68
5	50	52	55
6	37	40	44
7	24	28	33
8	8	12	20
9	-5	0	10
10	-15	-10	0

[1] Outside lane widths at 20 feet or greater should be considered as two lanes. Parking, shoulders, or bike lanes on both sides of a leg should be considered as an extra lane equivalent.

The addition of corner refuge islands breaks up the overall crossing distance. Corner islands can either be of a standard design which requires more head movement by the driver to check for oncoming pedestrians or a low-speed design which brings the right turns in closer to a right angle as seen in Exhibit 14-47. Low speed islands can be identified as having an approximate 60-degree angle between the refuge lane and the intersecting street. The standard island with 45 or less degree corners is typically not considered a low-speed design unless the turning radius of the refuge lane is very tight (less than 30 feet). The presence of a raised crosswalk will effectively slow vehicles traveling through the channelized right turn lane regardless of island design style which will afford a better likelihood for yielding to pedestrians.

Exhibit 14-47 Standard vs. Low-Speed Island Shape



A scoring adjustment is made for each corner refuge island crossed as seen in Exhibit 14-48. For example, if the overall crossing distance traverses a left turn lane, two through lanes in each direction, and a channelized right turn lane separated by an island then the total point would be based on six total lanes crossed (37 points) plus the refuge island (6 points) for a total of 43 points. Adjustments are also made for the traffic control for the channelized right turn lane depending on if there is signal control, yield control or no control (free-flow). Points decrease as the chance of a vehicle not stopping and/or not seeing the pedestrian increase. The point reduction is less if the island is a low-speed design type with a steeper entrance angle that forces vehicles to slow substantially as these offer better driver visibility of oncoming traffic and pedestrians.

Exhibit 14-48 Channelized Right Turn Refuge Adjustments

Corner Refuge Island Adjustment	**Points**
Each corner refuge island	6
Channelized Right Turn Lane Traffic Control Adjustment	
Signalized control	5
Yield control with low-speed island design and/or raised crosswalk	2
Yield control	-3
Uncontrolled (free-flow) with low-speed island design and/or raised crosswalk	0
Uncontrolled (free-flow)	-20

Curb Ramps

When a crossing lacks or has substandard curb ramps, the crossing can prevent use or make it difficult for disabled users potentially even forcing them out into the street and using nearby driveway ramps if available. A ramp built to ADA standards will have a flatter grade, a level landing, and a contrasting detectable surface for visually impaired pedestrians. Older ramps with short and/or steep grades (these almost never have any detectable surfaces) are considered equivalent to no ramp at all. The methodology restricts the total possible points by creating a maximum point threshold for an intersection leg based on the overall ramp quality as seen in Exhibit 14-49. For example, if an intersection leg did not have any curb ramps, then the maximum point value would be 37 (LOS E) regardless of other positive features and would be the controlling factor in this LOS. A good LOS is not possible with poor ramp conditions. The overall quality of the crossing is based on the condition of the curb ramps on either end of the crosswalk. If a curb ramp on one end of the crossing is good and the curb ramp on the other end is poor, the overall quality of the crossing is rated as poor as the crossing is not accessible. Determine the quality of each crossing based on the worst condition of the curb ramp pairs.

Exhibit 14-49 Maximum Point Thresholds and Curb Ramp Quality

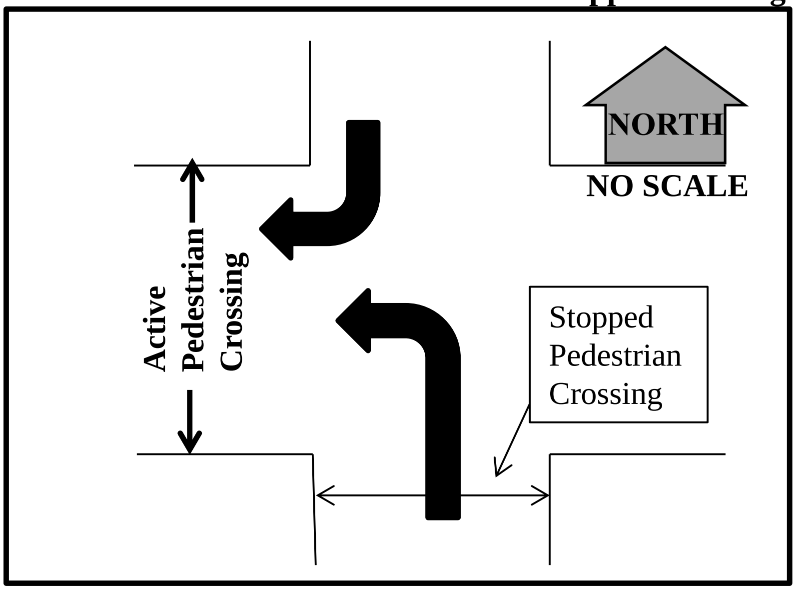
Rating	Description	Maximum Point Threshold
Good	ADA-standard; truncated dome insert, level unobstructed landing area, flat grades	None
Acceptable	Detectable (cross-hatched) ramp surface, level landing area, shallow grades	73
Poor	No detectable surface, obstructed/no level landing area, steep grades, or missing ramp	37

Signal Phasing and Timing

The second-most important element in the methodology is the effect of signal phasing and timing on potential pedestrian-vehicle conflicts. Signal phasing can remove, limit, or create these conflicts. Protected phases are best for minimizing left and right turn conflicts as the pedestrian phase is prohibited from coming up during the green arrow indication for turning vehicles. Permissive phases allow for the conflict to occur as the pedestrian path is crossed by turning vehicles, which are required to yield, but may not do so.

Left and right turn conflicts are based on turns crossing the active pedestrian path on the subject (analysis) leg as shown in Exhibit 14-50. The conflicts are coded for movements that cross the pedestrian path when departing the intersection, not arriving. Note that the pedestrian crossing on the departing side of the intersection is active (going with the through signal phases) while the arriving side is stopped. For example, in Exhibit 14-51, the northbound left turn conflict is coded to the west leg as this movement crosses the west leg crosswalk on green when departing the intersection, directly conflicting with pedestrians. While this movement also crosses the south leg crosswalk upon entering the intersection, this pedestrian movement is stopped. Each leg is analyzed in turn determining the potential conflicts allowed or not by geometry or phasing. Exhibit 14-51 shows the different kinds of left and right turning conflicts. Higher points are awarded for protected phasing than for permissive as shown in Exhibit 14-52 and Exhibit 14-55.

Exhibit 14-50 Vehicle Paths and Active/Stopped Crossings



Right turn-on-red is another source of conflicts, so points are increased when this conflict is eliminated (movement is prohibited by signing). In contrast to the right turn conflict, right turn-on-red conflicts are coded to the subject leg that contains the pedestrian path that this movement will cross to enter the intersection. The active pedestrian crossing is on the entering intersection leg instead of the departing leg. For example, in Exhibit 14-51, the eastbound right-turn-on-red movement is coded to the west leg as it directly conflicts with pedestrians.

Left turn-on-red movements are a special case as they only apply for intersections of two one-way streets or a two-way street and a one-way street. Points are assigned if a left turn can be made legally into the proper lane on (typically the curb lane unless otherwise marked) the one-way cross street or is prohibited by signing as shown in Exhibit 14-53. This adjustment does not apply and is skipped for intersections of two two-way streets or for incompatible lane configurations on approach legs. Left-turn-on-red conflicts are coded (the same way as right-turn-on-red conflicts) to the subject leg that contains the pedestrian path that this movement will cross to enter the intersection.

Exhibit 14-51 Pedestrian-Vehicle Crossing Conflicts

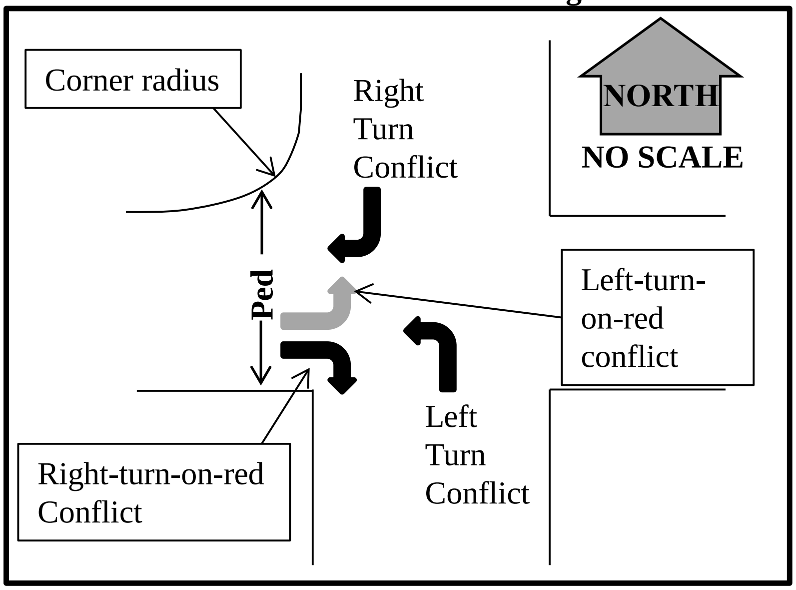


Exhibit 14-52 Left Turn Conflicts

Left Turn Lanes	Left turn phase	Points
1 – shared or exclusive	Permissive	-5
	Protected-permissive	0
1 or 2 - exclusive	Protected	10
2 – shared/ exclusive	Permissive	-10
No turn conflict – "T" intersection, one-way, mid-block crossing, or exclusive pedestrian phase		15
Left turn-on-red allowed[1]		0
Left turn-on-red prohibited (or no conflict)[1]		5

[1]Left turn-on-red adjustments are only considered for left turns going onto a one-way street from a one-way or two-way street. This adjustment does not apply in any other configuration.

The points are also adjusted for left turn conflicts coming from a two-way street onto a one-way street (Exhibit 14-53 and Exhibit 14-54) to account for the increased simultaneous exposure from turn conflicts across the entire width of the street. Even though vehicles executing a proper turn would turn into the curb lane, this is frequently ignored (or may be allowed by striping) so a left turning vehicle could end up in any of the receiving lanes. The left turn conflict for a one-way-to-one-way street or from left and right turn conflicts at a two-way to two -way street would only affect one part of the overall pedestrian crossing.

Exhibit 14-53 Two-Way to One-Way Street Conflicts

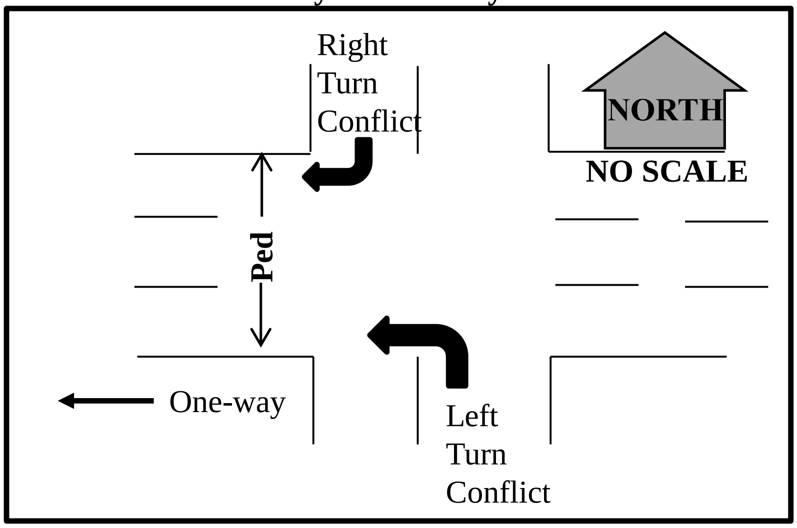


Exhibit 14-54 One-way Street Adjustment[1]

Left Turn Phase Type	Points
Permissive or Protected-permissive	-10
Protected	-2

[1]Only applies when the turn is made from a two-way to a one-way street.

Exhibit 14-55 Right Turn Conflicts

Right Turn Lanes	Right turn phase	Points
1 - shared or exclusive	Permissive	-5
1 - exclusive	Protected-permissive (Overlap)	0
1 or 2 - exclusive	Protected	5
2 – shared/ exclusive	Permissive	-12
No turn conflict – “T” intersection, one-way, mid-block crossing or exclusive pedestrian phase		15
Right turn-on-red allowed		0
Right turn-on-red prohibited (or no conflict)		5

Points are also given for different kinds of pedestrian signal treatments. Leading pedestrian phases where pedestrians start walking before vehicles get the green, countdown timers, or slower assumed walking speeds will all get higher points as shown in Exhibit 14-56.

Exhibit 14-56 Pedestrian Signal Displays

Display Type	Points
Standard (walk/don't walk)	0
Leading pedestrian phase	4
Countdown timer	5
Countdown timer and walking speed basis <3.5ft/s	8
Leading pedestrian phase & countdown timer	8
Leading pedestrian phase, countdown timer and walking speed basis <3.5ft/s	12

Pedestrian Delay

Pedestrians all expect to wait to cross for some period at a signalized intersection. However, an "excessive" amount of pedestrian delay can be incurred waiting to cross a busy intersection if multiple signal cycles are spanned and if a crossing cannot be done all at once. Long wait times, especially when experienced on a median island, can diminish the walking experience. Some complex intersections may require two or three separate crossings between islands where, if not carefully managed, the cumulative wait time could equal the walking time for a short trip. Too much delay may increase the likelihood of someone crossing at an unprotected midblock location or not obeying the signal indications which is a safety concern.

The pedestrian delay criteria are based on the tolerance for waiting which the maximum would be about 45 seconds (for the typical three-phase signal). Shorter cycle lengths for two-phase signals get increased points as potential violations would be substantially reduced while longer lengths for four-phase signals, signals with long cycle times over 120 seconds, or intersections/mid-block crossings that require more than one cycle length (cannot be crossed completely in one pedestrian phase) get negative points as shown in Exhibit 14-57. Signals that have pedestrian phases that end more than five seconds before the start of the yellow phase are regarded as being too short as the walk/don't walk phase could be a few seconds longer. These may incur additional delay for the pedestrian as they would have to wait for the next cycle. Use the cycle length corresponding to the analysis period used for other methodologies if cycle lengths vary over the day. Intersections with longer cycles than what is shown in the table for the number of phases should use the next higher value. For example, a three-phase signal that has a 110-second cycle should use the point value for the four-phase signal.

Exhibit 14-57 Pedestrian Delay

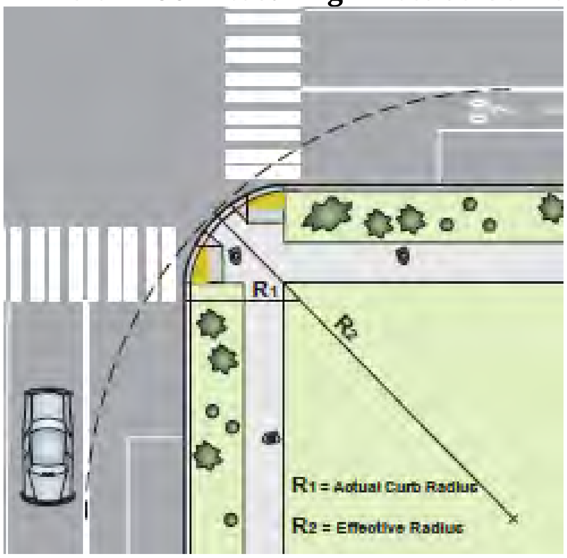
Signal Phases/Cycle length	Points
Two-phase (maximum 60 s)	5
Three-phase (maximum 90 s)	0
Four-phase (maximum 120 s)	-5
Any intersection with cycle length >120 s	-8
Adjustment for each extra cycle required for crossing	-5
Adjustment for pedestrian phase ending more than 5 seconds before start of yellow	-5

Corner Radius

Corner radius primarily impacts pedestrian safety and comfort by being a significant determinant of the speed at which vehicles are likely to cross the pedestrian path. At large values, it can also add to the crossing distance. The effect of wide lanes, bike lanes, & parking lanes on vehicle speed can be captured by considering the effective corner radius rather than simply the curb radius. Tighter corners with effective radii of 30 feet or less are rated best while wider effective radii of 50 feet or more impact the pedestrian enough to obtain negative values. The effective radius should be measured from edge of the bicycle lane around the corner to the edge of the bicycle lane on the cross-street as shown in Exhibit 14-58. Measurements can also be made from the edge of the travel lane if a bicycle lane does not exist, but parking does, or even from the curb is bicycle lanes and parking do not exist. The effective corner radius is assigned to the analysis leg corresponding to the right-turning movement departing the intersection (the same as a right turning conflict) as shown on Exhibit 14-58.

Exact measurements are not necessary as it only matters to be close enough to determine what radius category the corner falls into. Residential or central business district street intersections are likely 30 feet or less. Arterial street intersections will be likely in the 50-60 feet range especially if compound curves (more than one radius) are evident for accommodating larger vehicles. Collector –level intersections are likely in between. Effective radii in future alternatives should be measured from design plans, if available, for alternatives in active projects. Otherwise, coordinate with the appropriate ODOT Region, Traffic and/or-Roadway Section, or local jurisdiction design staff on assuming a typical radius to use for future improvements for a planning or design project. Exhibit 14-59 shows the point values assumed for each radius category.

Exhibit 14-58 Measuring Effective Corner Radii



Source: Oregon Bicycle & Pedestrian Design Guide, 2011, Fig 6-10, p.6-6.

Exhibit 14-59 Effective Corner Radius

Radius (ft)	Points
≤ 30	10
>30 and ≤ 40	5
>40 and ≤ 50	0
>50 and ≤ 60[1]	-10
>60[1]	-15

[1]May have compound curves present. Use approximate average radius between them when measuring.

If corner refuge islands are present instead of a regular corner radius, their effect on the score is based on the type of island (painted/raised), type of traffic control present for the channelized lane and whether the design forces vehicles to slow substantially as shown in Exhibit 14-60. This element, in contrast to the refuge island adjustments in Exhibit 14-48, considers the impact of the island on the departing leg crossing. Designs with lower speed channelized approaches have better visibility for oncoming traffic and pedestrians as less head movement is required. Higher speed approaches typically require the driver to look back over their shoulder to some degree and it is difficult to see pedestrians in this configuration.

Exhibit 14-60 Corner Refuge Island (in lieu of Corner Radius)

Island Type	Traffic Control	Right Turn Phasing Type	Points
Painted	Uncontrolled (free-flow)		-20
	Yield or signalized	All	-10
Curbed	Uncontrolled (free-flow)		-20
	Yield or signalized	Permissive or Permissive/Protected	0
	Signalized	Protected	5
Curbed – Low Speed	Yield or signalized	Permissive or Permissive/Protected	5
	Signalized	Protected	10

Crosswalk Treatment

The more visible a crosswalk is, the more awareness that a driver will have to the potential of pedestrians crossing the street. An unmarked crosswalk at a signalized intersection gets negative points on account of additional exposure required or more risk for the pedestrian. Crosswalk treatments beyond just the standard transverse or ladder striping garner positive points as shown in Exhibit 14-61.

Exhibit 14-61 Crosswalk Treatment

Crosswalk Treatment	Points
Unmarked	-5
Marked with transverse or ladder striping	0
Raised[1] and marked across entire approach	5
Raised[1] and marked across channelized right turn lane	5

[1]Raised crosswalks may not be appropriate on some state highway approaches or right turn lanes

Example 14-13 Signalized Intersection Pedestrian Level of Service

An urban intersection needs to be assessed for Signalized Pedestrian LOS as part of an analysis project. The intersection is at the junction of a north-south four-lane two-way street and a westbound two/three lane one-way street. Railroad tracks are adjacent to the intersection across the east leg and the pedestrian stop-bar is 25’ from the edge of the curb. There are no medians or refuges on any of the legs. Corner radius was measured at 15’ for all corners. All crosswalks are open and marked with standard transverse stripes. All curb ramps are of modern ADA design. A site inventory revealed the following additional signal phasing data:

- Three-phase (protected-permissive in the northbound direction with permissive turns on all other legs) signal operation running at a 65 second cycle length
- Countdown timers present on all legs

- Leading pedestrian phase on north and south legs
- Right-turn-on-red prohibited on east leg

Source: Google Earth, 2017.

The individual criteria scores were determined from the methodology for each leg and are shown below with the general reason behind the score. Since all curb ramps are of modern ADA design and are in good condition, there is no limitation on the maximum number of points that a leg can have.

Criteria	Score			
	North Leg	East Leg	South Leg	West Leg
Crossing Distance	5 lane equivalents = 50 pts[1]	2 lanes = 78 pts	5 lane equivalents = 50 pts[1]	3 lanes = 76 pts
Left turn conflicts	No conflict = 15 pts	No conflict = 15 pts	Yes (WB permissive turn) = -5 pts	Yes (NB protected-permissive turn) = 0 pts
Left turn-on-red	Does not apply		Allowed = 0 pts	Does not apply
One-way street adjustment	Does not apply			Yes = -10 pts
Right turn conflicts	Yes (WB permissive turn) = -5 pts	No conflict = 15 pts	No conflict = 15 pts	Yes (SB permissive turn) = -5 pts
Right turn-on-red	Allowed = 0 pts	WB prohibited = 5 pts	No conflict = 5 pts	No conflict = 5 pts
Pedestrian signal displays	Leading pedestrian phase & countdown timers = 8 pts	Countdown timers = 5 pts	Leading pedestrian phase & countdown timers = 8 pts	Countdown timers = 5 pts
Pedestrian delay	Three phase signal with >90 seconds cycle = 0 pts			
Corner radius	15' effective radius = 10 pts			
Crosswalk treatments	Transverse striping = 0 pts			
Leg Totals	78	128	83	81
Leg LOS	B	A	B	B
Intersection LOS	= (78 + 128 + 83 + 81) / 4 = 92		B	

[1]Note that the placement of the railroad tracks lengthens the overall crossing distance as there so safe place to wait between the tracks and the street for westbound pedestrians. The "official" waiting area also makes it more difficult for drivers to notice the pedestrians as they are more focused on the corner. Eastbound pedestrians are easier to see. The extra distance is approximately 24 feet but affects westbound more than eastbound, so one extra lane equivalent was added to the total crossing width of this leg

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14.16.2 Bicycle Signalized Intersection LOS

The major issues for a bicyclist crossing at a signalized intersection are the amount of separation from motor vehicles and the overall traffic speed. Most people desire the greatest separation as possible from faster moving vehicles to achieve the desired comfort (stress) level. Features that maximize separation between vehicles and bicyclists are awarded the most points while ones that have minimal or no separation are penalized.

Turning conflicts, crossing distance (exposure), and traffic signal features are somewhat less important, but still prominent for overall safety needs. The factors included in the methodology affecting the bicyclist focus on separation, speed, signal phasing, physical distances, intersection layout and turning conflicts, while volumes are handled implicitly.

Bicycle Facility & Adjacent Traffic Speed

The space dedicated to bicyclists including any separation from the motor vehicle traffic stream and the speed of that stream are the largest factors in determining the quality of the intersection crossing. Higher speeds (and related volumes) add difficulty for the bicyclist on intersection approaches, especially in the conflict areas around right turn lanes. Bicycle facilities that greater separate the bicyclist from adjacent traffic rate the best while conditions that force the bicyclist to share the lane rate the worst. Points decrease as roadway speed increases and/or separation decreases. Sharing roadway space works best when the speeds are low (20-25 mph) as the bicyclist can generally keep pace with traffic. Higher speeds at 30 mph or more require some sort of separation such as standard, buffered or separated bike lanes. Exhibit 14-62 illustrates the arriving and departing legs on an approach as to be used in Exhibit 14-63. Exhibits 14-63 through 14-67 show the point values for a given bike facility type on an arrival leg bike facility combined with different departing facility types and roadway speeds.

Exhibit 14-62 Arriving & Departing Legs for Approach Bike Facilities

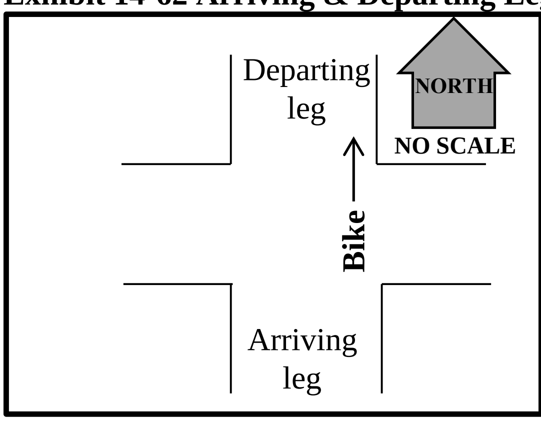


Exhibit 14-63 Arriving Leg - Shared Lanes

Shared Lane to Departing Leg Facility Type	Speed Limit (mph)	Points
To Shared Lane	≥ 40	-15
	30 - 35	10
	≤ 25	30
To Wide Outside Lane	≥ 40	0
	30 - 35	20
	≤ 25	35
To Bike Lane	≥ 40	15
	30 - 35	30
	≤ 25	40
To Buffered Bike Lane	≥ 40	30
	30 - 35	40
	≤ 25	45
To Separated Bike Lane	≥ 40	45
	30 - 35	50
	≤ 25	55

Exhibit 14-64 Arriving Leg - Wide Outside (Curb) Lanes

Wide Outside Lane to Departing Leg Facility Type	Speed Limit (mph)	Points
To Shared Lane	≥ 40	-5
	30 - 35	15
	≤ 25	30
To Wide Outside Lane	≥ 40	10
	30 - 35	30
	≤ 25	40
To Bike Lane	≥ 40	25
	30 - 35	40
	≤ 25	50
To Buffered Bike Lane	≥ 40	40
	30 - 35	50
	≤ 25	60
To Separated Bike Lane	≥ 40	55
	30 - 35	65
	≤ 25	70

Exhibit 14-65 Arriving Leg - Bike Lanes

Bike Lane to Departing Leg Facility Type	Speed Limit (mph)	Points
To Shared Lane	≥ 40	10
	30 - 35	25
	≤ 25	35
To Wide Outside Lane	≥ 40	20
	30 - 35	35
	≤ 25	45
To Bike Lane	≥ 40	40
	30 - 35	50
	≤ 25	60
To Buffered Bike Lane	≥ 40	50
	30 - 35	55
	≤ 25	70
To Separated Bike Lane	≥ 40	65
	30 - 35	70
	≤ 25	75

Exhibit 14-66 Arriving Leg - Buffered Bicycle Lanes

Buffered Bike Lane to Departing Leg Facility Type	Speed Limit (mph)	Points
To Shared Lane	≥ 40	20
	30 - 35	30
	≤ 25	40
To Wide Outside Lane	≥ 40	30
	30 - 35	40
	≤ 25	50
To Bike Lane	≥ 40	50
	30 - 35	55
	≤ 25	65
To Buffered Bike Lane	≥ 40	60
	30 - 35	65
	≤ 25	75
To Separated Bike Lane	≥ 40	70
	30 - 35	75
	≤ 25	80

Protected intersections are a new intersection treatment that has a wide appeal by separating the bicyclist from vehicles and pedestrians on all approaches. These allow for better visibility for drivers when turning right. These can be analyzed by using Exhibit 14-67 with a separated bike lane as the departing leg. If exclusive bicycle signal phasing exists, then no left or right vehicle turn conflicts can be additionally assumed.

Exhibit 14-67 Arriving Leg - Separated Bicycle Lanes

Separated Bikeway to Departing Leg Facility Type	Speed Limit (mph)	Points
To Shared Lane	≥ 40	35
	30 - 35	40
	≤ 25	45
To Wide Outside Lane	≥ 40	40
	30 - 35	45
	≤ 25	55
To Bike Lane	≥ 40	60
	30 - 35	70
	≤ 25	75
To Buffered Bike Lane	≥ 40	70
	30 - 35	75
	≤ 25	80
To Separated Bike Lane	≥ 40	80
	30 - 35	85
	≤ 25	90

Left Turn Conflicts

Left turn conflicts can be problematic for bicyclists as turning vehicles are typically travelling at higher speeds than right turns. Drivers may not see a bicyclist if they are concentrating on getting through a gap in traffic. The greatest chance of conflict is when the left-turn is permissive rather than purely protected. Intersection and signalization features that reduce or eliminate conflicts are rated the best and are the second contributing highest factor in the methodology. Exhibit 14-68 shows the bicycle-vehicle crossing conflicts and Exhibit 14-69 shows the point values for the opposing left turn conflicts. Left turn conflicts for bicyclists (note this is different from the facility types noted above) are coded to the approaching (arriving) leg for vehicles, so the southbound left turn conflicts with the northbound bicyclist in Exhibit 14-68 are coded to the north leg.

Exhibit 14-68 Bicycle-Vehicle Crossing Conflicts

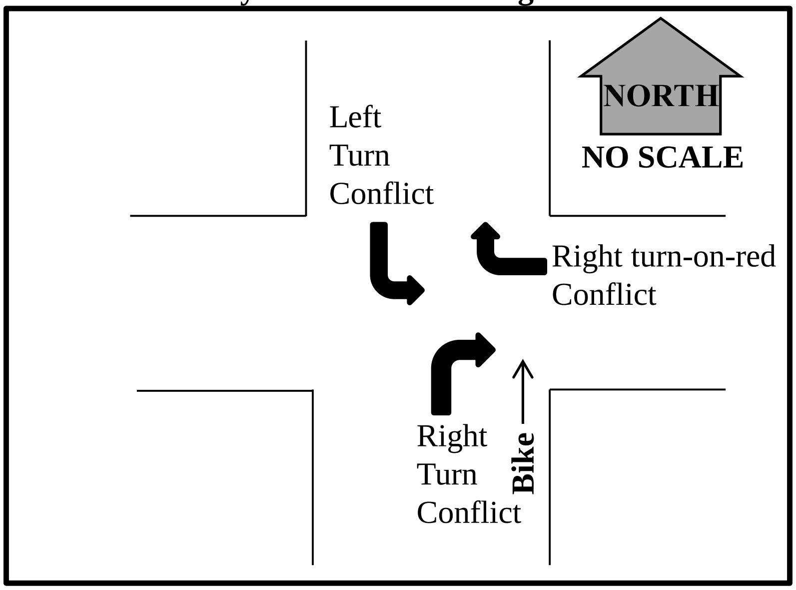


Exhibit 14-69 Left Turn Conflicts

Left Turn Phase Type[1]	Points
Permissive	0
Protected-permissive	5
Protected	15
No turn conflict – "T" intersection or one-way streets	15
Left Turn Adjustments	
Green "conflict area" paint	5
Two-stage turn box	10
Stop Bar Location	
Shared stop bar – vehicles and bikes stop at common point	0
Advance stop bar/bike box – bikes stop closer to intersection	10

[1]Left turn type that is opposing to the oncoming bicyclist.

Right Turn Conflicts

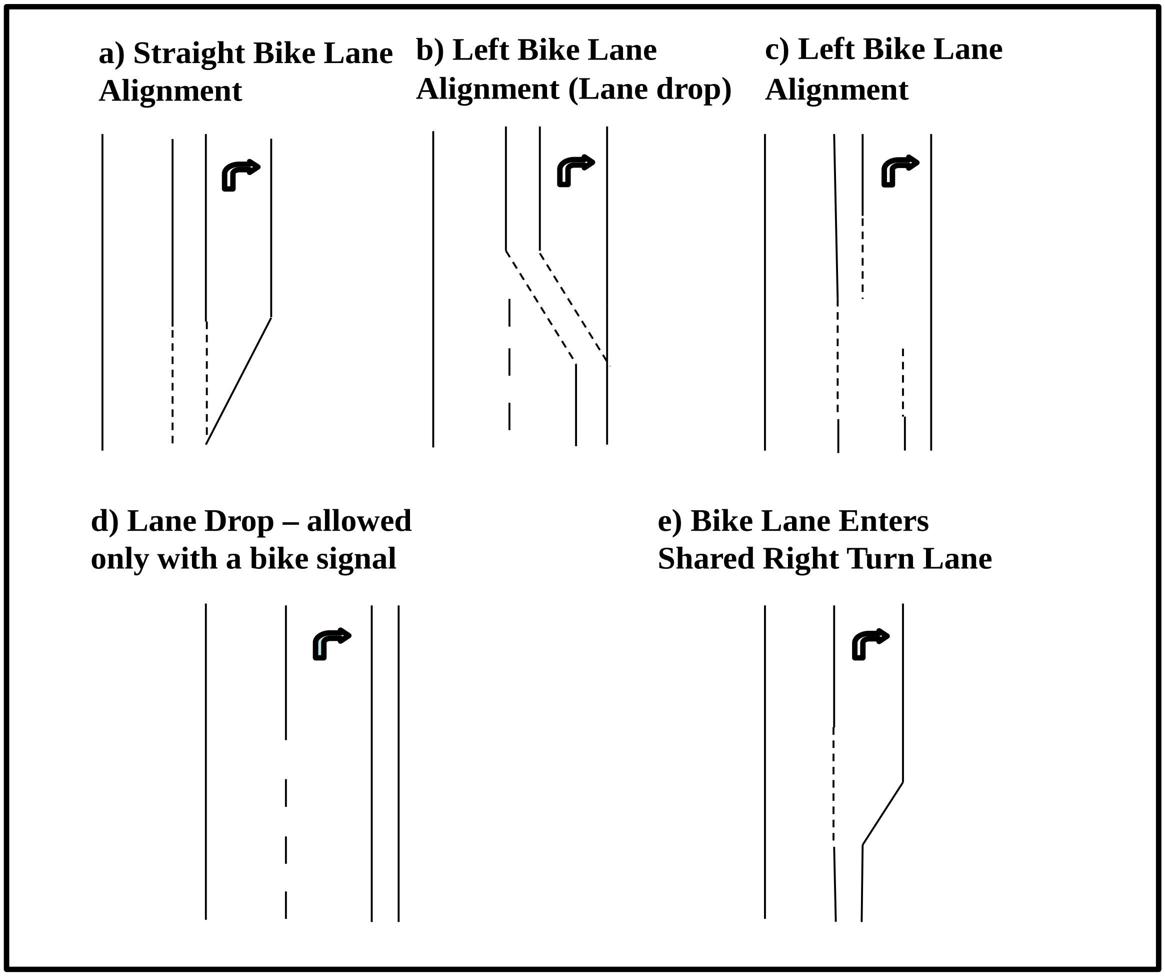
The most common right turn conflict involves a vehicle turning right and a bicycle heading straight ("right-hook"). The right turn conflict depends on the approach treatment at the intersection between the positioning of the right turn lane and the bicycle facility. Right turn conflicts are coded to the arriving approach leg for both vehicles and bicyclists, so as shown in Exhibit 14-68, so the right turn conflict on the northbound approach is coded to the south leg. The most desirable combination is to have the bicyclist traveling straight while the vehicle yields and merges to the right (bicycle lane is to the left of the right turn lane) as shown in Exhibit 14-70(a). Higher speeds and/or volumes may give bicyclists pause especially if there is a lack of yielding to bicyclists as vehicles cross over the bike lane to access the right turn lane. Longer right turn lanes, especially under higher volume conditions can create "sandwich" feeling for the bicyclist which can limit the actual use of the bike lane on the approach. It is recommended that coordination with either Region Traffic or Traffic-Roadway Section staff be done if higher speeds and volumes are prevalent in the study area as the point assignment may need to be modified.

Configurations that require the bicyclist to shift left (Exhibit 14-70(b) or Exhibit 14-70(c)) are awarded little or no points as the vehicle yielding behavior is less as there is too much of a straight shot into the right turn lane without a conscious need to move to the right. Exhibit 14-70(b) shows an older marking style while Exhibit 14-70(c) shows the current style. Both require bicyclists to shift to the left and are progressively more difficult to make as right turning volumes increase. Intersection approaches that do not have a bike facility require the bicyclist to share the vehicle lanes and shift left should also use Exhibit 14-70(e). Sharrow markings are optional for shared right turn lanes as shown in Exhibit 14-70(e).

The least desirable configuration is Exhibit 14-70(d) where the bike lane is to the right of the right turn lane. This configuration limits the ability of the driver to see any oncoming bicyclists as many drivers fail to check the right-hand mirror and blind spot which could result in a “right-hook” crash. This conflict can be removed with a presence of a bicycle signal by prohibiting right turning traffic while bicyclists have a green (and vice versa when vehicular traffic has a green). A bicycle signal can make this configuration significantly better than the standard design in Exhibit 14-70(a) by eliminating the turn conflict. However, this configuration should only be allowed with a bicycle signal. Adding bicycle signals will either require a significant bicycle volume, a high right turn volume or both. The presence of green paint in the conflict areas shows both drivers and bicyclist to be aware of potential conflicts and thus gets a positive point adjustment.

Adding bicycle signal phases will impact operation of the entire intersection as there will be less time for the other motor vehicle phases. Bicycle signal phases may also result in more delay for bicyclists as they would be prohibited from traveling with the vehicular phases. This tradeoff will need to be evaluated in context of all users, the local environment and the community.

Exhibit 14-70 Bike Lane Alignments



Right-turn on-red movements also create a safety issue as drivers are looking for approaching vehicles and may not see an oncoming bicyclist. Adjustments for right turn and right-turn-on-red conflicts are shown in Exhibit 14-71.

Exhibit 14-71 Right Turn Conflicts

Right Turn Conflict Type		Points
No exclusive right turn lane		5
Exclusive right turn lane	Right turn lane <500’ develops to the right of bike lane. Bike lane is left of right turn lane. (Exhibit 14.70a)	0
	Right turn lane ≥500’ develops to the right of bike lane. Bike lane is left of right turn lane. (Exhibit 14.70a)	-15
	Right turn lane drop, bike lane shifts left (Exhibit 14.70b or c)	-10
	Bike lane enters <100’ (including taper) shared right turn lane. Right turn lane volume <75 vph and at <25 mph (Exhibit 14.70e)	0
	Bike lane enters <100’ (including taper) shared right turn lane. Right turn lane volume <150 vph and at <25 mph (Exhibit 14.70e)	-5
	Bike lane enters >100’ (including taper) shared right turn lane or right turn lane volume >150 vph or >25 mph (Exhibit 14.70e)	-10
	No bike lane	0
	Bike lane right of right turn lane (Exhibit 14-70d)	-20
	Bike lane right of right turn lane with bike signal	15
No right turn conflict (“T” intersection, one-way street)		15
Right Turn Adjustments		
Green “conflict area” paint		5
Shared lane use marking in shared lane		5
Right Turn-on-red Conflict		
Allowed		0
Prohibited or no conflict		5

Crossing Distance

The wider the intersection, the greater exposure a bicyclist has to the cross-street traffic. Signal timing is often set for motor vehicle speeds and clearances, so wider intersections have a greater risk of having the bicyclist in the intersection in the phase change intervals. Exhibit 14-72 shows the adjustments for crossing distance based on total lanes (through and turn) crossed. If exclusive bicycle signal phasing exists on an approach, then this criteria does not apply.

Exhibit 14-72 Intersection Crossing Distance

Total Lanes Crossed	Points
≤ 3	0
4 – 5	-5
≥ 6	-10

Example 14-14 Signalized Intersection Bicycle Level of Service

An urban intersection was assessed for Signalized Bicycle LOS as part of an analysis project. The intersection is at the junction of an east-west two-lane street and a north-south three lane street. Bike lanes are present on the north, south and east legs while the bike share the lanes with vehicles on the west leg. The speed is 30 mph on both streets. The signalized intersection has protected left-turn phasing on all legs and the exclusive right turn lane on the north leg is controlled by a protected-permissive signal head.

Source: Google Earth, 2017.

The individual criteria scores were determined from the methodology for each leg and are shown below with the general reason behind the score.

Criteria	Score			
	North Leg	East Leg	South Leg	West Leg
Bicycle Facility and Traffic Speed	Bike lane to bike lane; 30 mph = 50 pts	Bike lane to shared lane; 30 mph = 25 pts	Bike lane to bike lane; 30 mph = 50 pts	Shared lane to bike lane; 30 mph = 30 pts
Left turn conflicts	Protected phasing = 15 pts	Protected phasing = 15 pts	Protected phasing = 15 pts	Protected phasing = 15 pts
Stop Bar Location	Shared stop bar = 0 pts	Shared stop bar = 0 pts	Shared stop bar = 0 pts	Shared stop bar = 0 pts
Right turn conflicts	Exclusive right lane; bike lane shifts left = -10 pts	No exclusive right turn lane = 5 pts	No exclusive right turn lane = 5 pts	No exclusive right turn lane = 5 pts
Right turn-on-red	Allowed = 0 pts	Allowed = 0 pts	Allowed = 0 pts	Allowed = 0 pts
Intersection Crossing Distance	3 lanes crossed = 0 pts	4 lanes crossed = -5 pts	3 lanes crossed = 0 pts	3 lanes crossed = 0 pts
Leg Totals	55	40	70	50
Leg LOS	C	D	C	D
Intersection LOS	= (55 + 40 + 70 + 50) / 4 = 54		D	

14.17 Transit LOS

14.17.1 Methodology Summary

Unlike the simplified Pedestrian and Bicycle LOS methods, there is no re-estimated Transit LOS. Instead, this is a streamlined version of the regular HCM Transit LOS methodology using simplifying assumptions and specific defaults. The full transit methodology involves calculating transit vehicle running time, delay, and speeds, then determining impacts caused by waiting times, stop amenities, and pedestrian access. Like with other MMLOS methods, the methodology is done separately for each direction of travel. This simplified method should only be applied to segments within the study area that have applicable fixed-route transit.

It also would be possible to estimate the Transit LOS within a travel demand model if transit routes were considered explicitly so the frequency could be captured. The travel times for the various route segment would need to be summed across each major segment and the other inputs likely defaulted with use of some custom variables or expressions.

The pedestrian LOS portion of the calculation could be computed from volumes, speeds and number of directional lanes which are common variables in a travel demand model and the sidewalk variable either assumed or based on field data.

The simplified methodology uses transit schedule speed, instead of calculating a transit travel speed, to consolidate the first three steps of the full MMLOS process. Schedule speed ultimately controls as transit vehicles will dwell at time points if they are ahead of schedule. Schedule speeds are also periodically reviewed and adjusted to account for ridership, dwell times, and traffic conditions.

For segments with heavy congestion, where travel speeds are substantially lower, the transit speed should be considered instead if the schedule does not reflect extra time for the regular peak hour congested conditions. Alternatively, the actual transit vehicle speed can be used if available from recent surveys or preferably from active GPS installations on the transit vehicles from the transit district. Note that other private data source travel times will only reflect the running speed of the average vehicle on the segment and will be too high for use as they will not reflect the transit stop delay and dwell times.

The re-estimated Pedestrian LOS (Section 14.9) is used in the Transit LOS calculation and is equated to an LOS score to avoid needing to calculate a full-detailed Pedestrian LOS score. The rest of the methodology uses reasonable defaults and assumptions using the HCM equations.

This method is applicable for urban street-running transit vehicles operating in an exclusive or a mixed-use lane. While the typical transit vehicle is a bus, it should not be assumed that this is always the case as this methodology also applies for bus rapid transit (BRT), streetcars, or light rail operating in mixed mode street-running conditions. Analysis of transit operating in a separated right-of-way such as adjacent to a street, in a median, or grade-separated is not covered under this methodology but in the companion *Transit Capacity and Quality of Service Manual*.

14.17.2 Transit LOS Criteria

LOS scoring threshold criteria for the transit mode is shown below in Exhibit 14-73. These are based on updated values of HCM Exhibit 18-2 and 18-3. The pedestrian LOS input has been converted from a range into an averaged single value for input into the final transit LOS equation.

Exhibit 14-73 Transit LOS Criteria

LOS	Pedestrian LOS Score, I_p	Transit LOS Score, I_t
A	0.75[1]	≤ 2.00
B	2.00	>2.00 – 2.75
C	3.00	>2.75 – 3.50
D	4.00	>3.50 – 4.25
E	5.00	>4.25 – 5.00
F	5.75[2]	>5.00

[1]The average score for LOS A is based on the minimum value of 0.00 and the highest score of 1.5.
[2]The average score for LOS F is based on the maximum value of 6.0 in HCM Equation 18-63 and the minimum value of 5.50.

Like with the other modes, these LOS scores are based on user perceptions (traveler satisfaction) and are graded from best (LOS A) to worst (LOS F). This kind of perception-based rating varied from the many test respondents (there is no one single definition of a multimodal LOS grade) and was eventually grouped into LOS ranges. Better conditions will result in better LOS scores. For example, more frequent transit service will rate better than less frequent service.

Transit LOS is heavily influenced by frequency, and frequency is influenced by land use density and availability of capital (vehicles) and operating (employees) funds. Therefore, a low LOS score may simply reflect the maximum feasible capability of a transit district on a particular route and should not be immediately equated with “poor” service. Better service (and LOS) may not be possible because of restricted funding and/or the land use is not dense enough to support it. The funding context of a particular transit district needs to be considered when reporting Transit LOS values.

Data Needs and Definitions

Transit Schedule Speed (S_t) – This is the speed (mph) calculated by dividing a known segment length by the difference of two adjacent time points published in the route timetable. If a segment covers parts of two sets of time points, then the resulting schedule speeds should be weight averaged. If there is more than one route on a given segment, then the schedule speeds should be averaged, or weight averaged if frequencies are different.

Transit Frequency (v_s) – This is the number of transit vehicles per hour on the directional segment. Sum up the frequency of all routes that may travel this segment. Start with the route that has the highest frequency during the analysis hour(s). For additional routes, note which times are offset versus ones that seem to duplicate.

For instance, a 60-minute route that runs on the same schedule as another 60-minute route will result in one vehicle following another and should only be coded as one vehicle per hour. If the two routes were offset (say one on the hour and the other on the half-hour), then code this as two vehicles per hour. The route duplication does not have the same frequency benefit for the rider compared to a more even time spacing.

For corridors with very frequent service, like with in-road bus rapid transit or light -rail transit, consider these routes to likely control the segment as the short headways (typically less than 10 minutes) makes it unlikely that additional routes will have times that do not duplicate, so there will not be any additional frequency benefit. Only consider “tripper” service (additional frequency and/or minor route changes to serve schools) if it is active during the chosen analysis period.

From the transit frequency, the headway factor (F_h) is computed from HCM Equation 18-56:

$$F_h = 4.00e^{-1.434/(v_s+0.001)}$$

All of the following inputs are defaults with the exception of needing the Pedestrian LOS data in Section 14.9.

Passenger Load Factor (a_1) – For this simplified methodology, this factor is assumed to be 1.0, which represents that, on the average of all transit vehicles using that segment in the desired period, they are 80% or less full (0.8 passengers per seat). On congested segments where passenger per seat ratios are higher than 80% and up to 100%, use passenger load factors in Exhibit 14-74. If overcrowding exists on the average where the numbers of passengers exceed the number of seats (presence of standees), please refer to the 3rd case of HCM Equation 18-59 for computing the appropriate passenger loading factor.

Exhibit 14-74 Passenger Load Factors[1]

Passenger to Seat Ratio	Passenger Load Factor(a_1)
<0.81	1.00
0.85	1.05
0.90	1.10
0.95	1.14
1.00	1.19
>1.00	Use HCM Equation 18-59

[1]Derived from *Highway Capacity Manual 6th Edition* Equation 18-59

Other general defaults are the threshold late time, which is the time that transit agencies typically consider a vehicle late, set at 5.0 minutes, and the proportion of transit vehicles arriving within the late time threshold, set at 0.75 (75% considered to be on-time). The late time threshold proportion value could be adjusted if it is desired to estimate reliability impacts. Both can be changed if more specific information is available from a transit district.

A large part of the full methodology involves calculating perceived travel time rates and factors which involves the transit speed, on-time ability, and stop amenities such as benches and shelters. Passengers generally view excess waiting time worse than slower travel speeds but may wait longer if there are amenities.

With the above default threshold late time and the on-time arrival percentage in HCM Equation 18-61, a fixed excess wait time of 1.6 minutes is calculated. The excess wait time is converted into the excess wait time rate of 0.41 min/mi by dividing the excess wait time by the average trip length of 3.8 miles based on reported Oregon transit system data in the National Transit Database. The proportion of shelters and benches in the full methodology has been found not to be very sensitive[14] to the results and is ignored in this simplified method. The perceived travel time rate (HCM Equation 18-58) equation can be simplified with the above defaults and simplifications to:

T_{ptt} (min/mi) = $60/S_t + 0.86$, where S_t is the transit schedule speed

Use the equation form T_{ptt} (min/mi) = $a_1[60/S_t] + 0.86$, where a_1 is the passenger loading factor, if the default 1.0 value was not used.

The perceived travel time factor (F_{tt}) is a combination of the perceived travel time rate and the base travel time rate, which is assumed to be defaulted at 4.0 min/mi for areas below five million in population. There also is a default ridership elasticity factor of -0.40 which considers changes in the travel time rate. Using the above defaults in HCM Equation 18-57, the equation reduces to:

$$F_{tt} = (5.6 + 0.6T_{ptt}) / (1.4T_{ptt} + 2.4)$$

The final Transit LOS score (I_t) is the combination of the wait-ride score and the Pedestrian LOS score (I_p) based on HCM Equation 18-63. The wait-ride score portion of the equation is the product of the perceived travel time factor F_{tt} and the headway factor F_h.

$$I_t = 6.0 - 1.5\,(F_h * F_{tt}) + 0.15I_p$$

The Pedestrian LOS (I_p) is calculated for the directional segment using the methodology in Section 14.9. Since the Pedestrian LOS from Section 14.9 is based on a probability of the entire range, the average LOS score is used for I_p in Exhibit 14-67, which is based on HCM Exhibit 18-3. If the Pedestrian LOS results in a range of levels, then the appropriate Pedestrian LOS scores in the second column in Exhibit 14-67 should be averaged together. The Transit LOS score (I_t) is then compared to the transit score range in the third column in Exhibit 14-73 to determine the final LOS for the directional segment.

[14] Carter, P., Martin, F., Nunez, M., Peters, S., Raykin, L., Salinas, J., et al. (2013). Complete Enough for Complete Streets? Senstivity Testing of Multimodal Level of Service in the Highway Capacity Manual. *Transportation Research Record: Journal of the Transportation Research Board, No,2395*, 36-37.

Example 14-14 Transit LOS

The same suburban arterial segment from the previous examples is used to continue the multimodal analysis. There are two transit routes on this roadway segment, one on 15-minute and one on 30-minute headways. The schedules are offset enough so the 30-minute route does not directly overlap the 15-minute route. For the analysis period, both routes are operating less than 80% full.

The first step is to calculate the schedule speed of the directional segment. For the 15-minute headway route, the available transit schedule shows seven minutes to travel from time point A to B. Measurement of the distance via aerial photos between A and B results in 5405 feet. The 30-minute route shows four minutes (apparently assuming less stops) between its time points C and D that bracket the analysis segment. The distance from C to D is measured as 6150 feet.

The schedule speed (S_{t15}) for the 15-minute route is calculated as:

S_{t15} = (segment length in feet / 5280 feet per mile) / (travel time in minutes / 60 minutes per hour)
= (5405 ft / 5280 ft/mi) / (7 min / 60 min/hr)
= 1.024 mi / 0.117 hr
= 8.75 mph

The schedule speed (S_{t30}) for the 30-minute route is calculated as:

S_{t30} = (segment length in feet / 5280 feet per mile) / (travel time in minutes / 60 minutes per hour)
= (6150 ft / 5280 ft/mi) / (4 min / 60 min/hr)
= 1.165 mi / 0.067 hr
= 17.48 mph

The average schedule speed (S_t) is weight averaged between the two routes as the 15-minute route has four vehicles per hour (67% of total) and the 30-minute route has two vehicles per hour (33% of total) for a total of six.

S_t = 8.75(0.67) + 17.48(0.33)
= 11.63 mph

Next, the headway factor (F_h) is computed from the overall transit frequency (6 veh/hr):

$F_h = 4.00e^{-1.434/(v_s+0.001)}$
$= 4.00e^{-1.434/(6+0.001)}$
$= 3.15$

The perceived travel time rate (T_{ptt}) is computed from the overall schedule speed:

$$
\begin{aligned}
T_{ptt} &= 60/S_t + 0.86 \\
&= 60/11.63 + 0.86 \\
&= 6.02 \text{ min/mi}
\end{aligned}
$$

The perceived travel time rate (T_{ptt}) is inserted into the simplified travel time factor (F_{tt}) equation:

$$
\begin{aligned}
F_{tt} &= (5.6 + 0.6T_{ptt}) / (1.4T_{ptt} + 2.4) \\
&= (5.6 + (0.6 * 6.02)) / ((1.4 * 6.02) + 2.4) \\
&= 9.21 / 10.83 \\
&= 0.85
\end{aligned}
$$

The final Transit LOS score (I_t) is calculated using the headway factor and the travel time factor from previous steps in addition to the Pedestrian LOS score (I_p) for the segment. The Pedestrian LOS was LOS C from the first example. This equates to an average LOS score in Exhibit 14-67 of 3.00.

$$
\begin{aligned}
I_t &= 6.0 - 1.5 (F_h * F_{tt}) + 0.15I_p \\
&= 6.0 - 1.5 (3.15 * 0.85) + 0.15(3.00) \\
&= 6.0 - 4.02 + 0.47 \\
&= 2.43
\end{aligned}
$$

Comparing the final LOS score with Exhibit 14-67 shows this segment to have a Transit LOS B (both directions the same).

Schedule Speed mph	Headway Factor	Perceived Travel Time Rate min/mi	Perceived Travel Time Factor	Pedestrian LOS Score	Transit LOS Score	Transit LOS
11.63	3.15	6.02	0.85	3.00	2.43	B

Some of the calculator inputs and intermediate calculations done above are shown below as an example. Note that some columns are hidden.

14.18 Transit Capacity and Quality of Service Manual

14.18.1 Overview

The *Transit Capacity and Quality of Service Manual* (TCQSM), 3rd edition (TCRP Report 165), is a comprehensive reference manual used to assist with public transit analysis and methods and is a companion document to the HCM. The purpose of this section is to give a high-level synopsis of the manual as generally it is not as well-known as the HCM.

Generally, the TCQSM is used when trying to determine the capacity and performance of transit services (i.e. bus, light and heavy rail, ferry etc.) rather than just the impact of the vehicles on the roadway system. Transit outside of the roadway right-of-way (i.e. bus rapid transit busways, heavy rail commuter service etc.) would need to be analyzed using TCQSM methodologies. While quality of service methods is still in the TCQSM, most of this analysis is covered already in other portions of Chapter 14, so there is more coverage of advanced transit topics.

Previous editions contributed service frameworks and filled in gaps in transit capacity and quality of service knowledge. This edition focuses on providing tools and techniques to evaluate critical aspects of transit operations which include analyzing capacity, speed, reliability, and quality of service for all transit needs on a consistent basis. These methodologies can be applied across different transit modes, such as buses and trains, and are important for decision-making in transit planning, infrastructure investment, and policy development. TCRP Project A-47, in progress as of 2025, was developing the TCQSM 4th edition, with publication expected in 2026.

14.18.2 Key Concepts

The key concepts in the TCQSM are quality of service, capacity, and speed/reliability. These key concepts are central to understanding and applying the manual's methodologies.

Quality of service refers to the overall experience of transit service as measured or perceived by the passenger. It involves factors like comfort, frequency, reliability, and convenience. While passengers typically seek high levels of service, the transit agency must balance this desire with practical constraints, such as budget limitations and demand. The quality of service is also influenced by the operational decisions of a transit agency, such as where and how often service is provided, and the level of service that is affordable or reasonable for the agency to deliver. Furthermore, these decisions are generally shaped by the agency's goals and objectives, like providing equitable access, reducing congestion, or improving sustainability.

Capacity is the maximum number of passengers, transit vehicles, or both, that can pass a given point in a specified time and under specific conditions. In the TCQSM, different types of capacity are discussed, focusing on both vehicles and passengers, depending on the type of transit system. In general, transit service should be designed for a practical ("design") capacity that reflects normal operating conditions and allows for some operational irregularity, rather than a maximum capacity achievable only under ideal conditions. Capacity is critical for understanding how much demand a system can handle, influencing decisions about route design, vehicle frequency, and infrastructure investment. Capacity limits are essential for planning purposes, helping agencies assess whether their systems will operate at or beyond their capacity.

Speed (or travel time) and reliability are vital for passenger satisfaction and influence overall ridership. Speed refers to how quickly a transit vehicle travels between locations, whereas reliability concerns how consistently the service adheres to its schedule. Both speed and reliability directly impact operating costs. For example, longer travel times or less reliable service require more vehicles and larger budgets to maintain service levels. These factors also influence the transit agency's ability to meet passenger demand efficiently. The manual highlights that factors impacting capacity also influence speed and reliability. For example, if the number of vehicles on a street approaches its capacity, it may lead to delays or slower travel times, reducing overall service quality.

These three concepts are fundamental to understanding how transit systems operate and how they can be evaluated. The TCQSM emphasizes that these concepts are interconnected and affect each other in various ways. For instance, decisions about capacity (such as increasing the number of vehicles on a route) can influence both speed (reducing delays) and reliability (increasing on-time performance). Similarly, increasing quality of service often requires adjustments in capacity, speed, and reliability to meet passenger needs effectively.

14.18.3 Manual Organization

The TCQSM is arranged into concepts and methods chapters, similar to the HCM. These chapters work together to provide both foundational knowledge and practical methodologies for evaluating transit systems' capacity, quality of service, and related performance measures.

Chapter 1 is a user's guide to the rest of the manual serving as an introduction to the chapters and concepts. This APM section is a summary of that chapter.

Concept Chapters 2 – 4 lay the groundwork by defining essential terms and presenting key ideas that are fundamental to understanding transit capacity, speed, reliability, and quality of service. Method Chapters 5 – 10 focus on providing computational methods for evaluating quality of service, transit capacity, and other performance metrics.

Mode and Service Concepts (Chapter 2): This chapter introduces the major types of transit modes addressed by the TCQSM—fixed route bus transit, demand-responsive transit (DRT) such as paratransit ("dial a ride"), and rail transit - along with their sub-modes (e.g., light rail, heavy rail, commuter rail). It also covers vehicle types and the general characteristics of fixed route and DRT services.

Operations Concepts (Chapter 3): This chapter explores how transit speed, capacity, and reliability are influenced by factors within and outside a transit agency's control. It covers topics like passenger demand patterns, dwell times, operating environments, and stop/station characteristics. Graphs are provided to show the relative impacts of these factors.

Quality of Service Concepts (Chapter 4): This chapter focuses on the role of transit in a community and the different perspectives of stakeholders regarding service performance. It emphasizes the passenger's point of view on quality of service, introducing key factors that influence satisfaction, such as availability, comfort, and convenience. The chapter also introduces the quality-of-service framework that will be expanded upon in Chapter 5.

Quality of Service Methods (Chapter 5): This chapter provides methods for evaluating fixed-route and demand-responsive transit availability, comfort, and convenience from the passenger's perspective. It also introduces a method for evaluating the transit level of service in a multimodal context.

Bus Transit Capacity (Chapter 6): This chapter discusses factors that affect bus capacity, speed, and reliability, including infrastructure and operational measures that can improve bus performance. It includes methods for evaluating bus capacity and speed and offers general information on the causes of bus unreliability, though no detailed forecasting methods are provided.

Demand-Responsive Transit (Chapter 7): This chapter covers the factors influencing DRT capacity and presents approaches and resources for estimating the number of vehicles and service hours required to meet DRT demand.

Rail Transit Capacity (Chapter 8): This chapter focuses on rail-specific capacity concepts, including train control, signaling, and operations. The chapter provides computational methods for estimating rail transit system capacity, with detailed guidance on measuring input values for calculations.

Ferry Transit Capacity (Chapter 9): This chapter describes ferry service-specific aspects like service planning, infrastructure, and scheduling. It includes methods for estimating ferry vessel capacity, including passenger and vehicle accommodation.

Station Capacity (Chapter 10): This chapter covers the design and capacity of transit stations, including access for people with disabilities, emergency evacuation, and security. The chapter discusses different types of stations and stops, and it provides methods for evaluating station features such as passenger circulation, vehicle circulation, and bike storage.

14.18.4 Applications

The TCQSM supports a comprehensive range of transit agency activities—from high-level policy and planning decisions to detailed operational and design analyses. One of the manual's most powerful contributions is its support for passenger-focused quality of service standards, evaluation, planning, and design. This allows strategic thinking around service quality from the passenger's perspective, enabling data-informed, rider-centered decisions.

Potential applications of the TCQSM methodologies include:

- **Transit mode guidance:** This includes transit mode concepts, operations, terms, and effects of transit treatments (i.e. transit signal priority).
- **Evaluation**: The methodology can be used to analyze and diagnose operational issues.
- **Comparisons**: Different transit modes (i.e. bus vs. light rail), alignments, frequencies, or fare collection types among others can be compared to answer "what-if" kinds of questions on the overall operations and quality of service.
- **Planning**: Either sketch-level or detailed operational methodologies can be applied to analyze a particular mode or alternative.
- **Design:** The manual can be used for supporting a wide range of sizing and capacity applications such as numbers of bus bays needed, platform lengths, passenger circulation areas, stop spacing, or line capacities.

15 TRAFFIC SIMULATION MODELS

This is the original APM v1 Chapter 7 text. An update is in progress.

15.1 Purpose

Traffic simulation models are complex tools that can provide valuable information on the performance and potential improvement of transportation systems. Traffic simulation models are in a constant state of improvement and accordingly this chapter attempts to be adaptive with the changes in the industry. This chapter currently presents instruction on calibration of microsimulation models created in Trafficware's SimTraffic and a brief overview of the other simulation models and parameters used in ODOT projects. Topics covered include:

- Traffic Simulation Modeling – General Calibration Instructions
- SimTraffic – Overview and Calibration Instructions
- VISSIM – Overview
- Paramics - Overview
- CORSIM – Overview

15.2 Traffic Simulation Modeling – General Calibration Instructions

Traffic simulation models are computer programs that simulate traffic movements over a user-defined transportation network and present the results via animation and reports. The degree of user control over the simulation and the types of facilities that can be modeled will vary depending on the program being used. These should not be confused with urban travel demand models (Chapter 6), which use current and projected land use and transportation network data to estimate current and future travel demand and traffic patterns.

Traffic simulation models (meso or microscopic) are complex tools that generally require more labor than programs that perform capacity analysis at a macro level. Because of this, they are generally only used when the use of other types of analysis tools will not be adequate for a given project. Simulation models offer a greater degree of flexibility than most programs designed specifically for capacity analysis and can be used for a wide range of analysis needs such as examining the interactions between different modes of transportation, modeling the operations of HOV lanes or bus priority systems, and evaluating operations through measures of effectiveness not offered by most other types of analysis programs. Simulation models are also very useful for presentations, especially for those given to audiences lacking technical knowledge of traffic analysis, because it provides a visual basis for evaluating operations that most people can easily relate to and understand.

Simulation models are commonly used by ODOT to analyze corridors or networks under congested conditions, where upstream or downstream operations have a significant influence on actual intersection operations (e.g., intersection blockage from queue spillback). It should be noted that simulation models use different methodologies for estimating queue lengths than other procedures described in this manual. These methodologies are typically based on observations of queues experienced during simulation, which are influenced by parameters such as driver characteristics, lane changing behavior and various traffic flow interactions. Capturing the impact of up and downstream operations on vehicle queues can make these models very effective at estimating queue lengths but underscores the importance of good model calibration. General guidelines for the application of simulation models have been published by the Federal Highway Administration, which can be found at the FHWA website under traffic analysis tools.

Depending on the specific program used, there may be numerous parameters that can be manipulated by the user to create a system that most accurately represents the one being analyzed. Before any simulation model is used to represent existing or future conditions, the existing conditions model created must be calibrated by adjusting operational parameters until the model provides a reasonable representation of existing conditions measured in the field. Existing conditions need to be replicated; otherwise future conditions will not be correct. Existing conditions should include only data, operations and measures known to currently exist in the project study area. Vehicle counts should be kept as close as possible to the original volumes obtained from the field. If all counts are available from the same day, vehicle counts used during calibration should be un-factored and unbalanced counts (this day should be as close to the 30th highest hour as possible). If counts cannot all be collected on the same day (or year), every effort should be made to collect counts at primary locations on a day that is on or closely represents, the 30th highest hour. The remaining counts can then be factored and balanced to this primary count day. If all counts occur on scattered days and none of the counts occur on the 30th highest hour or on a representative day, then short sample count should be conducted to factor the off- peak counts to the day the study area was visited. Use the seasonal factor methodology described in Chapter 5 to determine if the count is close enough to the 30th highest hour. If the primary counts for the study area occurred during a time that is less than 90% of the 30th highest hour for that area seasonal trend type, then a re-visit with a sample count is required for the calibration of the “existing” model.

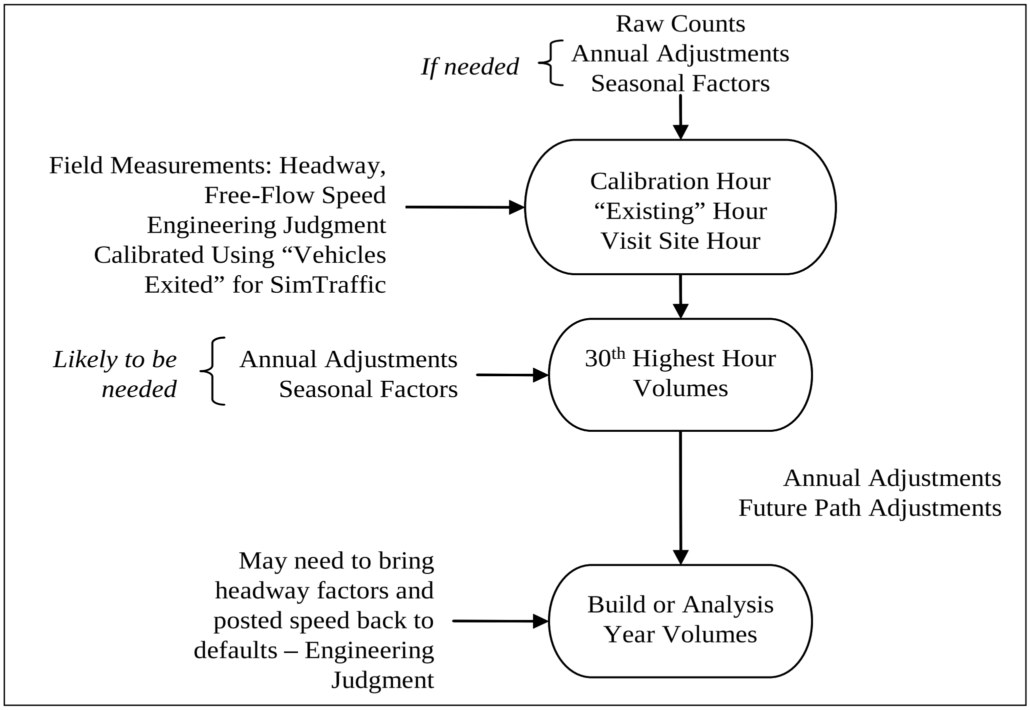
These rules are established to help ensure that calibration volumes 1) are near the 30th highest hour and 2) represent conditions that have been witnessed in the field. The emphasis is placed on witnessed, as the analyst needs to visit the study area on or near the count day (30th highest hour) so that the visual check of the simulation (the first step in calibration) is based on conditions that occurred in the field during the count. The Field Inventory Worksheet shows all the measures from the field that should be input into the simulation and visually checked in the animation to help analysts in the data collection process. Note that the worksheet is intended to be printed multiple times for a given project area. The collection of worksheets can be placed in a three-ring binder providing a hard writing surface. Each copy of the worksheet can be used for each intersection or area of interest in the study and all copies can be neatly organized in a single project binder (see Chapter 3).

The site visit should occur as close as possible to the 30th highest hour. After the site visit a calibration scenario can be constructed. For calibration, the peak hour volumes from the counts should be seasonally adjusted to the time of the site visit. The calibration network should include all measurements taken and all operational behavior witnessed. Many of the behavioral issues should be collected on the worksheet provided above. For Synchro and SimTraffic inputs refer to Chapters 12 and 13. Some of these sections refer specifically to Synchro/SimTraffic, but the list provided should include most of the measures that would have to be checked or adjusted in any software platform. Note that most microsimulations go into greater detail than SimTraffic, so there will likely be more measures to check and adjust. Also note that illegal behavior such as speeding, improperly using medians or shoulders as turn bays and improper lane changing distances should be accounted for in during calibration but should not be continued to be assumed in the future build scenarios. All non-calibration alternative analysis should assume that all drivers follow the rules of the road.

Once the “existing” inputs and behavior is coded into the simulation software, the analyst should run an animation to visually check the reasonability of the microsimulation. Any gross error like queues or blockages being much greater or much less than the field observations should be addressed by re-checking inputs. Further refinement may include measuring and adjusting saturation flow rates, driver reaction time and travel speed. A good place to start is by comparing simulated vehicle queues to those visually observed in the field. For some corridors, comparing simulated travel times or average speeds to actual observed conditions may be appropriate.

Good calibration is not only critical for accurate analysis but will establish credibility during presentations with technical advisory committees or public groups that have prior knowledge of existing problem areas. Exhibit 15-1 illustrates how the calibration process fits into the complete analysis. The calibration, existing and site visit hour refer to the same hour. In other words, the “calibration” data is collected in the study area in the “site visit” hour to represent “existing” conditions. For further information on calibration in general, consult the FHWA Analysis Toolbox. Section 15.3 has the detailed procedures on calibrating a SimTraffic model using SimTraffic for ODOT projects.

Exhibit 15-1 Simulation Construction and Application Flow Chart



15.3 SimTraffic

15.3.1 Overview

SimTraffic performs microsimulation and animation of vehicle traffic, modeling travel through signalized and unsignalized intersections and arterial networks, as well as freeway sections, with cars, trucks, pedestrians, and buses. SimTraffic includes the vehicle and driver performance characteristics developed by the Federal Highway Administration for use in traffic modeling. They were developed for CORSIM and Trafficware used them as they were published. Most of the input is entered through the Synchro program, but some parameters, such as the driver and vehicle characteristics, are modified through SimTraffic specifically.

SimTraffic can be used for all ODOT plans, projects and traffic impact studies. SimTraffic is primarily used by ODOT for the analysis of signal systems and vehicle queue estimation, especially in congested areas and locations where queue spillback may be a problem. For the estimation of signalized vehicle queues, SimTraffic is generally preferred in Regions 2 through 5 where v/c ratios exceed 0.70 and in Region 1 where v/c ratios exceed 0.90, but should always be used where v/c ratios exceed 0.90. SimTraffic should typically be used for the analysis of all coordinated signal systems. For isolated intersections, Synchro and SimTraffic should provide similar results. SimTraffic results will differ from Synchro most when the v/c ratio exceeds 0.90, when there are closely spaced intersections and other conditions that are not ideal. Overcapacity queues and metering conditions are identified in Synchro's Timing Window with a "#" or "m" symbol.

15.3.2 Simulation Calibration

As much as possible, operational field data should be obtained for the major facilities in the study area as close as possible to the design hour (see Appendix H). Beyond the field data listed in Chapter 3, additional field measures may be needed to achieve calibration of the microsimulation. If needed, saturation flow studies should be performed at the major intersections. Floating car travel time runs may need to be performed to ensure that observed and simulated travel times (and related speeds) are close. Free-flow link speeds using road-tube counters or speed guns (RADAR, LIDAR, etc.) may need to be collected and used in place of posted speed limits during calibration.

At the very least, the existing conditions network needs to be visually calibrated to the field conditions and the “vehicles exited” measure from SimTraffic should be reviewed. If everything is close, then the SimTraffic simulation should duplicate conditions seen in the field. Congested and free-flow areas in the field should be congested and free-flowing in the simulation.

If there is more congestion in the simulation than in the field, then one or more parameters may be off. For example, saturation flows and resulting headway factors may be too low, counts may be balanced too high, peak hour factors may be too low, link and turning speeds may be low, storage bays and taper lengths may be too short and intersection paths and lane change distances may be incorrect. If congestion is too low, then the reverse of these may be a cause.

To help determine the cause of inconsistencies with known conditions, any number of measures of effectiveness (MOE) may be reviewed, however as a minimum measure, “vehicles exited” needs to be checked to ensure that the model is calibrated.

“Vehicles Exited” represents the number of vehicles that make it through an intersection over a given period. This should equal the volume coded in the network for the “existing hour”. The calibration target for each intersection in the network is a tolerance of 1% over the analysis period based on the difference between the simulation and the input field-counted exiting (existing) hour volumes. However, at a minimum, the tolerances for any movement over 100 vph should be within 5% of the coded volume. Movements with less than 100 vph should be checked to make sure that the vehicles exiting is reasonable. These limits are required to achieve calibration for the calibration volume set (not required for the 30th highest hour or build year network). Exhibit 15-2 shows an excerpt from the Performance report showing the Vehicle Exited rates and calibration percentages. Note that all movements over 100 vph are under the 5% maximum tolerance and the entire intersection is under the 1% intersection tolerance.

Exhibit 15-2 Example Vehicles Exited from Performance Report

1001: Route 20 & Spring Hill Drive Performance by movement Entire Run

Movement	EBL	EBT	WBT	WBR	SBL	SBR	All
Total Stops	38	353	704	25	298	14	1432
Avg Speed (mph)	12	26	16	23	9	14	19
Vehicles Entered	39	1161	1198	476	317	19	3210
Vehicles Exited	39	1164	1200	476	320	19	3218
Hourly Exit Rate	39	1164	1200	476	320	19	3218
Input Volume	38	1169	1187	489	311	17	3211
% of Volume	103	100	101	97	103	112	100

Although calibration (fine-tuning) may take some time, it is necessary because if the existing conditions is not duplicating observed conditions, then the future conditions or build alternative performance will not be predicted very well. This is critical if any animated output is to be shown at public meetings. In achieving accurate calibration it is important that the SimTraffic parameter file is setup properly.

15.3.3 Simulation Preparation

In addition to setting up the SimTraffic parameter file, there are several Synchro settings that must be updated for simulations to work properly in SimTraffic. More signal timing detail must be added in the Phasing Window. These phasing details, settings and defaults are shown in Chapters 12 and 13. Project data needs to be entered into the Simulation Settings Window and the Detector Window. The Detector Window is covered under the Synchro sections in Chapter 13 because detector data is necessary if actuated signal functions are to be used in Synchro. The important Simulation Settings Window and the SimTraffic parameter data are included in Section 15.3.3 and 15.3.4, respectively. Earlier versions of SimTraffic only need to create the SimTraffic parameter file.

15.3.4 Simulation Settings Window

The following data is only used by SimTraffic and needs to be included for a proper simulation. This data allows for geometric refinement and operational behavior of the simulation. The data required by SimTraffic should be a part of the field collection/observation process and is included the Field Inventory Worksheet.

- **Storage Length (ft) –** The Storage Length is the length of a turning bay from the stop bar to the beginning of the taper. Storage Length is the area that can store vehicles and does not include tapers. If the Left or Right Turn lane goes all the way back to the previous intersection, enter "0". Storage Length data is used for analyzing potential blocking problems. Storage length is typically field measured or estimated from aerial photographs. If measurements are unknown or if the facility is new, the initial storage lengths of 100’ for urban and 150’ for rural can be used. SimTraffic outputs will be used to refine these lengths for build alternatives.
- **Taper Length (ft) –** The Taper Length is the remaining length of the turning bay from the end of the storage length to where the outer edge of the turning bay meets the outer

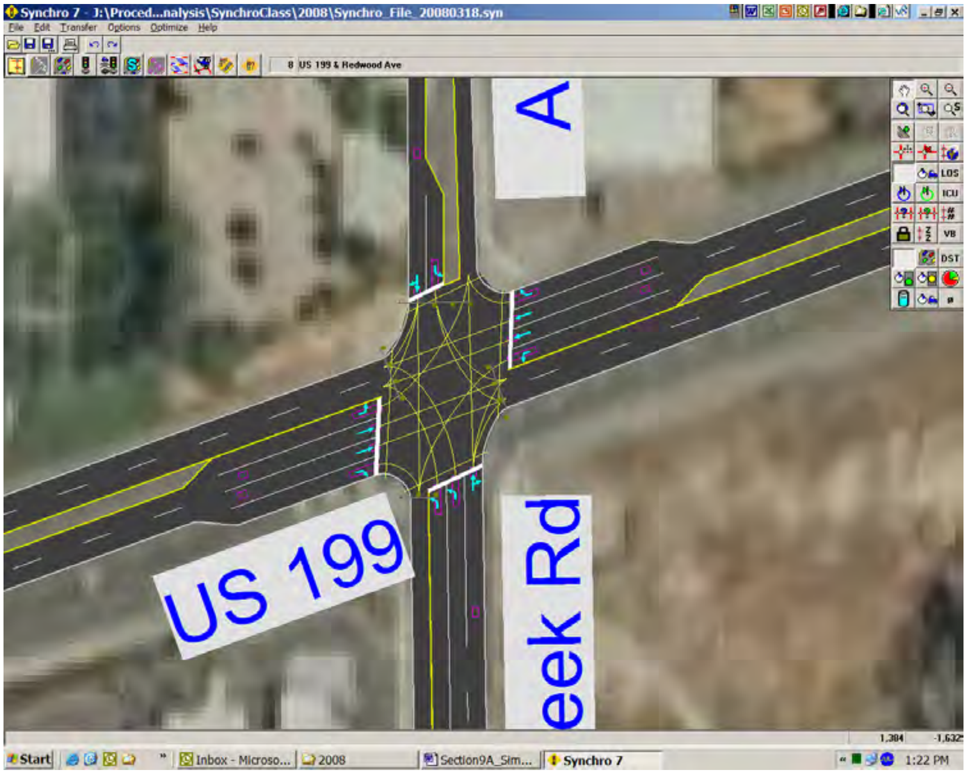
edge of the adjacent lane. This value is field-measured or estimated from aerial photographs. For state highways, the taper lengths can be obtained from the Highway Design Manual Figures 8-8 for right turn lanes and 8-9 for left turn lanes. This allows turning bays to store several more vehicles and allows a truer and a more consistent (with design) representation.

- **Lane Alignment –** The Lane alignment controls the vehicle paths in SimTraffic. When links are constructed, Synchro shows either a “Left” or “Right” alignment as default. This may not be correct especially if multilane approaches, skewed intersections, short links, free-flow ramp connections and merge/diverge/weaving sections make up a particular intersection.

  Other choices are “L-NA” and “R-NA” which will force the vehicle path either left or right. To check the lane alignment, the Intersection Paths box must be checked under the Map Settings window. The default color or zoom level will likely need to be changed to clearly see the paths.

  Exhibit 15-3 shows that Synchro defaults to single-lane turn lanes turning into a multilane leg with paths going to either departing lane. Unless lines are marked on the pavement guiding vehicles into different lanes Oregon vehicular code states that vehicles need to turn into the nearest lane. In most of these cases the Lane Alignment needs to be changed to “L-NA” or “R-NA” depending on the turn type.

Exhibit 15-3 Default Lane Alignment



For the existing calibrated network, the legal setting may not need to be followed if the majority of field-observed vehicles turn into both lanes (although itself an improper lane choice). Design alternatives should always be coded legally.

Note that the northbound dual left turn lane shown in Exhibit 15-3 has the correct paths. The southbound left still needs to be changed to limit traffic to the inside through lane. In cases of acceleration lanes, merging traffic should be forced right using “R-NA” and through traffic forced left using “L-NA.” This will keep through vehicles out of the acceleration lane.

- **Enter Blocked Intersection** – This setting controls whether mainline or side-street traffic can enter a blocked intersection. In earlier versions of SimTraffic, vehicles did not block intersections. Default is “No” for intersections and “Yes” for bend nodes and ramp junctions. This factor is best obtained through field observation.

  Along many busy roadways, minor intersections and driveways are frequently blocked by through traffic, so in this case the setting should be “Yes” for the through traffic. If “Do Not Block Intersection” signs exist, then the setting should remain “No” unless the signs are generally ignored. If there are intersections or accesses that are frequently blocked and through vehicles let side street vehicles out, then the side street movements can be set to “1 veh” which will allow one vehicle to enter. Use of the “2 veh” setting tends to cause the simulation to clog up.

- **Link Offset (ft) –** The Link Offset is used to set the roadway left or right of the natural centerline. This is typically used in creating “dogleg” or offset intersections without creating a second node.
- **Crosswalk Width (ft)** - this is the width of the crosswalk on an approach. This setting controls the placement of the stop bar which controls detector placement and link length. ODOT default crosswalk width is 12 feet (outside edge to outside edge) unless the adjoining sidewalk is wider. Local intersections should be measured.
- **Headway Factor** - The saturation flow rate in SimTraffic for intersection approaches is adjusted through the Headway Factor. The saturated flow rate calculated in Synchro is not used in SimTraffic; however, the corresponding headway factor is automatically calculated. In simulation calibration, the headway factor can be adjusted to help fine-tune (calibrate) the SimTraffic simulation. Exhibit 15-4 shows the equivalent headway factor for a given saturated flow rate. Earlier versions of Synchro/SimTraffic need to have the headway factor manually calculated in the Lane Window.

Exhibit 15-4 Headway Factors

Headway Factor	Saturated Flow Rate
1.2	1650 vphpl
1.1	1750 vphpl
1.0	1850 vphpl
0.9	2050 vphpl
0.8	2250 vphpl

- **Turning Speed (mph) –** This is the turning speed used by SimTraffic by movement. Higher speeds will increase the capacity of the SimTraffic simulation. Synchro default is 15 mph for left turns and 9 mph for right turns. The 9 mph right turn speed is too slow unless used for turning onto residential local streets or in a downtown central business district location.

  ODOT default is 15 mph for left and right turns. Non-standard turns at skewed intersections, channelized turns and interchanges should have different values and can be estimated by recording speeds while driving through the subject intersections or using a speed gun to capture turning vehicle speeds. Turning speeds are also needed for merge/diverge sections at interchanges or bend nodes.
- **Lane Change Distances** - Changes to these calculated values can help calibrate the vehicle lane-changing operation. Changes may be necessary if vehicles are having difficulty completing lane changes ahead of intersections or off-ramps or if vehicles are artificially clogging up at lane drops after an intersection or a two-lane ramp merging into a single lane. High heavy vehicle percentages combined with a higher number of long vehicles and/or a congested network increases the chances that modifications will be required. Closely spaced intersections will have short lane change distances while interchanges will have longer lane change distances as many drivers move into the desired lane considerably ahead of an off-ramp. The analyst will need to experiment with these values, either longer or shorter until the traffic is flowing consistent to the observed conditions or flowing smoothly for future conditions. Modifying ramp geometry so that

the ramps enter the mainline as turns rather than as a straight-through movement makes for smoother operation and less need to modify these distances.

There are two different types of lane change distances: mandatory and positioning. The Mandatory Distance is the distance measured from the stop bar at which a lane change must occur. The Positioning distance is the distance measured back from the Mandatory Distance where a vehicle first attempts a lane change. The Mandatory and Position Distance 2's are extra distance added if a second lane change is necessary. All these distances can extend around corners. Adding to the challenge of changing these variables, is that the driver types in SimTraffic have a range of a 50% (aggressive) to a 200% (passive) multiplier to the set distances.

15.3.5 SimTraffic Parameter File

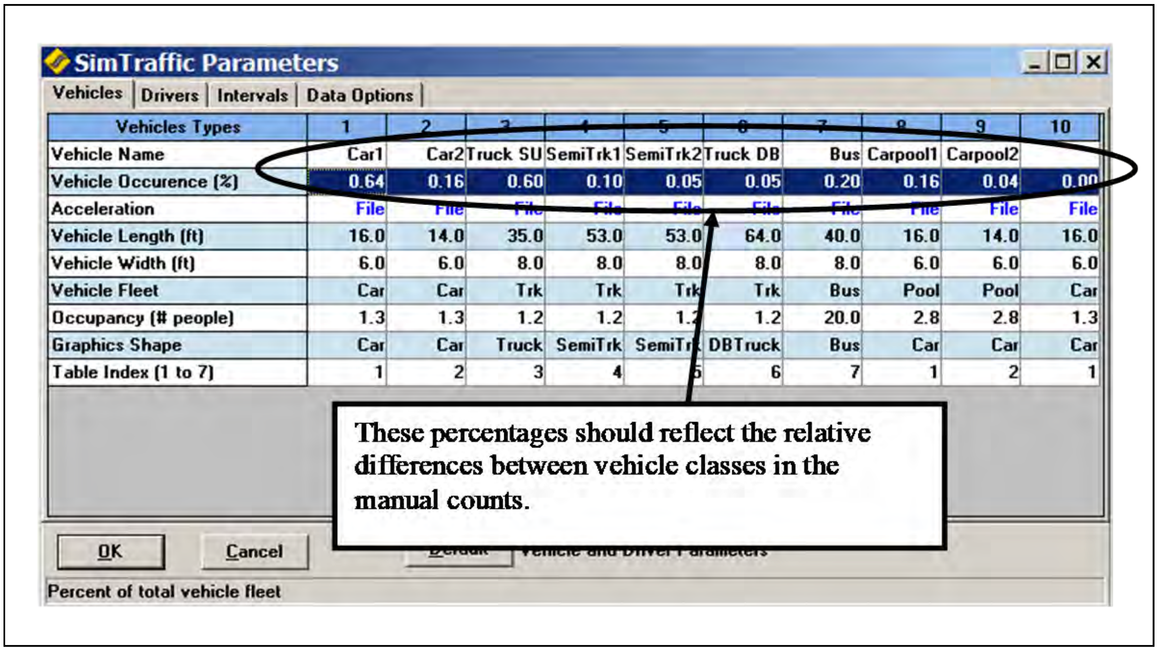
The SimTraffic parameter file controls the simulation operation and the defaults must be changed to reflect the proper impacts of queuing, travel time, etc. The parameter file has three major sections: Vehicles, Drivers and Intervals. The TPAU Analysis Tools webpage has a default SimTraffic template file with all the basic parameters set up. The following shows the variables that need to be changed. All other settings are left unchanged.

The Vehicles tab controls the type and physical vehicle characteristics.

- **Vehicle Occurrence (%) -** SimTraffic uses the Synchro heavy vehicle percentage to simulate the total number of heavy vehicles relative to all vehicles. When the simulation calls for a heavy vehicle, the vehicle type is represented by this factor which represents the percentage breakout of the global truck fleet. Likewise, when a car is called for, this factor will split the car types among the global car fleet percentages.
    - Earlier versions of SimTraffic defaulted to having the total vehicle percentages sum up to 100%.
    - SimTraffic 7 defaults total up to 100% for the car fleet and 100% for the truck (includes buses) fleet as shown in Exhibit 15-5.
    - Change the Vehicle Occurrence (%) for the different vehicle classes to match the composite average of your classification counts. If classification counts are unavailable, state highway vehicle classification segment data (available at https://highway.odot.state.or.us/cf/highwayreports/traffic_parms.cfm) can be used substituted. Average between multiple counts at the project boundaries and on different significant facilities both state and local. Note that while the heavy vehicle percentages per approach may vary largely, the heavy vehicle mix does not vary as much. The total truck fleet should total up to 100% and the total car fleet should total up to 100%.
        - Car1 represents the larger passenger vehicles in the fleet (i.e. SUV's, large pickups);
        - Car2 represents smaller passenger vehicles in the fleet;
        - TruckSU represents single unit trucks (i.e. delivery vans, dump trucks);
        - SemiTrk1 represents single tractor-trailer combinations;
        - SemiTrk2 represents shorter single tractor-trailer combinations;
        - Truck DB represents trucks with two trailers; Note: SemiTrk2 and Truck DB can be customized to fit other truck types like triple trailers.

- Bus represents buses in the fleet;
- Carpool1 & Carpool2 represents vehicles with the same characteristics as Car1 and 2 but with higher occupancies. Zero out the default Carpool1 and Carpool2 vehicles. These will have no effect on the simulation unless vehicle occupancy is used as an evaluation measure.

Exhibit 15-5 SimTraffic Default Vehicle Parameters

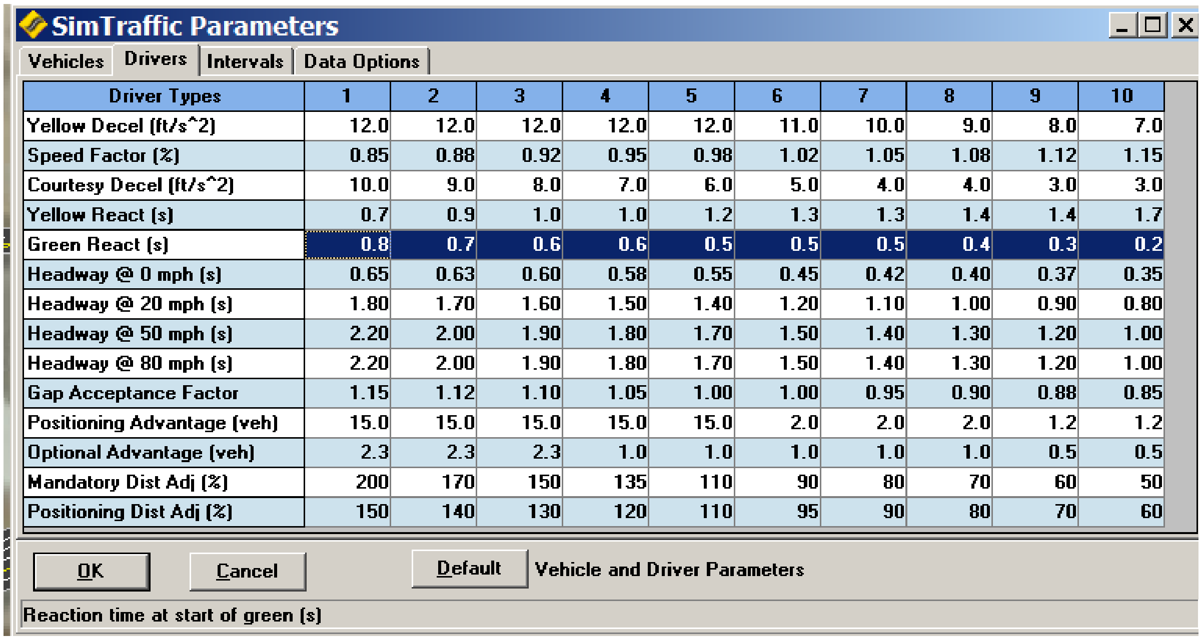


Vehicles Types	1	2	3	4	5	6	7	8	9	10
Vehicle Name	Car1	Car2	Truck SU	SemiTrk1	SemiTrk2	Truck DB	Bus	Carpool1	Carpool2	
Vehicle Occurence (%)	0.64	0.16	0.60	0.10	0.05	0.05	0.20	0.16	0.04	0.00
Acceleration	File	File	File	File	File	File	File	File	File	File
Vehicle Length (ft)	16.0	14.0	35.0	53.0	53.0	64.0	40.0	16.0	14.0	16.0
Vehicle Width (ft)	6.0	6.0	8.0	8.0	8.0	8.0	8.0	6.0	6.0	6.0
Vehicle Fleet	Car	Car	Trk	Trk	Trk	Trk	Bus	Pool	Pool	Car
Occupancy (# people)	1.3	1.3	1.2	1.2	1.2	1.2	20.0	2.8	2.8	1.3
Graphics Shape	Car	Car	Truck	SemiTrk	SemiTrk	DBTruck	Bus	Car	Car	Car
Table Index (1 to 7)	1	2	3	4	5	6	7	1	2	1

- **Vehicle Length (ft) –** This parameter directly affects queuing distances. Leaving the length unchanged will result in the queues being underestimated. Change the vehicle length in the following vehicle types:
    - Car1 = 20 ft;
    - Car2 = 16 ft;
    - TruckSU = 30 ft;
    - SemiTrk1 = 75 ft.

The Drivers tab (Exhibit 15-6) controls the behavior characteristics for the 10 different driver types that make up the simulation from the passive to the aggressive. For example, Driver Type 1 has 15% lower link speeds and will take 200% more distance when making a lane change while Driver Type 10 will travel 15% faster than the link speed and have lane change distances 50% of the coded values. All of the factors in the Drivers tab remain the same with exception of the Green React (s) setting. This setting reflects the time from when the signal turns green to the time that the vehicle begins to move. This value can be captured in the field and used as a calibration parameter. TPAU research indicates that Oregon values are substantially different than the defaults in SimTraffic. Change the Green React times to match Exhibit 15-7.

Exhibit 15-6 SimTraffic Default Driver Parameters



SimTraffic Parameters

Vehicles | Drivers | Intervals | Data Options

Driver Types	1	2	3	4	5	6	7	8	9	10
Yellow Decel (ft/s^2)	12.0	12.0	12.0	12.0	12.0	11.0	10.0	9.0	8.0	7.0
Speed Factor (%)	0.85	0.88	0.92	0.95	0.98	1.02	1.05	1.08	1.12	1.15
Courtesy Decel (ft/s^2)	10.0	9.0	8.0	7.0	6.0	5.0	4.0	4.0	3.0	3.0
Yellow React (s)	0.7	0.9	1.0	1.0	1.2	1.3	1.3	1.4	1.4	1.7
Green React (s)	0.8	0.7	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2
Headway @ 0 mph (s)	0.65	0.63	0.60	0.58	0.55	0.45	0.42	0.40	0.37	0.35
Headway @ 20 mph (s)	1.80	1.70	1.60	1.50	1.40	1.20	1.10	1.00	0.90	0.80
Headway @ 50 mph (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40	1.30	1.20	1.00
Headway @ 80 mph (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40	1.30	1.20	1.00
Gap Acceptance Factor	1.15	1.12	1.10	1.05	1.00	1.00	0.95	0.90	0.88	0.85
Positioning Advantage (veh)	15.0	15.0	15.0	15.0	15.0	2.0	2.0	2.0	1.2	1.2
Optional Advantage (veh)	2.3	2.3	2.3	1.0	1.0	1.0	1.0	1.0	0.5	0.5
Mandatory Dist Adj (%)	200	170	150	135	110	90	80	70	60	50
Positioning Dist Adj (%)	150	140	130	120	110	95	90	80	70	60

OK | Cancel | Default | Vehicle and Driver Parameters

Reaction time at start of green (s)

Exhibit 15-7 ODOT Green React Times

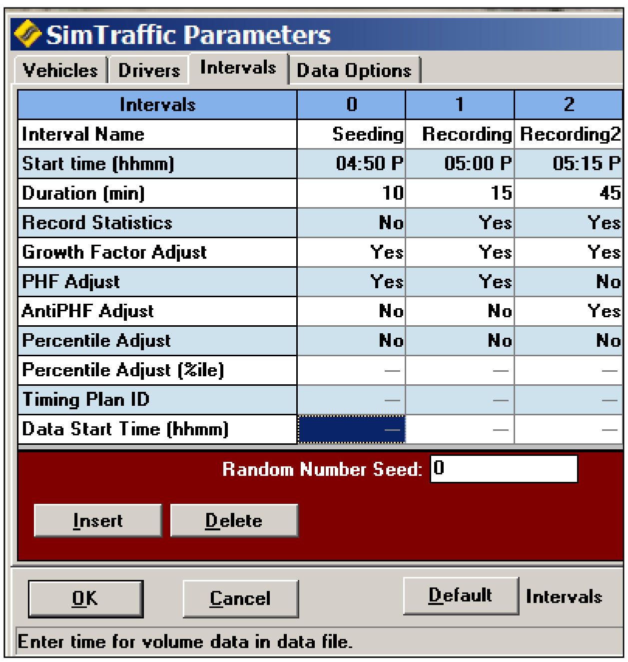
Driver Type	1	2	3	4	5	6	7	8	9	10
Green React (s)	2.0	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.5

The Intervals tab controls the actual operation and data recording of the simulation. Exhibit 15-8 shows the ODOT interval defaults.

- **Seeding "0" Interval** – The Seeding Interval fills the network before any statistics are recorded. This value must be long enough for vehicles to travel the length of the network. ODOT default is 10 minutes or the time to travel the longest trip on the network, whichever is longer.
- **Recording Intervals** – Simulation statistics are recorded in these intervals. The ODOT default uses at least two intervals, one 15-minute in length to represent the peak 15-minute period and one 45-minute interval to fill out the hour simulation period. However, you can have more intervals if you would like. For future analysis networks, the 15-minute interval is preferably placed as the first recording interval because it most represents the peaking in the output reports, regardless of where it occurs in the actual peak hour. However, for the calibration network, the 15-minute peak period should be coded to represent the actual peak 15-minute period as it occurred during the counts. The names of the recording intervals can be anything as they have no impact on the results.
- **Duration (min)** – Change to 10 minutes (time to cross the network if longer) for the seeding interval, 15 minutes for the first recording interval and 45 minutes for the second recording interval (or, if this is being applied to the calibration work, a distribution representing the peak as it occurred in the counts).

- **Start Time (hhmm)** – After Duration is specified, change the start time to reflect the hour being simulated.
- **Record Statistics** – Set to "Yes" for all recording intervals.
- **Growth Factor Adjust** – Set to "Yes" for all intervals.
- **PHF Adjust & AntiPHF Adjust** – The combination of these two settings creates a spike in the simulated hour. The PHF Adjust should be set to "Yes" during the seeding and the peak 15-minute intervals and the AntiPHF Adjust set to "No.". The AntiPHF Adjust should be set to "Yes" and the PHF Adjust set to "No" for all other recording intervals.
- **Percentile Adjust** - Set to "No" for all intervals. Use of this setting will overestimate the queuing in the simulation.
- **Random Number Seed** – SimTraffic uses nine different simulation scenarios (1 through 9). If it is desired to produce duplicate results, select a non-zero setting. ODOT default is to set it to '0' which will produce random arrival rates with each run.

Exhibit 15-8 ODOT Intervals Defaults



Intervals	0	1	2
Interval Name	Seeding	Recording	Recording2
Start time (hhmm)	04:50 P	05:00 P	05:15 P
Duration (min)	10	15	45
Record Statistics	No	Yes	Yes
Growth Factor Adjust	Yes	Yes	Yes
PHF Adjust	Yes	Yes	No
AntiPHF Adjust	No	No	Yes
Percentile Adjust	No	No	No
Percentile Adjust (%ile)	—	—	—
Timing Plan ID	—	—	—
Data Start Time (hhmm)	—	—	—

15.3.6 Simulation Execution

Once all Synchro and SimTraffic settings are completed, the simulation is ready to be executed. Upon starting the simulation, the "Errors and Warnings" window will appear. This shows anything that is outside of the value ranges what SimTraffic expects to find. Errors are split into fatal and non-fatal errors. Fatal errors will not allow the simulation to run and must be corrected. Fatal errors usually are related to lanes and lane groups where no lanes exist on a link.

Non-fatal errors still allow a simulation to be run, but these need to be reviewed and corrected if possible, for best results. Some examples of non-fatal errors that need to be corrected are:

- "Detector too close to stop bar";
- Minimum green /total split/pedestrian timing errors;
- Reference phase not in use errors;
- Storage lane and length errors.

Some examples of non-fatal errors that can be left alone as these are "how it is" are:

- "Angle between approaches less than 25 degrees." Small angles will lengthen out an intersection area and may cause unpredictable operation.
- Any error referencing vehicle extensions or minimum gaps exceeding 111% of travel time between detectors. Errors such as these indicate that actuated signal operation will be not as efficient.
- "Volume-delay operation not recommended with long detection zone." SimTraffic has issues generally with ODOT's default phasing variables.

ODOT standard is to average together at least five (5) random acceptably working (no system gridlock) runs. If you have a congested or a large network, it is advisable to have 7-10 runs to allow for "blown" runs which are caused by system gridlock so there are at least five good runs averaged together at the end. The system gridlock is typically caused by the improper actions of simulated vehicles that end up getting stuck. If every run or a majority of runs have gridlock, then the analyst should further refine the simulation settings, especially the headway factors, blocked intersection and lane change distance parameters.

It can take 20-40 minutes a run (depending on network size, congestion level and computer speed). Make sure you have adequate available storage. Each simulation file can be in excess of 1 GB. If you run out of space during a multiple recording session, SimTraffic will continue to run, but the simulations will stop being recorded.

Once the runs are completed, check each simulation run by selecting each number in the drop-down run number box to make sure it is free of any system gridlock errors and that the simulation reflects what is expected. If there are bad runs, make note of the run number, so it may be skipped in the report process.

15.3.7 Simulation Outputs

SimTraffic outputs are used for queue analysis, determination of storage lane lengths, travel times and other evaluation criteria. Many times in the evaluation of alternatives the typical v/c and LOS measures may have very small differences. It is common practice today to use additional MOE's to describe an operation of an alternative. These MOE's can include travel time, stopped delay, average speed and queue blocking. These are very useful in alternative comparisons because lower travel times, delays and stops coupled with higher average speeds, will indicate a more operationally efficient alternative.

Make sure before selecting any report to print or preview that a number is showing in the run number box. Otherwise, a message appears from SimTraffic saying that it needs to record the simulation again.

To preview a report, select the desired report(s) and make sure that the Multiple Runs box is checked. Select the desired .hst files (skipping any bad runs). Reports are generally broken down into sections by intersection and interval. The Summary of All Intervals section of the report is where information is pulled from for analysis. Check to make sure that content, headers and footers are correct before printing.

- **Simulation Summary Report –** Used to check whether that the runs look to have similar characteristics. Entering and exiting vehicles, total delay and total stops should be relatively consistent between runs. This is a second check of the run adequacy (the first is visual inspection). This report also gives system total MOE’s which can be used in alternative comparisons.
  Queuing and Blocking Report - The Queuing and Blocking report generates the 95th percentile queues which are used to design turn bay storage as well as document operation of the study area.

- Exhibit 15-9 shows a typical report.

  This report shows three different queues: maximum, average and 95th. The reported maximum queue is the highest queue calculated every two minutes. The average queue (50th percentile) is the average of the calculated two-minute queues. The 95th Queue is the 95th percentile of the reported maximum queue over the simulated period. With the Random Number Seed set to zero, the queues in this report will be different from those in another set of simulation runs. When reporting out the estimated queue lengths, round up to the next 25 feet.

Exhibit 15-9 Sample Queuing and Blocking Report

Queuing and Blocking Report
Baseline 2/7/2008

Intersection: 2: Redwood Ave & US 199, Interval #1

Movement	WB	WB	NE	NE
Directions Served	L	LT	R	R
Maximum Queue (ft)	49	99	139	132
Average Queue (ft)	11	18	43	48
95th Queue (ft)	61	91	142	167
Link Distance (ft)	592	592	382	382
Upstream Blk Time (%)				
Queuing Penalty (veh)				
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

Intersection: 2: Redwood Ave & US 199, Interval #2

Movement	WB	WB	NE	NE
Directions Served	L	LT	R	R
Maximum Queue (ft)	321	341	120	144
Average Queue (ft)	64	68	16	16
95th Queue (ft)	344	343	75	77
Link Distance (ft)	592	592	382	382
Upstream Blk Time (%)	3	5		
Queuing Penalty (veh)	25	49		
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

95th Percentile Queues

Intersection: 2: Redwood Ave & US 199, All Intervals

Movement	WB	WB	NE	NE
Directions Served	L	LT	R	R
Maximum Queue (ft)	341	374	155	157
Average Queue (ft)	51	56	22	24
95th Queue (ft)	299	300	96	106
Link Distance (ft)	592	592	382	382
Upstream Blk Time (%)	2	4		
Queuing Penalty (veh)	19	37		
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

The Upstream Block Time and the Storage Block Time are of particular interest in helping describe the overall impact of queuing. While the 95th percentile queues may show how long a queue is, the block time shows for how long of the simulated hour the queue will block intersections or storage bays.

Even if queue spillback into adjacent intersections is not occurring, storage bays may be overflowing, causing local problems such as the blockage of adjacent lanes. A queue blockage or spillback condition is considered a problem when the duration exceeds five (5) percent of the peak hour. Spillback may also be a sign of cycle failure as there may not have been enough green time available to serve all waiting vehicles. Signals do not recover instantly, so one spillback cycle could affect the operation of the next two or three cycles which can be a significant portion of hourly cycles.

- **Upstream Blk (Block) Time (%)** - This is an estimated percentage of the peak hour in which the queue from the subject node blocks an upstream node. This is especially useful when analyzing a complex Synchro network, to determine the extent of queuing on a system when reporting out results. It can also be used to determine if an alternative or option will provide the best progression.

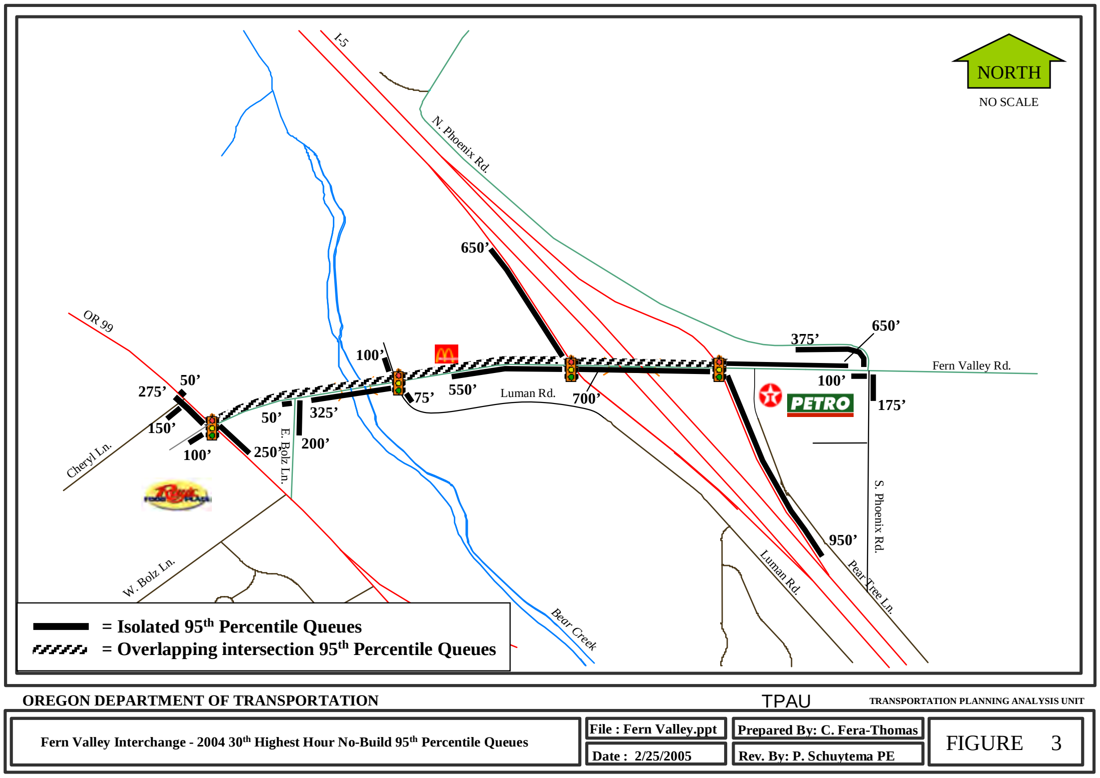
- **Storage Blk (Block) Time (%)** - This reports an estimated percentage of the peak hour in which the length of the through or turning queues exceeds the storage length. For build analysis, if your storage block time is significant (>5%), then it is recommended to enter a longer bay length, rerun the simulation, and continue this until you get a percentage less than 5%. Keep in mind that storage bays should adhere to the practical limit of 300 – 350 feet (most storage bays are 100 to 150 feet), so some alternatives and their simulations will still have significant storage block time.

Queues can be reported directly from the subject approach if the queue length is less than the link length. If a queue is longer than the link length, then the total actual queue length will be the link length(s) that are filled up plus the last queue length that does not exceed the link length. The analyst will need to trace the queue back from the intersection in question, so you will likely pass through multiple intersections and bend nodes to obtain the actual queue length. However, this queue is made up of contributions from other intersections that the subject queue spills back into which can make it hard to tell and report where exactly the queue originates. Queues are best reported graphically by identifying the queues under spillback conditions separately from the ones that do not exceed the link length.

Exhibit 15-10 shows a sample 95th percentile queuing diagram. To minimize reporting issues, link curvature should be used where possible to eliminate any unnecessary bend nodes.

The combination of the upstream and storage block times can also be used to report out the impacts of queuing at a higher level instead of reporting out the 95th percentile queues for intersection approaches.

Exhibit 15-10 Sample Queuing Diagram



- **Performance Report** - The Performance report (Exhibit 15-11) gives the MOE comparisons for each intersection by approach, movement or run; for each approach by run; or a total for the entire network. MOE’s are summed over the entire hour (i.e., hours of delay). During calibration, “vehicles exited” needs to be used to ensure calibration, see Section 15.3.1 for more instruction.

Exhibit 15-11 Sample Performance Report

SimTraffic Performance Report
Baseline 2/7/2008

2: Redwood Ave & US 199 Performance by movement Entire Run

Movement	WBL	WBT	NET	NER	All
Total Delay (hr)	2.8	2.6	0.0	1.6	7.0
Delay / Veh (s)	8.6	15.9	1.9	4.1	8.0
Total Stops	69	120	0	58	247
Travel Dist (mi)	147.4	72.6	0.4	112.2	332.5
Travel Time (hr)	6.2	4.3	0.0	4.3	14.8
Avg Speed (mph)	24	17	28	26	23
Fuel Used (gal)	6.4	3.3	0.0	3.8	13.5
HC Emissions (g)	259	121	0	155	534
CO Emissions (g)	8425	3399	11	4823	16658
NOx Emissions (g)	797	385	1	463	1645
Vehicles Entered	1176	597	9	1351	3133
Vehicles Exited	1177	598	9	1353	3137
Hourly Exit Rate	1177	598	9	1353	3137
Input Volume	1420	678	12	1475	3585
% of Volume	83	88	75	92	88
Denied Entry Before	0	0	0	0	0
Denied Entry After	0	0	0	0	0

- **Arterial Report –** The Arterial Report (Exhibit 15-12) is another version of the Performance report but reports out travel time, delay, and speed along a roadway section on a per vehicle basis. This roadway must have at least two nodes for this report to be available for it and the roadway must have the same road name without any special characters (i.e. dashes) along all the reported sections. The presence of a mixture of one-way and two-way sections along an arterial corridor may require segmenting and the individual results summed.

Exhibit 15-12 Sample Arterial report

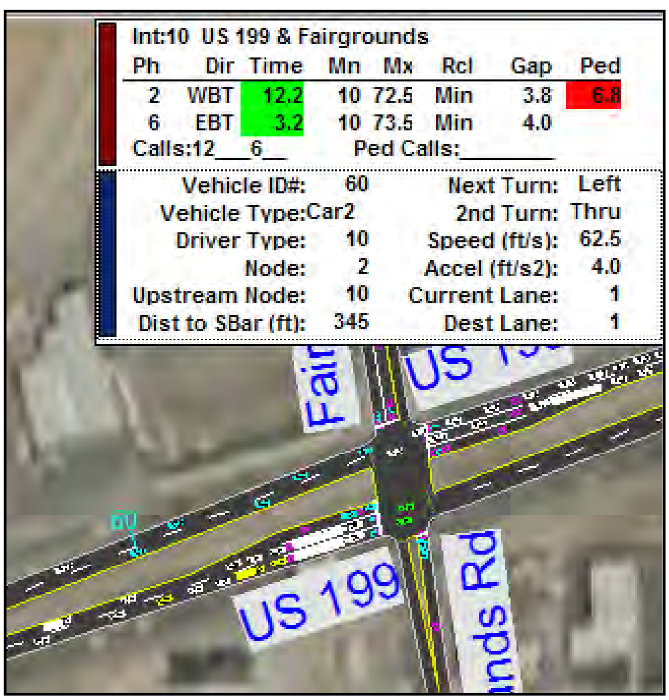
Arterial Level of Service: EB US 199, Entire Run

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed
Allen Creek Rd	5	18.5	26.2	0.1	12
Redwood Ave	8	8.6	22.5	0.2	27
US 199	2	4.4	11.0	0.1	27
Fairgrounds Rd	10	18.0	28.0	0.1	16
Total		49.5	87.8	0.5	19

Animated Tracking

In the SimTraffic simulation, clicking on a vehicle will bring up a box (Exhibit 15-13) showing speed, acceleration, distance to next turn, etc. this will allow the analyst to track vehicles as they travel through the network. Clicking on the vehicle again will remove the tracking box. In addition, signalized intersections can be clicked on showing the signalized operation in action as it goes through the phases. Both can be useful in debugging a simulation. It is recommended that the simulation speed be set to real time or slower for best viewing.

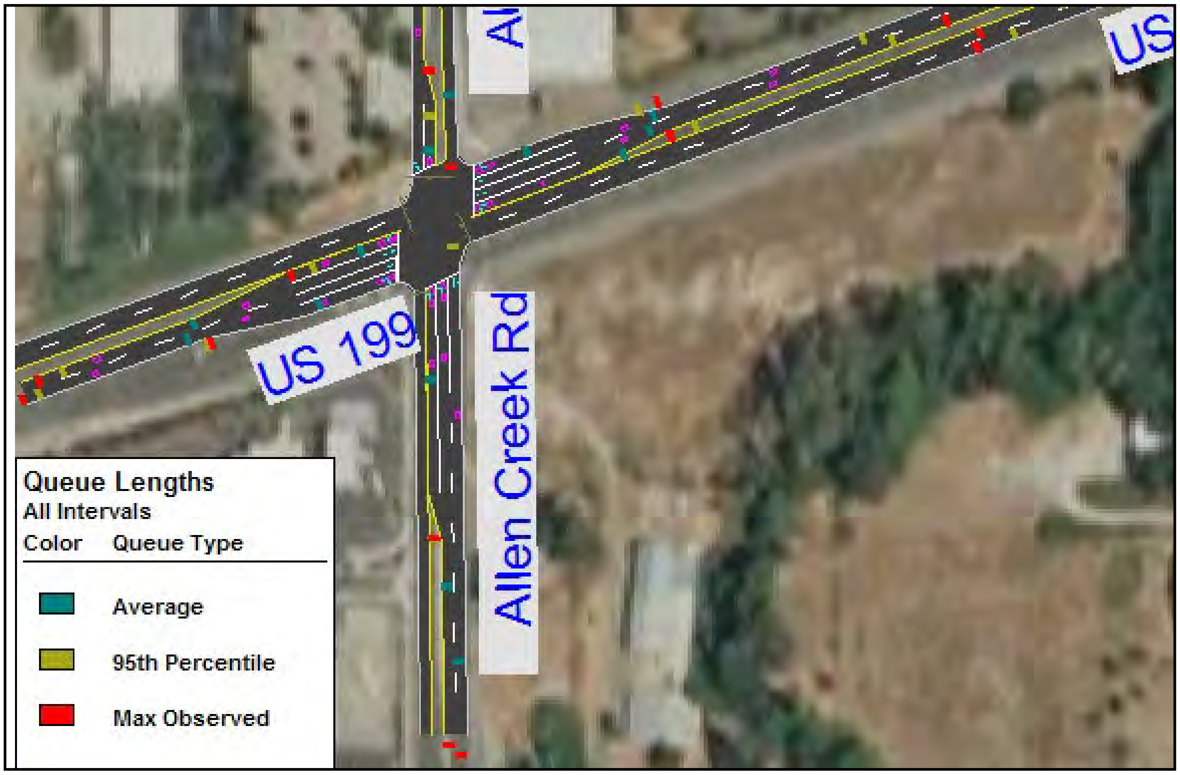
Exhibit 15-13 Animated Vehicle and Signal Tracking



Static Graphics

Other reports include the "Static Graphics" reports (Exhibit 15-14). Select the Graphics tab and you will get a box showing reports such as total delay, percent time blocked queues, etc. These reports are based on the same information that the previous comprehensive reports use, but display the information in graphical form, rather than a table of numbers. These report out just the run number selected rather than an average of runs of the regular reports. These can be used to quickly visualize the issues for the analyst or for others.

Exhibit 15-14 Example Queue Length Static Report



15.4 Vissim - Overview

Vissim is a simulation program that can model multi-modal traffic flows including cars, trucks, buses, heavy rail and light rail transit (LRT) as well as model traffic management systems (ramp meters, toll roads, and special lanes) and transit priority systems. Vissim can also model trip assignment, over fixed routes or dynamically, where vehicles change routes in response to specified events and can animate traffic movements in 3-D. Vissim is a program that can stand alone but is data intensive to create files for use on its own. Alternatively, the files can be created in Visum (a travel demand model assignment program) that can then import the files into Vistro or Vissim for analysis. Vistro can be used as a "front-end" to add in geometric lane configurations and traffic control details to an imported Visum file or as a start to an eventual Vissim file. See APM version 2 Appendix 8B for guidance on creating networks using PTV

Vision Suite software (Visum, Vissim, and Vistro). Because most ODOT region offices do not perform travel demand modeling, it is important to note issues both with and without Visum.

Other advantages of Vissim include the rail-roadway interface, which requires Vissim Level 3 or 4 in order to model the effect of rail crossing blockages on queues and roadway operations. Another advantage is that Vissim has the capability of "dynamic traffic assignment" (DTA), which will reroute a vehicle on the network in case of a crossing blockage or an overcapacity situation. Note that this strength of the software comes at the price of larger study areas to allow for correct dynamic assignment and to address effects occurring potentially outside of the focus of the study area. DTA will likely require more data, measures, and resources to properly calibrate (see APM version 2 Chapter 8 for more information).

APM version 2 Addendum 15A is a link to the ODOT Vissim Protocol which governs documentation and creation of all Vissim models created for ODOT plans and projects. Where applicable, the ODOT Synchro defaults should be implemented in the Vissim model to the extent possible.

Vissim has the capability of performing analysis directly on Visum traffic volume assignments and includes a post-processing function. The results of this type of analysis may be acceptable for certain applications, such as sketch planning and alternative screening. However, for most types of analysis, DHVs are required. The function in Vissim does not create DHVs, therefore the post-processing procedures outlined in APM version 2 Chapter 6 are still necessary.

Vissim has been used in some research projects to evaluate the potential effects of connected and autonomous vehicles (CAVs), including the pooled fund study *Planning-Level Capacities for CAVs in the Highway Capacity Manual*, led by ODOT with the participation of nine other state DOTs. The pooled fund study simulated CAV behavior by adapting a model developed by the California PATH program to test various intra-platoon gap settings. The research team developed a custom model to simulate cooperative adaptive cruise control (CACC) platooning, called the UC micro-sim tool. While Vissim 11 includes pre-defined driving behaviors to reflect automated vehicles, the research team found that a custom model was needed to simulate the complexities of actual CAV applications. Details about the modeling approach used in the pooled fund study are available in the Final Report for the pooled fund study, available on the ODOT Research website.

15.5 Paramics - Overview

Paramics and VISSIM share a lot of the same benefits in functionality and issues with complexity and time to achieve calibration. Paramics, like VISSIM, is a simulation program that can model multi-modal traffic flows including cars, trucks, buses, heavy rail and light rail transit (LRT) as well as model traffic management systems (ramp meters, toll roads and special lanes) and transit priority systems. Paramics can also model trip assignment, over fixed routes or dynamically, where vehicles change routes in response to specified events and can animate traffic movements in 3-D. Paramics is a program that can stand alone but is data intensive to create files for use on its own. Paramics does offer some importing functionality to bring networks in from other software, but it does not have a direct link to VISUM. However, all of Paramics' inputs are text files, making it easy to customize automations (macros, scripts, etc.) to take networks from other platforms and format the data into the text files Paramics requires. This creates many opportunities to bring networks from any software quickly into Paramics.

Arguably the biggest strength of any dynamic assignment software (like Paramics and VISSIM) is the "dynamic traffic assignment" (DTA) option, which will reroute a vehicle on the network in case of a rail crossing blockage or an overcapacity situation. Note that this strength of the software comes at the price of larger study areas to allow for correct dynamic assignment and to address effects occurring potentially outside of the focus of the study area. DTA will likely require more data, measures, and resources to properly calibrate.

Paramics has disadvantages like VISSIM since it does not produce signal coordination timing and can be very data intensive and time consuming to construct and calibrate a scenario, especially from scratch.

Some issues to consider when using Paramics for analysis are (based on ODOT's assessment of version 5.2):

- The flexibility of Paramics means the analyst is required, in most cases, to write their own reports. Paramics does have a set of limited standardized reports. This would require exporting the queuing data to Excel or comparable software; creating functions to calculate the maximum queues for each time period, calculating averages and standard deviations and then calculating the 95th percentile queue on each approach for each run.
- Paramics does not do signal coordination/progression, so the network must be constructed in Synchro (or similar software) to develop the timing and progression, which can then be incorporated into Paramics.
- Paramics should not be used in a TIS process as there are too many parameters to change and is likely out of the capable review range of most Region analysts.
- To date, none of the ODOT offices own the Paramics software. Paramics submittals by consultants should include the Paramics model translated into Synchro files to enable effective ODOT review. The ODOT Synchro defaults should be implemented in the Paramics model to the extent possible.

Currently, Paramics is not practical enough for most ODOT applications. Model development is data intensive, requires detailed knowledge on many input parameters and has limited standardized output reports. The use of Paramics input text file format can speed up some of the work but requires a custom import/export process which can be very time consuming to develop.

15.6 CORSIM - Overview

CORSIM is a microscopic traffic simulation program, applicable to surface streets, freeways, and integrated networks with a complete selection of control devices, i.e., stop/yield sign, traffic signals and ramp metering. CORSIM simulates traffic and traffic control systems using commonly accepted vehicle and driver behavior models and combines two traffic simulation models: NETSIM for surface streets and FRESIM for freeways. CORSIM allows for user control of trip assignment through the ability to set vehicle-type specific turn percentages and set predefined vehicles routes.

16 ENVIRONMENTAL TRAFFIC DATA

16.1 Purpose

Federal regulation requires, in some cases, that an air and noise study be completed to determine what impact, if any, will result from a proposed highway improvement and what measures will be taken to lessen these impacts. Certain projects may also require that greenhouse gas emissions (GHG) be quantified. This chapter presents the general outline for the needs and creation of common traffic data inputs requested for the Air Quality and Noise Analysis sections of the Environmental Impact Statement (EIS) or Environmental Assessment (EA) and for Categorical Exclusion (Class 2) projects as applicable. Traffic data needs for statewide GHG scoping, planning, and project-level analyses are also covered.

*The noise, air quality, and GHG methodologies in this chapter are only for the production of environmental **traffic data** needed to support calculation tool inputs. Other ODOT and consultant staff are responsible for using this environmental traffic data in noise, air quality and greenhouse gas tools and models. The actual noise, air quality and GHG analyses are outside the scope of this chapter and the APM.*

16.2 Induced & Latent Demand Project Considerations

An important consideration for projects that address capacity/congestion issues (e.g. adding travel lanes) is that assumptions regarding induced and latent demand need to be assessed (see Chapter 6). As part of the National Environmental Policy Act (NEPA) federal/state/local agency review process and the public hearing and comment periods, comments relating to the analysis and impacts of induced and latent demand are common.

Latent demand may cause the build alternative traffic to be higher than the no-build. In addition, if the project is on an edge of an urban/urbanizing area, travel demand model area, or is in an area with building land use and economic pressures (e.g. lack of affordable housing or available employment) the project alternative(s) need to be assessed for the potential of induced demand. Concerns such as these may facilitate development of bedroom communities that might add more demand to the system over time. Ideally, these kinds of expansions would also be accompanied with additional bike and transit networks and infrastructure to maximize mode share, but in cases where they are not increases the potential for latent and induced demand.

The results of the latent and induced demand assessment need to be clearly stated as part of the volume development for the alternatives. Depending on the roadway network and the extent of congestion, there may need to be separate forecasts created for the no-build and build. These separate project forecast assumptions need to be incorporated into the environmental traffic data creation process whether for noise, air quality or greenhouse gases.

Generally, a project should identify the number of new lane miles added including all auxiliary lane types (i.e. weaving, turning, etc.). Use Appendix 10A to isolate any sections of auxiliary lanes that are actually operating as through lanes. The analyst should also discuss with TPAU, ODOT Environmental Section, and their NEPA coordinator/staff any scoping needs or changes to the overall modeling approach to address potential risk from latent and induced demand.

16.3 Noise Analysis Traffic Data

ODOT is responsible for ensuring that state transportation projects are developed within the Federal Highway Administration's noise policies and procedures. To conduct the noise analysis necessary for measuring compliance, the ODOT Environmental and Hydraulic Engineering Section, or noise consultant, requires specific data from the project traffic analyst. This request is typically made through the Noise Traffic Data Request form in Appendix 16A which is filled out by the noise consultant or Environmental Section staff and delivered to the project traffic analyst. A traffic data request may be for a full or a screening-level noise study. Typically, the full study is requested, and its data requirements are discussed in the following sections. The screening level study uses a subset of the full data, so the same procedures apply. A calculation template workbook meant to streamline the noise traffic data production process in conjunction with the information in this section is available under the Volume Development section on the Technical Tools web page.

This traffic volume development process should typically only be done on the existing conditions, future no-build and final preferred alternative because of the time required to complete the work. Most times there is only one preferred alternative, but there have been cases of more than one final alternative requiring noise analysis. Any final alternatives need to be "frozen" and not be subject to changing designs, as that will incur excessive rework. Generally, it will take about three weeks for the existing conditions, two weeks for the future no-build, and two weeks for each build alternative for a medium to large-sized project.

The noise analysis needs to identify the times that have the highest impact, which would be the noisiest hour. This could be either the peak hour of all vehicles or the peak truck hour (total volumes could be much less but trucks typically make much more noise than other vehicles). This highest hour is also when the LOS C volumes occur, which is the maximum volume that can pass a point at the maximum (posted or 85th percentile) speed. Higher volumes will have slower speeds and faster speeds require low volumes and both cases will not have the highest noise levels. For example, at low volumes or densities, (LOS A or B) vehicles are independent and are not affected by others. Speeds will be higher, but volumes will be low thus the noise level will be relatively low. At high volumes or densities (LOS E – F), vehicles directly affect each other and resulting speeds are low which dampens the noise level. At LOS C (at the C/D threshold), vehicles may have to do some space adjusting but can still travel at the posted/85th percentile speed.

The noise analyst will obtain noise measurements in the field to establish the existing conditions. The traffic data supplied by the transportation analyst will be used by the noise analyst to calibrate the traffic levels to the existing noise levels. The noise measurements should be done under similar seasonal conditions (e.g. the PM peak hour used in a MPO area representing the 30th highest hour condition or use of a summer average weekday in a coastal community, etc.) as the traffic to minimize any seasonal pattern impacts that cannot be adjusted for (see Chapter 5). The difference in traffic volumes between the existing conditions and the future no-build will help establish the noise levels for the future no-build. The same goes for translating the future no-build to the future build. In a sense, this is analogous to post-processing with a travel demand model. The noise analysis will be summarized in a report which serves as the basis for recommending any noise mitigations such as soundwalls.

16.3.1 Input Data Needs

The traffic data requested will be the existing and the future design years for the no-build and the design year for the build alternative(s). The data will be provided in a directional segment or link basis. For each of the separate years, the following is required per link:

- Unique link identifier
- Link name
- Link length (mi.)
- Link type
- Posted speed (mph)
- 85th percentile speed (mph); if known
- LOS C volume (vph); Needed if any link volume for any analyzed year has a segment LOS D or higher, so it is highly advisable to include regardless to save time
- Peak hour and peak truck hour volume (vph)
- Percentages (decimal form) of vehicle groups:
    - Automobiles (FHWA Classes 1-3)
    - Medium Trucks (FHWA Classes 4-5)
    - Heavy Trucks (FHWA Classes 6-13)
    - Optional- Motorcycles (FHWA Class 1) and buses (FHWA Class 4)

Other information:

- Percentage of vehicles expected to stop on traffic signal approaches
- Existing and future zoning (or predicted/planned changes in land use from existing)
- Intersection turning movement diagrams (generally just for screening-level requests)

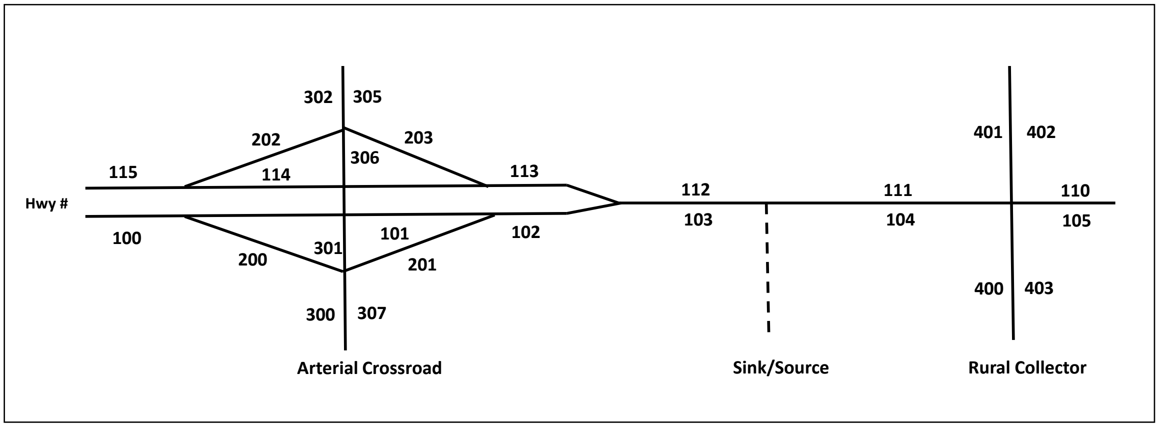
Links & Identifiers

The noise data process requires that the project area roadway network be broken into individual segments as everything will be recorded at the link level. This will be in the form of link diagrams depicting the study area roadway segments that will be included in the analysis for all considered no-build and build scenarios. These diagrams are not only useful for graphically relating the data provided to its location, but help in identifying links created, modified, or removed with each alternative and will facilitate the review and error-checking process. These can either be on paper or in an electronic format (i.e. Excel).

Each link must be given a unique number for identification purposes. Link numbers must be directional. Only roadways with collector and higher functional classifications should be included in a 500’ range from the subject facilities in the project area. Local streets and private driveways will have too low of speed and/or volume to make a significant noise impact and can be ignored. All count locations need to be represented as a node. Intersections with local streets and driveways should be noted as a sink/source location and should be a node (link break) if there was a count performed at that location. Creating a break at these locations will make the overall volume balancing process easier. Links representing the local street or driveway at this location should not be added, so the only intersection legs that would be represented would be on the mainline.

As much as possible, keep consistent numbering between the existing, no-build, and build link diagrams. This will mean adding extra links that have zero data into the existing and no-build future network that will accommodate the build alternative(s). It is best not to be completely sequential in the numbering by leaving gaps to accommodate extra links. This will avoid having out-of-order numbering if a new link is needed to be added later. For example, if the eastbound links were numbered 1 to 22, it would be better to start the westbound numbering at 30 instead of 23. The series numbers can also be changed to represent a different facility type for better clarity. For example, the mainline freeway could have the range of numbers being 1-50, the intersecting arterial could be in the 100’s and the ramps connecting the two could be in the 200’s. So in this case, Link 101 would be on the crossroad and Link 204 would be a ramp. The more consistent the diagrams are between all the scenarios, the easier it will be for the traffic analyst to create and troubleshoot and the noise analyst to follow. There is nothing wrong with having links with no data in the scenarios if they are labeled as not existing yet (i.e. “Future Link”) or not existing anymore (i.e. “Removed”). Exhibit 16-1 shows a sample project area with a wider highway that has access-control and medians transitioning to a two-lane highway section with at-grade intersections. Different numbering was used for the highway, ramp, arterial, and collector links. The link numbering was also non-consecutive to allow for the future alternative to keep numbering between scenarios consistent as possible.

Exhibit 16-1 Sample Link Diagram



Link Characteristics

Each directional link has a series of specific characteristics. This includes link name, length, type, and speed. Link names need to be descriptive with the street name, from-to, and direction. For example, this would be NB Main St (Elm St – Oak St) for a two-way roadway or like SB I5 to WB I105 Off-ramp for a one-way roadway. There should be no ambiguity on location between the description and the numerical identifier.

Link length is defined as the center-to-center intersection spacing. For state highway facilities (mainline, connections, or frontage roads), use of official roadway inventory data is preferred for existing and the future no-build conditions. Build alternative segment lengths should be determined from available single-line or detailed design drawings. Links on the edge of the network should have a default length of 0.25 mile.

Link type is a general identifier of the roadway environment. This is defined as either rural, urban street, or freeway. Study areas outside of an urban growth boundary should use the "rural" classification unless the facility is a freeway, then the "freeway" classification will override. Freeway ramps and related free-flow interchange connections should also use the "freeway" classification. Urban street interchange crossroads or other urban at-grade facilities should use the "urban street" classification.

Link speeds, at a minimum, are the posted speeds. Dual-posted highways with truck speed limits should weight-average the regular and the truck speed limit together using the heavy truck percentage. If the speed is weight-averaged, it should be rounded to the nearest whole number. If available for existing conditions, the 85th percentile operating speed should be included in addition to the posted speed limit. The 85th percentile operating speed is the highest overall speed at which a driver can travel on a given highway under favorable weather conditions, prevailing traffic conditions, and without at any time exceeding the safe speed as determined by the design speed. This can be estimated by using an analysis of average speeds or could be directly generated from probe speed data packages such as RITIS.

LOS C Volumes

In noise analysis, the LOS C volume is assumed to represent the maximum volume that can be sustained at free-flow speed resulting in the maximum noise condition. This is defined at the LOS C/D threshold as the maximum LOS C value. Because vehicle speeds typically affect noise levels more than vehicle volumes, this condition is often the most critical. In areas where peak period congestion is minimal or only occurs for a short time, allowing for continuously high speeds, the peak hour or peak truck hour may be critical. However, in areas where congestion is present for extended periods, lowering vehicle speeds, the LOS C volume may have a greater impact. It should be noted, however, that many links in a study area will not reach the LOS C level and some may exceed it.

LOS C volumes need to be determined on a link basis to establish a maximum volume threshold. If the peak hour or the peak truck hour volume exceeds the LOS C volume on a link, then the link volume is capped at the LOS C value. This way, the link volumes shown (and related vehicle percentages) will not exceed LOS C and will represent the maximum noise condition.

Capped link volumes will no longer balance across adjacent links. Links with capped volumes will need to reduce the number of vehicles such that the vehicle group (e.g. buses) percentages remain constant.

LOS C volumes need to be obtained from the latest version of the Highway Capacity manual for the respective facility type (e.g. freeway, multi-lane, two-lane highway, or urban street). Use the appropriate segment characteristics (number of lanes, widths, and other adjustment factors) to determine an initial value. This should have been done already as part of the project analysis so copies of the same software files can be used to start with. The LOS C volume is calculated using the HCM density-based methodologies, via an iterative process based on the project volumes (i.e. 2x, 1.5x, 0.75x, etc.), to identify the volume where the LOS C threshold occurs (at the top end of LOS C, adjacent to LOS D).

Roadway segments (e.g. merge section, freeway mainline) are relatively straightforward to iterate the initial LOS volume to reach the C/D threshold. Similarly, for signalized intersections, iterate until the overall intersection reaches the LOS C/D threshold. Roundabout and unsignalized intersection approach links for, the LOS C volumes will also need to be iterated using a “best approach” practice using the limiting leg (e.g. the leg with the lowest approach capacity) as it is a balancing activity between the mainline and side-street leg LOS. If mainline volumes were adjusted to LOS C/D, then the side street volumes would likely be LOS F and too high. Likewise, iterating side street volumes to LOS C/D probably will result in mainline volumes being too low. Trying to get the mainline and the side-street to LOS C/D at the same time results in a situation that would not occur unless large changes were made in the overall volume patterns which would result in inconsistences with other links not to mention with the volumes used in the rest of the project analysis. Iterate up or down based on the project volumes (i.e. 1.5x, 1.25x, 0.75x etc.) until the limiting leg reaches the LOS C/D threshold.

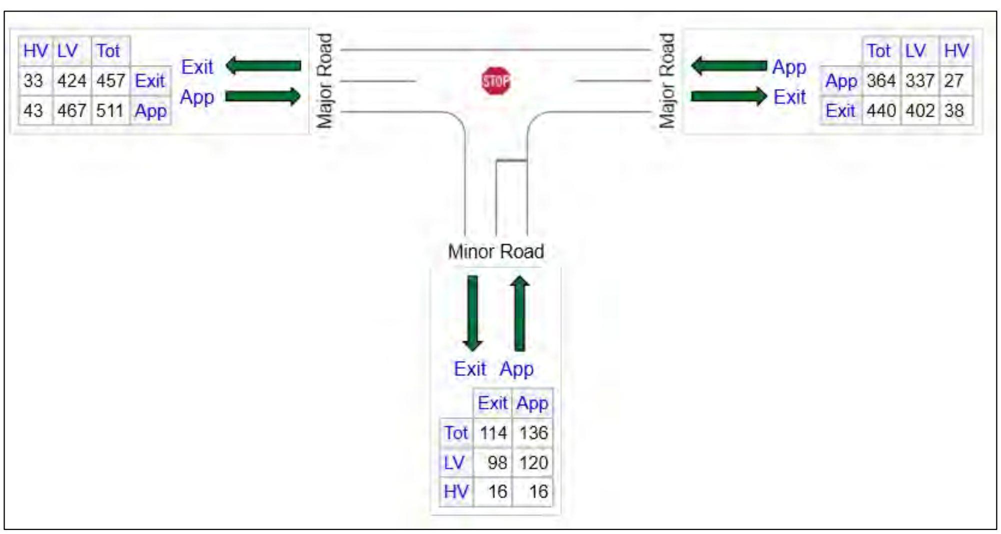
Report out all approach links at that point as LOS C volumes. Report out the departing link LOS C volumes if they end up being approach volumes for adjacent intersections. This will require that the LOS C volume from the departing link and the approach link to be averaged to create one LOS C volume per link. For documentation purposes, only the final software output with the final LOS C volumes is needed.

Use the following LOS C defaults only for links at the end of the network or for departing intersection links that are not also a calculable approach link at the next downstream intersection.

- For freeway free-flow ramp roadways and on-ramps, assume 1300 pcphpl. The analyst should consider effects of ramp metering on freeway ramp LOS C volumes, where applicable.
- For urban arterials, assume 600 pcphpl.
- For suburban arterials, assume 1000 pcphpl.
- For rural two-lane highways, assume 800 pcphpl.
- For rural multilane highways, assume 1200 pcphpl

Example 16-2 LOS C Calculations

As part of the noise analysis for a simple urban intersection upgrade project, LOS C volumes are needed for each exiting and entering leg of this two-way stop-controlled “T” intersection. Peak hour light and heavy vehicle volumes and other geometric/control details were entered into a SIDRA analysis (note that any intersection software could be used).



These inputs resulted in the below output. A review of the lane approach capacities indicates that the minor road approach is the limiting leg, so that is the controlling leg on the volume iteration to find the LOS C/D threshold. The starting LOS results show this approach to be LOS C, so initial volumes will need to be raised to have the LOS reach the C/D threshold.

Lane Use and Performance							
	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service
South: Minor Road							
Lane 1	136	12.0	$\boxed{335}$	0.405	100	22.9	LOS C
Approach	136	12.0		0.405		22.9	LOS C
East: Major Road							
Lane 1	364	7.5	1679	0.217	100	2.2	LOS A
Approach	364	7.5		0.217		2.2	NA
West: Major Road							
Lane 1	511	8.5	1691	0.302	100	0.1	LOS A
Approach	511	8.5		0.302		0.1	NA
Intersection	1011	8.6		0.405		3.9	NA

The volume iteration is started by assuming a volume multiplier (1.1x since the likely needed change is small) to apply to all entering and exiting volumes. After entering the new volumes the analysis reports that the minor leg is now at LOS D, so the 10% volume increase is a bit too high.

Lane Use and Performance							
	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service
South: Minor Road							
Lane 1	150	12.3	300	0.501	100	28.5	LOS D
Approach	150	12.3		0.501		28.5	LOS D

For the second iteration the volume multiplier was set at 1.05x which splits the difference between the starting volume and the first iteration. The analysis results still show LOS D, so again the volume multiplier would be cut in half to 1.025x for the third iteration.

Lane Use and Performance							
	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service
South: Minor Road							
Lane 1	142	11.5	318	0.448	100	25.2	LOS D
Approach	142	11.5		0.448		25.2	LOS D

The third iteration resulted in LOS C, so the 1.025x factor is a bit low, so the volume factor was raised to be halfway at 1.037x for the fourth iteration.

Lane Use and Performance							
	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service
South: Minor Road							
Lane 1	138	11.8	326	0.423	100	23.9	LOS C
Approach	138	11.8		0.423		23.9	LOS C

The fourth iteration also resulted in LOS C, so it is a bit closer, but a fifth iteration was done with the factor again splitting the difference at 1.043x.

Lane Use and Performance							
	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service
South: Minor Road							
Lane 1	140	11.6	323	0.434	100	24.4	LOS C
Approach	140	11.6		0.434		24.4	LOS C

The fifth iteration also resulted in LOS C with slightly lower capacities. One last iteration was performed with a volume factor of 1.047x.

Lane Use and Performance							
	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service
South: Minor Road							
Lane 1	141	11.5	321	0.440	100	24.7	LOS C
Approach	141	11.5		0.440		24.7	LOS C

The sixth and final iteration resulted in LOS C which was judged to be close enough to the threshold at 141 vph versus 142 vph from the 3rd iteration which resulted in LOS D. Typically five to seven iterations will be necessary to close in on a final value.

Lane Use and Performance							
	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service
South: Minor Road							
Lane 1	141	11.5	320	0.441	100	24.8	LOS C
Approach	141	11.5		0.441		24.8	LOS C

Once the LOS C/D threshold is determined to be reached then the approach volumes for the limiting and other legs are reported out as the LOS C maximum volumes. Departing leg volumes can also be used if they are also approach leg volumes for adjacent intersections.
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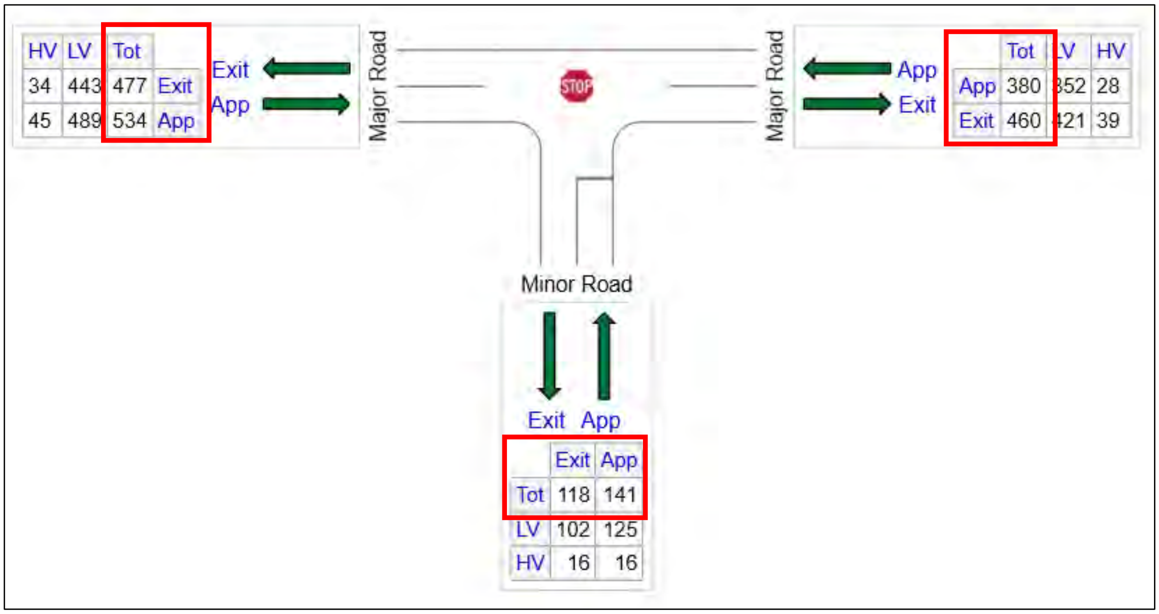


Peak Hour and Peak Truck Hour Volumes

Each link in the defined project area will need an assigned directional peak hour and peak truck hour volume. This will need to be done for the existing conditions, future no-build, and the future build (design) year scenarios.

Peak hour volumes should be based on the same counts, system peak hour, and factoring assumptions used to develop the project volumes following Chapter 5 and 6 methodologies. Any previously calculated peak hour project volumes for each of the three scenarios (existing, future no-build and future build) need to be converted from a turn movement basis to a link approach basis.

If it is known in the project scoping process that a noise and/or an air quality analysis will be eventually needed, then at least 16 or preferably 24-hour (FHWA 13 vehicle class) classification turn movement counts need to be ordered at all signalized intersections in addition to all intersections with substantial traffic volumes and/or heavy vehicle movements. These counts would also be used for the project volumes and analysis to maintain consistency with the later environmental analyses. Shorter duration counts (i.e. four hours) can still be used for peak hour volumes and be factored to daily volumes using factors from nearby long duration counts but will prove difficult for the calculation of the peak truck hour if they are not in the right period (typically morning). It is generally more efficient and less costly to have a single long duration count than multiple short duration counts.

A separate system peak hour is determined for the peak truck hour similarly as done for the peak hour (see Chapter 5) by noting when the highest amounts of trucks and buses occur (FHWA Classes 4- 13). The easiest way to do this is to add columns to a classification count export summing up the auto (FHWA Classes 1-3), medium truck (FHWA Classes 4 & 5) and heavy trucks (FHWA Classes 6-13) and the total trucks (medium + heavy). Identify the peak hour when the highest total trucks occur.

Exhibit 16-2 shows part of a 24-hour classification count with additional summary columns for the auto, medium truck, heavy truck, and total truck summaries. The total truck summary column allows for a quick identification of the truck peak hour when the highest total of medium and heavy trucks occurs. The highlighted hourly volume in the exhibit just happens to be the highest number of medium trucks in this count but that cannot be counted on to identify the truck peak hour by itself. In addition, while the identified truck peak hour in this count was from 12-1 PM, the actual truck peak hour could start on any 15-minute interval. This calculation is repeated for each count and then the resulting system peak truck hour is chosen from the count peak truck hours following the same process used in Chapter 5.

Exhibit 16-2 Peak Truck Hour Identification

	Multi Trailer Trucks (5 or less axles)	Multi Trailer Trucks (6 axles)	Multi Trailer Trucks (7 or more axles)	Auto	Med Trks	Hvy Trks		Class 4-13	
11:00	0	3	2	929	63	103		166	
11:15	0	0	0					0	
11:30	0	0	0					0	
11:45	0	0	0					0	
12:00	0	2	8	1055	**83**	101		**184**	
12:15	0	0	0					0	
12:30	0	0	0					0	
12:45	0	0	0					0	
13:00	0	1	4	1128	52	99		151	
13:15	0	0	0					0	
13:30	0	0	0					0	
13:45	0	0	0					0	
14:00	0	0	0	1110	60	102		162	

Truck peak hours typically occur in the mid-morning hours as this is when drivers are trying to minimize delays when going through an urban area or trying to time arrivals into or through a larger adjacent metropolitan area or are making deliveries within an urban area. The number of trucks and buses in the truck peak hour need to exceed the values in the peak hour. However, there likely will be links that will not exceed in the system peak truck hour versus the individual count truck peak hours because of the overall volume patterns in the study area. These exceptions are addressed in the calculation process. The peak truck hour volumes are created as part of the overall calculation process described later rather than independently/directly from the raw count values as done with the peak hour volumes. Note that the truck peak hour volumes are also “all vehicle” volumes which would be the sum of the auto, medium truck, and heavy truck columns in Exhibit 16-2 above. This should not be confused with the vehicle breakouts such as the “peak truck hour medium trucks” which would be the number of medium trucks in the truck peak hour.

Vehicle Classifications and Groups

The noise traffic data process will require percentages and related volumes for certain vehicle groups. All the 13 FHWA vehicle classes will need to be reallocated into these summary groups. This only needs to be done for the system peak hour and peak truck hours on each count to minimize the time and effort needed. Noise sources associated with transportation projects can include passenger vehicles, medium trucks, heavy trucks, and buses. Each of these vehicles produces noise, however, the source and magnitude of the noise can vary greatly depending on vehicle type. For example, while the noise from passenger vehicles occurs mainly from the tire-roadway interface and is, therefore, located at ground level, the noise from heavy trucks is produced by a combination of noise from tires, engine and exhaust resulting in a noise source that is approximately eight feet above the ground.

The 13 FHWA vehicle classes are shown below:

- ***Class 1: Motorcycles:*** All two- or three-wheeled motorized vehicles. Typical vehicles in this category have saddle type seats and are steered by handlebars rather than steering wheels. This category includes motorcycles, , mopeds, and three-wheel motorcycles. This category is not intended to include micro-mobility devices such as e-scooters and e-bikes.
- ***Class 2: Passenger Cars***: All sedans, coupes and station wagons manufactured primarily for the purpose of carrying passengers and including those passenger cars pulling recreational or other light trailers.
- ***Class 3: Other Two-Axle, Four-Tire Single Unit Vehicles***: All two-axle, four-tire vehicles, other than passenger cars. Included in this classification are pickups, sport utility vehicles, vans, and other vehicles such as campers, motor homes, ambulances, hearses, carry-alls and minibuses. Other two-axle, four-tire single-unit vehicles pulling recreational or other light trailers are included in this classification.
- ***Class 4: Buses***: All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires or three or more axles. This category includes only traditional buses (including school buses) functioning as passenger-carrying vehicles. Modified buses should be considered a truck and should be appropriately classified.
- ***Class 5: Two-Axle, Six-Tire, Single-Unit Trucks***: All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., with two axles and dual rear wheels.
- ***Class 6: Three-Axle Single-Unit Trucks***: All vehicles on a single frame including trucks camping and recreational vehicles, motor homes, etc., with three axles.
- ***Class 7: Four or More Axle Single-Unit Trucks***: All trucks on a single frame with four or more axles.
- ***Class 8: Four or Fewer Axle Single-Trailer Trucks***: All vehicles with four or fewer axles consisting of two units, one of which is a tractor or straight truck power unit.
- ***Class 9; Five-Axle Single-Trailer Trucks***: All five-axle vehicles consisting of

two units, one of which is a tractor or straight truck power unit.

- ***Class 10: Six or More Axle Single-Trailer Trucks***: All vehicles with six or more axles consisting of two units, one of which is a tractor or straight truck power unit.
- ***Class 11: Five or Fewer Axle Multi-Trailer Trucks***: All vehicles with five or fewer axles consisting of three or more units, one of which is a tractor or straight truck power unit.
- ***Class 12: Six-Axle Multi-Trailer Trucks***: All six-axle vehicles consisting of three or more units, one of which is a tractor or straight truck power unit.
- ***Class 13: Seven or More Axle Multi-Trailer Trucks***: All vehicles with seven or more axles consisting of three or more units, one of which is a tractor or straight truck power unit.

In reporting information on trucks the following criteria should be used:

- *Truck tractor units traveling without a trailer will be considered single-unit trucks.*
- *A truck tractor unit pulling other such units in a "saddle mount" configuration will be considered one single-unit truck and will be defined only by the axles on the pulling unit.*
- *Vehicles are defined by the number of axles in contact with the road. Therefore, "floating" axles are counted only when in the down position.*
- *The term "trailer" includes both semi- and full trailers.*

The summary vehicle groups are as follows:

- (Optional) Motorcycles (Class 1)
- Automobiles (Class 2 & 3): Passenger cars and other two-axle four-tire vehicles
- (Optional) Buses (Class 4)
- Medium trucks (Class 5): Two-axle six-tire trucks
- Heavy Trucks (Classes 6 – 13): Three-axle and greater single-unit trucks and all combination tractor-trailer trucks

The ODOT standard noise request combines the motorcycles into the automobile category and the buses into the medium truck categories, however all the categories may still be requested if there are substantial volumes of these vehicle classes. Separate motorcycle data is rarely needed in Oregon, but specific data related to bus volumes may be appropriate where the link could be experiencing higher than average bus traffic due to influence by a nearby school, bus barn, or tourist attraction. Noise requests from consultants may have the combined or all five categories listed. Generally, it is preferred to have a smaller number of classes to minimize the overall work, which can be very substantial for larger networks considering that the number of motorcycles and buses on a link basis tends to be small and which has limited impact on the results. Check with the project noise staff on whether all five or just the three main groups are needed before starting.

Other Information

For each approach link at a signalized intersection the percentage of vehicles expected to stop (arriving on the red phase) needs to be estimated. The proportion of vehicles arriving on red for uncoordinated/isolated signals is the inverse of the green time to cycle length (g/c ratio) calculated as:

$P_{red} = [1 - [(Total\ Split - yellow\ time - all\text{-}red\ time)/Cycle\ length)] \times 100$

If signals are coordinated or closely spaced then the proportion is based on the directional bandwidth available for the coordinated/progressed segment, which will extend across multiple links. Review the time-space diagram from (Synchro, SIDRA, etc.) and determine the arterial bandwidth for the desired direction. The total split, yellow and all-red times come from the upstream signal rather than the subject location as this overall calculation determines how much of the vehicle stream that leaves the upstream signal will stop at the subject location. Use the subject location if it is the first signal in the segment. In this case, the proportion of vehicles arriving on red is calculated for each approach link:

$P_{red} = [1 - [(Directional\ bandwidth / (Total\ split_{upstream} - yellow\ time_{upstream} - all\text{-}red\ time_{upstream})] \times 100$

Example 16-2: Proportion of Vehicles Expected to Stop Calculation

A highway signalized intersection has actuated (isolated) operation in the east-west direction and coordinated operation north-south. The cycle length is 100 seconds.

From the Synchro program file for the appropriate alternative and year containing the intersection, the timing window shows the needed total split, yellow and all-red times. The subject intersection times are used for the actuated isolated eastbound and westbound directions and the northbound coordinated time since it is first in this direction. The upstream intersection times are used for the southbound direction.

Subject Intersection

TIMING SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Total Split (s)	10.0	55.1	55.1	9.4	54.5	54.5	35.5	35.5	—	35.5	35.5	—
Yellow Time (s)	4.7	4.7	4.7	4.7	4.7	4.7	4.0	4.0	—	3.5	3.5	—
All-Red Time (s)	0.7	0.7	0.7	0.7	0.7	0.7	0.5	0.5	—	0.5	0.5	—

Upstream (Southbound) Intersection

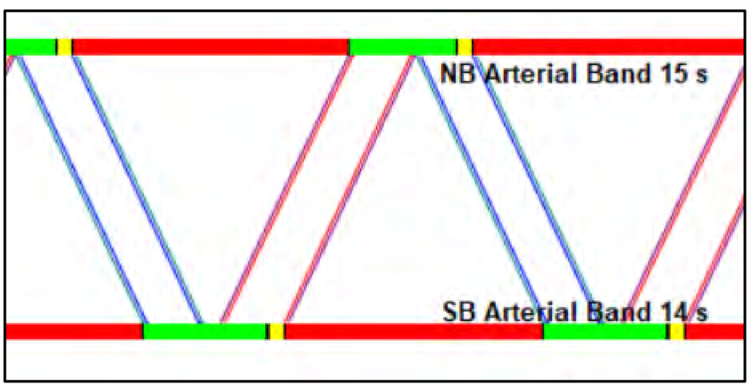
TIMING SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Total Split (s)	69.0	69.0	—	69.0	69.0	—	31.0	31.0	—	31.0	31.0	—
Yellow Time (s)	4.0	4.0	—	4.0	4.0	—	3.5	3.5	—	3.5	3.5	—
All-Red Time (s)	0.5	0.5	—	0.5	0.5	—	0.5	0.5	—	0.5	0.5	—

$P_{red} = [1 - [(\text{Total Split} - \text{yellow time} - \text{all-red time})/\text{Cycle length})] \times 100$

The total split, yellow and all-red times for the eastbound and westbound directions are entered into the equation above for isolated approaches:

$P_{red\text{-}EB} = [1 - [(55.1 - 4.7 - 0.7)/100] \times 100 =$ **50.3%**
$P_{red\text{-}WB} = [1 - [(54.5 - 4.7 - 0.7)/100] \times 100 =$ **50.9%**

For the northbound and southbound approaches, the coordinated equation is used:
$P_{red} = [1 - [(\text{Directional bandwidth} / (\text{Total split}_{upstream} - \text{yellow time}_{upstream} - \text{all-red time}_{upstream})] \times 100$
The time-space diagram from Synchro shows a 14 second bandwidth southbound and 15 seconds northbound. In this case there are only two signalized intersections north-south, so the link bandwidth is the same as the arterial bandwidth (normally this is not the case).



For the northbound direction, since the subject intersection is first, the timing from it is used in the equation:

$P_{red\text{-}NB} = [1 - [(15 / (35.5 - 4.0 - 0.5)] \times 100 =$ **51.6%**

The southbound direction uses the upstream intersection timing since it is first:

$P_{red\text{-}SB} = [1 - [(14 / (31.0 - 3.5 - 0.5)] \times 100 =$ **48.1%**

The overall zoning information is needed for the study area. This is not correlated with the roadway links. This is just a map showing existing zoning. A map showing future comprehensive plan zoning is needed if different from the existing.

Typically just for screening-level noise studies (check with the noise analyst to confirm if this will apply), a turning movement diagram is needed for each intersection in each of the three-year scenarios (existing year, future no-build, and future build alternative). This can be directly from an intersection analysis program, however if there are a larger number of intersections, a consolidated diagram showing multiple locations should be used to lessen the number of pages.

16.3.2 Calculations

The calculations necessary in the noise traffic data production are to generate the proportions of each of the summary vehicle groupings. These proportions will be used in the overall process to generate initial unbalanced volumes for each group, which are then balanced, and the proportions updated to create the final results. This will need to be done for the peak hour and the peak truck hour for all links.
For the peak hour, each of the vehicle groups are calculated by dividing the subject vehicle group volume by the total number of vehicles in the peak hour except for automobiles, which are simply subtracted:

$$Peak\ Hour\ Motorcycles\ Factor = \frac{Motorcycles\ in\ the\ peak\ hour}{Total\ peak\ hour\ vehicles}$$

$$Peak\ Hour\ Buses\ Factor = \frac{Buses\ in\ the\ peak\ hour}{Total\ peak\ hour\ vehicles}$$

$$Peak\ Hour\ Medium\ Trucks\ Factor = \frac{Medium\ trucks\ in\ the\ peak\ hour}{Total\ peak\ hour\ vehicles}$$

$$Peak\ Hour\ Heavy\ Trucks\ Factor = \frac{Heavy\ trucks\ in\ the\ peak\ hour}{Total\ peak\ hour\ vehicles}$$

$$Peak\ Hour\ Automobiles\ Factor = 1 - \sum Peak\ hour\ factors\ for\ all\ other\ groups$$

The peak truck hour is calculated from the relationship of the truck peak hour to the peak hour. This factor translates the peak hour volumes into peak truck hour volumes. Peak truck hour volumes are less than the peak hour volumes by definition, so the maximum value of the peak truck hour factor is 0.999. If any links have factors 1.000 or greater, they need to be capped at a maximum 0.999. Once the peak truck hour factor is created, then all the separate vehicle factors are based on it, which is different from the peak hour factors:

$$Peak\ Truck\ Hour\ Factor, \max 0.999 = \frac{Total\ truck\ peak\ hour\ vehicles}{Total\ peak\ hour\ vehicles}$$

$$Peak\ Truck\ Hour\ Motorcycles\ Factor = \frac{Motorcycles\ in\ the\ peak\ truck\ hour}{Total\ truck\ peak\ hour\ vehicles}$$

$$Peak\ Truck\ Hour\ Buses\ Factor = \frac{Buses\ in\ the\ peak\ truck\ hour}{Total\ truck\ peak\ hour\ vehicles}$$

$$Peak\ Truck\ Hour\ Medium\ Trucks\ Factor = \frac{Medium\ trucks\ in\ the\ peak\ truck\ hour}{Total\ peak\ truck\ hour\ vehicles}$$

$$Peak\ Truck\ Hour\ Heavy\ Trucks\ Factor = \frac{Heavy\ trucks\ in\ the\ peak\ truck\ hour}{Total\ truck\ peak\ hour\ vehicles}$$

$$Peak\ Truck\ Hour\ Automobiles\ Factor = 1 - \sum Peak\ truck\ hour\ factors\ for\ all\ other\ groups$$

16.3.3 Process – Existing Conditions

The following procedure is suggested. A spreadsheet should be created with rows for each link and columns for each of the link attributes for the existing conditions (A sample spreadsheet workbook with all of the sub-tables is available in Appendix A):

- Unique link identifier
- Link name
- Link length (mi.)
- Link type
- Posted speed (mph)
- 85^{th} percentile speed (mph); (optional)
- LOS C volume
- Peak hour volume
- Peak truck hour volume
- Automobiles peak hour %
- Medium truck peak hour %
- Heavy truck peak hour %
- Motorcycle peak hour % (optional)
- Bus peak hour % (optional)
- Peak truck hour factor
- Automobile peak truck hour %

- Medium truck peak truck hour %
- Heavy truck peak truck hour %
- Motorcycle peak truck hour % (optional)
- Bus peak truck hour % (optional)

Separate spreadsheets can be used for each of the three volume scenarios as using individual tabs and a single file will result in more than 50 or more which can be hard to follow and review if a good color scheme and formatting is not used. A tab containing the link diagram should also be added but it can be a separate document if desired. The link identifier through the 85th percentile speed columns are the same for all scenarios.

The overall process is as follows:

1. Add the unique link numbering scheme, link name/description, length, type and posted/85th speeds.

2. Calculate the LOS C for each link based on its facility type

3. Convert the peak hour existing volumes to a directional link basis
4. On a separate tab, create a linked table with the headers (See Exhibit 16-3):
   - Link identifier
   - Link name
   - Peak hour volume
   - Initial medium truck peak hour volume
   - Initial medium truck peak hour factor
   - Final medium truck peak hour volume
   - Final medium truck peak hour factor

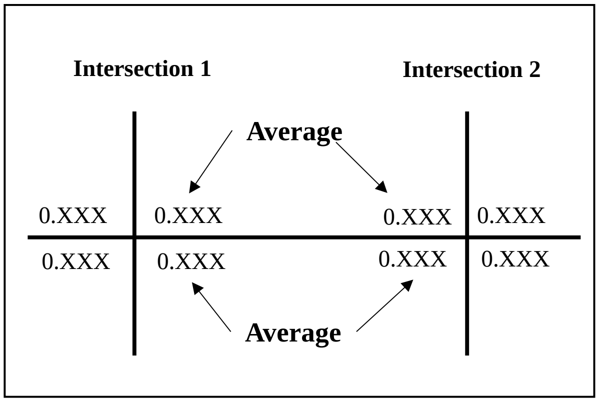
Exhibit 16-3 Example Medium Truck Factor & Volume Calculation Table

Link #	Link Name	Final 2017 (vph)	Initial Pk Hr Med Trk (vph)	Initial Pk Hr Med Trk Factor	Final Pk Hr Med Trk (vph)	Final Pk Hr Med Trk Factor
101	Beg Proj - 004CQ (SB)	1175	46	0.039	46	0.039
102	004CQ - 004CT (SB)	760	21	0.027	21	0.028
103	004CT - Rdwy 2 meets (SB)	820	26	0.031	26	0.032
104	Rdwy2 meets - N Uturn (SB)	820	26	0.031	26	0.032
105	N Uturn - Vandevert Rd (SB)	820	26	0.031	26	0.032
106	Vandevert Rd - Rdwy2 meets (SB)	695	20	0.028	20	0.029
107	Rdwy2 meets - S Uturn (SB)	695	20	0.028	20	0.029
108	S Uturn - 4 lanes (SB)	695	20	0.028	20	0.029
109	4 lanes - Sugarpine Butte (SB)	695	20	0.028	20	0.029
110	Sugarpine Butte - 2 lanes (SB)	695	20	0.028	20	0.029
111	2 lanes - USFS Boundary (SB)	695	20	0.028	20	0.029
112	USFS Boundary to end of project (SB)	695	20	0.028	20	0.029
201	Beg Proj - 004CV (NB)	895	62	0.069	62	0.069
202	004CV - 004CU (NB)	560	26	0.047	26	0.046
203	004CT - Rdwy 2 meets (NB)	610	28	0.046	28	0.046
204	Rdwy2 meets - N Uturn (NB)	610	28	0.046	28	0.046
205	N Uturn - Vandevert Rd (NB)	610	28	0.046	28	0.046
206	Vandevert Rd - Rdwy2 meets (NB)	585	27	0.046	27	0.046
207	Rdwy2 meets - S Uturn (NB)	585	27	0.046	27	0.046
208	S Uturn - 4 lanes (NB)	585	27	0.046	27	0.046
209	4 lanes - Sugarpine Butte (NB)	585	27	0.046	27	0.046
210	Sugarpine Butte - 2 lanes (NB)	585	27	0.046	27	0.046
211	2 lanes - USFS Boundary (NB)	585	27	0.046	27	0.046
212	USFS Boundary to end of project (NB)	585	27	0.046	27	0.046

5. The initial medium truck peak hour factor is computed by using the formula shown in Section 16.3.2 by dividing the total number of medium trucks in the peak hour by the total number of vehicles in the peak hour for a particular link from a single count. Values for factors should be to three decimal places. Repeat for all directional entering and leaving links at each count (i.e. intersection) location.

6. Two factor values will be generated on every link from Step 5, one leaving a previous intersection and one entering the next intersection unless it is an external link where there will be only one value. See Exhibit 16-4. Average these together so there is just one value per link and place in the medium truck factor table.

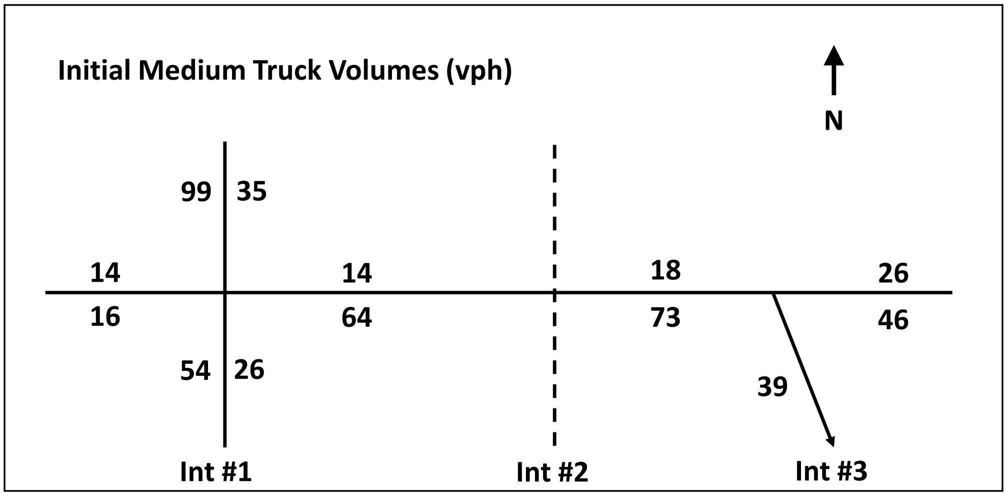
Exhibit 16-4 Averaging Entering & Leaving Factors



7. Multiply the initial medium truck peak hour factor by the peak hour volume to generate the initial medium truck peak hour volume. For example, from Exhibit 16-3, Link #101; the initial medium truck factor is 0.039 and the peak hour volume is 1175 vph. Multiplying these together gives an initial medium truck volume of 46 vph.

8. The initial medium truck peak hour volumes need to be balanced across all intersections. Balancing can be done via spreadsheet or by paper, but it is somewhat different than the balancing performed for project volumes as it is on a link basis rather than by turn movement. Balancing is done by summing the inbound and outbound volume and computing the difference. The difference in the in or out volumes is then spread around the intersection proportionately. It is generally best to split the difference between the ins and the outs to minimize the change at adjacent locations. Intersections with no minor legs (sink/source) included in the noise analysis scope only needs to reflect the drop or increase across the sink or source. This drop or increase is best reflected when it can be based on a count done at the location. These are also locations that can allow adjusting for differences where just an intersection balance cannot completely adjust for.

Example 16-3: Intersection Balancing



Medium truck peak hour volumes need to be balanced along a local east-west arterial shown above in the figure. This arterial intersects with another north-south arterial (Intersection #1), a local street (Intersection #2), and a ramp terminal (Intersection #3).

The intersecting roadways for Intersections #1 and #3 are also included in the noise analysis, but Intersection #2's are not. Intersection #2 had a count performed at this location, so this will be treated as a sink/source instead.

The first step will be to assess the differences between the in's and out's at Intersection #1:

Total In = 99 + 16 + 26 + 14 = 155 vph
Total Out = 35 + 14 + 54 + 64 = 167 vph
Difference = 167 – 155 = 12; so need to raise in's by 6 and lower out's by 6

The second step will be to determine the proportions of each in and out link as fractions of the total in and out volume checking to make sure that the proportions sum up to 1.000:

EB_{in} = 16 / 155 = 0.103 EB_{out} = 64 / 167 = 0.383
WB_{in} = 14 / 155 = 0.090 WB_{out} = 14 / 167 = 0.084
NB_{in} = 26 / 155 = 0.168 NB_{out} = 35 / 167 = 0.210
SB_{in} = 99 /155 = 0.639 SB_{out} = 54 / 167 = 0.323

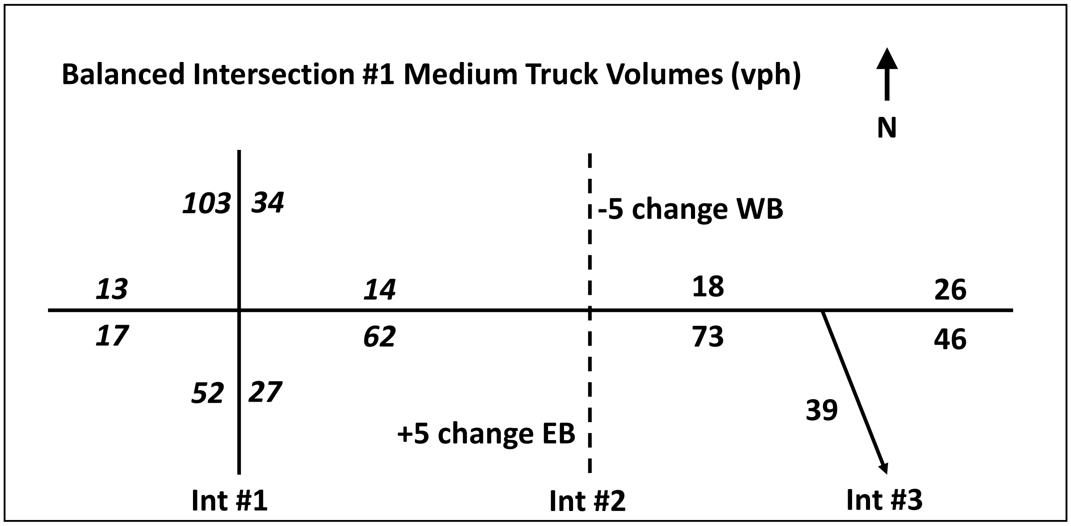
The third step will be to calculate the adjustments for the in's and out's, rounding the results to make sure the resulting total adds up to the total change for the in and out:

EB_{in} = 0.103 x 6 = 0.618 = 1 EB_{out} = 0.383 x 6 = 2.298 = 2
WB_{in} = 0.090 x 6 = 0.540 = 0 WB_{out} = 0.084 x 6 = 0.504 = 1

$NB_{in} = 0.168 \times 6 = 1.008 = 1$ $NB_{out} = 0.210 \times 6 = 1.260 = 1$
$SB_{in} = 0.639 \times 6 = 3.834 = 4$ $SB_{out} = 0.323 \times 6 = 1.938 = 2$

The last step will be to calculate the balanced Intersection #1 volumes for each in and out link which are shown as the italicized numbers in the figure below:

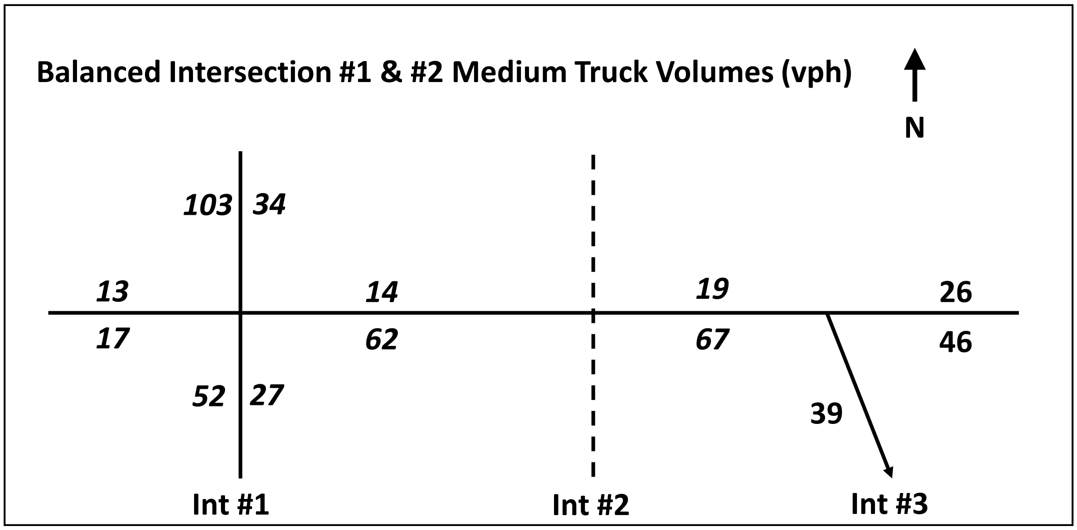
$EB_{in} = 16 + 1 = 17$ vph $EB_{out} = 64 – 2 = 62$ vph
$WB_{in} = 14 + 0 = 14$ vph $WB_{out} = 14 – 1 = 13$ vph
$NB_{in} = 26 + 1 = 27$ vph $NB_{out} = 35 – 1 = 34$ vph
$SB_{in} = 99 + 4 = 103$ vph $SB_{out} = 54 – 2 = 52$ vph



Since Intersection #1 is now balanced, it should be held constant when the adjustments for the Intersection #2 sink/source are done. The balancing at Intersection #1 increased the difference eastbound (i.e. from 9 to 11 vph) but did not affect the westbound difference of 4 vph. It is known from the count at Intersection #2, that the overall change across the intersection adds five vehicles eastbound and subtracts five vehicles westbound. In this case, add the 5 vph difference both to the EB_{out} and the WB_{in} of Intersection #1 to account for the change across Intersection #2:

Balanced $EB_{out} = 62 + 5 = 67$ vph Balanced $WB_{in} = 14 + 5 = 19$ vph

The resulting Intersection #2 italicized values are shown in the figure below.



The balancing for Intersection #3 is done the same as for Intersection #1, except that the EB_{in} and WB_{out} volumes are held constant.

Total In = 67 + 26 = 93 vph
Total Out = 46 + 19 + 39 = 104 vph
Difference = 104 – 93 = 11; so need to raise in's by 5 and lower out's by 6

Since there are only two inbound legs and one is held constant, all the inbound change is applied to the remaining inbound (WBin), so only outbound proportions are needed. Also, since the WB_{out} is held, its value needs to be subtracted from the total out to determine the split between the remaining outbound links.

Out Proportions:

EB_{out} = 46 / 85 = 0.541
SB_{out} = 39 / 85 = 0.459

In/Out Adjustments:

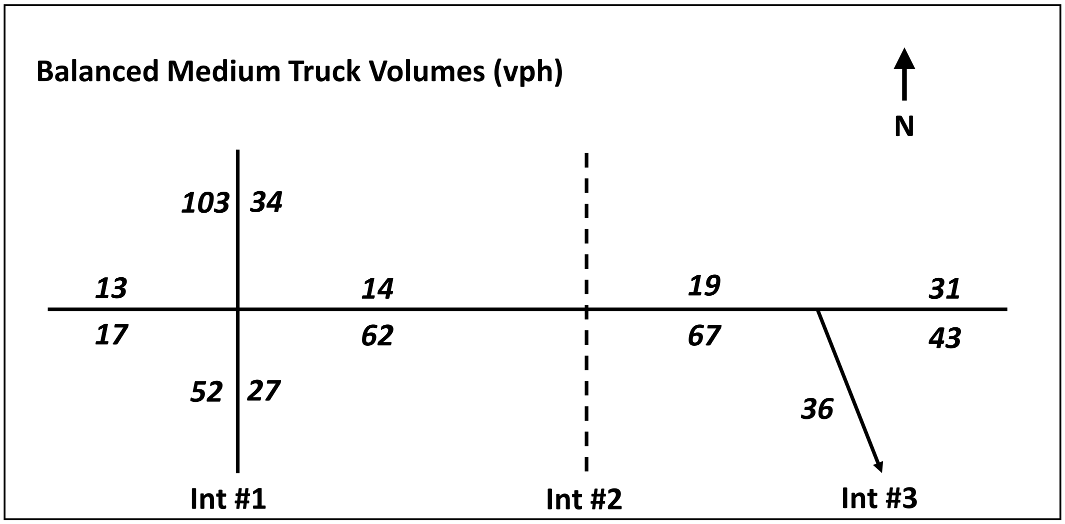
EB_{in} = No change
WB_{in} = 5

EB_{out} = 0.541 x 6 = 3.246 = 3
WB_{out} = No change
SB_{out} = 0.459 x 6 = 2.754 = 3

Resulting balanced volumes shown on figure below in italicized text:

EB_{in} = 67 vph (no change)
WB_{in} = 26 + 5 = 31 vph

EB_{out} = 46 – 3 = 43 vph
WB_{out} = 19 vph (no change)
SB_{out} = 39 – 3 = 36 vph



9. Once the medium peak truck volumes are balanced, enter the values as the final medium truck peak hour volume on the linked vehicle group spreadsheet. These are shown in the second column from the right in Exhibit 16-2 above.

10. Divide the final medium truck peak hour volume by the peak hour volume to obtain the final medium truck peak hour factor. This value should be linked back to the main spreadsheet tab. Following Link 101 in Exhibit 16-2; this would be dividing the final link medium truck volume of 46 vph by the link peak hour total traffic of 1175 vph to come up with the final medium truck peak hour truck factor of 0.039. In this case, the overall volume and factors did not change, as these links were held constant in the balancing process.

11. Repeat Steps 4 through 10 for the other peak hour volume groups: heavy trucks, motorcycles (optional), and buses (optional). Subtract the sum of the volume groups from 100% to obtain the automobile vehicle group percentages.

12. Like Step 4, create a linked table with the headers:
    - Link identifier
    - Link name
    - Peak hour volume
    - Initial peak truck hour volume
    - Initial peak truck hour factor
    - Final peak truck hour volume
    - Final peak truck hour factor

Exhibit 16-5 Example Peak Truck Hour Calculation Table

Link #	Link Name	Final 2011 30HV (vph)	Initial 2011 Pk Trk Hr (vph)	Initial Pk Trk Hr Factor	Final 2011 Pk Trk Hr (vph)	Final Pk Trk Hr Factor
1	OB Riley SB - N of OB Redmd	70	70	0.999	70	0.999
2	OB Riley SB - N of Cooley	80	80	0.999	80	0.999
3	OB Riley SB - S of Cooley	90	90	0.999	89	0.989
4	OB Riley SB - N of Empire	145	144	0.990	144	0.993
5	OB Riley SB - S of Empire	170	166	0.979	164	0.965
6	OB Riley NB - S of Empire	170	136	0.799	136	0.800
7	OB Riley NB - N of Empire	165	113	0.682	112	0.679
8	OB Riley NB - S of Cooley	65	61	0.938	63	0.969

13. From each of the counts, obtain the total link volume for the system peak truck hour chosen and enter the initial peak truck hour volume column (4th column from the left in Exhibit 16-5 above). Create a link to the previously calculated peak hour volumes and add those as shown in the 3rd column from left.

14. Create the initial peak hour factor by dividing the initial peak truck hour volumes from Step 13 by the peak hour volumes. If any values equal or exceed 1.000 then cap that link volume factor at 0.999 and readjust the initial peak truck hour volume to match. Note that in Exhibit 16-3 above this calculation would result in an initial factor of 1.000 after dividing the initial peak truck hour volume by the peak hour volume for Links #1 through #3. In this case the peak truck hour volume was the same as the peak hour volume. Links that reflect the actual peak truck hour will likely have volumes that exceed the peak hour volume which will result in overriding the calculation and adjusting volumes. The factor was adjusted from 1.000 to 0.999 but the volumes were small enough that the initial peak truck hour volume did not change.

15. Balance the peak truck hour volumes across the network in a similar fashion to Step 8.

16. Calculate the final peak truck hour factor to check that no values exceed 0.999. Modify the balanced volumes as needed until all factors are 0.999 or less. Note that in Exhibit 16-3 above, Link #1 and #2 needed the final peak truck hour factor calculation overridden back to 0.999, but that Link #3 had a slight balancing change which dropped the final factor to 0.989 which does not require adjusting.

17. Similar to Step 4 and Exhibit 16-2, create a linked table with the headers:
    - Link identifier
    - Link name
    - Peak truck hour volume
    - Initial medium truck peak truck hour factor

- Initial medium truck peak truck hour volume
- Final medium truck peak truck hour volume
- Final medium truck peak truck hour factor

This next set of calculations from this step through Step 20 repeats the medium and heavy truck factors (and any optional classifications) and volumes but this time for the truck peak hour.

18. Like what was done in Step 5, compute the initial medium truck peak truck hour factor by dividing the number of medium trucks in the truck peak hour by the peak truck hour volume computed in Step 16.

19. Repeat Steps 6 through 10 but for the medium trucks in the peak truck hour.

20. Repeat Steps 4 through 10 for the other peak truck hour volume groups: heavy trucks, motorcycles (optional), and buses (optional). Subtract the sum of the volume groups from 100% to obtain the automobile vehicle group percentages.

21. Check the final values for any errors by reviewing the number of buses (if used) and medium and heavy trucks in the peak truck hour to see if the values exceed the values in the peak hour. If not, the factors and volumes will need to be adjusted. Many times this may be caused by rounding errors. Typically, this will only be a few vehicle difference. With linked spreadsheets these changes are easily done and should reflect the final corrected values on the main tab.

22. Once all the vehicle group percentages are complete for both the peak hour and peak truck hour, compare the peak hour and the peak truck hour volumes with the LOS C volumes. Any volumes that exceed the LOS C volumes need to be capped at the LOS C volume. This means that any volume (that represents near, at, or over capacity conditions will likely require capping. Note that this capping process will result in volumes that will no longer balance with adjacent links, which is acceptable since the link network was completely balanced before this step.

The highlighted peak hour volumes shown in Exhibit 16-6 below are all higher than the corresponding LOS C volumes. These would all have to be reduced to be equal to the LOS C volumes as these represent the maximum volume that can be accommodated at LOS C. For example, for Link #27, the 2011 30^{th} highest hour volume of 1010 vph would need to be reduced to 1000 vph for the existing conditions and the 2036 DHV of 1560 vph would need also need to be reduced to 1000 vph for the upcoming similar steps for the future no-build.

Exhibit 16-6 Volume Capping

Link #	Link Name	2011 LOS C Volume (vph)	2016 LOS C Volume (vph)	2036 LOS C Volume (vph)	2011 30HV (vph)	2016 DHV (vph)	2036 DHV (vph)
25	3rd St (US20) NB - N of Empire	1200	1200	1200	535	590	855
26	US20 NB - One-way Conn	1000	1000	1000	540	595	870
27	US20 NB - One-way Conn	1000	1000	1000	1010	1115	1560
28	US20 NB - S of Robal	1043	1209	1249	1075	1190	1655
29	US20 NB - S of Cooley	1600	1600	830	935	1045	1475
30	US20 NB - S of Mountainview	1600	1600	1600	1035	1165	1625
31	US20 NB - S of OB Red	1600	1600	1600	1015	1145	1605
32	US20 NB - N of OB Red	1600	1600	1600	820	905	1195

23. Any capped peak hour or peak truck hour volumes need to have the vehicle group percentages modified to reflect the reduced volumes. Reduce the individual volumes and factors so that the total volume of the vehicle sub-groups equals the reduced total peak hour or peak truck hour volume and that the percentages still add up to 100%.

The highlighted columns in Exhibit 16-7 show the 2036 DHV that needs to be capped based on the LOS C volume column to the left and the affected medium and heavy truck peak hour volumes. The auto volumes are just the total link DHV minus the sum of the heavy and medium trucks. Exhibit 16-8 shows the result of the capped DHV and its effects on the medium and heavy peak hour truck volumes. Note that the sum of the auto + medium truck + heavy truck equals the capped DHV.

Exhibit 16-7 Volume Adjustments Before LOS C Capping

Link #	Link Name	2036 LOS C Volume (vph)	2036 DHV (vph)	Final Pk Hr Med Trk Factor	2036 Final Pk Hr Med Trk Volume (vph)	Final Pk Hr Hvy Trk Factor	2036 Final Pk Hr Hvy Trk Volume (vph)	2036 Final Pk Hr Auto Volume (vph)
27	US20 NB - One-way Conn	1000	1560	0.055	86	0.034	53	1421
28	US20 NB - S of Robal	1249	1655	0.056	92	0.033	55	1507
29	US20 NB - S of Cooley	830	1475	0.056	82	0.034	50	1342
30	US20 NB - S of Mountainview	1600	1625	0.054	88	0.050	82	1455
31	US20 NB - S of OB Red	1600	1605	0.052	84	0.054	87	1434
32	US20 NB - N of OB Red	1600	1195	0.049	58	0.068	82	1055
35	US97 SB - N of Bowery	3022	1705	0.048	83	0.054	92	1530
36	US97 SB - S of Bowery	3022	1715	0.048	83	0.054	92	1540

Exhibit 16-8 Volume Adjustments After LOS C Capping

Link #	Link Name	2036 LOS C Volume (vph)	Capped 2036 DHV (vph)	Final Pk Hr Med Trk Factor	2036 Final Pk Hr Med Trk Volume (vph)	Final Pk Hr Hvy Trk Factor	2036 Final Pk Hr Hvy Trk Volume (vph)	2036 Final Pk Hr Auto Volume (vph)
27	US20 NB - One-way Conn	1000	1000	0.055	55	0.034	34	911
28	US20 NB - S of Robal	1250	1250	0.056	70	0.033	42	1138
29	US20 NB - S of Cooley	830	830	0.056	46	0.034	28	755
30	US20 NB - S of Mountainview	1600	1600	0.054	87	0.050	80	1433
31	US20 NB - S of OB Red	1600	1600	0.052	84	0.054	87	1430
32	US20 NB - N of OB Red	1600	1195	0.049	58	0.068	82	1055
35	US97 SB - N of Bowery	3022	1705	0.048	83	0.054	92	1530
36	US97 SB - S of Bowery	3022	1715	0.048	83	0.054	92	1540

16.3.4 Process – Future No-build

The process to complete the noise traffic data for the future no-build is done in a similar manner to the existing conditions but relies on the data relationships within the existing conditions rather than going back to the actual raw counts. This is why it is important to completely finish and check the existing conditions before starting on the future no-build.

1. Use the same spreadsheet layout as the existing conditions, either in a separate workbook or set of separate tabs.

2. Link ID's, length, type and speeds are the same as the existing conditions

3. A new set of LOS C volumes is needed using the future no-build volumes.

4. Convert the future no-build volumes to a directional link basis.

5. Create the same linked tables for each of the vehicle groups for the peak hour and peak truck hour conditions except automobiles.

6. The initial factors for the peak hour vehicle groups start with the final factors from the existing conditions. These are used to develop the initial volumes for the future no-build which are then balanced by comparing the ins and outs at each intersection and used to modify the factors to reflect the future conditions. Sink/source locations should be consistent with the drop or increase of the existing conditions with the appropriate growth to the future.

7. The initial peak truck hour volumes are created by multiplying the directional link future no-build volumes by the final peak truck hour volume factor from the existing conditions. These volumes are then balanced across the network consistent with how it was done in earlier steps. Once balanced, create an updated peak truck hour factor by dividing the final peak truck hour volumes by the future

no-build peak hour volumes to make sure that no values exceed 0.999. If so, then readjust the factor and rebalance volumes until the 0.999 criteria is satisfied.

8. Create all the peak truck hour volumes and factors by using the existing year final factors as the initials as done in Step 6.

9. Check the bus, medium truck, and heavy truck peak truck hour volumes to see if they exceed the values in the peak hour. If not, then adjust factors and volumes until they do. Also, review all the volumes to make sure that the future no-build volumes exceed (or at least equal for no growth areas) the existing condition volumes.

10. Compare the future no-build peak hours and peak truck hour volumes with the future no-build LOS C volumes and cap off any that exceed. Any capped values need to have the vehicle group percentages/volumes adjusted to match the lower values.

16.3.5 Process – Future Build

The future build data is based off a pivot from the future no-build data. If the build alignment is the same (same link network) with differing number of lanes and/or traffic control, then the build data is done similar to the future no-build. Differences will be mainly in the LOS C volumes, so creation of the future build data will be very quick as the volumes and factors will be the same other than different (likely less) instances of where volumes are capped.

If the build design year volume is different from the no-build because of latent demand issues stemming from pent-up congestion, then the hourly volumes will change, and the process will be same as doing the future no-build. In this case, use the final factors from the no-build future to create the initial future build factors and follow the future no-build process.

The challenge with the future build data is when the build alignment or network layout is different from the no-build alignment as the relationships between links is muddied. The peak hour volumes would have already been re-distributed onto the build network for the project analysis. The analyst will need to figure out separately the routing of the vehicle groups (i.e. heavy trucks) if they do not follow the same patterns as the peak hour.

The process will be generally the same as for the no-build future starting with the no-build volumes factors to create initial volumes to be balanced. The balancing may be more substantial with larger changes in the factors than what was done with the future no-build. LOS C volumes will need to be re-done for the build volumes. The checks such as the 0.999 peak truck hour factor and “Do the peak hour trucks exceed the peak hour trucks” are very important when checking the build conditions.

16.3.6 Final Product Submittal

When all the data for the applicable analysis scenarios has been entered for each link, all errors have been fixed, the data is ready to be submitted to the contractor noise staff and the Environmental Section. Copies of the work should always be submitted to the ODOT noise staff in the Environmental Section for their records as they may be directly reviewing the noise outputs or at least the recommendations from the noise analysis. The entire noise workbook can be sent for documentation or just a copy of the values in the first (front) tab. Exhibit 16-9 shows part of the final link data table (this table is 27 columns wide). Make sure that the corresponding link diagrams are also included on a tab or in a separate document.

Exhibit 16-9: (Partial Table) Example Link Data

Link #	Link Name	BMP	EMP	Posted Speed (mph)	Link Length (mi)	2017 DHV (vph)	No Build 2041 DHV (vph)	Build V3 2041 DHV (vph)	2017 NoBuild Peak Hr Auto (vph)	2017 NoBuild Peak Hr Med (vph)	2017 NoBuild Peak Hr Hvy (vph)	2017 NoBuild Peak Truck Hr Auto (vph)
101	Beg Proj - 004CQ (SB)	152.57	152.82	65	0.25	1175	1850	1850	1090	46	39	469
102	004CQ - 004CT (SB)	152.82	153.40	65	0.58	760	1080	1080	706	21	33	467
103	004CT - Rdwy 2 meets (SB)	153.40	153.67	65	0.27	820	1165	1165	760	26	34	506
104	Rdwy2 meets - N Uturn (SB)	153.67	154.75	65	1.08	820	1165	1165	760	26	34	506
105	N Uturn - Vandevert Rd (SB)	154.75	155.50	65	0.75	820	1165	1205	760	26	34	508
106	Vandevert Rd - Rdwy2 meets (SB)	155.50	155.89	65	0.39	695	960	1040	640	20	35	446
107	Rdwy2 meets - S Uturn (SB)	155.89	156.17	65	0.28	695	960	1040	640	20	35	446
108	S Uturn - 4 lanes (SB)	156.17	157.73	65	1.56	695	960	960	640	20	35	446
109	4 lanes - Sugarpine Butte (SB)	157.73	158.57	65	0.84	695	960	960	640	20	35	446
110	Sugarpine Butte - 2 lanes (SB)	158.57	158.78	65	0.21	695	960	960	640	20	35	446
111	2 lanes - USFS Boundary (SB)	158.78	159.11	65	0.33	695	960	960	640	20	35	446
112	USFS Boundary to end of project (SB)	159.11	159.61	65	0.50	695	960	960	640	20	35	446
201	Beg Proj - 004CV (NB)	152.57	152.82	65	0.25	895	1390	1390	783	62	50	819
202	004CV - 004CU (NB)	152.82	153.33	65	0.51	560	760	760	489	26	45	525
203	004CT - Rdwy 2 meets (NB)	153.33	153.67	65	0.34	610	840	840	537	28	45	549
204	Rdwy2 meets - N Uturn (NB)	153.67	154.75	65	1.08	610	840	840	537	28	45	549
205	N Uturn - Vandevert Rd (NB)	154.75	155.50	65	0.75	610	840	880	537	28	45	549
206	Vandevert Rd - Rdwy2 meets (NB)	155.50	155.89	65	0.39	585	800	880	514	27	44	486
207	Rdwy2 meets - S Uturn (NB)	155.89	156.17	65	0.28	585	800	880	514	27	44	486
208	S Uturn - 4 lanes (NB)	156.17	157.73	65	1.56	585	800	800	514	27	44	486
209	4 lanes - Sugarpine Butte (NB)	157.73	158.57	65	0.84	585	800	800	514	27	44	486
210	Sugarpine Butte - 2 lanes (NB)	158.57	158.78	65	0.21	585	800	800	514	27	44	486
211	2 lanes - USFS Boundary (NB)	158.78	159.11	65	0.33	585	800	800	514	27	44	486
212	USFS Boundary to end of project (NB)	159.11	159.61	65	0.50	585	800	800	514	27	44	486

It generally is more efficient for project flow if the traffic data scenarios are sent off as they are completed so the noise staff can complete their calibration work rather than waiting until everything is completed. Many times the existing conditions and future no-build scenarios can be done relatively early in the project analysis, while the future build needs to wait until the preferred alternative(s) is chosen and all roadway design modifications are frozen a.k.a "pens down."

16.4 Air Quality Traffic Data[1]

Like noise analysis, ODOT is responsible for ensuring that state transportation projects are developed within the Federal Highway Administration's air quality policies and procedures. To conduct the air quality analysis necessary for measuring compliance, the ODOT Environmental Section, or air quality consultant, requires specific data from the project traffic analyst. This request is typically made through the Air Quality Traffic Data Request, which is filled out by the air quality consultant or the assigned ODOT air quality specialist and delivered to the project traffic analyst.

The types of air quality analyses required depend on the project location, type, pollutant, project funding source, traffic data, and NEPA class of action (Categorical Exclusions (CE), Environmental Assessments (EA), or Environmental Impact Statements (EIS)). The NEPA process does not always involve quantitative or even qualitative air quality analysis but always requires documentation of compliance. Qualitative and quantitative analyses have differing levels of traffic data required. The air quality analyses, and their related traffic data discussed in this section are for project-level conformity only, this does not cover regional air quality conformity analyses that might be done for a MPO area, for example.

Make sure that any scoping-level clarifying assumptions and requirements are clearly established by the ODOT air quality specialist and/or consultant air quality staff before starting on generating the air quality traffic data. Project-level air quality analyses are always directly based on the project traffic analysis. The volumes and analysis used for the project analysis must be consistent with the values and information generated for the air quality analysis. For example, if volume forecasts were post-processed for the project analysis from model volumes, then the same post-processed forecasts need to be used in the air quality analysis. Mixing of post-processed volumes (see Chapter 6) for the project and only model-based volumes for the air quality analysis would not be acceptable as they have completely different methodologies and would not be consistent between each other. Post-processing involves using the relationship (growth trend) between two different model scenarios and applying that relationship to actual ground counts to create design hour volumes on a directional link-by-link basis. This is much different than using model volumes from a scenario which are mathematically generated from household and employment data and their relationship with the transportation network. This means that the air-quality (along with noise) traffic data needs must be considered in the development of the overall project scope.

16.4.1 Local Carbon Monoxide (CO) Analysis

The purpose of the project-level local CO analysis is to estimate the highest localized CO concentrations resulting from each project alternative to show the project conforms to the

[1] Air Quality Manual – Project Level v 1.0, Geo-Environmental Section, Oregon Department of Transportation, October 2018.

Clean Air Act Amendments. The highest CO concentration usually occurs near the highest volume or congested intersections. Salem is the only remaining CO maintenance area that is subject to a CO conformity analysis. The CO conformity analysis must show that the project will not cause or contribute to new violation of the standard, increase the frequency or severity of an existing violation, or delay the timely attainment of any standard or transportation control measures.

CO analyses can be either qualitative or quantitative. If a project has intersections that fall into the range of LOS A-C only, then just a qualitative analysis will be needed. The air quality analyst will need to document the overall conditions with the relative impacts of v/c ratio, LOS, delay, or other traffic analysis results that should be obtained from the project technical memorandums or directly from the project traffic analyst. If there are signalized intersections on the project which operates at a LOS D or worse, then a quantitative analysis will be needed. Calculation/organizational templates for developing the required CO traffic data are available on the Technical Tools website under the Volume Development section.

Quantitative CO Analysis Data Needs

The overall scope of data needs for quantitative CO analyses are based on the overall ranking of the final design alternative(s) project-area signalized intersections. The project traffic analyst should separately rank the top three signalized intersections by LOS and total entering volume (TEV) for both the build year (year of opening) and the design year (20-yr projection). In addition, intersection delay and v/c should be included as extra information used for helping to pick locations if the LOS/TEV approach is not clear enough. Coordination with ODOT Air Quality staff can be helpful here as they could help to indicate the intersection(s) of interest which could reduce the analysis burden. While the same scoped data needs are also necessary for the no-build condition, only the build condition signalized intersections are ranked.

However, since CO ambient concentrations are well below the standards and there is no chance of violating the CO standard, usually a single intersection can be used to draw a conclusion regarding project impacts. All other affected intersections, given there are no substantive geometric differences compared to the no-build, can be qualitatively discussed based on LOS and TEV.

The following data elements are needed for the top one (1) to three (3) intersections for each of the LOS/delay or TEV cases for the build and design years for both the AM & PM peak hours (or periods as applicable):

- Intersection lane configurations
- Signal controller type: Pre-timed, semi-actuated, or actuated
- Approach grade (%) :This can be obtained from the TransInfo Vertical Grade Report for ODOT-owned approaches, local jurisdictions, or possibly project design staff.
- Lane saturation flow rates (vph); including permitted and protected rates for turns
- Traffic volumes by lane (vph)

- Effective green time (s)
- Yellow time (s)
- Red time (s) = Cycle length – Yellow time – Maximum green time
- Phase times and overall cycle length (s)
- Clearance lost time (s) = Also known as the “extension of effective green time”, generally assumed to be a default of 2 seconds, but could be longer for complex intersections.
- Free-flow speeds for each approach
- Arrival types for approaches (1 to 5):
    1. Worst progression; dense platoon at beginning of red
    2. Below average progression: dense platoon at middle of red
    3. Average progression: random arrivals
    4. Above average progression: dense platoon at middle of green
    5. Best progression: dense platoon at beginning of green (includes both HCM Arrival Type 5 and 6)

The volumes supplied by the traffic analyst should be balanced across intersections and links in a similar process to what is done for the noise analysis (See Section 16.3 or Chapter 6 for general balancing guidance). This should be the case if the volumes are directly taken from project analysis files.

Free-flow speeds can be obtained from available private speed data sources such as RITIS if enough historical data is available to project future speeds, post-processed from a travel demand model or estimated using posted speeds plus five mph if better data was not available. See Section 16.4.3 for the MSAT future no-build and build process steps for more information.

Most intersection parameters can be easily obtained or calculated by assembling a PDF file of the appropriate signalized input or output from Synchro, Vistro, Sidra Intersection, HCS, etc.

Arrival types can be estimated by viewing time-space diagrams on the progressed approaches or is best calculated using the platoon ratio. The platoon ratio indicates the quality of the signal progression. See the Platoon Ratio discussion in Chapter 19 of the Highway Capacity Manual for more information. The platoon ratio for the approach would be based on the exclusive and shared through lanes (i.e. through lane group) by using the green time of the through lane group movement. HCM Exhibit 19-13 is used to obtain the arrival type from the calculated platoon ratio.

The platoon ratio is defined as:

$R_p = P / (g/C)$

P = proportion of vehicles arriving during the through lane group green time (decimal); = 1 -P_{red} from Section 16.3.1 Other Information and Example 16-2.
g = effective green time (s)
C = cycle length (s)

Example 16-4: Arrival Type Calculation

This example is a continuation of Example 16-2. From Example 16-2 the proportions of traffic expected to stop (expressed as a decimal) for each isolated approach is:

$P_{redEB} = 0.503$
$P_{redWB} = 0.509$
The resulting proportion of traffic arriving on green is calculated as 1 – P_{red}:

$P_{greenEB} = 0.497$
$P_{greenWB} = 0.491$

The northbound and southbound movements are coordinated, but since the platoon ratio is based on the entire green time of the through movement, the isolated approach method is used to calculate the proportion arriving on green. If the coordinated method was used, it will result in platoon ratios exceeding the upper limit (i.e. 2.00).

P_{green} = (Total Split – yellow time – all-red time) /cycle length

From the data given in Example 16-2, the proportion of traffic arriving on green is calculated as:
$P_{greenNB} = (35.5 – 4.0 – 0.5) / 100 = 0.310$
$P_{greenSB} = (35.5 – 3.5 – 0.5) / 100 = 0.315$

From the Synchro timing window the effective green time and g/C ratio can be obtained. Since the g/C ratio is indicated it can be used as-is in the platoon ratio equation instead of doing the calculation with the effective green time and cycle length. The same consideration for coordinated signals in the north-south direction applies here where the subject intersection timing is used for the northbound approach and the upstream intersection is used for the southbound approach as both are the first signal in the respective directions.

Subject Intersection

TIMING SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Actuated Effct. Green (s)	6.0	66.7	66.7	6.3	58.6	58.6	23.4	23.4	—	23.4	23.4	—
Actuated g/C Ratio	0.06	0.67	0.67	0.06	0.59	0.59	0.23	0.23	—	0.23	0.23	—

The platoon ratio for each direction is computed as:

R_{pEB} = 0.497 / 0.67 = 0.74
R_{pWB} = 0.491 / 0.59 = 0.83
R_{pNB} = 0.310 / 0.23 = 1.35
R_{pSB} = 0.315 / 0.23 = 1.37

Comparing to the Platoon ratio thresholds in HCM Exhibit 19-13, the corresponding arrival type is:

Arrival Type EB = 2; (Unfavorable, below average)
Arrival Type WB = 2; (Unfavorable, below average)
Arrival Type NB = 4; (Favorable, above average)
Arrival Type SB = 4; (Favorable, above average)

The traffic data needs above included in the CO quantitative tool are then sent to the air quality analyst who uses two different modeling software packages. The first (MOVES) establishes the amount of pollutants that would be emitted by vehicle traffic in grams per mile and idling vehicles in grams/hour which uses the traffic volumes and free-flow speeds for each approach. The second (CAL3QHC) determines the concentration of the pollutant based upon a conservative dispersion algorithm in parts per million which uses the intersection-based signal timing data.

FHWA CO Categorical Hot Spot Analysis

The air quality analyst may also want to determine if the project could be classified under the FHWA "CO Categorical Hot-Spot Finding" to lessen the overall CO analysis needs. This requires additional data and has specific limits for each data parameter to determine if the project can use the categorical analysis methodology. More information on the parameters, scenarios and application of the finding is available here. The following traffic data elements are needed for both the build and the design year as shown in Exhibit 16-10 for each of the top three (or as determined by the air quality analyst) highest total entering volume (TEV) and worst LOS signalized intersections. The air quality analyst will need to provide the temperature and CO concentration parameters to complete the data requirements. The project traffic analyst should let the air quality analyst know if any of the data parameters below on any of the intersections are exceeded as that would invalidate the use of this option.

Exhibit 16-10 FHWA Categorical CO Traffic Data Parameters

Parameter	Acceptable Range	Notes
Analysis year	≥ 2022	Build and Design year
Area type	Urban or Rural	Minimum urban population of 5,000; otherwise use rural
Road approach grade (%)	≤ 6%	Maximum approach grade of all legs within 100' of stop bar
Truck percent (%)	≤ 20%	Highest approach percentage for heavy trucks (FHWA Classes 6-13)
Peak hour approach speed (mph)	15 to ≤ 45 mph	All approaches must be within limits
Peak hour approach volume (vph)	≤ 2640 vph	Highest approach volume
Peak hour LOS	A-E	Intersection LOS
Intersection approach angle	≥75°7	Smallest angle between the intersecting roadways
Number of approach through lanes	≤ 4	Maximum among all approaches
Number of approach left turn lanes	≤ 2	Maximum among all approaches
Lane width (ft)	≥10 ft	Minimum width of all lanes
Median width (ft)	≥ 0 ft (Any)	

The geometric data in Exhibit 16-10 should be available in the project inventory and the traffic data should be available as part of the project traffic analysis. The approach peak hour average speed could be obtained from the available traffic analysis output such as Synchro, Vistro, etc. or better, from a calibrated microsimulation previously done for the project such as SimTraffic or Vissim. Posted speeds or existing year probe speed data could be used as a proxy if no other better data is available.

The first step in a CO analysis traffic data development is to rank the signalized intersections by the highest LOS and total entering volume (TEV). The CO template tool includes a ranking table which is filled out and used for the rest of the traffic data development. It is recommended to coordinate with the ODOT Air Quality Unit to confirm the number intersections needed as frequently the number of required locations is less. It is common practice to only model the top intersection because CO concentrations have dropped significantly below standards in Oregon with no risk of violating the NAAQS (National Ambient Air Quality Standards). See the ODOT Air Quality Manual for more information. Example 16-5 uses a sample project location to illustrate the use of the available CO templates for the quantitative and categorical analysis methods.

Example 16-5 Sample Project Quantitative & Categorical CO Traffic Data using Templates

The sample project area below had four signalized intersections which were ranked for both the 2025 build (year of opening) and the 2045 design year. This analysis was only done for the PM peak; however the template does allow for inclusion of the AM peak which is commonly also included.

For the build year, the highest LOS was C which is LOS rank #1. There were two intersections with LOS B and so the v/c column was used to decide that the Redwood & Dowell Rd intersection was to be LOS rank #2 with the US199 & Allen Creek Road intersection being #3. These three intersections were also found to have the highest TEV, so these three locations would be the ones analyzed for this case.

Scenario	Peak	Intersection	v/c	Intersection Delay(s/veh)	Intersection LOS	TEV (vph)	LOS Rank	TEV Rank
		US199 & Allen Creek Rd	0.59	17.9	B	3,000	3	1
Build Year 2025	PM	US199 & Dowell Rd	0.60	25.6	C	2,540	1	2
		Redwood Ave & Allen Crk Rd	0.82	8.9	A	1,280		
		Redwood Ave & Dowell Rd	0.65	11.7	B	1,320	2	3
		US199 & Allen Creek Rd	0.58	24.7	C	3,920	3	1
Design Year 2045	PM	US199 & Dowell Rd	0.84	54.5	D	3,465	2	2
		Redwood Ave & Allen Crk Rd	1.10	75.1	E	1,750	1	
		Redwood Ave & Dowell Rd	0.80	16.1	B	2,065		3

For the design year, conditions have gotten worse as expected but the intersections are not in the same ranking order as in the build year. The highest LOS is E at the Redwood & Allen Creek Road intersection which gets LOS rank #1 (note that this location was unranked in the build year). The US199 & Dowell Road intersection is LOS rank #2 and US199 & Allen Creek Road is #3. For TEV, the highest intersections in the build year are also in the same order for the design year. In this case the intersections to be analyzed for CO will include all four as these encompass the top three LOS and TEV locations.

The intersection data is entered into the quantitative CO template sheet for each intersection (for each year and hour analyzed. This includes various geometric (e.g. lane configurations and approach grade), volume (e.g. volumes and saturation flow) and signal timing/phasing (e.g. cycle length, yellow/all-red time) elements. For simplification, only Intersection #1 (based on the 2045 ranking above), Redwood Avenue and Allen Creek Road, for the 2045 PM Peak is shown. The resulting approach red time and arrival type by lane is calculated by the spreadsheet.

[Please enlarge/zoom-in as necessary to view]

Intersection 1	Redwood Ave & Allen Creek Rd					PM Peak						
Signal Controller Type	Actuated-Coordinated											
Cycle length (s)	120											
Clearance lost time default (s)	2											
Approaches		North (SB)			East (WB)			South (NB)			West (EB)	
Approach grade (%)		0%			0%			0%			0%	
Movements	L	T	R	L	T	R	L	T	R	L	T	R
Shared lane configuration	n/a	TR	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Number of lanes	0	1	0	1	1	1	1	1	0	1	0	2
Lane saturation flow rates (vph)		1514		1496	1559	1339	1496	1575		1496		2333
Lane volumes (vph)		10		15	1080	5	5	5		5		155
Phase movement		SBTR		WBLTR	WBLTR	WBLTR	NBLT	NBLT		EBLR		EBLR
Phase time (max green) (s)		24		87	87	87	24	24		87		87
Yellow time (s)		4		4	4	4	4	4		4		4
Red time (s)	120	92	120	29	29	29	92	92	120	29	120	29
Actuated Effective green time (s) (Synchro calc.)		9.8		100.2	100.2	100.2	9.8	9.8		100.2		100.2
Free-flow approach speed (mph)		35			35			35			35	
All-red time (s)		0.5		0.5	0.5	0.5	0.5	0.5		0.5		0.5
Actuated g/C ratio (Synchro calc.)		0.09		0.92	0.92	0.92	0.09	0.09		0.92		0.92
Platoon Ratio (interim calculation)	#DIV/0!	1.81	#DIV/0!	0.75	0.75	0.75	1.81	1.81	#DIV/0!	0.75	#DIV/0!	0.75
Arrival Type	#DIV/0!	5	#DIV/0!	2	2	2	5	5	#DIV/0!	2	#DIV/0!	2

If required, the intersection data is also entered into the Categorical CO spreadsheet to see if all the parameter limits are satisfied. Any value outside of the required range will be highlighted in red. The figure below shows the inputs for both the 2025 year of opening and 2045 design year and shows that all values fit within the required parameters.

Intersection 1	Redwood Ave (E-W) & Allen Creek Rd (N-S)							
Area Type :	Urban							
	2025 Build Year				2045 Design Year			
Approach	N	E	S	W	N	E	S	W
Grade (%)	0	0	0	0	0	0	0	0
Heavy Trucks (%)	0	1	0	1	0	1	0	1
Peak Hour Speed (mph)	35	35	35	35	35	35	35	35
Peak Hour Volume (vph)	10	1100	160	10	10	1475	10	255
Skew angle (deg)	90	90	90	90	90	90	90	90
Number of through lanes	1	1	0	1	1	1	0	1
Number of left turn lanes	0	1	1	0	0	1	1	0
Lane width (ft)	12	12	12	12	12	12	12	12
Median width (ft)	0	0	0	0	0	0	0	0
Peak Hour Intersection LOS	A				E			

For example, if the E-W direction had a large skew at 45 degrees, this would be less than the minimum angle (i.e. 75 degrees 7 minutes). If any parameter is exceeded, then this intersection cannot use this method and must use the standard Quantitative CO template and method. Once completed, both spreadsheets for each method are forwarded to the responsible air quality staff for use in their modeling.

Intersection 1	Redwood Ave (E-W) & Allen Creek Rd (N-S)							
Area Type :	Urban							
	2025 Build Year				2045 Design Year			
Approach	N	E	S	W	N	E	S	W
Grade (%)	0	0	0	0	0	0	0	0
Heavy Trucks (%)	0	1	0	1	0	1	0	1
Peak Hour Speed (mph)	35	35	35	35	35	35	35	35
Peak Hour Volume (vph)	10	1100	160	10	10	1475	10	255
Skew angle (deg)	90	45	90	45	90	45	90	45
Number of through lanes	1	1	0	1	1	1	0	1
Number of left turn lanes	0	1	1	0	0	1	1	0
Lane width (ft)	12	12	12	12	12	12	12	12
Median width (ft)	0	0	0	0	0	0	0	0
Peak Hour Intersection LOS	A				E			

16.4.2 Particulate Matter (PM_{10} or $PM_{2.5}$) Hot Spot Analysis

Most PM analyses will be qualitative unless they are a project of local air quality concern [as defined in 40 CFR 93.123(b)(1)] and as such are required to have a quantitative analysis. Any project exempt from CO analysis is also exempt from PM analysis. Qualitative analyses will have a short discussion prepared by the air quality analyst based on the highest traffic volume links, diesel truck percentages, and LOS for both the no-build and build conditions. This data may be gleaned from project reports or obtained directly from the project traffic analyst. This data must be documented using the PM10 and PM2.5 Project Level Checklist (provided by ODOT Environmental Section staff) and each PM project taken to interagency consultation by ODOT Air Quality staff. The traffic engineer responsible for the traffic data inputs must also attend the interagency consultation.

Only certain project types are subject to needing a PM hot spot analysis. Calculation/organizational templates for developing the required PM traffic data are available on the Technical Tools website under the Volume Development section.
A quantitative analysis may be required if the project has a substantial number or increase of diesel-powered vehicles; or affects build alternative intersections that operate at LOS D or worse along with a substantial number of diesel-powered vehicles. Generally these types are:

- New highway projects with a substantial amount of diesel vehicles or expansion projects that would have a substantial increase of diesel vehicles. Examples would be a project on a highway with at least 125,000 total bidirectional AADT with 8% or more diesel trucks (10,000 truck AADT) or a new interchange connecting to a major bus or intermodal terminal. See Exhibit 16-11 for heavy truck diesel fuel factors to help determine the number of diesel trucks. Thresholds may differ based on interagency coordination requirements, so actual application of this element is flexible.

- Projects that affect intersections that are currently at LOS D-F with a substantial amount of diesel vehicles (10,000 truck AADT) or those that will change to LOS D-F because of increased traffic volumes from a substantial amount of diesel vehicles related to the project. An example would be a highway widening project that affects a poorly operating intersection that has a substantial amount of diesel trucks. This condition occurs relatively infrequently and generally would only occur in the Eugene-Springfield metropolitan area.

- New bus and intermodal terminals that have a substantial amount of diesel vehicles congregating at a single location or terminal expansions that significantly increase the amount of diesel vehicles congregating at a single location. Examples would be anything determined to be a "regionally significant project" or a large bus terminal which has the number of arriving diesel buses increase by 50% or more.

- Projects in or affecting locations, areas, or categories of sites which are identified in the $PM_{2.5}$ or PM_{10} applicable implementation plan or a new implementation plan submission, as appropriate, as sites of violation or possible violation. Projects within the urban growth boundary in these cities may trigger a quantitative analysis: Eugene-Springfield, Lakeview, and La Grande. Exceptions to the UGB limit are the entire Rogue Valley metropolitan area and the larger-than-UGB titled "$PM_{2.5}$ Nonattainment Boundary" for Klamath Falls and Oakridge.

Projects that are mainly intended to improve traffic flow and speed and that do not have increases in delays (i.e. increase of idling vehicles) or capacity can be exempted from the need to do a quantitative PM analysis. These include:

- Intersection channelization; especially those with physical separators
- Roundabouts
- Intersection signalization
- Roadway reconfigurations
- Auxiliary lanes under one mile in length
- Ramp metering

For a project to remain under a qualitative analysis, it must demonstrate that it is not a local air quality concern. This requires identifying the roadways with the highest AADT and diesel truck percentages that are acceptable to the interagency group. The Environmental Protection Agency's (EPA) PM guidance suggests that projects below 125,000 (bidirectional) AADT or 10,000 diesel truck AADT (8%) are typically not a project of air quality concern. This screening analysis (see Example 16-6) should be documented with ODOT's Air Quality PM10 and PM2.5 checklist (available from ODOT Environmental Section) and be done using the build alternative for the future (20-yr) design year. The screening-level AADT's should be reported for all project roadways. For example, if a project on I-5 exceeded 125,000 AADT on a couple mainline sections then all the freeway mainline and assorted ramps would be reported for consistency surrounding this location. Other roads (e.g. crossroads, parallel frontage roads, local arterials) in the project area with substantially less volume and trucks should also be reported for consistency and documentation requirements.

Each link in the screening analysis needs to have the AADT and the daily percentage of heavy-duty trucks (FHWA vehicle class 6-13). This percentage needs to be further modified by the diesel fleet fraction. Alternative heavy duty vehicle fuel sources are becoming more common, especially in the future. Exhibit 16-11 shows the diesel fuel fractions for heavy trucks that should be used to estimate the percentage of diesel trucks. Interpolate as necessary for years that fall in between the years in the table, but note that the relationship is not linear, so interim years will need to be estimated along the curve. Use the 2035 value for all future years beyond 2035.

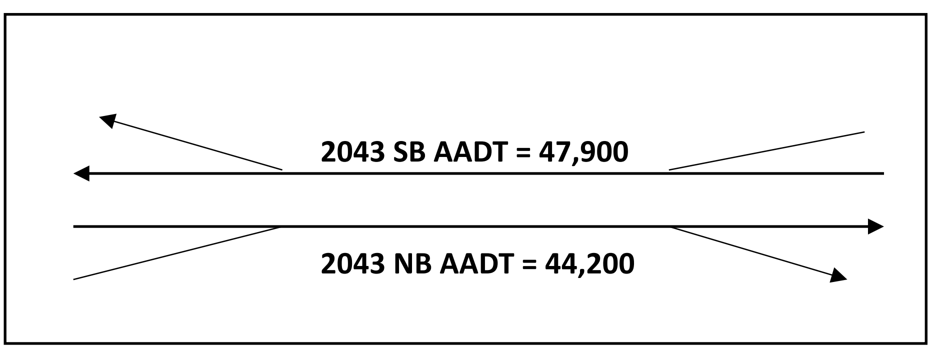
Exhibit 16-11 Heavy Truck Diesel Fuel Fraction

Year	Diesel Fuel Fraction
2020	1.000
2025	0.990
2030	0.941
2035+	0.880

An interagency coordination process must be initiated to confirm if a quantitative PM analysis is required. This includes the EPA, Federal Highway Administration (FHWA), Federal Transit Administration (FTA), and either Oregon Department of Environmental Quality (DEQ) or if the project is in Lane County, then the Lane Regional Air Protection Agency. This group will decide if a quantitative analysis is needed and the specific methodology and scope of any additional traffic data.

Example 16-6 Qualitative Particulate Matter Screening Analysis

An auxiliary lane project is proposed on a freeway which has a projected 2043 no-build daily volume of 92,100 from the Future Volume Tables. There is a directional split of 52% southbound and 48% northbound and 12.6% heavy trucks (FHWA vehicle classes 6 – 13) in this section. The figure below shows the general configuration and the 2043 directional volumes.



The project was modeled in a travel demand model scenario and resulted in corresponding 2043 daily model link volumes for the no-build of 105,431 and a build of 107,843. Using the build volume in the screening analysis allows consideration of any latent demand shifts when traffic diverts from other slower links to the improved build section.

Using the difference method (see Chapter 6) to post-process the section:

Build volume = No-build volume + (Model Future Build volume – Model Future No-build volume)

92,100 AADT + (107,843 – 105,431) = 94,500 (rounded) which is less than 125,000 AADT. All other roadway sections would be below this level as this auxiliary lane portion has the highest volume.

However, 94,500 x 12.6% heavy trucks x 0.88 diesel fuel fraction (See Exhibit 16-XX) = 10,500 (rounded) which is higher than 10,000 diesel truck AADT. This converts into 5,450 southbound and 5,030 northbound diesel trucks.

It would be up to the interagency coordination to determine if the amount of the heavy diesel trucks in the project would require a quantitative PM analysis.

Quantitative PM Process – Future Build/No-build Conditions

Quantitative PM analyses involve use of the MOVES air quality analysis tool with sufficient traffic data needed to properly characterize the conditions over a typical day. Interagency coordination would be typically used to determine the scope of the analysis.

The recommended best-practice analysis approach should start with the future build conditions to minimize effort and time. The results from these inputs would be used by the air quality analyst to determine if the desired $PM_{2.5}$ concentration are below the National Ambient Air Quality Standards (NAAQS). If they are met, then the interagency consultation will determine if the analysis is complete. The traffic consultant does not determine this. If the NAAQS are not met, then mitigation or control measures will be needed (i.e. adding vegetation along the roadway or some level of redesign, etc.). Then the build analysis is repeated which may involve modifications to the traffic data inputs. Depending on the scale of the measures used, if any, it also may be necessary to calculate the future no-build conditions and test if the resulting build $PM_{2.5}$ concentrations are less than the no-build. This would require another set of traffic data inputs for the future no-build. If the build concentrations were determined to be less than the no-build then the NAAQS could be met, however, passing this test can be difficult, so this could introduce a level of iteration with multiple requests for updating the traffic data inputs.

A spreadsheet should be created with rows for each link and columns for each of the link attributes for the future build conditions below. All project area links need to have the attribute data created. Links outside of the project area are optional (based on guidance from the interagency consultation). Links, identifiers, and characteristics are set up in the same manner as for the noise analysis in Section 16.2.1. Most projects that are big enough to trigger air quality analysis also will have a noise analysis, so the noise analysis directional link spreadsheet should be used as a starting point to expand upon to expedite this analysis.

Intersections should be located on a separate tab and the listed attributes reported for all intersections. The interagency consultation will determine what intersections need to be included and what data is needed. Data needs could be more specific/detailed than shown based on the context of the project area, screening results, etc.

Link attributes

- Unique link identifier
- Link name
- Link length (mi)
- Grade (%)
- MOVES road type 1-5; Off-network (1), rural restricted (i.e. access via interchanges) (2), rural unrestricted (3), urban restricted (4) or urban unrestricted (5)
- Design speed (mph)
- Prevailing operating average speed (mph)
- Peak hour volume for morning (6-9 AM) & afternoon (4-7 PM) periods
- Average hour volumes for midday (9 AM – 4 PM) and overnight (7 PM to 6 AM) periods
- Hourly K-factors
- Vehicle classification for each period: light vehicles (Class 1-3), medium trucks (Class 4-5), and heavy trucks (Class 6-13)

Intersection attributes

- Total entering peak hour (AM & PM periods) or average hour volume (midday & overnight periods)
- AM or PM K-factor
- ADT to AADT seasonal adjustment factor
- Future build year directional AADT
- Light, medium, and heavy vehicle daily percentages
- PM Peak hour LOS

The project future year design hour volumes following Chapter 6 are used as-is. This usually will be at least the PM peak and possibly the AM peak hours. The AM/PM peak hour volumes are representative of each larger morning or afternoon three-hour period. There is no need to create multiple hours for each peak period. Available 24-hr counts or ATRs need to be used to create volume profiles which are used to determine the proportion of the daily volumes that are included in each of the midday and overnight periods (see Example 16-7 below). The hours shown above for the period durations are typical, but if the specific project is different than these should be changed to match local conditions.

Ideally these are same counts as used in the project analysis, so the average hour volumes are completely consistent with the peak hour volumes on a link-by-link basis. If that is not possible due to count duration limitations, they should be in the same general area, on the same or similar facilities, and within five years. The summed proportion across each period would be multiplied by the link volume and then divided by the number of hours in each period to determine the average hourly volume. Like with the noise analyses, the volumes for all four periods should be balanced across the links.

In addition, this same process can be used to help determine AM peak hour volumes if they are not available. The proportions of daily volumes (i.e. K-factors) from the 24-hour count will be shown for the AM peak hour period. Depending on the known local context, either use the hourly proportion (K-factor) for the actual AM system peak hour if known from other counts, or the highest directional AM hourly K-factor if the actual AM peak hour is not known. If the highest K-factor representing a certain AM peak hour is taken it needs to be the same across all counts and roadway directions for consistency as this would be an AM system peak hour (see Section 5.3 for considerations on choosing system peak hours).

Example 16-7 Midday & Overnight Average Hour Volume Calculation

A 24-hour count from a project is shown below with each hour's volume as a proportion of the daily total volume. The midday and overnight hours are highlighted in gray.

Hour	EB Volume	Daily Proportion	WB Volume	Daily Proportion
0	226	0.012	170	0.008
1	137	0.007	136	0.007
2	125	0.007	111	0.005
3	98	0.005	113	0.006
4	80	0.004	238	0.012
5	179	0.010	604	0.030
6	340	0.018	1151	0.056
7	697	0.038	1499	0.073
8	1071	0.058	1437	0.070
9	822	0.044	1205	0.059
10	829	0.045	1118	0.055
11	939	0.051	1139	0.056
12	1109	0.060	1172	0.057
13	1146	0.062	1156	0.057
14	1332	0.072	1230	0.060
15	1506	0.081	1232	0.060
16	1520	0.082	1319	0.064
17	1541	0.083	1320	0.065
18	1348	0.073	1221	0.060
19	960	0.052	878	0.043
20	807	0.044	753	0.037
21	656	0.035	571	0.028
22	592	0.032	395	0.019
23	440	0.024	290	0.014
Total	18500		20458	

Summing the hourly proportions yields:

- Eastbound midday (9 AM – 4 PM): 0.415
- Westbound midday: 0.403
- Eastbound overnight (7 PM – 6 AM): 0.232
- Westbound overnight: 0.208

These proportions would be applied to applicable link volumes. If a link had 10,000 ADT eastbound then this would result in 0.415 x 10,000 = 4,150 vehicles assigned to the midday period and 0.232 x 10,000 = 2,320 vehicles assigned to the overnight period. The average hour for each period would be determined by dividing this ADT by the number of hours in the period.

- Eastbound midday average volume = 4,150 veh / 7 hrs = 595 vph
- Eastbound overnight average volume = 2,320 veh / 11 = 210 vph

The westbound direction would be handled similarly, so if there were 10,000 westbound then there would be 4,030 vehicles for midday and 2,080 vehicles for the overnight period. This would result in 575 vph for the midday average hour and 190 vph for the overnight average hour.

In addition, if the AM peak hour was not available from the project analysis, then the highest hourly proportion (i.e. K-factor) would be picked if the peak hour was not known. In this case, the highest proportion is 0.073 in the WB direction representing the 7-8 AM peak hour. The corresponding K-factor for the non-controlling EB direction for this peak hour would be chosen (0.038) for consistency.

If the build alignment is the same (same link network) with differing number of lanes and/or traffic control, but the volumes are the same then the future build would likely be the same as the future no-build. If the build design year volume(s) is different from the no-build because of latent demand issues stemming from pent-up congestion, then the volumes will change, and the process will be same as doing the future no-build.

The challenge with the future build data is when the build alignment or network layout is different from the no-build alignment as the relationships between links gets muddied. The peak hour volumes would have already been re-distributed onto the build network for the project analysis. The analyst will need to figure out separately the routing of the heavy truck group and resulting link percentages if they do not follow the same patterns as the peak hour.

Design speeds can be the posted speed plus five mph for the no-build, but actual design speeds should be available for the build alternatives. Operating/prevailing speeds for links will need to be based on deterministic (i.e. Highway Capacity Software) or a calibrated microscopic (i.e. SimTraffic) analysis tool outputs. If future conditions are overcapacity, then HCS output may not include speed, so micro-simulation output may be necessary. Alternatively, future speeds could be post-processed from travel demand models. This would need to be based on the probe-based speed data (i.e. RITIS) for the existing conditions and a model link speed factor (i.e. future year model build link speed divided by the model base or reference year link speed).

The overall limitation of this process is based on the overall ability of the model to account for congested conditions, so the reported speeds are as accurate as possible. The ideal case would be to calibrate the model to speeds in addition to volume, however that is a practice that is not currently performed, and its potential success is untested. In general, use of an activity-based model or a dynamic traffic assignment model scenario would be preferred, if available, over a regular small city or MPO-level travel demand model. Raw unadjusted model speeds are not acceptable as ground-truthing is needed either through post-processing or calibration.

Vehicle classification data for light, medium and heavy vehicles should be obtained from the project counts. This usually would be already done for at least heavy vehicles for the peak periods in the project analysis which means that the medium trucks would be additionally created. The total of the heavy and medium trucks can be subtracted from the total hourly volumes to create the light volume totals. These total volumes for each classification are divided by the total volume for each period to determine the decimal proportion for each. Alternatively, if noise analysis traffic inputs were created earlier then these proportions are likely available for the peak periods which will reduce the data needed to be created.

The intersection attributes are mostly summing up the individual entering approach links for the peak/average hours and the daily volumes. The daily vehicle classification groupings will need to be created from the applicable 24-hr classification counts. It is unlikely that all counts in a project will be 24-hr classification counts, so shorter duration counts, or volume-only counts will need representative classification counts assigned to them. Intersection LOS analysis is only needed for the PM peak hour and in most cases, should be taken from the future build/no-build project analysis results. This analysis should be consistent with the current HCM methodology.

A tab containing the link diagram should also be added but it can be a separate document if desired. The link identifier through the speed columns is the same for all scenarios.

The overall process is as follows:

1. Add the unique link numbering scheme and link name/description, link length, link grade and MOVES road type

2. Add the known build design speed (or use posted speed + 5 mph). Calculate or post-process the link prevailing/operating speeds.

3. Use the future build directional link hourly volumes from the project or noise analysis from the PM or (if available) the AM peak hours. Use available 24-hour counts to determine hourly volume proportions (see Example 16-8) along to calculate the average hour volumes for the midday and overnight periods and the AM peak hours if needed. Apply a growth factor or post-processing to convert the average midday and overnight volumes to future no-build volumes and then to future build volumes following patterns used to originally create the AM/PM future build volumes.

4. For each of the four periods, balance the hourly volumes across the network on a link basis following a similar process to Section 16.3.3. Balancing is performed by summing the inbound and outbound link volume and computing the difference. The difference in the in or out volumes is then spread around the intersection proportionately. If differences are large then both the incoming and the outgoing volumes can raised or lowered to minimize the change. This is repeated for all intersections.

5. Use the counts to compute the initial AM/PM peak hour medium and heavy vehicle volumes for each link by multiplying the directional link hourly volume by the corresponding vehicle classification percentage. Average the medium and heavy truck percentages across the midday and overnight hourly periods to obtain the initial medium and heavy vehicle volumes for those periods.

   Depending on what may been developed previously (i.e. vehicle classification link allocations from a future build noise analysis), these may have to start with the existing or future no-build conditions. These are used to develop the initial volumes for the future no-build which are then balanced by comparing the ins and outs at each intersection and used to modify the factors to reflect the future conditions. Sink/source locations should be consistent with the drop or increase of the existing conditions with the appropriate growth to the future. Once the no-build future factors are developed, then these need to be translated over to the build network to make the necessary modifications to the factor data. For this step through Step 7, the process is like the noise traffic development process in Section 16.3.5.

6. Balance these initial medium and heavy truck volumes across the build network for each period. Subtract the total of the medium and heavy truck volumes from the total volumes to determine the total light vehicle volume.

7. Divide the balanced light, medium and heavy vehicle volumes by the total link volume to determine the decimal proportion of each classification grouping. Repeat for each of the four periods.

8. For each project-area intersection identified to be included from the interagency consultation, create the total entering volume by summing the entering link peak or average hour volume (depending on the period) for each of the four periods. Keep the intersection calculations on a separate tab.

9. Choose either the AM or PM peak hour and apply the corresponding peak hour K-factor (from the volume profile work in Step 3) to compute entering link ADT. Then use the Seasonal Trend Table (see Section 5.5.4) to determine the appropriate trend factor to use. The Seasonal Trend Table shows ratios of AADT to monthly ADT, so choosing the month(s) of the counts used will determine the necessary conversion factor. Interpolate for counts not at the beginning or middle of the month. Alternately, if available, use the AADT developed previously from the project analysis or create AADT from on-site or characteristic ATRs (see Chapter 5). Sum the entering link AADT to create the total intersection AADT.

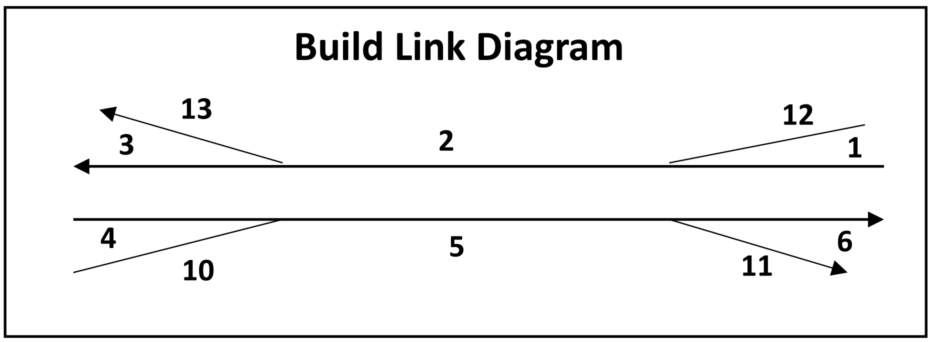
10. For each entering intersection link, determine the daily medium and heavy truck percentages from the classification counts or use percentages, if available, from the project or noise analysis for the future build year. Sum the medium and heavy trucks and subtract from 100% to determine the light vehicle percentages.
11. For each intersection but just for the PM peak period, record the intersection LOS

(D-F) from the future build (design) year traffic analysis on the intersection tab. If this has not been done (or available) then perform the intersection analysis following HCM methodology and APM Chapter 12 and 13.

12. Repeat Steps 1-12 for the future no-build if deemed necessary by the interagency coordination group.

Example 16-8 Quantitative PM Analysis

This example is a continuation of Example 16-7. From the interagency coordination discussions, it was decided that this auxiliary lane project should have a quantitative analysis of the future build conditions because diesel trucks exceeded 10,000 ADT. A build link diagram was established with a numbering scheme:



Link lengths and grades were obtained from ODOT inventory data. Design speeds were assumed to be the posted speed limit + 5 mph which would be 70 mph for the freeway mainline and 40 mph for the ramps. Since this the future 2043 build conditions, the operating speeds for the mainline links were developed using HCM freeway methodologies with the project area geometric assumptions [e.g. 12’ lanes, 6’ shoulders, three mainline lanes, a single auxiliary weaving lane, single lane ramps] and known project design hour volumes. Speeds for ramps were estimated from the design speeds.

Link #	Link Name	Link Length (mi)	Link Grade (%)	MOVES Road type	Design Speed (mph)	Prevailing Op. Speed (mph)
1	SB Fwy Upstream	0.18	0.0%	4	70	57
2	SB Fwy Between Ramps	0.99	0.0%	4	70	62
3	SB Fwy Downstream	0.28	2.0%	4	70	61
4	NB Fwy Upstream	0.46	-2.0%	4	70	63
5	NB Fwy Between Ramps	1.13	0.0%	4	70	63
6	NB Fwy Downstream	0.20	0.0%	4	70	60
10	NB on-ramp	0.25	-2.0%	4	50	45
11	NB off-ramp	0.13	0.0%	4	40	35
12	SB on-ramp	0.15	-1.0%	4	40	35
13	SB off-ramp	0.30	1.0%	4	50	45

Twenty-four hour counts on the freeway and ramp sections were used to develop volume profiles. An inspection of the hourly proportions (K-factors) revealed that the default AM peak (6-9 AM) was still correct but the PM peak was really 3-6 PM instead of the default 4-7 PM. The K-factors rise substantially in the peak periods then drop off afterwards as seen below in the figure. The ramp counts (not shown) also had the same peaking patterns.

Hour	Fwy NB Volume (vph)	Daily Proportion	Fwy SB Volume (vph)	Daily Proportion
0	264	0.008	303	0.009
1	239	0.007	192	0.006
2	221	0.007	217	0.007
3	292	0.009	258	0.008
4	506	0.015	357	0.011
5	1007	0.030	787	0.024
6	1781	0.054	1848	0.057
7	2199	0.066	2306	0.071
8	1889	0.057	1973	0.061
9	1900	0.057	1693	0.052
10	1788	0.054	1820	0.056
11	1791	0.054	1907	0.059
12	1927	0.058	1895	0.059
13	2023	0.061	1879	0.058
14	2068	0.062	1985	0.061
15	2381	0.072	2347	0.073
16	2565	0.077	2606	0.081
17	2447	0.074	2392	0.074
18	1777	0.053	1716	0.053
19	1258	0.038	1207	0.037
20	1122	0.034	956	0.030
21	984	0.030	714	0.022
22	504	0.015	565	0.017
23	352	0.011	373	0.012
Total	**33285**		**32296**	

Once the AM & PM peak periods were defined, then the remaining midday period and overnight period can be defined as 9 AM – 3 PM and 7 PM to 6 AM respectively. The AM & PM peak hours are used as representative hour for each period. The proportion of the day in the midday and overnight periods was summed and multiplied by the ADT and then divided by the total number of hours in each period to come up with an average hour value. This was repeated for all 24-hour counts. The resulting existing year volumes for each period were balanced across the network and then post-processed using NCHRP Report 765 techniques (see Section 6.12) to 2043 no-build future values. Then the 2043 no-build future was converted to 2043 build volumes using a factor developed by comparing the 2043 no-build and build model scenarios.

Lastly, the 2043 build volumes were balanced across the network to end up with the final peak and average hours shown below.

Link #	Link Name	2043 AM Pk Hr Volume (vph)	2043 PM Pk Hr Volume (vph)	2043 Ave Midday Volume (vph)	2043 Ave Night Volume (vph)
1	SB Fwy Upstream	2158	2728	2081	772
2	SB Fwy Between Ramps	2958	3451	2579	955
3	SB Fwy Downstream	2704	3226	2381	883
4	NB Fwy Upstream	2886	3252	2403	829
5	NB Fwy Between Ramps	3187	3605	2577	882
6	NB Fwy Downstream	2421	2839	2053	685
10	NB on-ramp	800	722	498	183
11	NB off-ramp	255	225	198	73
12	SB on-ramp	301	352	174	52
13	SB off-ramp	767	766	524	197

The available mainline freeway counts were used to develop the truck classification data for each period. The known AM/PM peak hour classification for medium (FHWA Class 4-5) and heavy vehicles (FHWA Class 6-13) was used directly as the peak hour is representative of the overall period. The medium and heavy vehicle classifications were each averaged across all hours consistent to how it was done for the volumes and converted into initial decimal proportions. There was no classification data available for the ramps, so it was assumed initially that these used the mainline percentages. These initial proportions were multiplied by the final build volumes to create a set of initial medium and heavy vehicle volumes with the AM peak shown as an example below.

Link #	Link Name	Final Build 2043 AM Pk Hr (vph)	Initial Med Trk Percent (decimal)	Initial Hvy Trk Percent (decimal)	Initial Med Trk AM Pk Hr (vph)	Initial Hvy Trk AM Pk Hr (vph)
1	SB Fwy Upstream	2158	0.039	0.131	84	283
2	SB Fwy Between Ramps	2958	0.039	0.131	115	388
3	SB Fwy Downstream	2704	0.039	0.131	105	354
4	NB Fwy Upstream	2886	0.040	0.119	115	343
5	NB Fwy Between Ramps	3187	0.040	0.119	127	379
6	NB Fwy Downstream	2421	0.040	0.119	97	288
10	NB on-ramp	800	0.040	0.119	32	95
11	NB off-ramp	255	0.040	0.119	10	30
12	SB on-ramp	301	0.039	0.131	12	39
13	SB off-ramp	767	0.039	0.131	30	100

Both the medium and heavy vehicles were then balanced across the network. The total of these was subtracted from the total hourly volume to compute the light vehicles (FHWA Class 1-3). Lastly, the final light, medium and heavy vehicle volumes were divided by the total hourly volume to determine the final vehicle proportions as shown below.

Link #	Final Balanced Lt Veh AM Pk Hr (vph)	Final Balanced Med Trk AM Pk Hr (vph)	Final Balanced Hvy Trk AM Pk Hr (vph)	Final Lt Veh Proportion (decimal)	Final Med Trk Proportion (decimal)	Final Hvy Trk Proportion (decimal)
1	1716	101	341	0.795	0.047	0.158
2	2455	115	388	0.830	0.039	0.131
3	2312	89	303	0.855	0.033	0.112
4	2490	99	297	0.863	0.034	0.103
5	2681	127	379	0.841	0.040	0.119
6	1963	115	343	0.811	0.048	0.142
10	690	28	82	0.863	0.035	0.102
11	207	12	36	0.812	0.047	0.141
12	240	14	47	0.797	0.046	0.156
13	656	26	85	0.855	0.034	0.111

After the link information is completed, the entering link volumes for the applicable intersections were copied into the intersection portion of the spreadsheet tool for each period as shown below. [Note: the intersection in this example is not an “standard intersection” but is used to illustrate the related connections and calculations. This simple case also could be considered equivalent to two one-way roadways intersecting.] The entering link volumes are summed to obtain the total entering volume for each of the four periods.

Intersection Name	Entering Link Dir	Entering Link Volume AM Peak Hr Build (vph)	Entering Link Volume PM Peak Hr Build (vph)	Entering Link Volume Midday Ave Hr Build (vph)	Entering Link Volume Overnight Ave Hr Build (vph)
Mainline & Ramp	NB	2886	3252	2403	829
	WB	800	722	498	183
	Total	**3687**	**3975**	**2900**	**1013**
		Peak Hour Int. LOS =	**C**		

The PM peak hour was chosen to create the total entering daily volume as the original volume development for the project was PM peak hour based. For the subject intersection, the corresponding peak hour K-factors for each entering link were chosen from the actual peak hour (4-5 PM) from the volume profile tab. Since AADT is desired, a conversion factor from the monthly to annual ADT is necessary. The Seasonal Trend Table (see Chapter 5) was referenced for the Interstate Urbanized trend and a mid-May count period to obtain the factor shown in the table below. The PM peak hour volume is divided by the K-factor first then multiplied by the ADT-to-AADT conversion factor second to calculate the total entering AADT.

Intersection Name	Entering Link Dir	Link K-factor PM	ADT to AADT Factor	Entering Link Volume Daily Build (AADT)
Mainline & Ramp	NB	0.077	0.9707	40968
	WB	0.079	0.9707	8904
	Total			**49872**

From the project build analysis, the corresponding daily medium and heavy truck percentages are added to the spreadsheet which also calculates the remaining light vehicle proportion as shown below.

Lastly, the PM peak hour LOS is added to the spreadsheet as shown in the first intersection table above.

Intersection Name	Entering Link Dir	Daily Lt Veh Percent (decimal)	Daily Med Trk Percent (decimal)	Daily Hvy Trk Percent (decimal)
Mainline & Ramp	NB	0.789	0.040	0.171
	WB	0.789	0.040	0.171
	Total			

16.4.3 Mobile Source Air Toxicity (MSAT) Analysis

Qualitative MSAT Analysis

Most projects will fall into FHWA category of ‘project with low potential for MSAT effects and only require a qualitative MSAT analysis. Example projects include those that improve operations without adding substantial capacity, minor widening, or new/revised interchanges. Projects can also increase emissions or relocate the roadway closer to sensitive areas such as residential areas, schools, churches, parks, sport fields, etc. if the future design year AADT is projected to be less than 140,000. Traffic data for the qualitative analyses require a narrative discussion of existing and future traffic volumes, vehicle mix and overall traffic routing patterns between the existing conditions, future no-build, and future build alternatives. Information for this discussion should be available from previously published traffic analysis technical memoranda or reports for the project.

To help with the qualitative discussion, some additional data may be requested:

- Regional annual vehicle miles traveled (VMT) and average regional speed, OR
- Project link ADT developed from short-term counts) and average speed
- Percent diesel by link

The regional annual VMT should be obtained from the applicable regional (metropolitan) travel demand model by summing up the VMT calculated for each link. This value needs to be using the daily link volume factored to an annual value and the link length. A model request should be completed for metropolitan areas with ODOT-constructed travel demand models (i.e. Albany, Bend, Corvallis, Grants Pass, and Medford). Model requests for the larger metropolitan areas (i.e. Portland Metro, Eugene-Springfield and Salem-Keizer) should be directed to the controlling organizations (i.e. Metro, Lane Council of Governments, and Mid-Willamette Council of Governments). The existing year can use the base model scenario, the future no-build can use the future model scenario, and the build future can use a future model scenario modified with the project. Existing and future model scenario years should be factored to match the actual project years.

Average regional speed can be obtained by averaging the calculated (i.e. link length divided by link travel time) link speeds from a daily volume assignment in a travel demand model across the entire regional area (e.g. a metropolitan area or city UGB). Alternatively, link speed can be obtained from dividing link VMT by link VHT (vehicle-hours traveled). This will not capture the full congestion effects, but any specific congested areas will be heavily diluted as a regional look will include lots of uncongested links. It is not practical to do a link-by-link speed assessment/post-processing across an entire regional area. This would be repeated for the existing, future no-build and future build model scenarios.

Most of this, if not all, could be done within the scope of the model request leaving just the scenario comparisons and discussion to the analyst. The resulting VMT and speeds for each year scenario should be relatively compared (versus directly using the actual link values) to create a meaningful discussion basis.

Project link ADT for all the scenario years (i.e. minimum of existing, future no-build, and future build) would be ideally obtained from the traffic analysis directly or by converting the balanced hourly volumes to directional links and factoring them like what is done for noise traffic data in Section 16.3. Alternatively, a travel model daily volume assignment for the project study area or specific affected facilities could be used from the closest base/reference year, future no-build, and future build scenarios.

The regional travel demand model can be used to create an average speed for each available period and for each scenario year as done for the regional process above but by using a project subarea (see Chapter 8) instead of the full model coverage. Alternatively, individual travel model link speeds could be averaged across the project study area or specific affected facilities. The existing year average speeds should be obtained for the same time periods from probe data sources such as RITIS on a link basis. Posted speeds should not be used for an MSAT analysis. The combination of these will post-process the model speed data between the existing year and the future no-build and between the future no-build and the future build. This will create the most accurate information and allows the direct use of absolute values in the discussion.

Percent diesel by link is determined by multiplying the diesel fuel fractions in Exhibit 16-12 by the vehicle classification percentages for automobiles and light, medium and heavy trucks (see Page 16-12 & 16-13). Interpolate as necessary for years that fall in between the years in the table, but note that the relationship is not linear, so interim years will need to be estimated along the curve. Consult the ODOT Air Quality Unit for values to be used with future years beyond 2035.

Exhibit 16-12 Auto & Light, Medium & Heavy Truck Diesel Fuel Factors

Year	Diesel Fuel Fraction			
	Auto (Class 1-2)	Light Trk (Class 3)	Medium Trk (Class 4-5)	Heavy Trk (Class 6-13)
2020	0.010	0.040	0.720	1.000
2025	0.010	0.030	0.680	0.990
2030	0.010	0.030	0.612	0.941
2035+	0.000	0.030	0.505	0.880

Example 16-10 Qualitative MSAT Analysis

This example is a continuation from Examples 16-7 & 16-9. Part of the interagency coordination also suggested that this operational auxiliary lane project perform a qualitative MSAT analysis. This project will widen the footprint of the freeway cross-section to the east and west by a lane which will bring it closer to a residential area that exists on the east side. Future projected build volumes (see Example 16-7) are less than the 140,000 AADT threshold, so a quantitative MSAT analysis will likely not be applicable.

While the previous examples used post-processed screening-level and project volumes, this example will use the regional travel demand model outputs obtained from a model request as the data source. The scope of most MSAT analyses is too large for an explicit link-by-link project assessment unless the volume data is already available, so a travel demand model would be the best tool choice. The regional VMT was obtained from the travel demand model for the existing, future no-build and build scenarios. From the table, the VMT increased about 1.7% per year from 2019 to 2043 which is commensurate with the increasing growth in the local area. The effect of the project increases the regional VMT by 0.23% over the no-build. For speed, the model daily link speeds for the project area links (see Example 16-8) were averaged together for all three scenarios. The improvements increase the daily average speed in the project area by 26% over the no-build and 8 % over the existing conditions.

Scenario	VMT (vehicle-miles)	Average Speed (mph)
2019 Existing	1,159,614	50
2043 No-build	1,635,794	42
2043 Build	1,639,504	54

Quantitative MSAT Analysis – Link Screening

Few projects will be applicable to a quantitative MSAT analysis. Currently, only the Portland Metro area has AADT's over 140,000 (sections of I-5, I-84, I-205 and US26) that could trigger this kind of analysis. Project impacts need to be in populated areas and:

- Create or significantly change a major intermodal facility that has the potential to concentrate high levels of diesel particulate matter in a single location by using a substantial number or increase of diesel vehicles, OR
- Create substantial capacity to urban roadways where the AADT is projected to be greater than 140,000 in the future design year.

The air quality analyst will need to submit a methodology memorandum to ODOT Environmental Section and FHWA. A link-based screening-level traffic assessment is required to determine the overall MSAT study area. This screening assessment should use available project-specific information consistent with the traffic analysis done for other parts of the project. This should be a comparison of the future no-build with the future build alternative. The screening criteria (for more information see FHWA's *"Frequently Asked Questions for Conducting Quantitative MSAT Analysis for FHWA NEPA Documents"*) used needs to be clearly described in the methodology memorandum and should use some form of the following criteria:

- AADT changes of +/- 5% or more on congested roadway links of LOS D or worse
- AADT changes of +/- 10% or more on uncongested links of LOS C or better
- Travel time changes of +/- 10% or more
- Intersection delay changes of +/-10% or more

The actual criteria used could use all or some of the above, or different equivalent metrics depending on the scope of the project (e.g. AADT changes of +/- 5% on all links). For example, performing a separate HCM-based intersection analysis to determine the effect of delay on adjoining links could be very intensive versus using simplified outputs from a travel demand model. However, while link travel time is an available transportation model output, the typical response to congestion is limited and may not show the full impacts of an alternative.

At a minimum, base (existing) link travel times would need to be established using RITIS (i.e. probe data) then post-processed with the base and future no-build transportation model link travel times to obtain the future no-build travel time. Then the build travel time could be factored up from the no-build by comparing the future no-build and build model link travel times. This post-processing could be skipped if the transportation model was calibrated for speed, but this is not current practice.

Because of the typical large project size and potential extensive impacts of a MSAT analysis, the link screening criteria shall be based on travel demand model outputs. The future no-build and build alternative volumes can be compared with a difference plot. to quickly identify the affected links. These plots can be used in multiple ways. Exhibit 16-

13 shows the change in volumes with one color representing increased volumes while the other color is a decrease. Equations can also be used with these, so one criterion could be showing any link with more than a 10% change in daily traffic or show a range for identifying demand-to-capacity thresholds to quickly highlight links. If the uncongested/congested sub-criteria are desired, then a 0.80 demand-to-capacity (d/c) threshold should be used to differentiate uncongested (LOS C or better) from congested (LOS D or worse) links.

Exhibit 16-13 Difference plot example

If a Visum-based model (whether trip-based or activity based, see Chapter 17) or sub-area (see Chapter 8) with some level of intersection detail (i.e. lane configurations, green time/cycle ratios, etc.) is used, then the intersection (i.e. nodal) delay could also be compared to identify all the adjoining affected links. The tagged links should be captured in a .csv formatted file so that further comparisons can be done in the model assignment software or in a spreadsheet can be done or can be used to as a start on the traffic data output.

This study area will include all the project roadway links and any additional links that experience substantial change in MSAT emissions. This is not a geographic-based study area as there could be roadways affected by the project considerably spaced from the actual project area especially if the project creates or enhances alternative routes or diversions. Projects that do not create substantial diversions or alternative routes such as a case of an isolated bridge crossing will have a smaller affected study area. These extra links could be wide-ranging and discontinuous from each other but should be in the general vicinity to be affected by the project. Identified links that are far removed from the project area should be investigated to see if it is a true impact or not.[2]

[2] FHWA HEP-15-056, FAQ on conducting Quantitative MSAT Analysis for FHWA NEPA Documents, 2016.

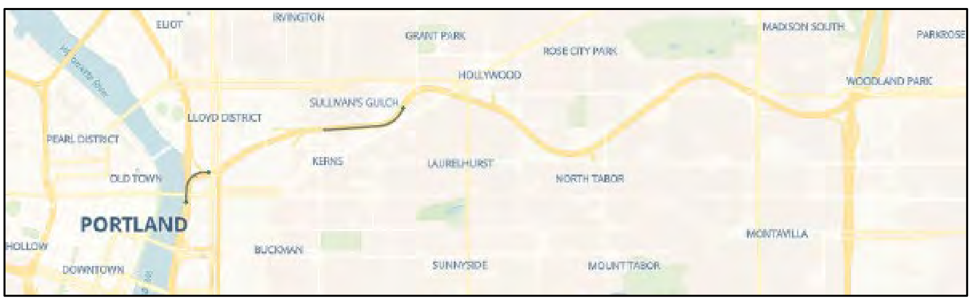
Example 16-11 Quantitative MSAT Screening

This and following MSAT examples use a “theoretical” widening of I-84 in Portland between I-5 and I-205 to four through lanes in each direction. This section of I-84 contains some of the highest volumes in the region at well over the 140,000 AADT MSAT analysis threshold. A project of this type that adds substantial system capacity to a congested section of freeway would be expected to have a noticeable impact on the surrounding area.

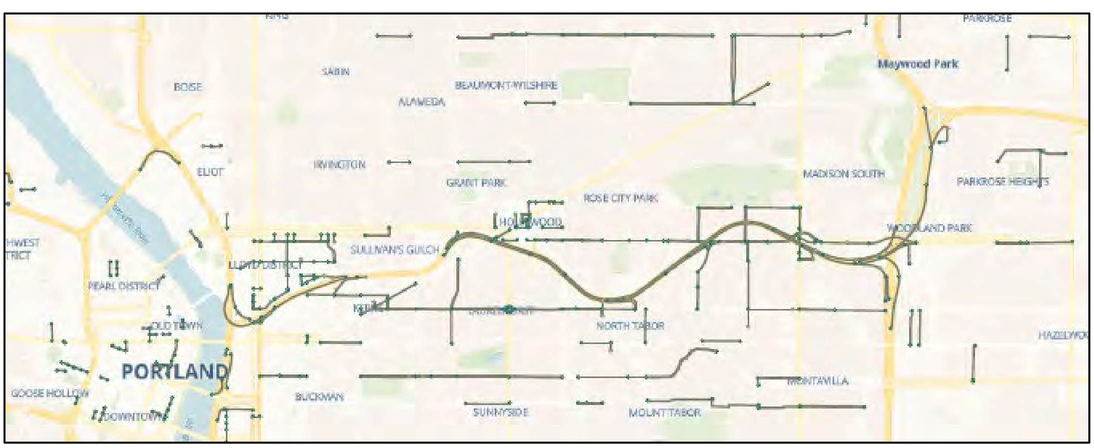
The (Portland) Metro travel demand model was used for the evaluation. The volume – level of service (LOS) criteria was used as this was thought to work the best with a travel demand model inputs and outputs. Link travel time could be generated from a model, but link congestion and the resulting delay tends to be understated so links could be inadvertently missed if this was the criterion used. The intersection volume delay criterion requires more detailed operational data than can be provided in a travel demand model.

A pair of volume difference plots were created which compare the 2045 no-build and build scenarios together with the capacity and volume change filtering conditions:

- Show links with demand to capacity ratios greater or equal to than 0.80 (approximates LOS C/D threshold) for the congested condition and where the daily volume changed by at least +/- 5% **OR**



- Show links with demand to capacity ratios less than 0.80 for the uncongested condition and where the daily volume changed by at least +/- 10%



Between the two screens, all the I-84 project links should be included in the quantitative (after all appropriate review and coordination) along with portions of I-5, I-205, I-405 and certain surrounding intersecting and parallel streets. For the following examples, a smaller subset of these roadway sections will be used to illustrate the quantitative method.

Quantitative MSAT Analysis

Once the affected directional links have been identified, the traffic analyst should coordinate with ODOT Environmental Section and the air quality analyst to review the overall traffic data request and finalize the MSAT methodology and data to be used.

> *The use of travel model-based data to simplify the overall calculation process for all scenarios (e.g. existing, future no-build, build) does mean that all the data is weekday-based. Obtaining weekend-based data would be very difficult and expensive for an MSAT-type scope. The most rigorous solution would have calibrated weekend model scenarios, but these would be special requests and likely not doable unless the model owner (or requestor) had access to numerous weekend-weekday counts (like 7-day hose tube counts) to do the calibration. Other than that, the same 7-day counts could be used to create conversion factors to translate weekday volumes to weekend before calculating VMT. This extra post-processing step would be required for all model scenarios and would be likely not be as accurate as trips and trips patterns are not the same on the weekend, so a simple conversion factor will not be able to fully account for all the changes.*

Data needs for quantitative MSAT analyses include:

- Unique link identifier
- Link length (mi)
- AADT
- Vehicle-miles traveled (VMT)
- MOVES road type (1-5).
  - Off-network (1)
  - Rural restricted (i.e. access via interchanges) (2)
  - Rural unrestricted (3)
  - Urban restricted (4)
  - Urban unrestricted (5)
- Average speed hourly fractions (%)
- VMT hourly fractions (%)

- Percent light vehicles (Class 1-3), medium trucks (Class 4-5), and (Class 6-13) heavy trucks (for use in determining MOVES “source types”)

The following sections show the process for creating the VMT data for the existing conditions, future no-build, and future build scenarios. Example 16-11 started in the previous section is extended through all the scenarios to illustrate the calcaution process. A .zip file of all of the Excel workbooks used to create these examples is available on the Technical Tools page under the Volume Development dropdown for use as a sample file. The sheer scope of an MSAT-type analysis will likely require different workflows, tools, and input/output formats beyond these sample files, so these are not intended for use as step-by-step guides. Appendix 16C shows sample future-year traffic data inputs for MOVES showing VMT fractions by various dimensions.

Quantitative MSAT Analysis – Existing Conditions

The overall process is as follows:

1. Create a spreadsheet/database with the unique directional link numbering scheme, link name/description, link length and roadway type.

2. Determine the link VMT for all tagged links (or pre-determined districts or subareas if doing an area-wide analysis) by using a model daily volume assignment. The model year used should be the base or reference year closest to the project base year. If the model year used matches the project base year, then the model link values can be used as-is. If the model year is different, then the model year link volumes should be adjusted/interpolated to match the project base year.
3. Run a peak period assignment for each of the identified periods in the model and calculate VMT for the same links as for the daily computed in Step 2. Activity-based models (ABM) will have five periods (Early AM, AM peak, Midday, PM peak, and evening). Trip-based models will have at least two periods but could have more depending on the jurisdiction (AM peak, PM peak, Midday if available, other periods if available).
4. For ABMs, sum the peak period VMT in Step 3 and subtract it from the daily VMT in Step 2 to determine the night period VMT for each link. For trip-based models subtract the sum of the link VMT in Step 3 from the daily VMT in Step 2 to calculate the VMT daily remainder (up to 22 hours’ worth) for each link.

5. For ABMs, use an averaged volume profile (or profiles as necessary to capture study area variations across the different facility types/functional classes) ideally generated from multiple representative (i.e. on-site or characteristic-based; see Chapter 5) ATR or 24-hour count locations to split each of the six link VMT periods (i.e. the five daily periods from Step 3 and the night period from Step 4) into hours. For trip-based models use the volume profile(s) described previously

to split the VMT remainder into the remaining hours not covered by the available hourly assignments.

Counts should be within five years of the existing analysis year, so overall volume patterns are representative. Since proportions are what are needed, the actual count value does not matter. These counts should be directional, if possible, to best match up with the known model assigned hours and have at least hourly breakdowns. Ideally, volume profiles should be developed for each facility type (i.e. freeway, ramps, arterials, etc.). More could be developed based on magnitude of volumes, functional class, or number of lanes if desired. Counts with similar base characteristics should be averaged together.

The daily volume profiles will need to be readjusted to exclude the periods or hours covered in Step 3, so the proportions just cover splitting up a particular VMT period or remainder. The adjusted volume profiles should be expressed in decimal percent fractions (e.g. 0.034) for each hourly fraction of the day and must sum to 1.000. The OTMS count report "Traffic Distribution by Hour" is the daily profile which is exportable in csv or Excel format which could be used to streamline these calculations.

Add the determined link VMT by hour to the spreadsheet to set up for the further fractional splitting in following steps. While formatting these VMT calculations can work by having the hours by columns, it will be more efficient to have the hours by rows as most data (e.g. count volumes, vehicle classification, speed data) is by column. This will avoid spending time needed to transpose the data.

6. Assign vehicle class counts to each specific facility or representative facility groups (i.e. subgroups like local arterials). Where possible, these should be the same counts obtained in Step 5 above. Ideally, single facilities should have a count for every section between major freeway or highway junctions. Vehicle class counts are ideally based on the FHWA 13-classes (See Chapter 3), but lesser number of classes can also work if motorcycles/passenger cars/passenger trucks (i.e. light vehicles; Classes 1-3), medium trucks/buses (Classes 4 & 5) and heavy trucks (Classes 6-13) can be identified. Vehicle class sources (e.g. counts, automatic vehicle classifiers) should have the longest duration possible (ideally 24-hr).

7. Determine the light vehicles (Class 1-3), medium trucks (Class 4-5) and heavy trucks (Class 6-13) vehicle percentages for each facility section or facility group by hour (if possible). Full day counts can determine specific percentages for each hour and allow creation of relative percentages through a vehicle class daily profile for applying to count locations or facility groups that do not have full daily coverage (this would allow use of short-term counts common on local streets). The three class groupings must sum to 100%. Average percentages from multiple counts for a facility group. Express the percentages in decimal form.

8. Multiply the hourly link VMT from Step 5 by each of the three hourly vehicle class group percentages determined in Steps 6 & 7 to determine the hourly VMT by vehicle class groupings.

9. For each of the specific facility sections or facility groups noted above use available probe-based data (e.g. RITIS), determine an average speed by hour for representative groups of links corresponding to a roadway section. Representative RITIS XD segment groups should be identified per direction (XD segments are directional) that correspond to roadway sections between major intersections or interchange ramps, to reduce as much as possible the number of required link speed calculations. Saving the XD segment selection will reduce time when repeating this with historical speeds in the future no-build. See the Oregon RITIS Handbook for more information on the analytical tools and settings.

   The speed calculations should be based on an average month. The ATRs in the project area, a representative ATR, or applicable seasonal trends should be checked to verify when to see when seasonal adjustments would be at 1.0 (typically April or October) which indicates average daily conditions.) Within RITIS, group together applicable XD segments as much as possible with links, but weight averaging by length is acceptable if a link bridges two or more XDs.

   Average speed can be best obtained using the Probe Data Analytics Performance Charts tool. Using the Map function to create a polygon around the selected road section, then deleting certain XD segments is best for identifying the directional analysis sections. Export the speed results for the selected segment, month, and time periods as an XML file which can be opened and edited in Excel.

   The Performance Charts tool will also produce a graphical speed profile if desired. Alternatively, the Massive Data Downloader (MDD) tool can be used if scripting and data analysis tools (outside of Excel as there will be too many records) are to be used to determine speeds over a large quantity of links. This will create a speed profile for each section or group.

10. Identify the applicable speed bin (labeled #1 through #16 below) for each hour for each link from the list below based on the speeds determined in Step 9. This is easily done using a lookup table in Excel to match up the reported average speeds and the speed bins.
    1. Speed < 2.5 mph
    2. 2.5mph ≤ Speed < 7.5mph
    3. 7.5mph ≤ Speed < 12.5mph
    4. 12.5mph ≤ Speed < 17.5mph
    5. 17.5mph ≤ Speed <22.5mph
    6. 22.5mph ≤ Speed < 27.5mph
    7. 27.5mph ≤ Speed < 32.5mph
    8. 32.5mph ≤ Speed < 37.5mph
    9. 37.5mph ≤ Speed < 42.5mph

    10. 42.5mph ≤ Speed < 47.5mph
    11. 47.5mph ≤ Speed < 52.5mph
    12. 52.5mph ≤ Speed < 57.5mph
    13. 57.5mph ≤ Speed < 62.5mph
    14. 62.5mph ≤ Speed < 67.5mph
    15. 67.5mph ≤ Speed < 72.5mph
    16. 72.5mph ≤ Speed

11. For each link, parse the hourly link VMT by vehicle class from Step 8 into each of the 16 speed bins. This represents the finest level of VMT disaggregation by link.

12. Consolidate the binned link VMT by each roadtype. Sum the VMT for an individual hour and classification across all links of that roadtype. For example, for Roadtype 4 (urban restricted), light vehicle class, sum all link VMT for 12-1 AM for each of the 16 speed bins. Then repeat for 1-2 AM throughout the rest of the day. Then the same process would be repeated for the medium and heavy vehicles for Roadtype 4. This would continue to be repeated through all applicable roadtypes for the project.

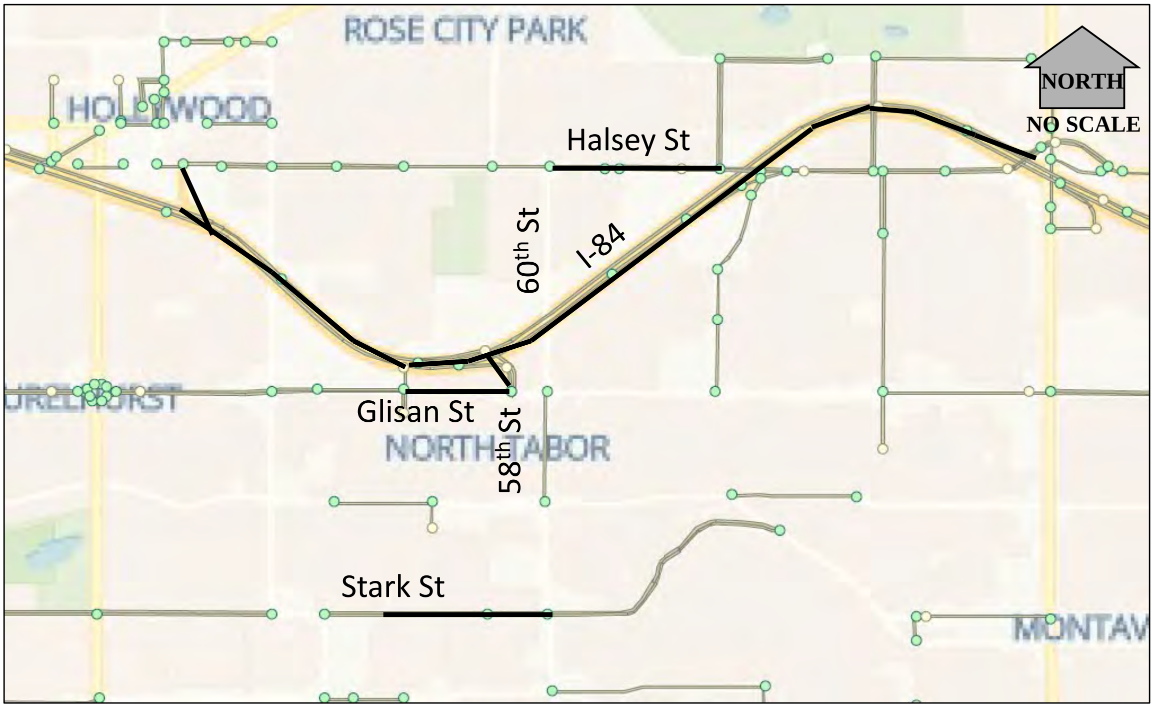
13. For each roadway type, sum the VMT across all the classes and speed bins to create an hourly VMT total. Sum this hourly VMT to create a roadtype VMT subtotal. Divide each roadtype VMT subtotal by the total sum of all the roadtype VMT's to create decimal fractions for each.

14. Sum up each of the hourly VMT by roadtype and classification records created in Step 12 to create an hourly total.

15. Divide the hourly VMT by roadtype, classification, and speed bin by the total hourly VMT by roadtype by classification from Step 13 to determine the VMT speed-based fractions in decimal form. Each hour in the resulting table needs to sum to 1.0000 (i.e. 100%).

Example 16-12 Quantitative MSAT Existing Conditions

This example continues the sample project started in Example 16-11. Traffic data will need to be developed for all chosen sections. A selection of roadway sections on mainline I-84, ramps and parallel roadways (shown in black) will have the VMT data developed in the next set of examples as shown in the figure below. Steps indicated in this example below correspond with the steps listed for the overall process above. Note that this only a small portion of the total number of links (shown in gray) that could be selected for this sample project.



Step 1

For each roadway link, a unique link ID was created (see Section 16.3.1 for link numbering guidance) along with the corresponding link description and roadway type. Note that ramps are treated the same as freeway links. From the travel model output, the link length and daily volume for each link were recorded as shown in the table below. Some roadway links are made up of multiple model links, so ADT was weight-averaged across those links.

Link ID	Link Description	Road type	Link length (mi)	ADT (vpd)
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp	4	0.76	97000
101	I-84 EB; 58th St off-ramp to 60th St on-ramp	4	0.40	85002
102	I-84 EB; 60th St on-ramp to Halsey St off-ramp	4	0.21	94344
200	I-84 WB; Halsey St on-ramp to 58th St on-ramp	4	1.40	88645
201	I-84 WB; 58th St on-ramp to 43rd St off-ramp	4	0.37	101779
500	58th St EB off-ramp	4	0.18	11998
501	58th St WB on-ramp	4	0.27	13134
502	60th St EB on-ramp	4	0.19	9342
503	Halsey St on-ramp	4	0.19	5825
504	43rd St off-ramp	4	0.36	11999
600	Glisan St EB; 55th St to 58th St	5	0.24	4684
601	Glisan St WB; 58th St to 55th St	5	0.24	5415
602	Halsey St EB: 60th St to 67th St	5	0.38	2862
603	Halsey St WB; 67th St to 60th St	5	0.38	5320
604	Stark St EB; 55th St to 60th St	5	0.50	1914
605	Stark St WB; 60th St to 55th St	5	0.50	2344

Steps 2-4

The link length and the ADT are multiplied together to obtain the total daily VMT for each link. From this value, known hourly VMT from each peak period needs to be subtracted. For the Metro model, any hour can be assigned, however, while doing 24 individual hours of assignments are possible, it is not practical from the modeler perspective since each hourly run takes almost a day. Representative hours were chosen for each of the five peak periods (Early, AM peak, Midday peak, PM peak, and Evening) as a balance between the model request effort and the analysis fidelity.

The hourly volume for each link for each of the hours is multiplied by the link length to determine VMTs shown in the table below. Links made up of the multiple model links were weight-averaged. The sum of the five hours is subtracted from the daily VMT calculation to determine the 19 hours' worth of VMT remainder that needs to be allocated over the rest of the day.

Link ID	DVMT (veh-mi)	Early VMT (veh-mi)	AM VMT (veh-mi)	Mid VMT (veh-mi)	PM VMT (veh-mi)	Eve VMT (veh-mi)	Rem VMT (veh-mi)
100	73720	2826	3995	4280	4572	3289	54758
101	34001	1292	1854	1982	2168	1925	24779
102	19812	736	1078	1146	1269	1011	14573
200	124103	7512	8448	7064	7113	5426	88539
201	37658	2177	2398	2182	2202	1713	26987
500	2160	88	112	122	107	124	1607
501	3546	140	121	230	235	203	2617
502	1775	52	94	95	118	92	1323
503	1107	47	57	69	70	59	806
504	4320	192	149	274	285	237	3183
600	1124	15	29	60	118	55	848
601	1300	70	149	63	56	30	930
602	1088	8	32	65	179	26	777
603	2021	75	303	95	118	33	1398
604	957	6	35	61	145	26	683
605	1172	40	154	61	84	23	811

Step 5

The next step is to identify representative ATR (either on-site or characteristic-based) or available 24-hour counts in the project area that can be used to develop a volume profile. A 24-hour count was located on I-84 in the example section along with all the identified ramps. ADT-only counts were found on the arterial streets which would be not sufficient to develop a volume profile, so a substitute count on US26 (Powell Blvd) will be used to develop factors for the surface streets. The ramp volume proportions were divided into eastbound and westbound directions and averaged together. All the three directional volume facility profiles (e.g. freeway, ramps, and arterials) were adjusted by removing the five assigned hours and re-calculating so the totals still summed to 100%. The hourly volume profile factors were then applied to the remaining VMT portion to distribute it across the other 19 hours as shown in in the partial table below.

Link ID	Link Description	Remaining DailyVMT	12AM	1AM	2AM	3AM	4AM
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp	54758	970	554	501	605	994
101	I-84 EB; 58th St off-ramp to 60th St on-ramp	24779	439	251	227	274	450
102	I-84 EB; 60th St on-ramp to Halsey St off-ramp	14573	258	147	133	161	265
200	I-84 WB; Halsey St on-ramp to 58th St on-ramp	88539	1407	824	686	1092	2506
201	I-84 WB; 58th St on-ramp to 43rd St off-ramp	26987	429	251	209	333	764
500	58th St EB off-ramp	1607	13	8	8	8	12
501	58th St WB on-ramp	2617	22	14	14	15	40
502	60th St EB on-ramp	1323	11	6	6	6	10

Steps 6-8

The next step was to work towards creating the classification breakouts for light (Class 1-3), medium (Class 4-5) and heavy vehicles (Class 6-13). The same directional counts on I-84 that provided the volume profile also had the full FHWA 13-class breakdown, so it will be used for the freeway segments. This will allow the vehicle proportions to vary by the hour. The number of light, medium, and heavy vehicles in each hour were divided by the total number of vehicles in each hour to create each proportion. This proportion was multiplied by the total hourly VMT to create the VMT by class as shown below in the partial table. It should be noted that the data was transposed to better match up with available inputs, so that the hourly data is by row instead of by column in the previous step.

Link ID	Link Description	Time	VMT by Hr	VMT by Class by Hr		
				Light	Medium	Heavy
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp					
		12AM	970	923	16	30
		1AM	554	500	14	40
		2AM	501	445	14	42
		3AM	605	542	14	49
		4AM	994	913	24	56
		5AM	2826	2626	76	124
		6AM	2983	2794	99	90
		7AM	3995	3767	128	99
		8AM	4050	3811	151	88

The ramp counts were volume only, but there was some high-level daily truck AADT information available for single and multiple unit trucks. In this case, single unit trucks were assumed to be equivalent to medium trucks and multiple-unit trucks were assumed to be equivalent to heavy trucks. The ramp counts were averaged together as they were generally similar which results in 2.8% medium trucks and 1.2% heavy trucks. Ramps have a mix of characteristics of the freeway and connecting arterials so a comparison average of the arterial and freeway segments together has 2.3% medium and 1.4% heavy trucks which is consistent (if no ramp truck information were available then this value could be used). In this case, the vehicle proportions will be flat across the day although the number of vehicles will change according to the available daily ramp volume profile. These proportions were multiplied by the daily volume ramp profile to create the ramp VMT by class as shown below.

Link ID	Link Description	Time	VMT by Hr	VMT by Class by Hr Light	Medium	Heavy
500	58th St EB off-ramp					
		12AM	13	13	0	0
		1AM	8	7	0	0
		2AM	8	7	0	0
		3AM	8	7	0	0
		4AM	12	12	0	0
		5AM	88	84	2	1
		6AM	86	83	2	1
		7AM	112	107	3	1
		8AM	162	155	5	2

For the arterial system, there was no available classification information. The same count on Powell Boulevard (US 26) that was used for developing the volume profile also had the full hourly classification data. Overall, it can be expected that truck proportions on city streets will be around a few percent. This count had an average daily truck percentage of 2.4% which is in the expected range, so it can be used as a proxy for the other sites on an hourly basis. The vehicle proportions were computed the same way as for the freeway segments. Note that variations in rounding based on calculated or value-only spreadsheet cells will cause some lines in the below table not to add up cleanly.

Link ID	Link Description	Time	VMT by Hr	VMT by Class by Hr Light	Medium	Heavy
600	Glisan St EB; 55th St to 58th St					
		12AM	14	13	0	0
		1AM	8	8	0	0
		2AM	8	7	0	0
		3AM	6	6	0	0
		4AM	5	4	1	0
		5AM	15	13	1	0
		6AM	21	19	1	0
		7AM	29	28	1	0
		8AM	65	62	2	0

Step 9

From a nearby ATR, April was determined to be the representative average month. The RITIS Performance Chart tool was used to develop the average speed by hour for April 2023 as this matches up the best with the available volume profile and classification data. The tool was run with matching as much as possible the link with the identified directional XD segments.

Step 10

Some links match up perfectly with the XD segments such as for Link 100 (I-84 EB from the Cesar Chavez Boulevard on-ramp to the 58th Street off-ramp). Other links do not, such as the remaining two eastbound I-84 links (Link 101 and 102) are only covered by a single XD segment. The westbound directional link (Link 200) is covered by three XD segments. The data was downloaded to Excel and a lookup table was used to match the speed bins with the reported average speeds as shown below in the partial table.

Link ID	Link Description	Time	Ave Speed (mph)	Speed Bin ID#
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp			
		12AM	61.5	13
		1AM	61.4	13
		2AM	60.6	13
		3AM	61.4	13
		4AM	62.8	14
		5AM	62.8	14
		6AM	61.5	13
		7AM	58.6	13
		8AM	49.4	11

Step 11

Once the hourly speed bins were identified for all the links, the light, medium and heavy vehicle hourly VMT totals were parsed into each of the 16 speed bins for each link. The partial table below shows the distribution for light vehicles for Link 100 between 12- 8 AM.

Link ID	Time	VMT Light Veh	VMT by Hr by Speed Bin - Light Veh						
			8	9	10	11	12	13	14
100									
	12AM	923						923	
	1AM	500						500	
	2AM	445						445	
	3AM	542						542	
	4AM	913							913
	5AM	2626							2626
	6AM	2794						2794	
	7AM	3767						3767	
	8AM	3811				3811			

Step 12

Next, the link VMT data was consolidated down to the roadtype by classification level. All the individual hours (i.e. 12-1 AM, 1-2 AM, etc.) were summed for each roadtype and classification (i.e. Roadtype 4 & Light vehicle, Roadtype 4 & Medium vehicle, etc.) as shown in the partial table below.

Roadtype	Class	Time	VMT by Hr by Speed Bin 5	6	7	8	9	10	11
4	Light	12AM	0	13	0	47	7	10	0
4	Light	1AM	0	7	0	30	4	6	0
4	Light	2AM	0	0	7	30	0	10	0
4	Light	3AM	0	0	7	33	0	11	0
4	Light	4AM	0	0	12	85	12	9	0
4	Light	5AM	0	0	84	185	180	50	0
4	Light	6AM	0	0	83	424	0	68	7594
4	Light	7AM	171	107	10425	0	90	0	0
4	Light	8AM	63	7561	0	0	128	0	6550

Step 13

The hourly VMT for each roadtype from Step 12 was summed across all the speed bins to create a subtotal. Each hourly subtotal was summed across the day to create a VMT total by roadtype. Each roadtype VMT total was divided by the total sum of all roadtype VMTs to create a decimal fraction.

Roadtype	Roadtype VMT Fraction
4	0.9754
5	0.0246

Steps 14-15

Each hour in the table above was summed across all speed bins to create a total. This was used to create the decimal speed fractions by dividing the hourly roadtype by classification by speed VMT data by the total hourly by roadtype by classification VMT. The partial table below shows the corresponding speed fractions for the previous table. For example, the highlighted 10,425 VMT for 7-8 AM for Speed Bin #7 (27.5 to 32.5 mph) results in a speed fraction of 0.6018. This means that for Roadtype 4 (urban restricted; freeway) between 7-8 AM for light vehicles (Class 1-3), 60.18% are traveling between 27.5 and 32.5 mph. Each row in the table will sum to 1.0000 (100%).

Roadtype	Class	Time	VMT by Hr by Speed Fractions 5	6	7	8	9	10	11
4	Light	12AM	0.0000	0.0039	0.0000	0.0140	0.0021	0.0030	0.0000
4	Light	1AM	0.0000	0.0037	0.0000	0.0161	0.0021	0.0032	0.0000
4	Light	2AM	0.0000	0.0000	0.0045	0.0191	0.0000	0.0064	0.0000
4	Light	3AM	0.0000	0.0000	0.0031	0.0147	0.0000	0.0049	0.0000
4	Light	4AM	0.0000	0.0000	0.0025	0.0180	0.0025	0.0019	0.0000
4	Light	5AM	0.0000	0.0000	0.0059	0.0129	0.0126	0.0035	0.0000
4	Light	6AM	0.0000	0.0000	0.0054	0.0277	0.0000	0.0044	0.4968
4	Light	7AM	0.0099	0.0062	0.6018	0.0000	0.0052	0.0000	0.0000
4	Light	8AM	0.0043	0.5212	0.0000	0.0000	0.0088	0.0000	0.4515

Quantitative MSAT Process – Future No-build

The future no-build computation process is similar to the existing conditions. The future no-build data is based off relational pivots from the existing information. This mainly involves updating the VMTs from future model data, and estimating changes to hourly volumes, vehicle classifications and speeds.

The overall process is as follows:

1. Copy the unique directional link numbering scheme, link name/description, link length and roadway type to a new spreadsheet or database file from the existing conditions.

2. Determine the link VMT for all tagged links (or pre-determined districts or subareas if doing an area-wide analysis) by using a model daily volume assignment. The future model year used should be the closest to the project future (horizon) year. If the future model year used matches the project future year, then the model link values can be used as-is. If the model year is different, then the model year link volumes should be adjusted/interpolated to match the project future year.

3. Run a peak period assignment for each of the identified periods in the model and calculate VMT for the same links as for the daily computed in Step 2. Activity-based models (ABM) will have five periods (Early AM, AM peak, Midday, PM peak, and evening). Trip-based models will have at least two periods but could have more depending on the jurisdiction (AM peak, PM peak, Midday if available, other periods if available).

4. For ABMs, sum the peak period VMT in Step 3 and subtract it from the daily VMT in Step 2 to determine the night period VMT for each link. For trip-based models subtract the sum of the link VMT in Step 3 from the daily VMT in Step 2 to calculate the VMT daily remainder (up to 22 hours' worth) for each link.

5. Obtain past hourly counts at each of the ATR or count sites used in the existing conditions analysis going as far back in the past as readily available. Ideally, this would be at least 10-15 years in the past or as far back as going forward to the future year (i.e. obtaining a count from 2003 corresponding to a 2023 base year and a 2043 future year. Skip the 2020 and 2021 pandemic years as counts will be substantially low and patterns significantly different. These counts ideally would be the same month and day of week as the ones done for existing conditions for as much consistency as possible, but holidays and known major regional events should be avoided. Using counts that can be averaged over multiple days would be best. While one past count is the minimum, having additional years reduces the potential for large shifts or negative values if the hourly proportions are shown to decrease over time. Any negative values will need to be manually corrected back

to zero which will mean that the profile will only sum close to 100% and not equal it. These cases would be likely in small volume hours so the overall impact would not be substantial. Convert each hour into a daily fraction expressed in decimal percents. Using linear extrapolation of the historic daily fractions, forecast the future hour-based daily fractions. If the forecasted future daily fractions are not significantly different from the existing, then continue to use the existing conditions daily fractions for the future-no-build analysis.

6. For ABMs, use the averaged volume profile hourly fractions from Step 5 to split each of the six link VMT periods (i.e. the five daily periods from Step 3 and the night period from Step 4) into hours. For trip-based models use the volume profile(s) described previously to split the VMT remainder into the remaining hours not covered by the available hourly assignments. The volume profiles should be expressed in decimal percent fractions (e.g. 0.034) for each hourly fraction of the day.

7. Add the determined link VMT by hour in Step 6 to the spreadsheet to set up for further fractional splitting in following steps.

8. Obtain past years (the same historic interval used in Step 5) state highway vehicle classifications for segments that match the existing year vehicle classification count locations from ODOT's TSM unit. These counts ideally would be the same ones that were used to develop the hourly profiles in Step 5. Using linear extrapolation (e.g. using Excel's FORECAST.LINEAR function) with available past and existing year vehicle classification group percentages, forecast the future year percentages. Relatively steep positive or negative trendlines between the historic reference year and the existing year may result in proportions higher than 1.000 or less than zero. These will need to be manually adjusted to correct the errors. Having more than two count years may mitigate some of these errors but will increase the data and time needs. Continue to use the existing year percentages if the forecasted future is not significantly different. Alternatively, if a noise analysis has been conducted, use that project build final link vehicle classifications in this step for consistency.

9. Use the vehicle class groupings determined in Step 8 and multiply by the hourly link VMT from Step 6 to determine the hourly VMT by vehicle class.

10. As was done with the existing conditions, use RITIS to find the earliest year available (e.g. 2016) and use the same XD segments. Within RITIS, group together applicable XD segments as much as possible with links, but weight averaging by length is acceptable if a link bridges two or more XDs. Average speed can be best obtained using the Probe Data Analytics Performance Charts tool. Using the Map function to create a polygon around the selected road section, then deleting certain XD segments is best for identifying the directional analysis sections. Export the speed results for the selected segment, month, and time periods as an XML file which can be opened and edited in Excel. The

Performance Charts tool will also produce a graphical speed profile if desired. Alternatively, the Massive Data Downloader (MDD) tool can be used if scripting and data analysis tools (outside of Excel as there will be too many records) are to be used to determine speeds over a large quantity of links. This will create a speed profile for each section or group.

11. Use forecasting functions like was done with the vehicle class with the existing year speed and the earliest base speed available and project the estimated future year speeds. Currently with around 10 years of available past speeds, projections will be generally reliable for around 10 years in the future (i.e. 2016 base speeds and 2025 existing year could produce 2035 future year speeds). For travel demand models, extrapolated data is generally acceptable to around five years extra (i.e. to 2040). However, speed projections do not necessarily continue at a linear rate over time, but the rate will tend to decrease, so the estimates may be too high or low, so the maximum future year (e.g. 2040 in this case) may be thought of as equivalent to horizon years beyond this date (e.g. 2045 for the corresponding 20-year design/horizon year that would normally be with a 2025 existing year).

12. Calculate the free-flow speed for each facility by adding five mph to each roadway speed limit (e.g. 60 mph for a 55-mph facility). Compare the free-flow speed with the projected future year speeds from Step 11 and reduce any speeds that exceed that value to the free-flow speed. In addition, use a practical crawl speed limit of 7 mph for any speeds that are projected to be less than this value.

13. Identify the applicable speed bin for each hour for each link from the list below based on the refined speeds determined in Step 12. This is easily done using a lookup table in Excel to match the reported average speeds and the speed bins.

1. Speed < 2.5 mph
2. 2.5mph ≤ Speed < 7.5mph
3. 7.5mph ≤ Speed < 12.5mph
4. 12.5mph ≤ Speed < 17.5mph
5. 17.5mph ≤ Speed <22.5mph
6. 22.5mph ≤ Speed < 27.5mph
7. 27.5mph ≤ Speed < 32.5mph
8. 32.5mph ≤ Speed < 37.5mph
9. 37.5mph ≤ Speed < 42.5mph
10. 42.5mph ≤ Speed < 47.5mph
11. 47.5mph ≤ Speed < 52.5mph
12. 52.5mph ≤ Speed < 57.5mph
13. 57.5mph ≤ Speed < 62.5mph
14. 62.5mph ≤ Speed < 67.5mph
15. 67.5mph ≤ Speed < 72.5mph
16. 72.5mph ≤ Speed

14. For each link, parse the hourly link VMT by vehicle class from Step 8 into each of the 16 speed bins. This represents the finest level of VMT disaggregation by link.

15. Consolidate the binned link VMT by each roadtype. Sum the VMT for an individual hour and classification across all links of that roadtype. For example, for Roadtype 4 (urban restricted), light vehicle class, sum all link VMT for 12-1 AM for each of the 16 speed bins. Then repeat for 1-2 AM throughout the rest of the day. Then the same process would be repeated for the medium and heavy vehicles for Roadtype 4. This would continue to be repeated through all applicable roadtypes for the project.

16. For each roadway type, sum the VMT across all the classes and speed bins to create an hourly VMT total. Sum this hourly VMT to create a roadtype VMT subtotal. Divide each roadtype VMT subtotal by the total sum of all the roadtype VMT's to create decimal fractions for each.

17. Sum up each of the hourly VMT by roadtype and classification records created in Step 15 to create an hourly total.

18. Divide the hourly VMT by roadtype, classification, and speed bin by the total hourly VMT by roadtype by classification from Step 16 to determine the VMT speed-based fractions in decimal form. Each hour in the resulting table needs to sum to 1.0000 (i.e. 100%).

Example 16-13 Quantitative MSAT Future No-build Conditions

This example continues the sample project analysis from Example 16-12. The same link sections and attributes such as link number, description and length were copied from the existing conditions. Indicated steps in the example correspond with the steps in the procedure above.

Steps 1-4

Daily and hourly volumes for the five assigned periods (5AM, 7AM, 12PM, 5PM, and 7PM) were captured from a future no-build model assignment. The link length and the ADT were multiplied together to obtain the total daily VMT for each link as shown below. From this value, the known hourly VMT from the peak periods was subtracted from the daily total as was done for the existing conditions to come up with the remaining VMT to be distributed over the remaining 19 hours of the day.

Link ID	DVMT (veh-mi)	Early 5A VMT (veh-mi)	AM 7A VMT (veh-mi)	Mid 12P VMT (veh-mi)	PM 4P VMT (veh-mi)	Eve 7P VMT (veh-mi)	Rem VMT (veh-mi)
100	91889	4150	5261	5428	5533	3938	67580
101	42865	1966	2508	2496	2628	2289	30978
102	24694	1097	1425	1428	1526	1202	18016
200	148333	8683	9828	8282	8641	5943	106956
201	44868	2562	2778	2583	2640	1896	32409
500	2474	98	117	162	128	145	1824
501	4134	195	131	288	260	237	3023
502	1988	59	98	106	133	103	1489
503	1705	58	68	127	135	69	1248
504	5125	273	224	310	322	276	3721
600	1120	14	21	61	104	54	866
601	1058	44	138	67	51	33	725
602	1249	9	33	84	187	24	911
603	2090	52	281	112	150	35	1460
604	765	4	22	46	108	20	565
605	844	19	112	43	63	12	595

Step 5

Past counts were located for the same sites for the existing conditions. The earliest counts were 2008 for the ramps and freeway segments and 2012 for the arterial segments. These will be used to create a trend line to project existing condition attributes with the first being the volume profile.

Step 6

For each of the freeway, ramp and arterial volume profiles, the hourly proportions were calculated for the earlier counts as done for the existing conditions. Then the proportions were adjusted so they summed to 100% after removing the five assigned hours since the VMT is known for those hours. These adjusted proportions for 2008 (for freeway and ramps) or 2012 (arterials) along with the 2023 adjusted proportions done for the existing conditions were used to establish a linear forecast trendline to 2045 (i.e. using the "FORECAST.LINEAR" Excel function). The partial table below for the freeway volume profile shows the final adjusted 2045 proportions which also summed to 100%. The black lines represent the known VMT hours (i.e. 5-6 AM and 7-8 AM) in the partial table.

	Trend to 2045			Trend to 2045		
	2008	2023	2045	2008	2023	2045
Hour	WB			EB		
0	0.011	0.016	**0.024**	0.015	0.018	**0.022**
1	0.007	0.009	**0.013**	0.010	0.010	**0.011**
2	0.006	0.008	**0.010**	0.009	0.009	**0.010**
3	0.009	0.012	**0.017**	0.008	0.011	**0.016**
4	0.021	0.028	**0.039**	0.016	0.018	**0.021**
5						
6	0.101	0.090	**0.074**	0.056	0.054	**0.052**
7						
8	0.050	0.066	**0.088**	0.068	0.074	**0.083**

In general, the 2045 profiles made sense as the hourly fractions generally decreased in the peak periods representing further flattening or spreading of the peak periods. This was heavily influenced by direction (i.e. inbound/outbound) and facility type. Having more years in the trend may improve these relationships, but that must be weighed against the additional time and data required. There was one case for the eastbound ramp profile in the 1-2 AM period that ended up with a slightly negative 2045 proportion (-0.001). This was caused by a substantial decrease in the inputs from 0.009 in 2008 to 0.005 in 2023. In this case, the value was manually changed to 0.000 which would represent no volume in that period but could be true for these lighter-volume ramps at that hour. This did cause the sum of the hourly eastbound ramp VMT not to match the total VMT, but the overall VMT difference was 6 to 7 vehicles-miles which translates to just a 0.3% error.

Step 7

The hourly volume profile factors were then applied to the remaining VMT portion to distribute it across the other 19 hours as shown in in the partial table below. This table can be compared with the one in Example 16-12 to see the differences between 2023 and 2045.

Link ID	Link Description	Total DailyVMT	Hourly VMT (veh-mi) 12AM	1AM	2AM	3AM	4AM
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp	91889	1511	742	675	1072	1404
101	I-84 EB; 58th St off-ramp to 60th St on-ramp	42865	693	340	310	492	644
102	I-84 EB; 60th St on-ramp to Halsey St off-ramp	24694	403	198	180	286	374
200	I-84 WB; Halsey St on-ramp to 58th St on-ramp	148333	2391	1175	1069	1697	2222
201	I-84 WB; 58th St on-ramp to 43rd St off-ramp	44868	725	356	324	514	673
500	58th St EB off-ramp	2474	4	0	3	8	15
501	58th St WB on-ramp	4134	16	13	14	7	54
502	60th St EB on-ramp	1988	3	0	3	7	12

Steps 8-9

A similar forecast process was used to create the 2045 no-build vehicle class proportions using the same classification counts used to develop volume profiles.

Freeway Segments	Westbound								
	Light			Medium			Heavy		
Hour	2008	2023	2045	2008	2023	2045	2008	2023	2045
0	0.905	0.925	**0.953**	0.027	0.021	**0.012**	0.067	0.054	**0.034**
1	0.890	0.896	**0.905**	0.026	0.042	**0.064**	0.084	0.063	**0.031**
2	0.888	0.846	**0.784**	0.031	0.069	**0.125**	0.081	0.085	**0.091**
3	0.858	0.890	**0.937**	0.041	0.042	**0.043**	0.101	0.068	**0.019**
4	0.917	0.930	**0.949**	0.028	0.031	**0.035**	0.055	0.039	**0.016**
5	0.956	0.958	**0.960**	0.019	0.019	**0.019**	0.025	0.023	**0.021**
6	0.961	0.953	**0.941**	0.014	0.021	**0.030**	0.025	0.027	**0.029**
7	0.961	0.948	**0.928**	0.015	0.025	**0.040**	0.023	0.027	**0.032**

After the initial computation of proportions, any 2045 values over 1.000 or negative were flagged. These are caused by a significant positive or negative trend between the input years. While the totals still add to 100%, they may not after doing the manual adjustments. Adjustments ideally were taken from the past count looking at adjacent hours to identify potential values that are a spike or a dip. Any changed value had cell comments added to document the original value and reasons for the adjustment. The 2023 values were not adjusted for consistency as these are already used within the existing year analysis.

Arterial Segments									
	Westbound								
	Light			Medium			Heavy		
Hour	2012	2023	2045	2012	2023	2045	2012	2023	2045
0	0.938	0.965	**1.018**	0.040	0.035	**0.027**	0.023	0.000	**-0.045**
1	0.964	0.963	**0.961**	0.036	0.037	**0.039**	0.000	0.000	**0.000**
2	0.935	0.982	**1.077**	0.047	0.009	**-0.066**	0.019	0.009	**-0.010**
3	0.957	0.991	**1.059**	0.011	0.000	**-0.022**	0.032	0.009	**-0.038**
4	0.964	0.979	**1.010**	0.020	0.017	**0.010**	0.016	0.004	**-0.020**

Steps 10-11

Like what was done for the existing condition, the RITIS Performance Chart tool was used to develop the average speed by hour for April 2016 which is the earliest available data. The tool was run with matching as much as possible the links with the identified directional XD segments. The 2016 data was downloaded to Excel and forecasting functions were used to project to 2035. The combination of 2023 existing and a 2016 base year allows for a projection of seven years forward to 2030 based on a seven year backwards look. Data can also be reasonably extrapolated beyond this point up to about five years to 2035. Year 2035 was also deemed equivalent to 2045 speeds considering the tendency to overstate the actual rate of change. Trials using years beyond 2035 in the projection calculations created too many unrealistic minimum and maximum speeds.

Steps 12-13

Free-flow speeds were calculated for each facility section and lookup tables were used to limit maximum speeds to the free-flow level and to identify the appropriate speed bin. Lastly, any speeds that were projected to be lower than seven mph were changed to this minimum speed and speed bins were changed to match as necessary. The partial table below shows the 2016 actual speeds (earliest year available), the 2023 existing year actual speeds, and the projected 2035 speeds. The highlighted 2035 speeds for Link 100 were projected to greatly exceed the free-flow and thus were limited to 60 mph based on the 55-mph speed. While the 2023 speeds also exceed the free-flow speeds, these are actual reported values and it is unknown exactly what conditions will be in 2035 for volumes, vehicle class, etc. so it is best to limit these speeds to the free-flow level.

Link ID	Link Description	Time	2016 Ave Speed (mph)	2023 Ave Speed (mph)	2035 Ave Speed (mph)	Speed Bin ID#
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp					
		12AM	56.0	61.5	60.0	13
		1AM	56.7	61.4	60.0	13
		2AM	57.3	60.6	60.0	13
		3AM	57.4	61.4	60.0	13
		4AM	56.9	62.8	60.0	13
		5AM	58.8	62.8	60.0	13
		6AM	59.1	61.5	60.0	13
		7AM	58.0	58.6	59.6	13
		8AM	57.7	49.4	35.2	8

Step 14

The rest of the process is the same as done for the existing conditions. Once the hourly speed bins were identified for all the links, the light, medium and heavy vehicle hourly VMT totals were parsed into each of the 16 speed bins for each link. The partial table below shows the distribution for light vehicles for Link 100 between 12- 8 AM.

Link ID	Time	VMT Light Veh	8	9	10	11	12	13	14
100									
	12AM	1268						1472	
	1AM	568						660	
	2AM	519						602	
	3AM	908						908	
	4AM	1129						1129	
	5AM	3502						3502	
	6AM	2845						2845	
	7AM	4120						4120	
	8AM	4595	4595						

Step 15

Next, the link VMT data was consolidated down to the roadtype by classification level. All the individual hours (i.e. 12-1 AM, 1-2 AM, etc.) were summed for each roadtype and classification (i.e. Roadtype 4 & Light vehicle, Roadtype 4 & Medium vehicle, etc.) as shown in the partial table below.

Roadtype	Class	Time	VMT by Hr by Speed Bin 5	6	7	8	9	10	11
4	Light	12AM	19	0	4	18	0	0	0
4	Light	1AM	11	0	3	14	0	0	0
4	Light	2AM	3	0	34	0	0	0	0
4	Light	3AM	7	0	15	0	0	0	0
4	Light	4AM	23	14	103	0	0	0	0
4	Light	5AM	0	56	258	204	0	0	0
4	Light	6AM	0	79	291	0	280	0	68
4	Light	7AM	0	10164	0	242	0	0	0
4	Light	8AM	7621	2472	292	4595	0	3376	0

Step 16

The hourly VMT for each roadtype from Step 12 was summed across all the speed bins to create a subtotal. Each hourly subtotal was summed across the day to create a VMT total by roadtype. Each roadtype VMT total was divided by the total sum of all roadtype VMT to create a decimal fraction. The VMT roadtype fractions did not change from the existing in this case because of the linear projections used to develop the future no-build data.

Roadtype	Roadtype VMT Fraction
4	0.9754
5	0.0246

Steps 17-18

Each hour in the table above was summed across all speed bins to create a total. This was used to create decimal speed fractions by dividing the hourly roadtype by classification by speed VMT data by the total hourly by roadtype by classification VMT. The partial table below shows the corresponding speed fractions for the previous table. Each hourly VMT row in the table sums to 1.0000 (100%).

Roadtype	Class	Time	VMT by Hr by Speed Fractions 5	6	7	8	9	10	11
4	Light	12AM	0.0037	0.0000	0.0008	0.0035	0.0000	0.0000	0.0000
4	Light	1AM	0.0047	0.0000	0.0014	0.0060	0.0000	0.0000	0.0000
4	Light	2AM	0.0013	0.0000	0.0168	0.0000	0.0000	0.0000	0.0000
4	Light	3AM	0.0019	0.0000	0.0044	0.0000	0.0000	0.0000	0.0000
4	Light	4AM	0.0051	0.0031	0.0226	0.0000	0.0000	0.0000	0.0000
4	Light	5AM	0.0000	0.0035	0.0161	0.0127	0.0000	0.0000	0.0000
4	Light	6AM	0.0000	0.0067	0.0249	0.0000	0.0239	0.0000	0.0058
4	Light	7AM	0.0000	0.5721	0.0000	0.0136	0.0000	0.0000	0.0000
4	Light	8AM	0.4113	0.1334	0.0158	0.2480	0.0000	0.1822	0.0000

Quantitative MSAT Process – Future Build

If the build alignment is the same (same link network) with differing number of lanes and/or traffic control, but the volumes are the same then the build data would likely be the same as the future no-build. If the build design year volumes are different from the no-build because of latent demand issues stemming from pent-up congestion, then the daily volumes will change, and the process will be same as doing the future no-build.

The overall challenge with the future build data is when the build alignment or network layout is significantly different from the no-build alignment, the relationships between links get muddied. The peak hour and/or daily volumes would have already been re-distributed onto the build network for the project analysis. The relative relationships between the no-build and build link volumes and VMT for the project study area would need to be determined so that they could be applied to the analysis. The analyst will need to figure out separately the routing of the separate vehicle classes and resulting VMT fractions if they do not follow the same patterns as in the future no-build. New links will need traffic redistributed onto them along with the likely redistribution of all the needed vehicle classes.

The overall process is as follows:

1. Modify the future no-build spreadsheet/database file with changed or new links, updated build descriptions, lengths and roadway types.

2. Determine the link VMT for all tagged links (or pre-determined districts or subareas if doing an area-wide analysis) by using a model daily volume assignment as shown in Steps 3 through 5 for the future build scenario. The future model year used should be the closest to the project future (horizon) year. If the future model year used matches the project future year, then the model link values can be used as-is. If the model year is different, then the model year link volumes should be adjusted/interpolated to match the project future year.

3. Run a peak period assignment for each of the identified periods in the model and calculate VMT for the same links as for the daily computed in Step 2. Activity-based models (ABM) will have five periods (Early AM, AM peak, Midday, PM peak, and evening). Trip-based models will have at least two periods but could have more depending on the jurisdiction (AM peak, PM peak, Midday if available, other periods if available).

4. For ABMs, sum the peak period VMT in Step 3 and subtract it from the daily VMT in Step 2 to determine the night period VMT for each link. For trip-based models subtract the sum of the link VMT in Step 3 from the daily VMT in Step 2 to calculate the VMT daily remainder (up to 22 hours' worth) for each link.

5. Compare the future build and the no-build scenarios to determine relative changes on links to create conversion factors for each applicable period to modify the no-

build VMT into the build. For example, a link loses 30% of its volume to a new route in a certain period so a 0.70 conversion factor would be applied to the no-build VMT for all hours in that period (i.e. midday, night, etc.) on that link to convert it into the build VMT.

6. Determine the specific hours to be grouped/assigned with the known hourly assignments (i.e. using 10 PM – 5AM as the hours that will use the 5 AM early period assignment) by inspecting the future volume proportions to see if there are any natural breaks for the peak periods. Some groupings may be arbitrary as proportions may be relatively even across multiple hours. Multiply the conversion factor by the no-build VMT for each link and hour to compute the build VMT values. [Note: use of the no-build VMT here has the no-build hourly volume proportion of the no-build remaining daily VMT built in, so no extra work is needed with the volume profiles developed for the no-build.] Subtract the hourly VMT from the remaining daily build VMT to check for the overall error as it will not match exactly due to rounding and the general simplification of using groups of conversion factors. Add the determined link VMT by hour to the spreadsheet to set up for the further fractional splitting in following steps.

7. Project links that are new or have been identified to have substantial volume shifts through the previously completed project analysis may also have substantial changes in vehicle classification percentages. Relationships between the no-build and build links need to be identified in the original project analysis. If a noise analysis has been completed, use those resulting final link vehicle classification percentages for project build links. The project build links can be used to identify representative facilities (i.e. similar types and characteristics such as functional class, volume ranges and number of lanes) to estimate the future vehicle class changes on roadways outside of the build study area. If build vehicle classification percentages are not available, follow a similar process as in Section 16.3 with starting with the future no-build percentages and the link total volumes to determine initial numbers of vehicles in each classification grouping. Modify these initial volumes with those found relationships/knowledge/estimation to create final volumes. Use the final vehicle class group volumes and the total link volumes to create the final resulting vehicle class percentages. If there is no substantial change in the VMT (i.e. changes less than +/- 20% such as VMT conversion factors in the 0.80 to 1.20 range from Step 6) on a link from converting the no-build future to the build future or if there is insufficient data to determine changes to the build conditions, then continue to use the future no-build vehicle classification percentages. The 20% limit is from the potential 10% error range in data from travel demand models plus the typical 10% needed for a noticeable or substantial change.

8. Use the vehicle class groupings determined in Step 7 and multiply by the hourly link VMT from Step 6 to determine the hourly VMT by vehicle class.

9. Post-process the future build and build link travel times and link length from the travel demand model scenarios to calculate a link speed. Model travel times do not completely account for control and congestion delays so model-based speeds would not match up well if used directly with field-based speeds. For each hourly scenario, sum up the model link travel times for each facility link in this analysis. Use the same hourly groupings used in Step 6 and assign link travel times to other hours to fill out the whole 24-hour period. Convert link travel times to speeds using the link length and create a no-build to build conversion (i.e. post-processing) factor. Multiply this factor with the future no-build average speeds to obtain the future build average speed. Limit maximum speeds to the free-flow speed and minimum speeds to the future no-build default (i.e. 7 mph).

10. Identify the applicable speed bin for each hour for each link from the list below based on the speeds determined in Step 9. Use the same lookup tables used in the future no-build.
    1. Speed < 2.5 mph
    2. 2.5mph ≤ Speed < 7.5mph
    3. 7.5mph ≤ Speed < 12.5mph
    4. 12.5mph ≤ Speed < 17.5mph
    5. 17.5mph ≤ Speed <22.5mph
    6. 22.5mph ≤ Speed < 27.5mph
    7. 27.5mph ≤ Speed < 32.5mph
    8. 32.5mph ≤ Speed < 37.5mph
    9. 37.5mph ≤ Speed < 42.5mph
    10. 42.5mph ≤ Speed < 47.5mph
    11. 47.5mph ≤ Speed < 52.5mph
    12. 52.5mph ≤ Speed < 57.5mph
    13. 57.5mph ≤ Speed < 62.5mph
    14. 62.5mph ≤ Speed < 67.5mph
    15. 67.5mph ≤ Speed < 72.5mph
    16. 72.5mph ≤ Speed

11. For each link, parse the hourly link VMT by vehicle class from Step 8 into each of the 16 speed bins. This represents the finest level of VMT disaggregation by link.

12. Consolidate the binned link VMT by each roadtype. Sum the VMT for an individual hour and classification across all links of that roadtype. For example, for Roadtype 4 (urban restricted), light vehicle class, sum all link VMT for 12-1 AM for each of the 16 speed bins. Then repeat for 1-2 AM throughout the rest of the day. Then the same process would be repeated for the medium and heavy vehicles for Roadtype 4. This would continue to be repeated through all applicable roadtypes for the project.

13. For each roadway type, sum the VMT across all the classes and speed bins to create an hourly VMT total. Sum this hourly VMT to create a roadtype VMT

subtotal. Divide each roadtype VMT subtotal by the total sum of all the roadtype VMT's to create decimal fractions for each.

14. Sum up each of the hourly VMT by roadtype and classification records created in Step 12 to create an hourly total.

15. Divide the hourly VMT by roadtype, classification, and speed bin by the total hourly VMT by roadtype by classification from Step 13 to determine the VMT speed-based fractions in decimal form. Each hour in the resulting table needs to sum to 1.0000 (i.e. 100%).

Example 16-14 Quantitative MSAT Future build Conditions

This example continues the sample project analysis from Example 16-13. Steps mentioned in the example correspond to the steps shown above in the procedure.

Step 1

The same link sections and attributes such as link number, description and length were copied from the future no-build conditions.

Steps 2-4

Daily and hourly volumes for the five assigned periods (5AM, 7AM, 12PM, 5PM, and 7PM) were captured from a future build model assignment. The link length and the ADT were multiplied together to obtain the total daily VMT for each link as shown below. From this value, the known hourly VMT from the peak periods was subtracted from the daily total as was done for the existing and future no-build conditions to come up with the remaining VMT to be distributed over the remaining 19 hours of the day.

Link ID	DVMT (veh-mi)	Early 5A VMT (veh-mi)	AM 7A VMT (veh-mi)	Mid 12P VMT (veh-mi)	PM 4P VMT (veh-mi)	Eve 7P VMT (veh-mi)	Rem VMT (veh-mi)
100	91889	4150	5261	5428	5533	3938	67580
101	42865	1966	2508	2496	2628	2289	30978
102	24694	1097	1425	1428	1526	1202	18016
200	148333	8683	9828	8282	8641	5943	106956
201	44868	2562	2778	2583	2640	1896	32409
500	2474	98	117	162	128	145	1824
501	4134	195	131	288	260	237	3023
502	1988	59	98	106	133	103	1489
503	1705	58	68	127	135	69	1248
504	5125	273	224	310	322	276	3721
600	1120	14	21	61	104	54	866
601	1058	44	138	67	51	33	725
602	1249	9	33	84	187	24	911
603	2090	52	281	112	150	35	1460
604	765	4	22	46	108	20	565
605	844	19	112	43	63	12	595

Step 5

The future build link VMT was divided by the future no-build link VMT to create a series of volume conversion factors as shown below. These will allow the future no-build volume profile to be translated to the build conditions. Links that have higher volumes in the build will have factors greater than 1. Links that have decreased volume in the build will have factors less than 1.

This is a good opportunity to check to see if the changes make sense. Inspection of the table shows about 10-20% increases for the mainline links (i.e. 100 & 200 series) on I-84 with higher changes on the ramp connections (i.e. 500 series) depending on the time of day. Local roadways (i.e. 600 series) have decreased volumes by around 25%, however some have higher increases in the overnight hours. Since capacity is being added to I-84 in this scenario, it would make sense to see higher volumes on I-84 and lower volumes on the close parallel routes.

Link ID		Early 5A (vph)	AM 7A (vph)	Mid 12P (vph)	PM 4P (vph)	Eve 7P (vph)
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp	1.12	1.22	1.23	1.21	1.13
101	I-84 EB; 58th St off-ramp to 60th St on-ramp	1.10	1.21	1.20	1.21	1.12
102	I-84 EB; 60th St on-ramp to Halsey St off-ramp	1.09	1.18	1.17	1.18	1.11
200	I-84 WB; Halsey St on-ramp to 58th St on-ramp	1.14	1.18	1.15	1.16	1.06
201	I-84 WB; 58th St on-ramp to 43rd St off-ramp	1.16	1.17	1.16	1.16	1.06
500	58th St EB off-ramp	1.31	1.30	1.45	1.26	1.18
501	58th St WB on-ramp	1.28	1.08	1.21	1.20	1.05
502	60th St EB on-ramp	1.01	0.88	0.93	0.97	1.03
503	Halsey St on-ramp	1.40	1.17	1.63	1.96	1.05
504	43rd St off-ramp	1.28	1.15	1.10	1.11	1.04
600	Glisan St EB; 55th St to 58th St	1.94	0.41	0.81	0.82	0.98
601	Glisan St WB; 58th St to 55th St	1.08	0.96	1.22	0.81	2.78
602	Halsey St EB: 60th St to 67th St	0.96	0.63	0.93	0.87	0.73
603	Halsey St WB; 67th St to 60th St	0.62	0.90	0.79	0.85	0.99
604	Stark St EB; 55th St to 60th St	0.92	0.42	0.61	0.81	0.87
605	Stark St WB; 60th St to 55th St	0.50	0.76	0.69	0.75	1.00

Step 6

These volume factors for the known assigned hours were used to translate the no-build volume profile to the build volume profile. For example, for Link 100, the 1.12 factor for the early 5 AM period was assigned to all hours for the early period. This required that certain hours be grouped together to be able to assign a growth factor. Inspection of the overall volume profile showed that from 12 AM – 6 AM should be grouped with the Early 5AM assignment, 7 – 8 AM for the 7AM Peak, 9 AM to 3 PM for the 12PM Mid, 4 – 6 for the 5 PM Peak, and 7 PM – 12 AM for the 7 PM Evening periods.

The no-build hourly VMT for each link (which is made up of the hourly proportion for each facility type multiplied by the no-build remaining daily VMT) is copied into the partial build spreadsheet as shown below. The 5AM values are italicized as these are already build VMT values from the known assignment.

Link ID	Link Description	Hourly VMT (veh-mi)					
		12AM	1AM	2AM	3AM	4AM	*5AM*
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp	1302	640	582	924	1209	*4150*
101	I-84 EB; 58th St off-ramp to 60th St on-ramp	601	295	269	427	559	*1966*
102	I-84 EB; 60th St on-ramp to Halsey St off-ramp	355	175	159	252	330	*1097*
200	I-84 WB; Halsey St on-ramp to 58th St on-ramp	2134	1049	954	1515	1983	*8683*
201	I-84 WB; 58th St on-ramp to 43rd St off-ramp	644	317	288	457	599	*2562*

The calculated no-build to build conversion factors are assigned to each of the applicable hours. The 5AM column is blacked out as it had a specific hourly assignment, so no conversion is necessary.

Link ID	Link Description	Assigned No-build to Build Conversion Factors					
		12AM	1AM	2AM	3AM	4AM	5AM
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp	1.12	1.12	1.12	1.12	1.12	
101	I-84 EB; 58th St off-ramp to 60th St on-ramp	1.10	1.10	1.10	1.10	1.10	
102	I-84 EB; 60th St on-ramp to Halsey St off-ramp	1.09	1.09	1.09	1.09	1.09	
200	I-84 WB; Halsey St on-ramp to 58th St on-ramp	1.14	1.14	1.14	1.14	1.14	
201	I-84 WB; 58th St on-ramp to 43rd St off-ramp	1.16	1.16	1.16	1.16	1.16	

The no-build VMT for each link and hour is multiplied by the corresponding no-build-to-build conversion factor shown above to calculate the build VMT for each link as shown below. The total hourly VMT was checked against the total daily VMT and there was about a 1% error between the two which was deemed acceptable because of rounding and use of these simplified blocks of conversion factors. With more known hourly assignments, the overall error could be less, but this will be an overall tradeoff with the sheer number of assignment runs needed.

Link ID	Link Description	Hourly Build VMT (veh-mi)					
		12AM	1AM	2AM	3AM	4AM	*5AM*
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp	1453	714	649	1031	1350	*4150*
101	I-84 EB; 58th St off-ramp to 60th St on-ramp	660	325	295	469	614	*1966*
102	I-84 EB; 60th St on-ramp to Halsey St off-ramp	388	191	173	275	361	*1097*
200	I-84 WB; Halsey St on-ramp to 58th St on-ramp	2440	1199	1090	1732	2267	*8683*
201	I-84 WB; 58th St on-ramp to 43rd St off-ramp	745	366	333	528	692	*2562*

Steps 7-8

The assigned hourly VMT conversion factors were averaged over the entire day to determine the average change to weigh the potential of vehicle class changes from the no-build to the build as shown in the table below with some selected hours shown. Bolded values indicate links with a greater than a 20% change. Most links (63%) had a change of less than 20% including all of the freeway links and most of the ramp links. These links also carry the vast majority of the total VMT (97.5%). The local streets have the smallest percentages of medium (around 3%) and heavy vehicles (0.5%). Changes in vehicle class on these remaining links will likely not have any substantive impact overall. In addition, there was a lack of available vehicle class data for the ramps and local streets, so those already have a higher built-in level of estimation.

In all, it was decided to keep the hourly vehicle class mix at the projected future no-build levels.

Link ID	Link Description	4AM	6AM	8AM	10AM	Ave Factor Change
100	I-84 EB; Cesar Chavez on-ramp to 58th St off-ramp	1.12	1.12	1.22	1.23	1.17
101	I-84 EB; 58th St off-ramp to 60th St on-ramp	1.10	1.10	1.21	1.20	1.15
102	I-84 EB; 60th St on-ramp to Halsey St off-ramp	1.09	1.09	1.18	1.17	1.14
200	I-84 WB; Halsey St on-ramp to 58th St on-ramp	1.14	1.14	1.18	1.15	1.13
201	I-84 WB; 58th St on-ramp to 43rd St off-ramp	1.16	1.16	1.17	1.16	1.14
500	58th St EB off-ramp	1.31	1.31	1.30	1.45	**1.32**
501	58th St WB on-ramp	1.28	1.28	1.08	1.21	1.19
502	60th St EB on-ramp	1.01	1.01	0.88	0.93	0.98
503	Halsey St on-ramp	1.40	1.40	1.17	1.63	**1.44**
504	43rd St off-ramp	1.28	1.28	1.15	1.10	1.15
600	Glisan St EB; 55th St to 58th St	1.94	1.94	0.41	0.81	1.18
601	Glisan St WB; 58th St to 55th St	1.08	1.08	0.96	1.22	**1.45**
602	Halsey St EB: 60th St to 67th St	0.96	0.96	0.63	0.93	0.87
603	Halsey St WB; 67th St to 60th St	0.62	0.62	0.90	0.79	**0.79**
604	Stark St EB; 55th St to 60th St	0.92	0.92	0.42	0.61	**0.77**
605	Stark St WB; 60th St to 55th St	0.50	0.50	0.76	0.69	**0.71**

Step 9

Obtaining the 2045 build speeds requires post-processing speeds calculated from travel demand model link travel times as this is the only source of information for the build alternative. The link travel times were converted into link speeds using the known link lengths. The projected 2035 no-build future speeds from extrapolated RITIS data are deemed equivalent to 2045 considering the tendency to overstate the actual rate of change. These "2045" values were used as the field input into the calculation. The process multiplied the extrapolated field speed data by a build/no-build ratio factor to obtain the build speeds.

Step 10

The same free-flow speeds from the future no-build were used for each facility section and lookup tables were used to limit maximum speeds to the free-flow level and to identify the appropriate speed bin. Speeds improved as expected as no hours fell below the minimum speed level, so no extra adjustments were needed. The partial table below shows the "2045" (i.e. equivalent to 2035) no-build future projected speeds from RITIS data, the 2045 no-build and build model link travel times and calculated speeds using the link lengths determined for the existing conditions, the post-processed 2045 build speeds and the resulting identified speed bin. The no-build projected average speeds from RITIS were previously limited to free-flow maximums, and in these cases, the build/no-build ratio (e.g. 46.5/41.5 for 12 to 6 AM below) would likely be greater than 1.0 which would create build speeds in excess of free-flow, so these speeds were also limited to the free-flow speed as shown below.

Link ID	Time	"2045" No-bld Ave Speed (mph)	2045 No-Build Link Trvl Time (min)	2045 No-build Link Speed (mph)	2045 Build Link Trvl Time (min)	2045 Build Link Speed (mph)	2045 Ave Speed (mph)	Speed Bin ID#
100								
	12AM	60.0	1.11	41.1	0.98	46.5	60.0	13
	1AM	60.0	1.11	41.1	0.98	46.5	60.0	13
	2AM	60.0	1.11	41.1	0.98	46.5	60.0	13
	3AM	60.0	1.11	41.1	0.98	46.5	60.0	13
	4AM	60.0	1.11	41.1	0.98	46.5	60.0	13
	5AM	60.0	1.11	41.1	0.98	46.5	60.0	13
	6AM	60.0	1.11	41.1	0.98	46.5	60.0	13
	7AM	59.6	1.48	30.8	1.21	37.7	60.0	13
	8AM	35.2	1.48	30.8	1.21	37.7	43.0	10

Step 11

The remainder of the process is the same as done for the existing and future no-build conditions. Once the hourly speed bins were identified for all the links, the light, medium and heavy vehicle hourly VMT totals were parsed into each of the 16 speed bins for each link. The partial table below shows the distribution for light vehicles for Link 100 between 12- 8 AM.

Link ID	Time	VMT Light Veh	8	9	10	11	12	13	14
100									
	12AM	1416						1416	
	1AM	634						634	
	2AM	579						579	
	3AM	1013						1013	
	4AM	1260						1260	
	5AM	3909						3909	
	6AM	3175						3175	
	7AM	5013						5013	
	8AM	5592			5592				

Step 12

Next, the link VMT data was consolidated down to the roadtype by classification level. All the individual hours (i.e. 12-1 AM, 1-2 AM, etc.) were summed for each roadtype and classification (i.e. Roadtype 4 & Light vehicle, Roadtype 4 & Medium vehicle, etc.) as shown in the partial table below.

Roadtype	Time	VMT by Hr by Speed Bin - Light Veh							
		4	5	6	7	8	9	10	11
4	12AM	0	24	6	22	0	0	0	0
4	1AM	0	14	0	23	0	0	0	0
4	2AM	0	3	34	8	0	0	0	0
4	3AM	9	7	17	3	0	0	0	0
4	4AM	0	47	0	132	0	0	0	0
4	5AM	0	0	56	337	262	0	0	0
4	6AM	0	0	79	374	359	0	0	95
4	7AM	159	126	112	215	9123	2578	0	0
4	8AM	392	0	556	8702	2611	1474	5592	2567

Step 13

The hourly VMT for each roadtype from Step 12 was summed across all the speed bins to create a subtotal. Each hourly subtotal was summed across the day to create a VMT total by roadtype. Each roadtype VMT total was divided by the total sum of all roadtype VMT to create a decimal fraction. The results show about a 0.6% VMT increase in Roadtype 4 (freeway) from the no-build (0.9754) and a corresponding decrease in the arterial share (0.0246). This reflects the greater freeway capacity of the build alternative and a general shift toward it from the surface arterial system.

Roadtype	Roadtype VMT Fraction
4	0.9813
5	0.0187

Steps 14-15

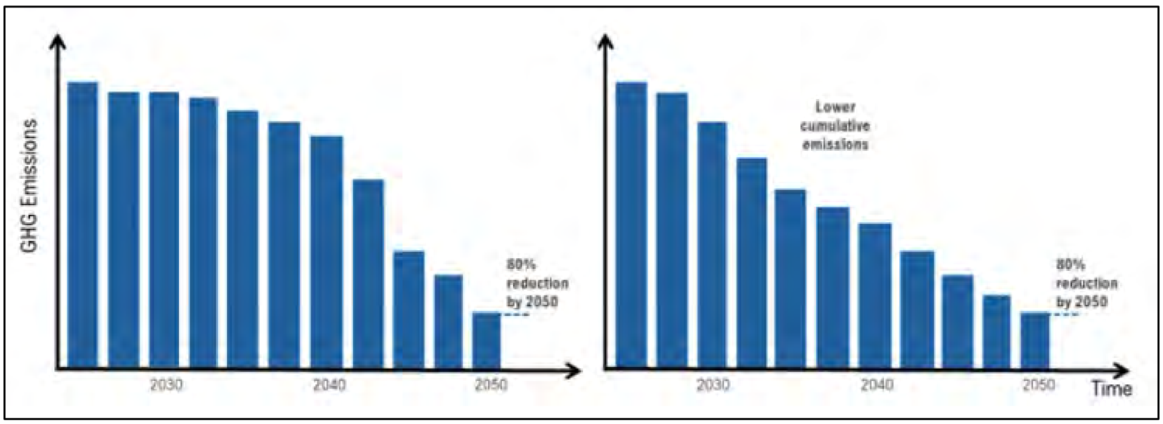
Each hour in the table above was summed across all speed bins to create a total. This was used to create decimal speed fractions by dividing the hourly roadtype by classification by speed VMT data by the total hourly by roadtype by classification VMT. The partial table below shows the corresponding speed fractions for the previous table.

Roadtype	Class	Time	VMT by Hr by Speed Fractions						
			5	6	7	8	9	10	11
4	Light	12AM	0.0044	0.0011	0.0041	0.0000	0.0000	0.0000	0.0000
4	Light	1AM	0.0053	0.0000	0.0089	0.0000	0.0000	0.0000	0.0000
4	Light	2AM	0.0012	0.0159	0.0038	0.0000	0.0000	0.0000	0.0000
4	Light	3AM	0.0017	0.0044	0.0007	0.0000	0.0000	0.0000	0.0000
4	Light	4AM	0.0090	0.0000	0.0256	0.0000	0.0000	0.0000	0.0000
4	Light	5AM	0.0000	0.0031	0.0185	0.0143	0.0000	0.0000	0.0000
4	Light	6AM	0.0000	0.0059	0.0281	0.0270	0.0000	0.0000	0.0071
4	Light	7AM	0.0060	0.0053	0.0102	0.4329	0.1223	0.0000	0.0000
4	Light	8AM	0.0000	0.0253	0.3960	0.1188	0.0671	0.2544	0.1168

16.5 Greenhouse Gas Emission (GHG) Analyses

GHG emissions (e.g. carbon dioxide, methane) measure the accumulation of carbon in the broader atmosphere that threatens the environment/climate system. GHG emission analyses may use similar tools and methods as used in the air quality analyses described in previous sections but GHGs are substantially different from the standard air quality pollutants (e.g. carbon monoxide). GHGs can persist for decades while most other pollutants disperse over a few minutes to days. GHGs are well mixed and do not have hot spots thus specific emission locations do not apply. GHGs have a resultant global impact which can impact precipitation patterns and long-term climate changes while air quality pollution tends to only affect local or regional areas population through respiratory issues. Capturing the impact of GHGs needs to go beyond just the on-road impacts of tailpipe emissions but also the accumulation over time which can have different impacts even if the desired target is met at the end as illustrated in Exhibit 16-14.

Exhibit 16-14 Two Accumulated Emission Trajectories[3]



This is normally done by considering the overall lifecycle emissions of the transportation system.[4] By calculating the GHG emissions in this way, they can be compared to Oregon's state and regional GHG reduction targets (e.g. 2024-27 STIP Handout).

GHG analyses include the three main types of annual (or daily) emissions:

- Fuel cycle (operational)
- Infrastructure cycle (construction, materials, and long-term maintenance)
- Vehicle cycle (manufacturing)

These three lifecycles and their interactions are illustrated in Exhibit 16-15. The fuel cycle captures operational emissions from on-road vehicles including the fuel efficiency

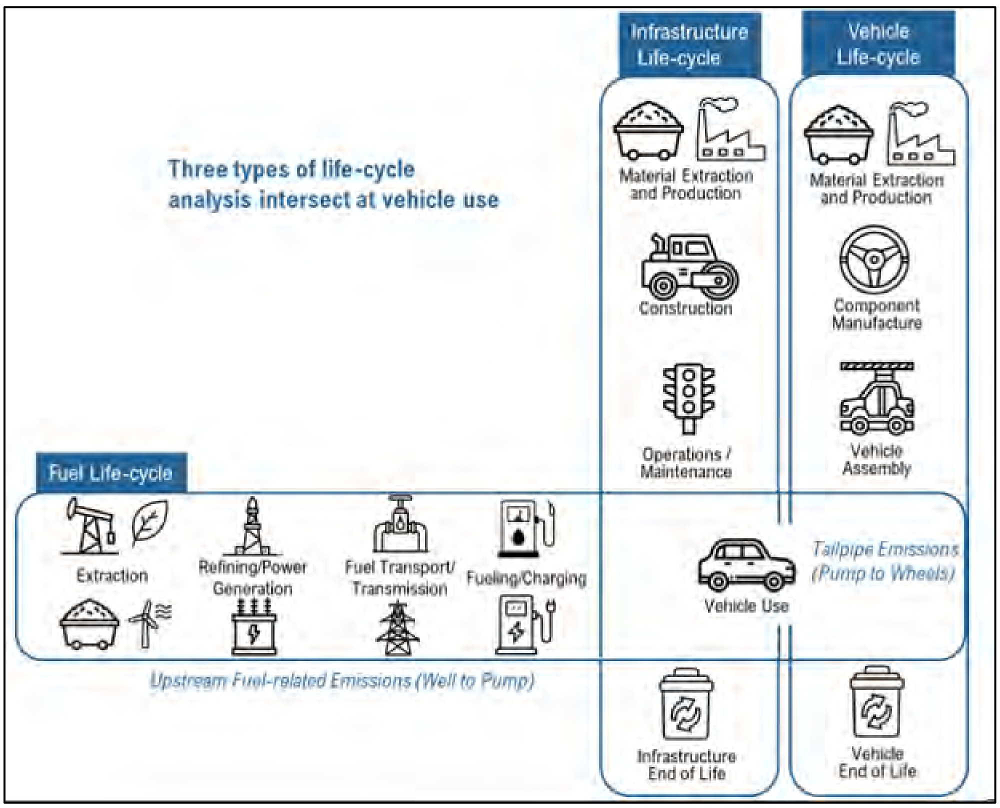
[3] From "Overview of Methods for Conducting GHG Analysis in the Transportation Planning Process", Michael Grant, ICF (TRB Annual Meeting, 2025).

[4] From "Overview of Methods for Conducting GHG Analysis in the Transportation Planning Process", Michael Grant, ICF (TRB Annual Meeting, 2025).

effect of recurring congestion if substantial. The infrastructure cycle captures emissions from construction and post-construction maintenance activities. This includes construction vehicles and materials used including any increased traffic congestion while the project is being built. Maintenance emissions are from vehicles used to perform maintenance activities (e.g. paving) periodically throughout the life of the project. The vehicle cycle captures manufacturing emissions from the impacts of materials used and following production such as steel making, battery lifecycle, and related automotive subassembly processes.

Both fuel (operational) and infrastructure (construction/maintenance) lifecycle emissions are quantified as accumulated impacts over time. The vehicle cycle with its emphasis on manufacturing is not covered by the analyses in this chapter.

Exhibit 16-15 GHG Emission Lifecycles[5]

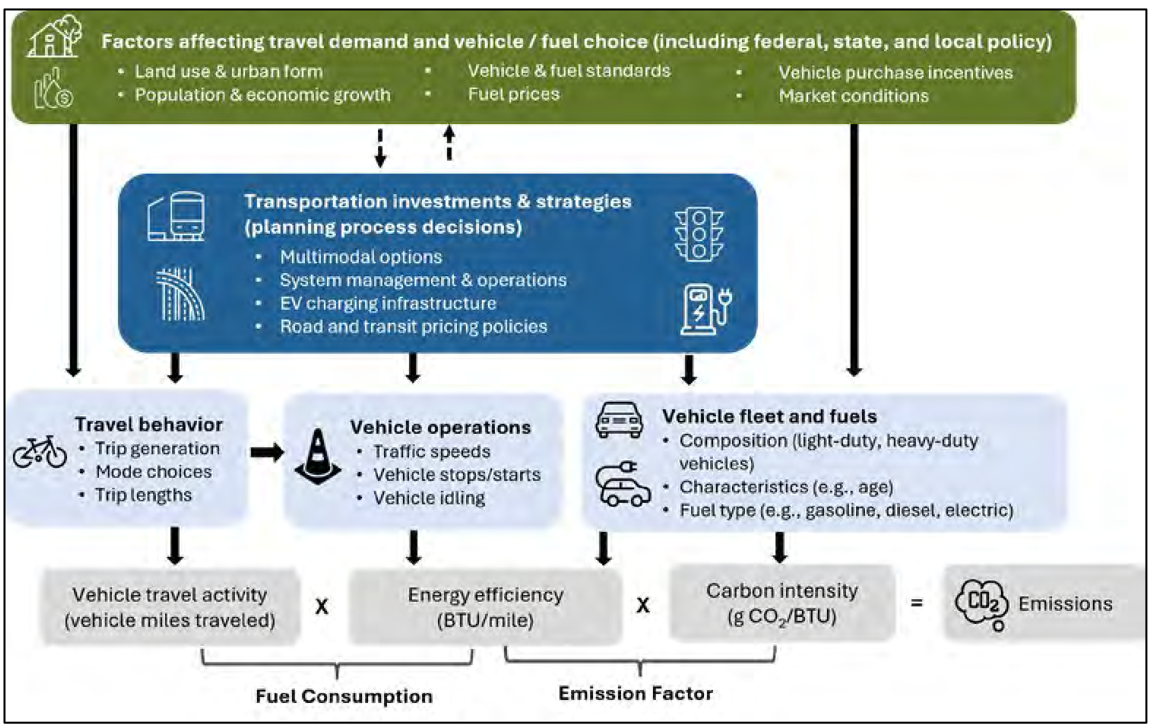


[5] From "Overview of Methods for Conducting GHG Analysis in the Transportation Planning Process", Michael Grant, ICF (TRB Annual Meeting, 2025).

Both tailpipe and lifecycle emissions (operational/user and construction emissions that result from fuel extraction, refining, and transport prior to use) are covered in the GHG calculation process in ODOT's Air Quality Manual and thus out of scope of this chapter.

Exhibit 16-16 below shows the key factors and policies that feed into the overall GHG calculation. The focus of the APM for GHG is on the production of the traffic data inputs consistent with the rest of the chapter. This means data, tools, and methods needed to support calculation of vehicle-miles traveled (VMT) for the traffic activity portion and speed-related data used that is combined with the emission factors. Application of emission factors and performing the resultant GHG calculations uses data provided by others (i.e. Air Quality/GHG analyst, Oregon Department of Environmental Quality MOVES staff) as noted throughout this chapter.

Exhibit 16-16 GHG Calculation Process Inputs[6]



6 From "Overview of Methods for Conducting GHG Analysis in the Transportation Planning Process", Michael Grant, ICF (TRB Annual Meeting, 2025).

16.5.1 GHG Methodologies

The GHG guidance in this chapter (Section 16.5.2 through 16.5.4) shall only be used for planning-level analysis of ODOT projects outside of the NEPA process. The Section 16.6.5 MOVES-based emission rate method can also be used at a planning level.

ODOT conducts technical analyses of climate change and GHG emissions during project planning. A climate lens is beginning to be employed in early project planning such as during STIP scoping of ODOT projects, largely based on what is included in the project scope and location-based attributes. In general, a planning-level analysis uses high-level estimates of activity and general design descriptions.

At the planning-level (non-NEPA), to account properly for the accumulated GHG impacts, this will still require a quantitative analysis. For example, a quantitative GHG assessment analysis will need to be performed when a project is proposed to be placed into the State Transportation Improvement Plan (STIP) for construction phase funding.

GHG planning-level analyses require a substantial amount of traffic data inputs. These inputs vary depending on the level of analysis and specific tools used along with the overall context of the project(s) being analyzed. The traffic analyst and the effort lead staff (whether being from ODOT or a consultant) will need to initially coordinate with the ODOT Environmental Section to determine the overall approach, level of analysis, tool requirements, data needs, staff resources, and schedule for the effort. Coordination should be ongoing through the life of the effort as ODOT Environmental staff will be reviewing the provided data, coordinating the latest emission rates with Oregon Department of Environmental Quality (DEQ), using the data in the determined tools and reviewing the output for consistency with expectations.

16.5.2 Tools and Project Types

Project-level GHG analyses require application of multiple tools to address the fullest range of project types. Tools can range from available individual software and spreadsheet calculators, combinations of tools in an overall process, to the custom creation of new tools or approaches. Exhibit 16-17 shows the GHG tools along with their corresponding project types. Some project types may have more than one applicable tool that could be used. Sketch tools are more appropriate for quicker analysis when less detail is available. GHG tools may either create traffic data (to be used in emission tools) or may calculate emissions directly.

In Oregon, the VisionEval tool is often used to estimate planning-level fuel lifecycle GHG emissions. As a strategic planning model, it represents “categories” of projects, and while it has an aggregate demand model, it does not have a network to code up individual projects. It is typically used to set a high-level roadmap, strategy, or vision, with more detailed tools used in implementation of long-range plans and projects. It can be used as a screening tool for quickly testing specific scenarios like doubling transit, impact of ITS operational programs, and alternative population growth or alternative fuel price or income forecasts. (See APM Chapter 7 for more on VisionEval/GreenSTEP family of tools).

Exhibit 16-17 Primary Planning-level GHG Tools and Corresponding Project Types

<table>
<tr><th>Tool</th><th>Tool Type[5]</th><th>Project Type[6]</th></tr>
<tr><td>SWIM[1]</td><td>Traffic</td><td rowspan="3">Roadway widening
New roadways
Roadway reconfigurations
Turn restrictions
Tolling</td></tr>
<tr><td>Regional Travel Model</td><td>Traffic</td></tr>
<tr><td>MOVES[2]</td><td>Emission</td></tr>
<tr><td>MOVES[2]</td><td>Emission</td><td>Roadway widening
New roadways
Roadway reconfigurations
Channelization
Turn lane improvements
Traffic control changes
Shoulder improvements
Resurfacing
Horizontal & vertical realignments</td></tr>
<tr><td>CMAQ[3]</td><td>Emission</td><td>Adaptive traffic control systems
New sidewalks, bike lanes, mid-block crossings, etc.
New traffic signals
Signal turn phasing changes
New signalized left turn lanes
Signal coordination/synchronization
Conversion to single/multilane roundabouts
Adding two-way left turn lanes
Roadway reconfigurations
Electric vehicles and charging infrastructure
Transit increased frequency, added stops, and new vehicles
Transit bus upgrades/replacement with low/no emission vehicles
Variable message signs
Variable speed limits</td></tr>
</table>

ICE[4]	Emission	New roadways Roadway widening Lane widening & shoulder improvements Roadway realignments Pavement reconstruction and resurfacing Bridges (new, reconstruction, or widening) New parking facilities (surface lot or structure) New rail facilities (heavy or light; either underground, at-grade or elevated) Bus rapid transit (new or lane conversions, stations) New or resurfaced multi-use paths New sidewalks, or restriped bicycle lanes

[1]Statewide Integrated Model
[2]Motor Vehicle Emission Simulator
[3]Congestion Mitigation & Air Quality
[4]Infrastructure Carbon Estimator
[5]The traffic-based tools need emission data and the emission-based tools need VMT & speed data to compute GHG.
[6]Curb ramp projects are a common project type but these are not covered by any existing tools and will require new approaches to assess emission impacts.

VMT data for these tools most commonly comes from SWIM or other travel demand models. VMT can also be estimated on a “on-road” basis from HPMS (i.e. daily volume times segment length) or DEQ-provided odometer data, the upcoming ODOT Research household-based VMT, or household survey data, etc. Assumptions could be made on speed data to help calculate the factors used in Exhibit 16-21. While most of the alternative VMT sources mentioned here are likely better suited to the sketch-level methods, other applications are possible. If any alternative VMT source is desired to be used, then these need to be investigated by the proposer for suitability and approved for use by the ODOT Air Quality Unit and/or the Climate Office.

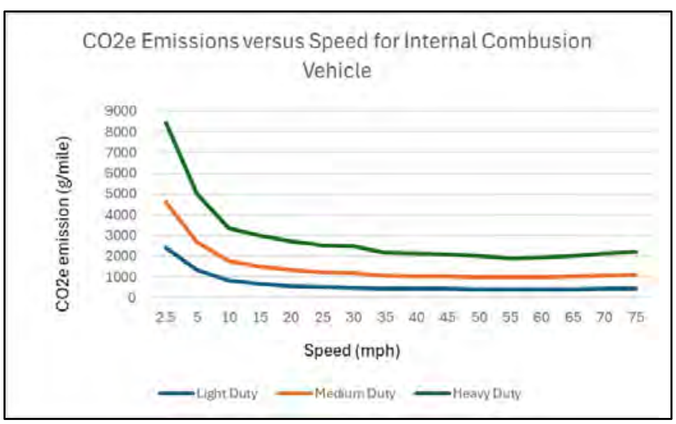
16.5.3 GHG Emission Elements

Information presented here in this section provides background and context relating to the fuel cycle (operational/user emissions) and the infrastructure cycle, (construction/materials/maintenance emissions) and their related inputs to calculate accumulated impacts that are used to develop the supporting GHG traffic data.

1. Fuel Cycle (Operational) Emissions

Operational emissions are sensitive to traffic volume, speed, roadway type (e.g. freeways), and vehicle classification. Exhibit 16-18 shows the impact of speed on carbon dioxide (GHG) emissions. Higher speeds and heavier vehicles produce higher emissions with heavy trucks producing the most per mile (approximately three times passenger cars based on MOVES data). This chart represents the operations of a fully internal combustion engine and would be expected to be less dynamic with hybrid and battery electric vehicles, and even vehicle electric stop/start technology. Speeds will be ideally based on probe data (e.g. RITIS) for existing conditions and post-processed travel demand model speeds for future conditions (see Section 16.4.3 for more detail on speed post-processing). Activity-based models (ABM) or dynamic traffic assignment (DTA) techniques can further improve the post-processed results. See Chapter 8 for more information on DTA and Chapter 17 for travel demand models including ABM. Ideally, the travel demand model would be calibrated to speed, but this is not in practice currently.

Exhibit 16-18 Smoothed & Congested Speeds vs GHG Emissions for internal combustion vehicles[7]



*Includes fuel cycle emissions; derived from EPA MOVES5

[7] Barth, M. and K. Boriboonsomsin. Real-World Carbon Dioxide Impacts of Traffic Congestion. Transportation Research Record: Journal of the Transportation Research Board, No. 2058, 2008, pp 163-171.

Calculation of annual accumulated emissions require several time periods both over a year (i.e. seasonal as emissions do vary and are accounted for in the emission rates) and time of day (i.e. by hour). Calculation of the required VMT fractions by speed, time, roadway, and vehicle classification are shown in Section 16.4.3 (MSAT analyses) and in Section 16.5.7. Typically a travel demand forecasts an average day with VMT by speed across the various hours of that day, which is then annualized by multiplying by 365.

Study area boundaries: Analysis boundaries should extend beyond the actual physical improvement limits as effects usually extend further, especially under congested conditions. Boundaries on arterials, expressways or freeways should extend to the next signalized intersection or interchange. If the extent of congestion is known either from available documentation or from RITIS tools, then the boundaries should encompass the congested extent. If time permits, it may be best to create a model scenario and track the substantial change (i.e. +/- 10% or greater change) from a roadway project such as a roadway reconfiguration or expansion to determine the overall extent.

Traffic volume forecast growth: The relative change between the existing year and the future and/or interim years for the build or no-build can be captured by breaking down the growth into yearly fractions forecasted by the travel demand model to create an unique volume set for each standard year (i.e. existing/base, opening/build, and design) and all desired interim years. If a calibrated model sub-area analysis was used, then the model volumes produced could be used directly as outputs. If a full travel demand model scenario was used to develop the original project volumes, then the relative change could be post-processed (following Chapter 6 and NCHRP Report 765) using available existing year AADT or hourly counts to tie the forecasted volumes to a realistic base value. Alternatively, for large scale/county/regional analyses, like for the MSAT process, growth would be generally based on the applicable travel demand model base and future no-build and build scenarios.

Calculating VMT in five-year increments is important to properly capture the substantial change in emission rates per mile as the expected transition to electric vehicles during the next 30 years occurs. VMT can be assumed to be a linear interpolation, which would be combined with the non-linear forecast of emission rates reflecting the vehicle adoption curve. This is discussed further in Step 3 (Combined Accumulated Emissions). An example of interpolating modeled VMT to match the emission rate dimensions (speed bin and vehicle type) and then calculating the accumulated emissions over the project lifecycle are included in Section 16.5.7.

Travel models should use the standard assumptions for adding new capacity (new through lanes) in build scenarios. Certain auxiliary lanes may add capacity such as turn lanes and longer weaving lanes, while some will not such as climbing/passing lanes, accel/deceleration lanes and shorter weaving lanes. See Appendix 10A for guidance on determining capacity thresholds for auxiliary lanes. See Section 6.12.2 for determining the risk of your project for latent and induced demand, leading to adjustments in the analysis approach to capture those effects.
Travel demand models may have difficulty in assessing the full benefits of improvements

depending on detail level (i.e. ABMs would likely have less issues than a standard trip-based model). These may need to be augmented by sketch-level analyses and related more-detailed facility-level assumptions.

Analysis of VMT data in metropolitan areas has specific GHG targets (OAR 660-044-020/025) which work out to be VMT per capita targets, as they assess local actions "beyond vehicle and fuel technology". As such, while these targets were set with the VisionEval model, progress can be assessed with a travel demand model, without combining VMT with emission rates, as noted in this chapter. See Appendix 17B for the methods assumed in the TSP analysis, including how to pull the specific household-based VMT defined in these rules.

Reoccurring and non-reoccurring congestion: Regardless of methodology, the AADT and VMT values should reflect reoccurring and non-reoccurring congestion so the effect of volumes, incidents, events, and severe weather can be captured. Use of peak spreading techniques or potentially dynamic traffic analysis if warranted or available (See Chapter 8) should be used if the project area has persistent congestion when peak periods extend across multiple hours or portions of the day. In general, traffic data needs to be provided by speed, roadway type (i.e. freeway, arterial, or ramp), vehicle type (i.e. light vehicles, medium trucks, and heavy trucks) and by individual hours which are the same requirements for an MSAT analysis in Section 16.4.3. This will enable calculation of accumulated emissions over all hours and all days of the year and interpolating that data into interim years.

Under congested conditions the initial base speed inputs should always be using the prevailing operating speed rather than a posted speed limit. Posted speeds should not be used in a GHG analysis as they are not generally representative of conditions. Posted speeds could easily overstate speeds in congested areas while understating the same areas under free-flow (e.g. night operation) conditions. These could be based on a combination of private probe data sources (e.g. RITIS) for the existing conditions with travel demand models either with a full model or a sub-area defining the changes over time (typically in five-year increments) for future years. FREEVAL (see Chapter 11), a calibrated micro-simulation, a calibrated-for-speed travel demand model, or post-processing travel demand model link speeds will help calculate speeds that could be used in the GHG emissions analysis.

Travel demand models typically only account for reoccurring congestion because of roadway capacity limitations. Regional MPO models usually do a better job estimating speeds as the overall process is more rigorous with activity-based models providing better values than trip-based ones. Regardless of model type, travel model speeds should not be used directly as these models do not validate speed outputs as direct use of these could over/underestimate the potential GHG emissions. Ideally, a travel demand model would need to be calibrated for speed or at the very least model link speeds must be post-processed with known probe speeds, so the reoccurring congested speeds are correctly represented (see Section 16.4.3 in the MSAT future no-build and build calculation steps) over each modeled period. Even with post-processing, resulting speeds may be too high or low and thus need to have bookend speed values established and speeds modified to match those upper and lower bounds. Future year and interpolated years have a high uncertainty for speeds as congestion does not grow in a linear matter over time.

For analyses with substantial non-reoccurring congestion, using RITIS Probe Data Analytics Congestion Scan or Causes of Congestion tools can quickly evaluate the existing year conditions over that entire year to for substantial impacts from incidents, weather, construction/other full/partial closure events that are beyond the typical reoccurring congestion patterns. These impacts can be evaluated on travel time, user delay or speed basis which can be incorporated into the analysis (e.g. assuming a lower operational average speed).

Alternatively, operational models (see Chapter 11) at the meso or micro scale can be constructed to evaluate the effects of incidents, special events, and adverse weather conditions (see Appendix 11F for guidance on developing related adjustment factors). These models can be complex and time-consuming to construct, however, if the operational analysis used these reliability methodologies already, extra time would not be necessary. However, if it desired to include the effects of weather on future no-build and build alternatives, including interim years, then this will require the extra effort to modify these models for projected future impacts.

The ODOT Climate Office can help identify information sources/guidance to better include future severe weather risks due to climate change impacts. These models also better capture the effects of queue spillback in reoccurring congestion. Micro-simulation is too detailed for a reliability analysis for a regional area but could be developed to provide congested speeds on a facility-level basis. There is potential for mesoscopic simulation to help fill in the non-reoccurring data knowledge gap for larger areas, but those tools have not been explored at the time of this writing.

Planning-level analyses covering larger areas involving many roadways, can use the SHRP2 reliability screening methodology shown in Section 11.5 for any freeway, multilane highway and urban arterial facility. This process could be potentially semi-automated with the use of consistent input/output datasets and scripts to provide more robust reporting of the variation in speeds so important in GHG analysis.

Emission rates: These usually include the effects of different fuels and fleet mixes at different speeds, roadway type, vehicle/fuel types in the overall rate. Section 16.5.5 provides approach for MOVES-travel model methods. The emission rate method can be applied by the travel modeler with emissions data provided by ODEQ. For more involved MOVES inventory methods additional coordination is needed with the AQ analyst. Emission rates are applied outside of the traffic data development process, which for the 16.5.6 is a straight-forward post-processor on the travel model outputs (Figure 16-21).

Contact the ODOT Air Quality lead for more details and any seasonal requirements needed (i.e. identification of specific periods to be used).

Uncertainty: Given the general low-level of details available, especially at the scoping level even for construction phase projects where many of the GHG analyses are done, and the companion inherent assumptions that also must be taken (e.g. disaggregation to speed bins and vehicle types, EV adoption and vehicle size assumptions in MOVES forecasts) will mean that there could be considerable uncertainty in the final results. Traffic volumes can easily vary 10% across days with similar characteristics. Travel demand models usually have an uncertainty range of 10-20% as volume calibration is only done on certain links with most in higher functional classes, so it is unlikely to be able to have full confidence in values in less than +/- 10% - 20%. Where possible, reporting results over a range may be better than an explicit value. When full details are available as part of a refinement planning effort it should be possible to have confidence in values down to a +/- 10% range.

2. Infrastructure (Construction, Materials & Maintenance) Emissions

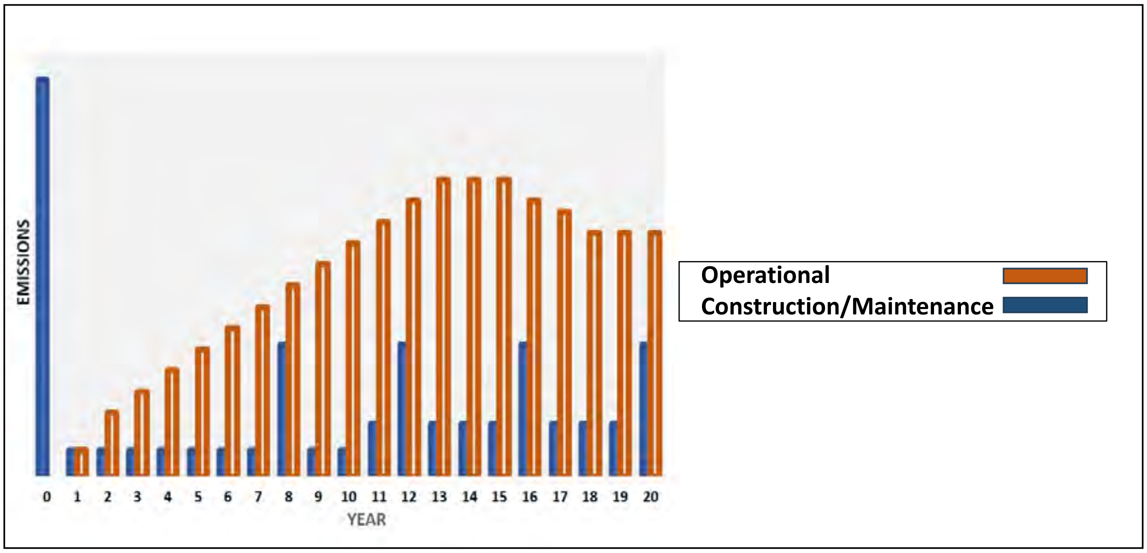
Construction emissions include the embodied emissions used in the materials used to build the project as well as from construction traffic delays. Maintenance emissions are from fuel and materials used in routine activities such as re-surfacing. Both types of emissions are best calculated in FHWA's sketch-level Infrastructure Carbon Estimator (ICE) spreadsheet tool. The tool requires a minimal set of data to calculate yearly GHG estimates (See Section 16.5.4). The tool can output total cumulative emissions over the project horizon (e.g. could be any duration; 20 or 30 years is typical), or alternatively, the annual average of the time period selected. Traffic data inputs should be created for each analysis year using the same assumptions as used for the fuel cycle (operational) emissions for consistency.

3. Combined Accumulated Lifecycle Emissions

Emissions from both the operational/user and construction/maintenance aspects need to be combined cumulatively for the life of the project as it is the accumulation of lifecycle GHG emissions over time that has the impact on the climate, instead of just relying on the typical base, year of opening, and future year data points. These accumulated emissions represent the overall project impact which is the difference between the no-build and the build conditions. For GHG reporting, these emissions will need to be converted to lifecycle, rather than tailpipe emissions, including upstream fuel cycle and electricity emissions. This fuel lifecycle accumulated GHG allows tracking progress against the state's 2050 GHG reduction goals represented in the Statewide Transportation Strategy (STS) and the Oregon Transportation Emissions Website.

Exhibit 16-19 shows how the fuel cycle (operational) and infrastructure (construction/maintenance) emissions occur over time. Initially there is a large construction emission as a project is built, but then maintenance emissions occur at regular intervals as part of normal minor or major maintenance activities. The operational emissions track the normal growth of traffic volume over the project horizon. The exhibit shows the GHG emissions that accumulate from a single build scenario. A no-build scenario would be similar but without the initial construction emissions and would have differing levels of operational and maintenance emissions following. Reported accumulated emissions are normally the change between the no-build and build conditions.

Exhibit 16-19 Accumulated Emissions[8]



Typically, this calculation of project impact (build vs. no-build) is suggested to be done in five-year increments via a series of interim years covering the time span between the base year and 2050, or project year horizon. The interim years are necessary since fleet electrification and thus GHG emission rates will not follow a linear curve over time. In contrast, traffic data (VMT by vehicle type) for these interim years can be assumed to be more linear which allows for interpolation to simplify the analysis burden. Accumulated impacts should be calculated for all analysis levels (see Sections 16.5.4 through 16.5.6).

The air quality analyst will take the interim year traffic data inputs and use the appropriate emission tools (e.g. CMAQ sketch -level, ICE, or MOVES regional or project-level) to produce an estimate of GHG lifecycle fuel (operational) and infrastructure (construction/material/maintenance) emissions for the same interim years All interim years would be summed to estimate the total accumulated effect for fuel cycle (operational) and infrastructure (construction/maintenance) lifecycle GHG emissions.

Creation of discrete volumes for interim years would be preferred as this would have less

[8] From "Overview of Methods for Conducting GHG Analysis in the Transportation Planning Process", Michael Grant, ICF (TRB Annual Meeting, 2025).

consistency issues as growth change may not be the same across the period. Discrete volumes allow for multiple inflection points to be added which could have more or less growth than previous or subsequent periods. However, project budget, schedule and data availability may not allow this level of work, so interpolation becomes the next best choice. Having five-year interpolated increments will allow for reasonable inflection points that will track the emission rate change appropriately without creating a huge level of effort.

No-build scenario interim year AADT/VMT and vehicle fractions should be interpolated from existing (base year) & future no-build years. The same can be done between build year (year of opening) and future build (design) year for the build scenario. Section 16.5.7 shows a process that can be used to interpolate interim year values for VMT by speed and vehicle type. Generally, each project operational analysis would have scenarios for an existing base year, a future no-build and a future build as each of these three builds on each other. Since interpolation will be needed to create interim years for the build scenario, a build year (year of opening) scenario will also be needed. Without a year of opening scenario the future no-build growth factors would need to be used to extrapolate the interim build years, but these no-build growth factors will not be the same as the build growth factors which would produce inconsistent results

Unlike volumes, speeds are generally not linear, so straight-line interpolation or extrapolation may not be as accurate. The interpolated interim year volumes can be input into a calibrated micro-simulation, travel demand model, or a deterministic facility analysis software tool (e.g. FREEVAL, Highway Capacity Software, etc.) to calculate corresponding speeds. These calculated speeds could be combined with other volume-based sources (e.g. ATRs, roadway tube classification counts) so that the relationship of speed and volume can be shown ideally in the project or local area so a likely curvilinear projection or speed profile can be developed to parse the VMT data of each vehicle type into each speed bin. Speed changes, especially for trucks, can show substantial GHG reduction even if VMT is not changed. Given the complexity in interpolating both speeds and vehicle type, it is important to check for reasonableness in the resulting values, especially if the project is designed to have different effects on different vehicle groups, such as a truck-only lane.

Currently, the available historic probe speed data will not cover the required 20+ year set of future congested link speeds to determine the resulting interim years in a typical project via regression, so alternative sources will be needed. A future forecast can be supported by looking back just as far as going forward (i.e. 10 years of historical data could support a projection 10 years in the future). Stretching the projections beyond the historical basis for all the desired interim years will likely not be accurate as the representation of the past is not necessarily what the future will be reflecting (i.e. adoption of electric vehicles, impact of CAVs, etc.). Also, speeds could be easily over or underestimated with long-term extrapolation and may need to be capped or minimum speeds established (see the MSAT future no-build section Steps 11 & 12 for more information).

Post-processing travel demand model speeds will be likely the most straightforward way of obtaining multiple interim future year speeds for multiple segments. This will require a robust existing field speed base from probe data (e.g. RITIS) which will be applied to a speed factor from the model (i.e. future model speed divided by the base model speed for each needed segment). The overall limitation of this method is the ability of the model to capture link and nodal congestion. An activity-based model (see Chapters 7 & 17) or a dynamic traffic assignment model scenario (see Chapter 8) would be superior to a standard three or four-step travel demand model, but these are much more complex to create.

Other Oregon locational (or national although this is not preferred) data for use in speed projection could be used if nothing else is available, but ideally the base area/facility characteristics should be as close as possible. For Oregon sources, use of the ATR characteristic seasonal trend groupings (see Chapter 5) can help identify the most applicable areas including any known caveats. One of the important caveats that likely directly applies to this section is that Interstate/freeway sources should not be used on other facility types as it is known that the characteristics are considerably different.

16.5.4 Sketch-level Methods [for Non-NEPA Planning Analyses]

Fuel Cycle (Operational) Emissions

The "Handbook for Analyzing Greenhouse Gas Emission Reductions in Western States" is an additional source of sketch-level project type applications for computing GHG reductions. The GHG methodologies are modified from California and uses state-specific data used to support planning-level analyses in Arizona, Colorado, New Mexico, Oregon, and Washington. The guidance covers all the emission sectors, not just transportation. Within the transportation sector, measures and related calculation methodologies, default values and related guidance are broken into seven groupings:

- *Land use – includes transit-oriented development and street connectivity*
- *Trip reduction programs – includes transportation demand management volunteer and mandatory programs, ridesharing, vanpools*
- *Parking/Pricing management – includes EV charging, on-street parking costs*
- *Neighborhood design – includes pedestrian/bike network improvements, bike facilities, car/bike/scooter share programs*
- *Transit – includes transit network coverage/frequency improvements, bus rapid transit, shelter improvements, transit-supportive roadway treatments (queue jumps, bus lanes, transit signal priority, etc.)*
- *Clean vehicles and fuels – includes EV, natural gas/propane, ethanol and biodiesel-fueled fleet purchases*
- *School programs – includes school bus programs, Safe Routes to School projects*
- *Additional project and project element types are included in Appendix 16D.*

Data requirements, like the CMAQ & ICE tools, are generally light with defaults or guidance on recommendations given. These may include general planning-level data such as number of miles of bike lanes or percentages of bus routes or data elements that are traditionally available from travel demand models such as average trip length. Appendices show specific state defaults and input data for additional reference.

The sketch-level method is used for simple applications where full MOVES-level details are not needed. For federal reporting, FHWA maintains a CMAQ (Congestion Mitigation & Air Quality) toolkit webpage where the tools and reference materials are stored. These Excel-based spreadsheet tools are based on national vehicle fleets and fuel source assumptions, however many of the modules below can accommodate specific vehicle mixes (e.g. heavy truck percentages) or fuel/VMT sources to customize the tool as much as possible to the specific area. There will be differences between the built-in assumptions and Oregon's such as for the proportion of electric vehicles assumed in the fleet used to calculate the GHG values. It is always a tradeoff on level of effort vs level of detail on deciding whether to accept a default or use a specific value, but these differences are generally acceptable at this sketch-planning level in order to save on effort. If available, Oregon-based data is preferred. CMAQ methods can be supplemented, particularly for Oregon-reporting, with other sketch-tool methods included in this section.

The most common project types that may come up in a scoping, programming, or operational process that also have CMAQ tools available. The below tool list is not inclusive:

- Adaptive Traffic Control Systems
- Bicycle and Pedestrian Improvements – Adding new sidewalks, bike lanes, mid-block crossings, etc.
- Congestion Reduction & Traffic Flow Improvements – New traffic signals, adding turn phases, adding signalized left turn lanes, traffic signal coordination, single/multilane roundabout intersection conversions, adding two-way left turn lanes (TWLTL). Roadway reconfigurations (e.g. road diets) can also be assessed using a combination of the TWLTL and bicycle/pedestrian modules.
- Electric Vehicles (EV) & EV Charging Infrastructure – replacement of conventional vehicles and development of EV charging infrastructure
- Transit Service & Fleet Expansion – Increased frequency, added stops, and new vehicles
- Transit Bus Upgrades & System Improvements – replacement of conventional buses with electric vehicles or replacement/retrofit with cleaner/alternative fuels
- Travel Advisories - VMS (Variable message signs) and VSL (variable speed limits)

These tools generally require a moderate number of inputs and the air quality staff doing the GHG calculations can handle some of them (e.g. Project evaluation year). Depending on the specific tool and project type, there are many inputs that the project traffic analyst will need to provide as shown for each following tool. Tool modules can be combined as many project types will have multiple elements. The data inputs or related calculations needed to create them for each project analysis should be kept on separate tabs with the Excel spreadsheet tool for organization purposes. Some default inputs are based on national data, however these generally do not match too well with Oregon, so state-based data is preferred where possible. The new 2025 Oregon Travel Survey data results should be used, or if not available, data from the earlier 2010 Oregon Household Activity Survey.

Traffic data inputs need to be calculated for all analysis years (base, future and all needed interim years) to be able to assess for accumulated lifecycle GHG impacts.

Outputs depend on the tool, but most modules have both traffic performance and emission output summaries. Performance outputs can include items such as VMT, capacity, volume, speed, and travel time across peak, off-peak or daily conditions for both the existing and proposed conditions. All the tool modules below output CO_2 and (equivalent) CO_{2e} reductions that reflect the changes between the existing and proposed conditions. The CO_{2e} emission outputs can be used directly in reporting the GHG results. The performance outputs can be used to help explain and add context to the GHG results (e.g. reduced delay or VMT) by the appropriate air quality/GHG/environmental staff.

It should be noted that the CMAQ tool, since it was developed for air quality pollutants, does not typically account for the full lifecycle emissions. Lifecycle emissions are beyond the scope of this tool and should be coordinated with the ODOT Air Quality analyst before starting work. In some cases the emission rate data tables (Section 16.5.6, Figure 16-20) may be useful. If warranted, future guidance may provide more standardized methods to expand to lifecycle. Even with just the tailpipe portion, emissions accumulated over the lifetime of the project are still possible.

CMAQ Adaptive Traffic Control System Traffic Inputs

This toolkit is intended for application for projects that implement a new adaptive traffic signal control system on an existing signalized corridor.

- **Area type –** Rural or urban
- **Corridor length (mi) –** Distance from center of first signalized intersection to last signalized intersection for each adaptive system extent
- **Number of signalized intersections –** Number of consecutive signalized intersections included in the adaptive system. If there is an intersection gap in the proposed corridor, then each section should be analyzed separately.
- **Total peak hours per day –** Total average weekday peak hours across all peak periods (i.e. AM, PM, etc.). Ideally, this would be based on a visual inspection of a volume profile determined from 48-hr roadway tube classification counts (at least a single or an averaged set of multiple counts if volumes change by more

than 10% along the corridor) in the corridor. Future and interim year conditions could use the peak hour spreading techniques in Chapter 8 to modify the volume profile either with or without a travel demand model.

- **Free-flow speed or posted speed limit (mph)** – A probe-data based free-flow speed is preferred for existing conditions. If probe data is not available, see HCM 7th Edition Equation 18-3 for estimating urban street segment free flow speed. This is a function of speed limit, median type, curb presence, access density and parking. If the posted speed changes along the corridor, weight-average the calculation. Future and interim conditions should use estimation methods or defaults shown in APM Appendix 11A to estimate the free-flow speed if the parameters in the HCM equation cannot be estimated sufficiently.
- **Total volume on corridor (vph) –** Average of both directions of the weekday peak and non-peak hourly volumes. Summing the volumes for the peak and non-peak periods and average to a single hour for both based on the counts used to determine the total peak hours per day above. Average daily volumes (ADT) can be used with a representative K-factor to create peak hour volumes if count-based peak hour volume are unknown. For future and interim conditions, the weekday peak and non-peak volumes will need to be projected to the desired future year following Chapter 6 procedures.
- **Existing total corridor delay (s/veh) –** Calculate total corridor delay as an average of both directions using existing conditions probe data travel times compared to the free-flow travel time using the and the free-flow speed above. Future and interim year delays will need to be estimated from future projected volumes and deterministic analysis software to estimate average speeds and or travel time and intersection control delay (see Chapters 11 through 13) or microsimulation (See Chapter 15) if a separate project analysis is available. Travel demand models (See Chapter 17) can be used as a last resort for speed/travel time determinations if the produced speeds can be accurately calibrated to existing conditions or at least shown that they are representative of actual observed speeds, especially for congested areas.
- **Truck percentage –** Calculate the heavy truck percentage (FHWA Class 6-13) for peak and non-peak periods from a classification count done on the corridor or a representative one elsewhere that has similar characteristics (see Chapter 5). The count, ideally, would be the from the same group of counts used along the corridor that was used to develop other values for this tool. These values need to be an average of any classification counts done on the corridor and with both directions averaged together. Future conditions should use the same percentages as calculated for existing.
- **Corridor delay reduction per vehicle (s/veh) –** Value is optional. This should only be used if there are actual delay reductions available such as from a deployed system or from a microsimulation of a deployed system from a project-level analysis. Like with the total corridor delay, this value needs to be shown for the peak and non-peak periods and based on an average of both directions along the entire corridor.

CMAQ Bicycle and Pedestrian Improvements

This toolkit is intended for any project type that is estimated to divert trips from the automobile mode to the bicycle or pedestrian mode. It is recommended that these projects are included in a travel demand model scenario so that the potential mode shift can be accurately calculated.

> *It is recommended that Bicycle and Pedestrian LTS (See APM Chapter 14) be evaluated for projects under this toolkit to assess the impact on those modes, such as from longer crossing distances and higher AADT.*

- **Daily individual motorized trips by mode –** Number of one-way trips by passenger vehicles (FHWA Class 2 & 3) on a daily basis before and after the project. Small point-level (i.e. intersection) projects can use an available classification count (See Chapter 3) or existing classification data for state highways. Multiply trips by two if directional volumes are not available. Larger corridor and area-type projects should be modeled in travel demand model scenarios with and without the project. These scenarios should include the entire project area plus an appropriate buffer area equal to the typical regional modal trip length (i.e. ¼ mile for walking or a maximum five miles for biking) to capture any trip diversions.
- **One-way trip distance source –** For small projects or ones where travel demand models are not used, choose "Average" but for any projects that use travel demand models to calculate the number of diverted trips above, choose "Distribution."
    - **Average trip distance (mi)** – For any project with the Average trip distance source, use Oregon-based travel survey data over the default national values available.
    - **Distribution of trip distances (%) –** Use the available model trip distribution for determining the diverted trips for the following distance bins: < 1 mile, $1 \le x < 2$ miles, $2 \le x < 3$ miles, $3 \le x < 4$ miles, and $4 \le x \le 5$ miles.

CMAQ Congestion Reduction & Traffic Flow Improvements

This toolkit contains several different project types separated into different modules: intersections, traffic signal synchronization, roundabouts, and two-way left turn lanes (typically added in rural to urban cross-section roadway upgrades or reconfigurations).

The MOVES-based methods shown in Section 16.5.5 are best for roadway expansion and some smaller roadway projects over use of this CMAQ tool.

The intersection module is intended for a single typical four-legged intersection. Intersections with three legs or more than four will be likely more of an approximation. The module covers the following intersection project types:

- New traffic signal replacing two-way or all-way stop control.
- Adds or modifies left or right turn phasing
- Adds a new left turn lane with corresponding left turn phasing

It is recommended that Bicycle and Pedestrian LTS (See APM Chapter 14) be evaluated for projects under this toolkit to assess the impact on those modes, such as from longer crossing distances and higher AADT.

Intersections

- **Area type** – Rural or urban
- **Business district** – Select “yes” if located within a central business district (CBD).
- **Total peak hours per day** (hrs) – Total average weekday peak hours across all peak periods (i.e. AM, PM, etc.). Ideally, this would be based on a visual inspection of a volume profile determined from 48-hr roadway tube classification counts (at least a single or an averaged set of multiple counts if volumes change by more than 10% along the corridor) in the corridor. Alternatively, probe speed data (e.g. RITIS) could be used to determine the peak periods for existing conditions. Future and interim year conditions could use the peak hour spreading techniques in Chapter 8 to modify the volume profile either with or without a travel demand model. If unknown, the default four hours (assuming two-hour AM & PM peaks) can be used.
- **Existing intersection traffic control type** – Signalized or un-signalized
- **Existing AADT, Roadway 1 (vpd)** – Sum of the AADT for both approach directions for the main subject roadway. Volumes can be obtained from the Transportation Volume Tables or created from ADT or peak hour volumes with seasonal and K-factors as appropriate (see Chapter 5).
- **Existing peak hour volume (vph)** - Average of both directions of the weekday peak period volumes. Sum the volumes for each individual peak hour (if known) and average to a single hour. Average daily volumes (ADT) can be used with a representative K-factor to create peak hour volumes if count-based peak hour

volumes are unknown. For future and interim conditions, the weekday peak and non-peak volumes will need to be projected to the desired future year following Chapter 6 procedures.

- **Existing number of lanes –** Total number of through lanes in one approach direction only. If the number of through lanes is different per direction, use the highest value.
- **Truck percentage (%) –** Calculate the heavy truck percentage (FHWA Class 6-13) for peak and non-peak periods from a classification count done on the corridor or a representative one elsewhere that has similar characteristics (see Chapter 5). The count, ideally, would be from the same group of counts used along the corridor obtained from OTMS that was used to develop other values for this tool. These values need to be an average of any classification counts done on the corridor and with both directions averaged together. Future conditions should use the same percentages as calculated for existing.
- **Existing delay per vehicle for Roadway 1 (s**) - Calculate total delay per vehicle in terms of the greater delay value of the two directions. Ideally, this value would be from using existing conditions probe data (e.g. RITIS) travel times compared to the free-flow travel time using. Future and interim year delays will need to be estimated from future projected volumes and deterministic analysis software to estimate average speeds and or travel time and intersection control delay (see Chapters 11 through 13) or microsimulation (see Chapter 15) if a separate project analysis is available. Travel demand models (see Chapter 17) can be used as a last resort for speed/travel time determinations if the produced speeds can be accurately calibrated to existing conditions or at least shown that they are representative of actual observed speeds especially for congested areas.
- **Existing left and right-turn phases for Roadway 1 –** Choice of “yes” or “no”. If intersection is unsignalized, use “no.”
- **Proposed cycle length (s) –** Use the known cycle length for the intersection. Otherwise, defaults of 60, 90 or 120 seconds for two, three or four-phase intersections, respectively can be used. The tool default is 90 seconds.
- **Proposed number of left-turn lanes for Roadway 1 & 2 –** Enter in the number of left turn lanes for both roadways that will be constructed.
- **Proposed left and right turn phases for Roadway 1 & 2 –** Choose “Yes” or “No” depending on whether protected left and right turn phases for both roadways will be used.
- **Green time to total cycle time (g/C) for Roadway 1 & 2 –** Enter in the estimated g/C ratio for both roadways. The sum of the two roadways must sum to 1. Ideally, this will come from the existing deterministic project traffic analysis or can be estimated using an HCM-compatible analysis using appropriate defaults and values consistent with the other tool inputs.

Volumes, cycle length, g/C ratio or delay may need to be modified by the analyst if computed existing v/c is over 1.0 to arrive at a v/c ratio ≤1.0 as tool does not produce any emission reductions at v/c's that exceed capacity.

Traffic Signal Synchronization

This module is only intended for coordination of existing traffic signals, and not for adding new ones. New signals are covered in the section above on intersections.

- **Road type –** Choice of rural or urban
- **Corridor length (mi) –** Length of the signal coordination project. The default is one mile if the exact length is unknown.
- **Number of signalized intersections –** Number of signalized intersections affected by the project with a required minimum of two sites.
- **Number of lanes -** Use the number of through lanes for one approach direction.
- **Posted speed limit (mph) –** Use the weight-averaged posted speed limit along the project corridor.
- **Average cycle length (s) –** Average the cycle length for the traffic signals along the corridor using the existing timing. Defaults of 60, 90 or 120 seconds for two, three or four-phase intersections, respectively can be used.
- **Truck percentage (%)** - Calculate the heavy truck percentage (FHWA Class 6-13) for the peak periods from a classification count done on the corridor or a representative one elsewhere that has similar characteristics (see Chapter 5). The count, ideally, would be the from the same group of counts used along the corridor that was used to develop other values for this tool. These values need to be an average of any classification counts done on the corridor and with both directions averaged together. Future conditions should use the same percentages as calculated for existing.
- **AADT (vpd)** - AADT volumes for both directions and all lanes from the Transportation Volume Tables or created from ADT or peak hour volumes with seasonal and K-factors as appropriate (see Chapter 5). This should be the average value along the corridor if there are multiple data points. Alternately, travel demand model link daily volumes (averaged across all links that make up a particular segment by direction) can be used for existing and future volumes if actual count data is unavailable.
- **Peak hour volume (vph)** - Averaged over the length of the segment, for both directions and all lanes by direction for the weekday peak hour volumes. Average daily volumes (ADT) can be used with representative seasonal factors, daily K-factor and directional D-factors to create weekday peak hour volumes if count-based peak hour volume are unknown (see Chapter 5). For future conditions, the existing volumes will need to be projected to the desired future year following Chapter 6 procedures. Alternately, travel demand model link volumes (averaged

across all links that make up a particular segment by direction) can be used for existing and future volumes if actual count data is unavailable.

- **Existing corridor travel time (min) –** Calculate total corridor travel time as an average of both directions using existing conditions probe data (e.g. RITIS) travel times.
- **Total peak hours per day (hrs)** - Total average weekday peak hours across all peak periods (i.e. AM, PM, etc.). Ideally, this would be based on a visual inspection of a volume profile determined from 48-hr roadway tube classification counts (at least a single or an averaged set of multiple counts if volumes change by more than 10%) in the corridor. Alternatively, probe speed data (e.g. RITIS) could be used to determine the peak periods for existing conditions. Future and interim year conditions could use the peak hour spreading techniques in Chapter 8 to modify the volume profile either with or without a travel demand model. If unknown, the default four hours (assuming two-hour AM & PM peaks) can be used.

Roundabouts

This module is for a new roundabout installation replacing a typical stop or signal-controlled intersection. Single or double lane roundabouts with three or four legs can be analyzed with this tool.

It is recommended that Bicycle and Pedestrian LTS (See APM Chapter 14) be evaluated for projects under this toolkit to assess the impact on those modes, such as from longer crossing distances and higher AADT.

- **Area type –** Rural or urban
- **Business district –** Select "yes" if located within a central business district (CBD).
- **Existing total peak hours per day (hrs)** - Total average weekday peak hours across all peak periods (i.e. AM, PM, etc.). Ideally, this would be based on a visual inspection of a volume profile determined from 48-hr roadway tube classification counts (at least a single or an averaged set of multiple counts if volumes change by more than 10% along the corridor) in the corridor. Alternatively, probe speed data (e.g. RITIS) could be used to determine the peak periods for existing conditions. Future and interim year conditions could use the peak hour spreading techniques in Chapter 8 to modify the volume profile either with or without a travel demand model. If unknown, the default four hours (assuming two-hour AM & PM peaks) can be used.
- **Existing intersection traffic control type –** Choice of either signalized or unsignalized
- **Existing approach AADT (vpd) –** Approach (directional) AADT volumes from the Transportation Volume Tables or created from ADT or peak hour volumes with seasonal and K-factors as appropriate (see Chapter 5). Alternately, travel

demand model link approach daily volumes can be used for existing and future volumes if actual count data is unavailable.

- **Existing approach peak hour volume (vph) –** Calculate or use the peak hour volume for each approach. Average daily volumes (ADT) can be used with a representative seasonal factors, daily K-factor and directional D-factors to create peak hour volumes if count-based peak hour volume are unknown (see Chapter 5). For future conditions, the existing volumes will need to be projected to the desired future year following Chapter 6 procedures. Alternately, travel demand model link approach volumes can be used for existing and future volumes if actual count data is unavailable.
- **Existing approach truck percentage (%) -** Calculate the heavy truck percentage (FHWA Class 6-13) ideally for the peak periods but could also be based on daily volume from a classification count done on the corridor or a representative one elsewhere that has similar characteristics (see Chapter 5), HPMS data, or other accepted source. The count, ideally, would be the from the same group of counts used along the corridor that was used to develop other values for this tool. Future conditions should use the same percentages as calculated for existing unless project traffic projections are available that show it differently. Otherwise the default six percent value can be used if there is no other information available.
- **Existing approach delay (s) –** Enter the existing and future no-build delay for the existing intersection. Actual field data for each intersection is highly preferred, which can be gathered through a special study or through a probe-data tool such as RITIS. If field data is not available, then a calculated value can be obtained from an HCM-compatible intersection traffic analysis which might be available for the project.
- **Existing approach number of lanes –** Choice of either one or two approach lanes
- **Existing approach left and tight turn lane percentage –** Calculate from a turning movement count done at the subject intersection for existing conditions, from future no-build projected traffic (see Chapter 6) or alternatively from a travel demand model-based select link analysis (see Chapter 17).
- **Proposed number of circulating lanes –** Choose one or two circulating lanes that the proposed roundabout will have. If unknown, use the default one lane as most roundabouts are single lane.

Two-Way Left Turn Lanes

This module is intended for conversion of an unseparated median segment between any two major intersections (either stop or signal-controlled) into a two-way left turn lane (TWLTL). This project type is commonly part of rural to urban upgrades or as part of a roadway reconfiguration (e.g. road diet). If volumes change more than 10% or the number of lanes change across an intersection, then that location should be considered a “major” intersection for the purposes of this tool and the segment broken into new segment(s). A separate set of input data is required for each separate segment. A single segment may include any number of accesses or minor roadways.

It is recommended that Bicycle and Pedestrian LTS (See APM Chapter 14) be evaluated for projects under this toolkit to assess the impact on those modes, such as from longer crossing distances and higher AADT.

- **Area type** – Rural or urban
- **Segment length (mi)** – Length of project converting median to TWLTL or the default value of 0.25 mile can be used if specific detail is unknown.
- **Number of lanes** – Number of through lanes in both directions (either two, four or six).
- **Free-flow speed (mph)** – Use the weight-averaged speed limit along the length of the segment. This is slightly different from other free-flow definitions, but it represents the mid-block speed between signalized intersections
- **Total peak hours per day (hrs)** - Total average weekday peak hours across all peak periods (i.e. AM, PM, etc.). Ideally, this would be based on a visual inspection of a volume profile determined from 48-hr roadway tube classification counts (at least a single or an averaged set of multiple counts if volumes change by more than 10% along the corridor) in the corridor. Alternatively, probe speed data (e.g. RITIS) could be used to determine the peak periods for existing conditions. Future and interim year conditions could use the peak hour spreading techniques in Chapter 8 to modify the volume profile either with or without a travel demand model. If unknown, the default four hours (assuming two-hour AM & PM peaks) can be used.
- **Number of access points** – Use aerial imagery to determine the total number of access points which include all minor streets, driveways, and parking lot entrances on the right (curb) side for a single direction. Both directions will need to be quantified separately.
- **Average percent of left and right turning vehicle per access point** (%) – The percentage of vehicles that turn right and left into each access point on the segment. These need to be averaged across each segment by direction. This typically will not be available unless full specific traffic analysis and volume development has been performed for the project. In many cases, an access point by access point turning movements will not be available save for special high-detailed efforts (e.g. access studies, safety studies, or possibly microsimulations). This can be estimated from available turning movement counts for each segment, or representative counts from similar areas or from travel demand model node intersections with zone centroid connectors using a select-link analysis to determine the turning movement percentages (see Chapter 17).
- **Truck percentage (%)** - Calculate the heavy truck percentage (FHWA Class 6-13) ideally for the peak periods but could also be based on daily volume from a classification count done on the corridor or a representative one elsewhere that has similar characteristics (see Chapter 5). The count, ideally, would be the from the same group of counts used along the corridor that was used to develop other

values for this tool. These values need to be an average of any classification counts done on the corridor and but done separately by direction. with both directions averaged together. Future conditions should use the same percentages as calculated for existing unless project traffic projections are available that show it differently.

- **Peak hour traffic volume (vph) –** Averaged over the length of the segment, by direction for the peak hour volumes. Average daily volumes (ADT) can be used with a representative seasonal factors, daily K-factor and directional D-factors to create peak hour volumes if count-based peak hour volume are unknown (see Chapter 5). For future conditions, the existing volumes will need to be projected to the desired future year following Chapter 6 procedures. Alternately, travel demand model link volumes (averaged across all links that make up a particular segment by direction) can be used for existing and future volumes if actual count data is unavailable.
- **AADT (vpd) –** Directional AADT volumes from the Transportation Volume Tables or created from ADT or peak hour volumes with seasonal and K-factors as appropriate (see Chapter 5). Alternately, travel demand model link daily volumes (averaged across all links that make up a particular segment by direction) can be used for existing and future volumes if actual count data is unavailable.

CMAQ Electric Vehicles & EV Charging Infrastructure

Replacing conventional internal combustion vehicles with electric versions will reduce operating (fuel cycle) emissions. This toolkit covers the replacement of conventional vehicle fleets with electric vehicles along with the development of charging infrastructure for restricted or full access across a couple of modules. Multiple light and heavy vehicle types can be evaluated outside of transit buses (see the transit bus upgrade section below).

EV Purchases/ Restricted Access EV Charging Infrastructure

This module can calculate emissions from an EV purchase project and/or development of restricted (non-public) charging infrastructure. The user can either use the default VMT and/or vehicle replacement number data or enter in their own specific estimates.

- **Restricted access infrastructure checkbox -** Where public vehicle charging is not permitted.
- **Replacement vehicle type –** Corresponds with MOVES vehicle types: passenger car/truck, school bus, refuse truck, single-unit short/long haul truck and combination short/long haul truck. Will need to do multiple calculations if multiple vehicle types are included.
- **Model year of conventional fuel vehicle –** Use a weight-averaged or representative single year if multiple years are included. Alternatively, individual years can be done separately.
- **Conventional fuel type –** Either gasoline or diesel

- **Annual VMT of vehicles to be replaced (miles)-** Use an estimate of how many miles each vehicle type will travel per day, aggregated to an annual basis (may or may not include holidays and weekends) and summed across all vehicles of that type. Travel demand model output may be used to estimate VMT along with external vehicle classification data from classification counts or fleet registration data to estimate vehicle fractions to estimate VMT by vehicle type.
- **Number of conventional vehicles to be replaced-** Estimate number of vehicles to be replaced

The information below is only needed if restricted-access charging infrastructure is desired:

- **Vehicle type to be charged at the facility –** Could be the same type as above. Corresponds with MOVES vehicle types: passenger car/truck, school bus, refuse truck, single-unit short/long haul truck and combination short/long haul truck. Will need to do multiple calculations if multiple vehicle types are included.
- **Model year of EVs-** Use a weight-averaged or representative single year if multiple years are included. Alternatively, individual years can be done separately.
- **Charging access –** Indicate whether the distance to the charging infrastructure will increase or decrease from the conventional fueling infrastructure.
- **Change in total annual VMT (miles)-** Estimate the change in total annual VMT traveled for charging at the restricted access location(s).

Unrestricted Access EV Charging Infrastructure

This module should be used only for the development of public vehicle charging projects. This module assumes that conventional vehicles are replaced by electric vehicles on a 1 to 1 basis.

- **Total vehicle count in study area (veh) –** The study area is variable. This can range from one specific charging site to a whole regional corridor. Vehicles within the study area should be assumed to mostly travel and fuel within the study area. Travel demand model zonal/demographic relationships could be used to determine the number of passenger vehicles based on household income and population otherwise use of aggregated vehicle registration data could be used. Commercial vehicle numbers could be estimated from aggregated vehicle registration data assuming that data is available. Volume growth projections from travel demand models, SWIM, or even historical volumes could be used as proxies to help estimate future years.

- **EV market share –** Estimate vehicle market penetration for the evaluation year. Exhibit 16-20 shows estimated EV penetration from DEQ for various vehicle types other than transit buses. Multiple years will require multiple iterations of the calculation.

Exhibit 16-20 Estimated EV Share

Year	Estimated EV Share (%)			
	Auto (Class 1-2)	Light Trk (Class 3)	Medium Trk (Class 4-5)	Heavy Trk (Class 6-13)
2020	3.0	0.0	0.0	0.0
2025	12.1	3.0	1.0	1.0
2030	28.7	11.9	7.1	5.0
2035+	52.0	31.0	19.2	11.0

- **(Vehicle) Source type distribution for vehicle activity and populations –** Can use the default annual VMT and vehicle count as-is or ideally modify the input table with known data. Travel demand model output may be used to estimate VMT along with external vehicle classification data from classification counts or fleet registration data to estimate vehicle fractions and to estimate VMT by vehicle type. Activity-based model output could be used to estimate individual vehicle populations as these track trips on an individual person basis.

CMAQ Transit Bus & Fleet Expansion

This toolkit is intended for any transit project that has the potential to divert travel from automobiles to transit with new routes, schedules, stops, and vehicle purchases being the most common. It includes a change in both passenger vehicle (mode shift) and transit vehicle miles travelled. A travel demand model scenario is preferred to create the vehicle-miles traveled (VMT) and mode shifts for the transit and automobile modes.

The VisionEval model (both regional and state versions) provides a good mi-level tool for assessing the impact of transit fleet and fuel changes. To learn more about these tools contact the ODOT Climate Office Data & Analysis group.

- **Transit bus VMT before and after project –** Travel demand model scenario-based VMT of the transit vehicles for the no-build (closest reference or base year) case and for a model scenario containing the transit project(s). Transit miles should be increased to represent both in-service miles (with passengers), and non-service miles. Transit provider GTFS schedule data may also be useful to identify bus VMT for existing schedules and/or future schedules created with the ReMIX tool (i.e. for transit route planning, schedule building and visualization).

- **Allocations of (transit vehicle) analysis years –** Transit vehicle analysis year VMT / total fleet VMT distribution. Vehicle model year distributions including any specific route assignments or general use assumptions (i.e. only used in cease of breakdowns) will need to be obtained from the transit agency/operator or can use the national defaults in the toolkit. These distributions can be applied to the total VMT to determine the overall VMT distributions by model year.
- **Allocation of transit vehicle fuel types –** Fuel types include gasoline, diesel, CNG (MOVES-based emission rates) and a full range of alternative fuels (based on US Department of Energy AFLEET alternative fuel factors that modify the MOVES rates) such as biodiesel, battery electric, and hybrids (not exhaustive). Fuel Type VMT/ Total Fleet VMT distribution. Vehicle fuel type distributions will need to be ideally obtained from the transit agency/operator or can use the values shown for medium trucks in Exhibit 16-20 as buses fall into the vehicle Class 4 type. These distributions can be applied to the total VMT to determine the overall VMT distributions by fuel type. These can be modified across different scenarios to show the effect of transitioning buses from internal combustion to electric power. Fuel assumptions should be consistent with DEQ's fleet fuel allocations.
- **Allocation of road types –** Road type VMT / Total Fleet VMT distribution. The two base road types considered are grade-separated highways and all other roads (i.e. arterials, collectors and local streets). Both road types need to be also split between rural and urban areas.
- **Passenger Vehicle Activity Type –** Input choice of either VMT or total number of trips.
- **Passenger Activity (VMT or trips) -** Annual travel demand model derived VMT or annual total number of automobile trips in the transit district service area. This may require creating a model scenario with districts to group the transportation analysis zones together to simplify the quantification of the information. Historic activity may be available from the National Transit Database (NTD).
- **Average One-Way Trip Distance (miles) –** Travel demand model derived average trip distance for the automobile mode in the transit district service area. Use values from the Oregon Travel Survey to determine this. In addition, the National Transit Database may have trip lengths for the desired transit agency.

CMAQ Transit Bus Upgrades & System Improvements

This multiple module toolkit is intended for projects that involve replacing diesel and compressed natural gas (CNG) buses with cleaner/alternative fuels and direct replacement of conventional buses with battery electric no emission vehicles.

The VisionEval model (both regional and state versions) provides a good mi-level tool for assessing the impact of transit fleet and fuel changes. To learn more about these tools contact the ODOT Climate Office Data & Analysis group.

Electric (EV) Transit Bus Replacement

This module is used for determining the emission impacts from replacing conventional transit buses with EV transit buses. The user can either use the default VMT and/or vehicle replacement number data or enter in their own specific estimates.

- **Restricted access infrastructure checkbox -** Where public vehicle charging is not permitted and if this component is part of the project.
- **Model year of current transit buses –** Use a weight-averaged or representative single year if multiple years are included. Alternatively, individual years can be done separately.
- **Fuel type of current transit buses –** Either diesel or CNG
- **Number of transit buses to be replaced-** Number of vehicles to be replaced; should be available from the transit district if not directly available in the project description.
- **Total annual VMT of transit buses to be replaced-** Use an estimate of how many miles the replaced buses travel per day, aggregated to an annual basis (may or may not include holidays and weekends) and summed across all vehicles. These estimates may be available from the subject transit district or could be estimated using service days, schedules, and typical actual routes used mapped in GIS. Travel demand model output could also be used to estimate VMT for the transit mode if available. Otherwise use transit provider GTFS schedule data.
- **Model year of replacement transit buses -** Use a weight-averaged or representative single year if multiple years are included. Alternatively, individual years can be done separately.
- **Change in fueling distance –** Indicate whether the distance to the charging infrastructure will increase or decrease from the conventional fueling infrastructure.
- **Distance changed for fueling (miles) -** Estimate the change in total annual VMT traveled for charging at the restricted (no public use) access location(s).

The below data items are only required if restricted-access fueling infrastructure is included in part of the project. Model year and fuel type of replacement buses would also be needed for this section, but that is the same data as required above.

- **Change in fueling distance –** Indicate whether the distance to the new restricted

access fueling infrastructure will increase or decrease from the current fueling infrastructure.
- **Distance changed for fueling (miles) -** Estimate the change in total annual VMT traveled for fueling at the restricted (no public use) access location(s).

Non-EV Transit Bus Replacement (or drop-in fuel changes such as renewable diesel)

This module is used for determining the emission impacts from replacing conventional transit buses with newer diesel, CNG or alternative-fuel (non-EV) transit buses. The user can either use the default VMT and/or vehicle replacement number data or enter in their own specific estimates.

- **Restricted access infrastructure checkbox -** Where public vehicle fueling is not permitted and if this component is part of the project.
- **Model year of current transit buses –** Use a weight-averaged or representative single year if multiple years are included. Alternatively, individual years can be done separately.
- **Fuel type of current transit buses –** Either diesel or CNG
- **Number of transit buses to be replaced-** Number of vehicles to be replaced; should be available from the transit district if not directly available in the project description.
- **Total annual VMT of transit buses to be replaced-** Use an estimate of how many miles the replaced buses travel per day, aggregated to an annual basis (may or may not include holidays and weekends) and summed across all vehicles. These estimates may be available from the subject transit district or could be estimated using service days, schedules, and actual routes mapped in GIS. Travel demand model output could also be used to estimate VMT for the transit mode if available. Otherwise use transit provider GTFS schedule data.
- **Model year of replacement transit buses-** Use a weight-averaged or representative single year if multiple years are included. Alternatively, individual years can be done separately.
- **Fuel type of replacement transit buses –** Diesel, CNG, biodiesel, renewable diesel (drop-in), hybrids, liquified natural gas, dual fuel vehicles or hydrogen fuel cell. Multiple fuel types will require multiple runs of this calculation.

The below data items are only required if restricted-access fueling infrastructure is included in part of the project. Model year and fuel type of replacement buses would also be needed for this section, but that is the same data as required above.

- **Change in fueling distance –** Indicate whether the distance to the new restricted access fueling infrastructure will increase or decrease from the current fueling infrastructure.
- **Distance changed for fueling (miles) -** Estimate the change in total annual VMT traveled for fueling at the restricted (no public use) access location(s).

Transit Bus Diesel Retrofit

The toolkit also has a module for retrofitting diesel-powered 40’ low-floor transit buses with varying emission reducing exhaust and filtering technologies. These will address CO, NOx, and particulate matter pollutants and not CO_2/CO_{2e}, so it does not apply to GHG estimation.

CMAQ Travel Advisories

This toolkit is intended for projects that implement variable message signs (VMS) or variable speed limits (VSL). Travel advisories can modify driver behavior and smooth overall operations as drivers prepare for slowdowns due to weather, congestion or incidents which can contribute to lowering emissions.

- **Period of activity –** Choice of operational hours: either peak hours, or more commonly, peak and non-peak hours.
- **Hours of activity (hrs) –** Average number of peak hours and (optional) non-peak hours that the advisory is active or projected to be active per day. Alternately, the six-hour default can be used. An average/typical value should be obtained from the ODOT Region or applicable city/county operational centers.
- **Peak and non-peak volumes before conversion (vph) –** Average hourly vehicles per hour in a single direction across all lanes for peak and non-peak hours of advisory operation. Average daily volumes (ADT) can be used with a representative daily K-factor and directional D-factors to create peak hour volumes if count-based peak hour volumes are unknown. Non-peak hour volumes would need to be averaged from on-site or representative 16+ hour counts or automatic traffic recorders or used to create a non-peak hour daily conversion factor to be applied to a source of ADTs (or AADTs such as the Transportation Volume Tables if seasonal adjustments were applied to convert these into ADTs). Alternatively, peak hours could be obtained from a travel demand model if the project was in an urban area and the values were post-processed. The ability to easily get non-peak hours from a travel demand model will depend on the model used, so it may be easier to rely on physical count sources instead.
- **Peak and non-peak speeds before conversion (mph) –** For current conditions, probe speed data (e.g. RITIS) should be used to determine the peak speeds and optional non-peak speeds in the study section. Alternatively, post-processed travel demand model speeds, available project micro-simulations or the posted speed limit can be used for future speeds.
- **Peak and non-peak volumes after conversion (vph); Optional –** Only needed if volumes are predicted to be significantly different during the hours of operation after project completion. This will require available micro-simulations or a post-processed travel demand model scenario (assuming that this project is part of a larger effort with substantial modellable elements) or an external source that indicates the potential volume diversion/change that could be obtained with a VMS/VSL project.

- **Peak and non-peak speeds after conversion (mph); Optional -** Only needed if speeds are predicted to be significantly different during the hours of operation after project completion. This will require available micro-simulations or a travel demand model scenario (assuming that this project is part of a larger effort with substantial modellable elements) or an external source that indicates the potential speed improvements that could be obtained with a VMS/VSL project.
- **Corridor length (mi) –** Total number of miles along the corridor that the VMS/VSL project impacts
- **Heavy duty traffic (%) –** Truck percentage on affected VMS/VSL corridors obtained from available traffic counts, automatic vehicle classifiers, HPMS sample data, etc.

Infrastructure Cycle Emissions

Infrastructure emissions are in addition to the Fuel Cycle (Operational) emissions noted above.

Infrastructure Carbon Estimator (ICE)[9]

The Infrastructure Carbon Estimator (ICE) from FHWA is a comprehensive sketch-planning tool for evaluation of the lifecycle energy and emissions of transportation infrastructure construction, maintenance, and use. The tool includes considerations for material production and transport, energy and fuel used in construction equipment, routine maintenance, and impacts of construction-caused vehicle traffic delay. It is typically assessed in addition to user emissions from other sections (e.g. 16.5.4 or 16.5.5).

- ICE can cover multiple roadways and related project types such as new roadways (not including sidewalks or bike lanes)
- Roadway widening (adding new lanes)
- Roadway rehabilitation (standalone maintenance)
- Lane widening & shoulder improvements
- Roadway realignments
- Pavement reconstruction and resurfacing
- Bridges (new, reconstruction, or widening)
- Culverts
- New parking facilities (surface lot or structure)
- New rail facilities (heavy or light; either underground, at-grade or elevated)
- Bus rapid transit (new or lane conversions, stations)
- New or resurfaced multi-use paths
- New sidewalks, or restriped bicycle lanes
- Signing and lighting

[9] From "Overview of Methods for Conducting GHG Analysis in the Transportation Planning Process", Michael Grant, ICF (TRB Annual Meeting, 2025).

These infrastructure cycle (construction) emissions would be added to fuel cycle (operational) emissions of the project, calculated through the separate GHG analyses and tools noted earlier in this section. GHG results can be presented on an annualized or accumulated basis, viewed by material, life-cycle phase, or by infrastructure type.

ICE tool traffic inputs are limited to establishing the total number of lane and/or centerline miles, number of interchanges, number of bicycle and pedestrian lanes or sidewalks, VMT, and speed of each applicable facility type and any parking spots (i.e. carpool lot) needed for each project. The tool also needs input on mitigation strategies. These include alternative fuels and vehicle hybridization, vegetation management, snow fencing and removal, and in-place roadway recycling. The baseline, business-as-usual, and maximum potential deployment for each strategy need to be specified.

Directional ADT (see Chapter 5) is also needed for determining construction delays on any roadway segments that will require lane closures as part of the subject project. The ADT could be based on project-level volumes, or it could use scoping level AADT values if more specific ADT was not known. If AADT is used as the source, then appropriate seasonal factors are needed to convert these to the peak month ADT. Estimated project-days required for lane closures and percentage of lanes closed are also needed to estimate construction delays. Input on lane closures will likely require coordination from ODOT Region design/workzone or District maintenance staff depending on the specific project.

Workzone/construction delays can also be calculated through several other tools and methods which bring in additional details beyond the ICE tool such as workzone reduced speeds and capacity factors (e.g. closed lanes or shoulders, reduced lane widths or side clearances) which could produce more specific local values.

Simpler methods of calculating construction delay are shown in Section 10.6.8 for a specific roadway section assuming hourly demand volume and capacities are known. Specific workzone -reduced capacities can be added along with durations to determine daily delays. These can be computed by using HCM-based tools (see Chapter 11) or sometimes these are defaulted to specific workzone capacities that are specified by ODOT Traffic Section or region traffic analysts. This would need to be repeated for different volume scenarios (i.e. weekday vs. weekend, or seasonal impacts) to expand the delay calculations to cover an extended period. The PPEAG screening method also shown in Section 10.6.8 could also be used to calculate section delays. A more detailed methodology for specific roadway sections would be the HCM reliability methodology using the Highway Capacity Software (HCS) tool (see Section 9.3.6 and Chapter 11).

16.5.5 Use of MOVES-based Methods

The two methods for estimating fuel cycle emissions using the EPA MOVES software tool are Emission Rates and Inventory methodologies. Details about each methodology are in subsequent sections. Coordination with ODOT’s Air Quality Unit in the

Environmental Section will be necessary to discuss the necessary context and determine and document which methodology applies.

- **MOVES Emission Rates Method** (CO_{2e}) - This methodology may be preferred for regional, area, or scoping-level, planning analyses after coordination with ODOT Environmental Section staff. There is the potential of only the largest projects affecting the final GHG results if done on a regional or system-wide scale, so supplementing this with the sketch-level methodology may be preferable to understand contributions of individual projects. This methodology is applicable to most of the project types listed in Exhibit 16-17 and could also be mixed with the sketch level methods, e.g. to add in elements that are not captured in a travel model, such as EV chargers, many bike/ped improvements or TDM programs. This method is not intended for use in air quality conformity as that must follow certain federal regulations.
- **MOVES Inventory Method** - This methodology is generally compatible with larger-scale infrastructure projects. Like with the MSAT air quality analysis, substantial differences between the no-build and build scenarios may only be noticeable with larger-scope projects or substantial bundles of smaller projects. Smaller projects, and projects outside non-attainment areas, are best served by the sketch or emission rate methods.

Use of either methodology requires that emissions are accumulated over the planning horizon to determine the overall lifecycle-based GHG. Accumulated emissions will require repeating the analysis for multiple interim years (i.e. five-year intervals). See Section 16.5.3 for more details on lifecycle accumulated emissions.

16.5.6 MOVES Emission Rate Method (CO_{2e})

The MOVES Emission Rate method is one of the two methods used to calculate fuel cycle emissions using the EPA MOVES software tool. Project-specific information is processed after running MOVES to develop lookup tables of emission rates. Generally, the number of MOVES runs are less with this method without loss of precision as local travel and emissions data (e.g. VMT) is used instead of widespread use of national defaults. The ODOT and ODEQ staff are available to provide emission rate look-up tables so local analysts do not have to run MOVES. MOVES or other GHG calculating tools, or other external sources are not discussed in the APM other than providing context. Please refer to the Air Quality Manual or the Air Quality Unit for more information on MOVES, project-level GHG calculations, etc.

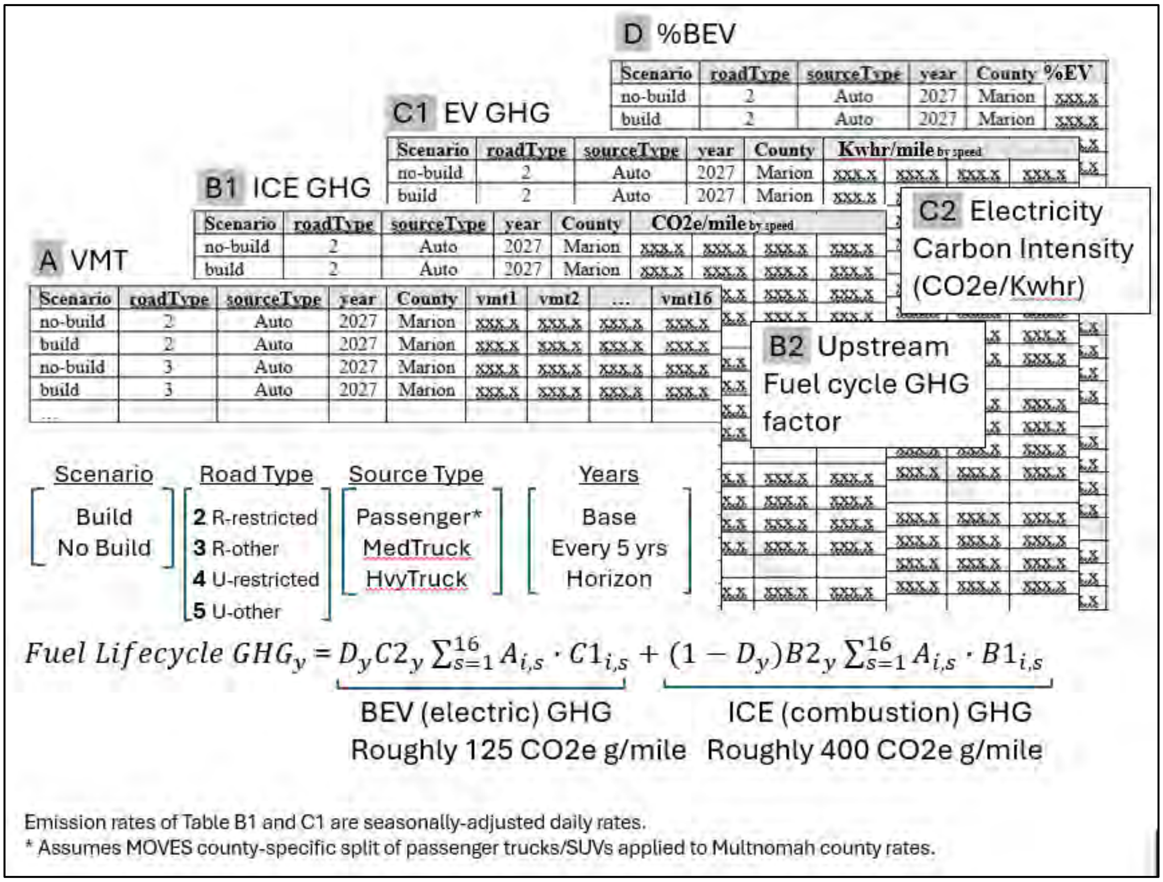
The method follows the steps summarized in Figure 16-21 below (highlighted in gray boxes) to estimate fuel cycle lifecycle emissions from a project. Table A (VMT) data for each yearly build and no-build scenario comes from the travel demand model and is developed following the quantitative MSAT methodology minus the link screening portion shown in Section 16.4.3. Tables B (ICE GHG)-D (%BEV) including the related B2 & C2 factors come from ODOT's Air Quality Unit in coordination with ODEQ. Table A data is multiplied by emission rates for combustion vehicles developed from Table B1 and the B2 upstream fuel factor and for electric vehicles from Table C1 and the C2 local electricity carbon intensity factor, and the share of EVs (Table D).

Table A would be generated by the traffic analyst starting with travel demand model data and ending with aggregated VMT to the dimensions indicated. This MOVES data is broken down by:

- **Scenario –** build versus no-build, or alternative future scenarios
- **Road typ**e **–**MOVES type 2-5; rural and urban with restricted/unrestricted access.
- **Source Type** (vehicle classification) – Automobiles, medium and heavy trucks; typically based on project/regional traffic counts as this is not available from a travel demand model. HPMS data and related counts can also be used for full regions.
- **Years –** base (existing) year and future years, and five-year increment linear interpolation between base and future year. Interim years are used to estimate the accumulated GHG impact on the environment over the life of the plan.
- **Speed –** VMT is split into 16 speed bins, so this requires use of RITIS to obtain existing year, forecasting to obtain future no-build (or model post-processing), and model post-processing of speeds to obtain future build years.

See Section 16.4.3 for the overall VMT development process, data needs, and examples. Section 16.5.7 has additional detail on developing and formatting the VMT data for MOVES-based analyses. Exhibit 16-22 has more details on the VMT dimensions needed.

Figure 16-21 Summary of "Emission Rates" Method for Fuel Lifecycle Emissions



The Table B1 and C1 emission rates per mile are generated from the EPA MOVES model by the air quality analyst. The Table D estimate of on-road share of EVs are included in the default advanced vehicle and fuel technology (AVFT) data in MOVES5. MOVES5 is the better representation of Oregon vehicle and fuel regulations, as one of the California Clean Air Act exception states, while Oregon data in the MOVES4 (most recent past version) model is more representative of national vehicle standards. For the latest EV adoption assumptions contact ODOT and/or ODEQ MOVES staff.

Because the impact of GHG on the environment is due to its accumulation in the atmosphere, it is important to move from tailpipe from internal combustion engines for a single year to fuel lifecycle emissions of all vehicles. This requires adding in emissions from electric vehicles (C1 and C2) and the upstream emissions for extraction of fuel (B2). Fuel lifecycle emissions is consistent with Oregon's Emissions Website, including the Statewide Transportation Strategy (STS) roadmap for GHG reduction progress towards state targets of ground transportation sector. Metropolitan GHG targets (OAR 660-0020/0025) are pulled from this statewide strategy (for more information see ODOT's Metropolitan Transportation Planning website and Appendix 17B for the VMT per capita calculation process). Infrastructure emissions estimated using the ICE tool (see Section 16.5.4) can be added to this lifecycle fuel emissions.

To fully capture the accumulated GHG impact on the environment, the method should be repeated for interim years, and emissions (fuel, infrastructure, and vehicle-if using) summed across the design life of the project or plan. Interim years are required to capture the non-linear adoption of electric vehicles (Table D) and associated change in emission rates over time (Table C1). VMT (Table A) can be assumed to increase in a more linear fashion between base and future year, allowing linear interpolation for interim years. Guidance on developing the VMT for this methodology is included in Section 16.5.7.

The "Emission Rates" method can be applied to a build vs. no-build scenario of a single project (summing associated VMT from impacted link segments in the designated project area) or a system level (e.g. aggregating links within a regional or city boundary for an RTP or TSP, or set of project locations, like a statewide STIP project list).

Example 16-15 is a recent application of the method at the system-level for ODOT's STIP. The system-level application also aligns with the Oregon Transportation Plan's Key Performance Targets for safety, equity, and climate by providing a methodology to evaluate the emissions impact of investments in a project prioritization process.

Example 16-15: 2024-27 STIP GHG Quantification

The GHG impact of the Agency's Statewide Transportation Improvement Plan (STIP) was quantified, in response to a Governor's directive, i.e., the accumulated GHG estimate from all ODOT-led or administered construction phase projects through 2050. This included the fuel (operational) and infrastructure (construction) lifecycle GHG impacts. Many projects were unable to be modeled given small changes to VMT and operational GHG, while nearly all the projects included construction GHG.

This system-level analysis had the fuel cycle GHG calculated with the MOVES Emission Rate method. It relied on the partnership coordination between ODOT and DEQ regarding available data, particularly as it attempted to capture Oregon vehicle and fuel regulations that had just passed, in advance of MOVES5. This effort was fed by data from travel demand models and supplemented with sketch-level tool methods for selected projects and bicycle/pedestrian elements that were not well estimated in these tools. The MOVES assumptions reflected recent Oregon vehicle and fuel regulations and added upstream fuel emissions as well as electricity emissions for a more complete estimate of lifecycle emissions. Infrastructure cycle emissions (construction) were also estimated but are not discussed here.

The methodology involved coding specific STIP projects into TPAU's Highway Economic Reporting System (HERS) software (now phased out of use) and/or the Statewide Integrated Model (SWIM) as applicable and then running the tools to produce outputs in a format compatible with the MOVES model (i.e. VMT by speed bin by vehicle class fraction). Using both tools allowed capturing a wider range of project types. HERS was a segment-based analysis tool using HCM-consistent planning methodologies with each segment being independent, whereas SWIM is a travel demand tool in which upstream links have influence on downstream links.

The first step required that the possible project types needed to be matched to the HERS and SWIM analysis tools for evaluation. Adding/removing through lanes, new turn lanes, new signals, wider shoulders, lane width changes, and median barrier additions-type projects could be analyzed directly in HERS. New roadway segments and lane additions/removal-type projects could be done in both SWIM and HERS. Turn restrictions and projects that re-route or modify demand (i.e. future tolling) or projects under congested conditions required that they were processed first with SWIM to get the capacity-constrained volumes and related latent demand shifts set before exporting into HERS for the final analysis. The HERS SHRP2 reliability post-processor was able to generate more-detailed VMT speed distributions, which is important since emission rates are highly speed sensitive. Other project types in the STIP list that could not be evaluated with SWIM and /or HERS, were evaluated by sketch-level methods (see Exhibit 16-17), which were mostly the simpler CMAQ tools. Some, like curb ramp projects, prompted the development of new custom sketch-level methods.

Once each project was matched to a corresponding tool, the necessary input data was gathered to properly code each tool with the project build and no-build condition details. Generally, at the STIP scoping level, the project descriptions provided did not have enough detail to code the specific projects which required contacting project staff to provide the remaining details.

Projects that modified roadway capacity either by enhancement (e.g. widening, longer auxiliary weaving lanes) or reduction (e.g. roadway reconfiguration) were coded into SWIM before running in HERS. A crosswalk script was written to automate the linking of the SWIM network to the HERS dataset which enabled model data to be transferred back and forth. After all the network changes were completed, the SWIM assignment is run based on the 30-yr period.

The results of the no-build and build SWIM runs were reviewed for reasonableness regarding road segment VMT and speed changes, especially where volume shifts occurred outside of project extents. These external to the project changes were identified and noted. Since HERS receives its VMT exogenously, it is important to pull in any travel model link changes into the corresponding HERS segments. For example, sometimes only a specific project section saw a decrease or increase in VMT. However, other times-adjacent links exhibited diversion-impacted VMT, leading to more comprehensive VMT changes in HERS.

After all the extra roadway segments were noted in HERS, the horizon year (i.e. 2050) no-build and build AADTs from SWIM were transferred to the HERS segments. Non-SWIM projects (i.e. the ones that did not have potential for diversion through latent demand shifts), had the same volumes for both the no-build and build conditions. The HERS software was executed, and the output files containing VMT by class and average speed for each five-year period were passed to a post-processor to calculate the VMT by the different speed bins. The HERS SHRP2 reliability post-processor created a summary of each project with VMT by speed bins for automobiles (FHWA Class 1-3), medium/single-unit trucks (FHWA Class 4 & 5), and heavy/combination trucks (FHWA Class 6-13) in five-year increments over the 30-year period between the base and 2050 horizon years.

The VMT data was then combined with MOVES and lifecycle factors (essentially Tables B-D of Exhibit 16-21) by ODOT's Air Quality Unit to produce the initial set of fuel lifecycle GHG outputs and accumulated emissions. This included a lookup table developed from prior MOVES runs for Multnomah County and estimates of electricity rates, statewide upstream fuel cycles, and electricity carbon intensity factors. Both VMT and GHG results were run through a quality control review to make sure that the overall trends lined up (e.g. if VMT increased for a section then it would be expected that GHG would also rise), magnitudes of change, and direction of changes were reasonable. In some cases GHG changes were the result of speed changes and/or vehicle mix changes alone, with no change in VMT across these segments.

The change in fuel lifecycle (operational) GHG from each project (build vs. no build) was summed and added to selected additional impacts evaluated in sketch tools, such as bike/ped CMAQ work and estimates of GHG reductions from EV chargers. A separate exercise estimated the infrastructure cycle GHG emissions of these same projects. The accumulated fuel and infrastructure lifecycle emission change from the projects was then estimated by summing across all interim years out to 2050.

16.5.7 MOVES Inventory Method (CO_{2e})

The MOVES Inventory Method is the second of the two primary emission computation methodologies. In this method, in contrast to the emission rates, the project specific data is processed before running MOVES.

Full project-level VMT broken down by roadway type, speed, and vehicle classification (i.e. MOVES-compatible traffic data) would be produced by the traffic analyst following the quantitative air quality MSAT analysis covered in Section 16.4.3 with two main exceptions. First, there is no regional link screening process and related AADT thresholds as any project could be evaluated with this method, and second, the analysis will need to be repeated at five-year intervals to support the computation of lifecycle emissions. Once completed, this information will then be passed off to the air quality analyst for running in MOVES and eventual GHG calculations. In comparison with Exhibit 16-20, running MOVES for this method would replace the Table A-B1-C1-D workflow and factors B2 & C2 would be applied to the MOVES output.

16.5.8 Developing and Formatting VMT Data for MOVES-based Analysis

Prior sections have discussed how to properly estimate project level volume and VMT data. To obtain GHG and other AQ measures, such as in the GHG MOVES Inventory and the Air Quality MSAT analysis methodologies, the VMT data must be processed into the input format that MOVES requires.

There are five key dimensions that the project must "bin" the VMT data into. Those dimensions are listed and described as follows:

- Project Scenario – This will typically be a comparison of two scenarios:
    1. No-build
    2. Build
- Road Types – MOVES uses four road type codes or categories (each project segments/links must be assigned or cross-walked via lookup tables to these designations):
    1. 2 – Rural Restricted (highway / access controlled)
    2. 3 – Rural Unrestricted (arterial, or no access control)
    3. 4 – Urban Restricted (highway / access controlled)
    4. 5 – Urban Unrestricted (arterial, or no access control)
- Vehicle Types – This level of MOVES analysis uses three vehicle (source) types:
    1. Light Vehicles – All light passenger vehicle types (Motorcycles, Passenger cars and other two-axle four-tire vehicles; FHWA Class 1-3)
    2. Medium Truck (Single Unit Trucks (SUT) or Two-axle six-tire trucks; FHWA Class 4 & 5)
    3. Heavy Truck (Multi-Unit Trucks (MUT) or Three-axle and greater single-unit trucks and all combination tractor-trailer trucks; FHWA Class 6-13)
- Speed bins – the VMT needs to be assembled into 16 different speed bins:
    1. Speed < 2.5 mph
    2. 2.5mph ≤ Speed < 7.5mph
    3. 7.5mph ≤ Speed < 12.5mph
    4. 12.5mph ≤ Speed < 17.5mph
    5. 17.5mph ≤ Speed <22.5mph
    6. 22.5mph ≤ Speed < 27.5mph
    7. 27.5mph ≤ Speed < 32.5mph
    8. 32.5mph ≤ Speed < 37.5mph
    9. 37.5mph ≤ Speed < 42.5mph
    10. 42.5mph ≤ Speed < 47.5mph
    11. 47.5mph ≤ Speed < 52.5mph
    12. 52.5mph ≤ Speed < 57.5mph
    13. 57.5mph ≤ Speed < 62.5mph
    14. 62.5mph ≤ Speed < 67.5mph
    15. 67.5mph ≤ Speed < 72.5mph
    16. 72.5mph ≤ Speed
- Year – Years need to start at the opening or base analysis year and then include each five-year increment for an approximate 25 to 30-year period to cover for the accumulated impacts of the project. Accumulated horizon years, for example, are

currently set at 2050 in this time of writing in 2023. If the opening year of the project is 2027, then the required years would be 2027, 2030, 2035, 2040, 2045, and 2050. Most projects do not include specific analysis for every five years into the future, so these values will commonly need to be linearly interpolated, which is discussed further below.

After the project has assembled all this information, it needs to be processed into the correct format for MOVES. The format consists of rows of data, where each row contains the 16 VMT speed bins for each unique scenario, road type, vehicle type, year, and county as shown in Exhibit 16-22.

Exhibit 16-22: MOVES Data Format Sample Table Excerpt

Scenario	roadType	sourceType	year	County	vmt1	vmt2	…	vmt16
no-build	2	Light Veh	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	2	Light Veh	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
no-build	3	Light Veh	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	3	Light Veh	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
…								
no-build	2	Med Truck	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	2	Med Truck	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
no-build	3	Med Truck	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	3	Med Truck	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
…								
no-build	2	Hvy Truck	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	2	Hvy Truck	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
no-build	3	Hvy Truck	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	3	Hvy Truck	2027	Marion	xxx.x	xxx.x	xxx.x	xxx.x
…								
no-build	2	Light Veh	2030	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	2	Light Veh	2030	Marion	xxx.x	xxx.x	xxx.x	xxx.x
no-build	3	Light Veh	2030	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	3	Light Veh	2030	Marion	xxx.x	xxx.x	xxx.x	xxx.x
…								
no-build	2	Light Veh	2035	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	2	Light Veh	2035	Marion	xxx.x	xxx.x	xxx.x	xxx.x
no-build	3	Light Veh	2035	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	3	Light Veh	2035	Marion	xxx.x	xxx.x	xxx.x	xxx.x
…								
no-build	4	Hvy Truck	2050	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	4	Hvy Truck	2050	Marion	xxx.x	xxx.x	xxx.x	xxx.x
no-build	5	Hvy Truck	2050	Marion	xxx.x	xxx.x	xxx.x	xxx.x
build	5	Hvy Truck	2050	Marion	xxx.x	xxx.x	xxx.x	xxx.x

Note that the final resulting table is much longer than this; two scenarios x four road types x three source types x six year bins = 144 rows of information with each row containing 16 columns of VMT by speed bin data.

As mentioned above under the "Year" description, most to all projects will not have a VMT analysis for each five-year increment between the opening (build) year and future analysis (design/horizon) year. Possibly, a project may have an interim mid-point year in addition to the base and future bookend years most likely because of a need for an additional phase or scoped support for needed noise/air-quality/GHG data. The current recommended practice given the huge amount of complexity in assembling all needed inputs and factors for the existing and future horizon years is to linearly develop (i.e. interpolate) the VMT by speed bin for years in between these end points. As with other environmental traffic data development, the resulting math is relatively simple but repeated many times which adds overall complexity.

Example 16-16: VMT Data Interpolation

Using Exhibit 16-20 (Table A) and Exhibit 16-21 above as the start, assume that the starting year (opening year/build year) in the analysis is 2027 and the horizon year is 2045 (18 years apart). Under this assumption, the analyst could populate 2 scenarios x 4 road types x 3 vehicle types x 2 years = 48 of the rows required but would still need 4 years of information (96 rows).

To develop year 2030 (24 needed rows), the VMT cells from the 2045 years are compared to the VMT cells for the 2027 years as follows starting with the first VMT speed bin, "vmt1":

Scenario	roadType	sourceType	Year	County	vmt1
no-build	2	Light Veh	2027	Marion	10

Scenario	roadType	sourceType	Year	County	vmt1
no-build	2	Light Veh	2045	Marion	28

2030 vmt1 = ((Future Year VMT – Base Year VMT) / # Years between Future and Base * # of Years from base to new year) + Base Year VMT

2030 vmt1 = ((28 – 10) / 18 * 3) + 10

2030 vmt1 = 13

Scenario	roadType	sourceType	Year	County	vmt1
no-build	2	Light Veh	2030	Marion	13

This can similarly be done for 2050 (just extrapolation for the five years out past 2045)

Scenario	roadType	sourceType	Year	County	vmt1
no-build	2	Light Veh	2050	Marion	33

This interpolation calculation needs to be done for each of the 16 VMT bins, scenario bins, road type bins, and vehicle type bins independently. It is recommended that an Excel spreadsheet analysis or scripting should be used to greatly automate and simplify the task of building all the interim years.

If the project had a midpoint year available, this would shorten the interpolation ranges. For example, if 2037 was available then both 2027 and 2037 would be used as the base and future years, respectively, to develop the 2030 and 2035 interim years. Developing the 2040 interim year and the 2050 horizon year would use 2037 and 2045 as the data basis.

16.5.9 Coordination & Review

Coordination

Any project GHG analysis **requires** initial communication and ongoing coordination with all parties (e.g. ODOT Environmental Section, project staff, consultants). After the responsible parties produce the VMT traffic data estimates, it is passed to the ODOT Environmental Section's Air Quality Unit (or applicable consultant for an active project) to create the GHG emission values from the supplied inputs. The GHG emission estimates would then need to be reviewed along with the traffic data inputs (i.e. VMT, speed, etc.) inputs to determine if overall trends and comparisons were consistent with expectations.

> *It is important to allow enough time to allow for coordination between the analyst and regional project team/leader/planner contacts, in addition to any additional staff that will be running tools (e.g. TPAU-only tools such as SWIM) to avoid making any wrong project assumptions in the* model *coding process.*

Timing

For system-level planning evaluations, like the STIP assessment, it will take considerably longer and have a higher error-potential across multiple steps as more assumptions and "reverse-engineering" will be needed if the GHG analysis is left to do after the project is closed. Retroactively assessing projects for their GHG impact would limit available information which would require document searches and assorted staff inquiries to re-build an understanding of the construction and impact limits of each project. For some evaluation efforts, there may not be time to do the full level of direct project coordination required to cover all the project details without substantial assumptions. If possible, GHG analyses should be done of projects when they are active (i.e. in project development/scoping stages with project activities occurring, etc.)

Projects are part of a complex system and changes to one project may affect others along with user's responses to those changes. Ideally, impacts should be assessed on an accumulated basis (e.g. coding projects into a travel demand model to capture impacts of shifting demand) when time allows although not all project types are capable of being assessed this way.

Review Considerations

Regardless of the underlying methodologies and tools used, all the inputs to be used and the resulting traffic-based outputs need review. Section 19.5.3 contains considerations and elements that should be reviewed.

Sketch-level Method Reviews

For sketch-level methodologies, the inputs should be reviewed including data sources, basis of assumptions, and interim input calculations. Many of the required inputs such as volumes, heavy truck percentages, number of peak hours, delays, and speeds require substantial calculations and assumptions and need to follow other APM methodologies or RITIS best practices. Errors in input data may result in output data not making sense or being of the correct magnitude.

While the deterministic sketch-level CMAQ/ICE tools produce CO_2 emissions and related performance outputs such as speed and travel time, effort needs to be made to ensure that the resulting emission change makes sense with the change of performance data. For example, if delay and vehicle travel decrease, then it would be expected that emissions would also decrease.

Caution should be taken in combining GHG from different sources. ODOT GHG requirements report the full fuel lifecycle GHG (See Exhibit 16-14), which is ideally the accumulated GHG over the project design life. Sketch tools are built from air quality methods that only capture tailpipe elements of fuel cycle emissions. The travel demand model-based MOVES methods instruct on how to add in lifecycle. The ICE tool allows adding the "infrastructure cycle" GHG emissions of a project as well.

MOVES-based Emission Rate and Inventory Method Reviews

Each project requires traffic data review by the traffic data reviewer to ensure that the VMT total and by each vehicle class, speed bin and road type for both No-Build and Build conditions make sense. Input data such as calculated volumes, vehicle classifications, travel demand model scenario assumptions, and speed profile/RITIS information should be checked. The resulting VMT fractions should also be checked to make sure that they sum properly to the total (e.g. 100%) across all the major input categories: road type, vehicle class, and speed bin. Since these efforts involve many data elements, where possible, reasonableness checking should be automated.

Traffic reasonableness checks:

- Model limitations should be documented. Are there key relationships, feedback, or effects that limit the overall completeness? Is there bias in the direction and magnitude that prevent certain results from being used?
- Does the trend based on the comparison between the no-build and build conditions reflect the expectation from the project? If not, check that the analysis process was followed correctly, investigate further, and consult with other staff or project/local contacts
- Is total VMT increasing over the years for both automobiles and trucks? More lane-miles will increase VMT, reduced lane miles will decrease VMT.
- Within each vehicle class category, do the VMT and speed trends make sense? Are the changes the same magnitude and direction across all classes?
- Relative magnitude of VMT increase/decrease – The largest VMT change should reflect the largest capacity or travel time change. Mainline lane-mile changes should be larger than auxiliary lane change which in turn are larger than operational travel-time-only changes.

The results of the traffic review must be documented. Ideally, the traffic review portion would be documented in a technical memorandum clearly indicating what was checked and any changes that were post-review. This could be done in a table or checklist as part of an appendix.

After the traffic review is complete, the VMT file is passed to the ODOT Environmental Section's Air Quality Unit for MOVES analysis. The air quality analyst will provide traffic data (typically done by the traffic analyst) to calculate emission rates by VMT/vehicle class, speed bin and road type to the consultant. The resulting GHG values should be compared with the VMT, speed and travel time changes to make sure changes are consistent and reasonable.

GHG reasonableness checks:

- Does the GHG impacts require the use of a combination of methods (e.g. sketch-level and the Inventory MOVES-based) to get a more complete picture?
- Are the methods capturing the same "cycle" emissions? Do some methods need to be extended to "lifecycle" emissions? Should the analysis report accumulated emissions over project life or plan horizon?
- Transfers between tools, initial results, and computed emission rates should be checked for reasonableness.

VMT & GHG reasonableness checks:

- Does the VMT and resulting GHG changing in the same direction and magnitude? Tolling assumptions, different impacts on higher emitting trucks vs. lower emitting automobiles, and toll diversion onto other roadways of different speeds may create inconsistencies.
- Is the difference between the no-build and build scenarios an actual change discernable beyond versus model random variation (i.e. noise)?
- After GHG emissions are applied, VMT and GHG may not change in the same direction and magnitude, reflecting higher emissions by trucks and speeds. Does the analysis approach miss key relationships, feedback, or effects that limit the completeness of the results that should be noted?
- VMT and GHG results should be compared and reconciled with existing/current project work and messaging

Like with the traffic review, the results of the VMT-GHG review also need to be documented showing what was checked and any inconsistencies noted. This is needed so the traffic analyst and supporting staff can review specific data elements, assumptions, or scenarios to either fix or explain through discussion on why the results are correct. Any project that has inconsistent VMT – GHG trends must be double-checked through all the analysis steps especially for any coding, assumptions, and defaults used. Each project should have the GHG changes (e.g. from no-build to build) for each project documented on whether they appear reasonable (or not because of limitations in the modeling, or data, etc.).

APPENDICES

Appendix 16A – Noise Traffic Data Request Form

Appendix 16B – Air Quality Traffic Data Request Form

Appendix 16C- Sample MOVES Traffic Data Input

Appendix 16D- Western States Guidebook Project Types & Elements

17 TRAVEL DEMAND MODELING

17.1 Purpose

The purpose of this chapter is to provide an overview for non-modelers of building and applying travel demand forecasting models. Modeling practice continues to evolve as methods and tools vary in their level of maturity at any given time. For example, ABM (Activity Based Models) which are discussed further below, are just starting to be created and used in Oregon.

This chapter focuses on travel demand modeling tools currently used in Oregon in TSPs, corridor plans, refinement plans, and project development. Post-processing of model volumes is addressed in Chapter 6. System level modeling for statewide applications, RTPs or Scenario Planning is discussed in Chapter 7. Mesoscopic modeling topics such as focusing, windowing, dynamic traffic assignment, and peak spreading are addressed in Chapter 8. For information on more advanced model topics, contact TPAU.

Many additional sources of information and training on modeling are available for those interested, such as on the ODOT Technical Tools website and FHWA's Travel Model Improvement Program (TMIP). Another source is the Greenhouse Gas Emissions Reduction Toolkit, which focuses on modeling tools used in Scenario Planning.

17.2 Travel Demand Models

Travel demand models are an important tool in analyzing proposed plans, projects and policies. Information from travel demand models is used by decision-makers to identify and evaluate different approaches to addressing transportation issues and to select policies and programs that most closely achieve a desired future vision.

Travel demand models represent travel decisions that are consistent with the actual travel trends and patterns. The travel demand model attempts to quantify the amount of travel on the transportation system. Through careful studies of data on people's travel behavior, mathematical relationships have been developed to predict how many trips people will make, where they will go, by which mode of transportation, and by which specific route. These decisions are influenced by the available transportation system, the spatial location of households and employment, household socioeconomics, and travel costs. Known Oregon travel behavior and relationships from household surveys are used to simulate the impacts on the actual transportation system.

Travel demand models can be used to forecast future travel patterns and demands due to changes in:

- Transportation system changes
  i.e., new roads, increased transit frequency, wider roads with more capacity, new pedestrian connection, closed roads, introduction of connected and automated vehicles (CAVs), etc.

- Land use changes
  i.e., more residential development, a new industrial site, etc.

- Demographics changes
  i.e., more or less people in a specific area, aging population, etc.

The model makes general forecasts of travel patterns 15 to 25 years into the future. Travel demand forecasting can test the impacts of critical “what if” questions about proposed plans and policies. Model outputs can provide users with a variety of information on travel behavior and travel demand for a specified future time frame, such as forecast of highway volumes for roadways, transit forecasts, or the effects of a proposed development or zoning change on the transportation system. In addition, travel demand models may help analysts understand the range of potential impacts from emerging trends and technologies such as CAVs. Appendix 6B provides guidance on CAVs and their potential effects, focused on the methods in the Highway Capacity Manual 7th Edition (HCM) for adjusting roadway capacity for the presence of CAVs in the traffic stream on freeways and at signalized intersections and roundabouts. Model results allow planners to analyze the effects of latent demand and other unanticipated impacts to the system.

Travel demand models vary in what they model and the level of detail they model. All model motor vehicle travel. Some model pedestrian, bicycle, commercial vehicle, transit (MPO areas), and/or freight travel. Some model policies, strategies, programs, investments, and/or emerging technologies such as connected and autonomous vehicles or vehicle miles traveled (VMT per capita. Some model operations, peak spreading, and reliability. The Portland Metro model is the most sophisticated in Oregon, with the ability to model many of the above variables. Some key limitations to be aware of in applying models are.

- Active modes including transit, bike and walk, require a Metropolitan Planning Organization (MPO) model.
- Transit system planning typically requires accurate forecasts of ridership at the route level. Due to insufficient observed data at the required level of resolution, TPAU-created MPO models do not output route ridership estimates.
- Bicycle and pedestrian facility planning: most travel forecasting models are not developed to estimate bicycle and pedestrian demand at the network or facility level. The models produce estimates of zone-to-zone bicycle and pedestrian person trips. They have been calibrated at a regional level for these modes but not at a zonal level or network/route level.
- Transportation Demand Management (TDM) planning: There is frequently the need to know how many trips could be reduced, particularly SOV trips, through workplace TDM strategies, such as preferential parking for rideshare, subsidies for transit riders, transportation allowances, and parking management programs. Trip-based models cannot provide this information because the model is structured at the Transportation Analysis Zone (TAZ) level and does not contain the variables that would allow these strategies to be reflected.

17.2.1 Model Types

The models discussed in this chapter are primarily urban or regional travel demand models. ODOT maintains and operates all the Oregon Small Urban Models (OSUM) and the following MPO/regional models: Corvallis-Albany-Lebanon (CALM), Bend-Redmond (BRM), Southern Oregon (covering the Rogue Valley and Middle Rogue MPOs; a.k.a. SOABM).

The three largest MPOs in Oregon maintain and operate their own travel demand models: the Portland Metro MPO model, the Salem/Keizer MPO model, and the Central Lane MPO model in Eugene - Springfield. All of the current ODOT and non-ODOT travel demand models are shown on the Oregon Travel Demand Models Map.

The trip-based travel demand model methodology has been the best practice standard for many decades. Activity Based Models (ABM) are a new approach that is being implemented.

The key difference between a trip-based travel demand model and an ABM is the treatment of travelers. Trip-based models estimate behavior and travel decisions for zones or groups of travelers. Activity based models work from a synthesized and discrete population for the area, using information and characteristics about individual travelers to estimate travel behavior and decisions made throughout the day. This higher level of detail adds complexity to ABMs but allows more detailed questions to be tested and more information to be provided for a variety of questions. ABMs allow for equity information to be better assessed; how different individuals are specifically impacted. Pricing strategies are also better tested and answered with ABMs.

While trip-based models will continue to be used in the foreseeable future, Oregon is moving towards implementing ABM models in MPO areas. The initial deployment is the SOABM model in southern Oregon, encompassing the Middle Rogue and Rogue Valley MPOs, with other MPOs being planned. The Statewide Integrated Model (SWIM) is also a unique version of an ABM.

17.2.2 Trip Based Models

There are two types of trip-based models used in Oregon: four-step and three-step. Four-step models consist of trip generation, trip distribution, mode choice, and trip assignment. Four-step models such as Joint Estimation Model in R (JEMnR) are used for MPO's. Oregon Small Urban Models (OSUM) are three step models used for non-MPO cities; they do not include the mode choice step.

The following sections further describe the trip-based model steps.

Trip Generation

A travel demand model divides the study areas into transportation analysis zones (TAZ). The main goal of trip generation is to predict the number of trips that are generated by and attracted to each zone in the study area. This stage of the model development process is only concerned with the number of trips that start or end in each zone, not with making the connection between zones. A trip is a one-way movement from an origin (which is always the beginning point of the trip) to a destination (which is always the ending point of the trip).

All the necessary inputs for trip generation are produced using a set of household sub-models that stratifies households by the number of workers, household size, and number of workers by household size. In this pre-generation process, estimates for the following demographic information are prepared for each zone:

- Number of Workers
- Presence of Children
- Auto Ownership

Trips produced by households in a zone are generated by applying trip production rates to the zone's household demographics. The trip production rates are developed from household travel behavior surveys and are applied by trip purpose (i.e., work, school, shop, etc.). An example trip production table is shown in Exhibit 17-1. In this example, the number of daily home-based shopping trips are shown by household size (number of people in the household) and number of workers.

Exhibit 17-1 Example Trip Production (Generation) Table

No. of Workers in HH	**Household Size**			
	1	**2**	**3**	**4+**
0	0.752	1.088	1.188	1.323
1	0.438	0.853	1.182	1.528
2	0	0.652	1.112	1.120
3+	0	0	0.563	1.046

Trip purpose is differentiated to support developing scenarios for different strategies or policies. Patterns of trips are different overall for different purposes: different distributions, different travel times, etc. Some examples of trip purposes are home based work and home-based school trips.

It is important to understand the difference between trip generation rates provided by the Institute of Transportation Engineers ITE Trip Generation manual (refer to Chapter 6) and model-based trip generation.

- ITE trip generation numbers are vehicle trips; model trips are generated as person trips and later converted to trips by mode.
- Generally for a single-family dwelling unit ITE estimates ten vehicle trips per day, which includes all trips made by the household occupants plus delivery or service provider trips. A travel demand model estimates approximately seven home-based vehicle trips per day involving the household occupants; delivery and service trips are captured separately as employment trips.

- Trip generation values in JEMnR are person trips, which are later converted to vehicle trips in the mode choice model. OSUM initially produces person-vehicle trips which are divided by average vehicle occupancy to convert to vehicle trips.
- For more information on ITE-based trip generation, refer to Chapter 6.

Trip Distribution, trip matrices

The trips produced during the trip generation process are distributed by trip purpose to the proper attraction zones. Trip distribution evaluates the probability of travel between each pair of zones based on factors including travel time, travel costs, and size and type of attractions at the destination. The distribution process explains where the trips produced in each zone will go, and how they will be divided among all other zones in the study area (i.e., trips from Zone A to Zone B, A-to-C, A-to-D, etc.). As an example, 100 trips from one zone might be defined as 50 work trips to Zone 'X', 20 shopping trips to zone 'Y' and 30 other trips to zone 'Z'. Additional information on trip distribution is available in Section 6.7.

An external sub-model deals with trips that either begin or end outside the study area. Internal-to-External refers to trips that begin inside but end outside the study area. External-to-Internal is the case where trips begin outside but end inside the study area. External-to-External is for trips that begin and end outside the study area but pass through the study area.

External trips are based on counts and projections of future growth based on historical trends or other methods. SWIM can be used in travel demand model development to estimate growth at external stations, where available (typically state highways and major local facilities).

Refer to Chapter 6 for information on manual (non-model) trip distribution methods.

Mode Choice (JEMnR Only)

Modes are used to categorize the transportation alternatives available for making trips in the study area. Mode choice converts trip demand to trips by mode assigned by time of day (TOD). TOD assigns daily demand by mode to the right hour of the day for production-to-attraction (P-A) and attraction-to-production (A-P) directions. For JEMnR, the time-of-day periods considered are peak and off-peak. JEMnR typically reports out seven different modes as listed below, based on household, socioeconomic and system accessibility attributes.

- Drive Alone – single occupant vehicle (SOV)
- Driver Passenger – The driver in a high occupancy vehicle (HOV)
- Passenger – passenger in an HOV
- Bike – bike mode
- Walk – walk mode

- "busWalk" – a trip that includes a transit vehicle, where the only mode to and from the transit vehicle is the walk mode (not by auto and not by bike)
- Park and ride to bus – a trip that includes a transit vehicle, where some portion of the trip to or from the transit vehicle is by other than walk mode – i.e., consists of park and ride, kiss and ride, or bike and ride trips

Trip Assignment

Trip assignment is the process of allocating vehicle/transit trips for each origin-destination pair in the trip tables onto the roadway/transit network. The allocation consists of identifying routes or paths through the network. The assignment process may be mode-specific, for example, paths for single occupant vehicles may be determined using different criteria than paths for non-SOVs.

Several methods exist for assigning trips from a trip table onto a network, including capacity-restrained, user equilibrium, and all or nothing (AON). Capacity-restrained and user equilibrium assignment methods account for congestion impacts by reducing travel speeds as traffic volumes increase using volume-delay functions, such as the Bureau of Public Roads (BPR) equation. For more information on the trip assignment process, refer to the Supplemental Materials section and consolidated document on the APM web page.

Oregon's models are built to interface with commercial traffic assignment software packages. Currently, Visum (PTV) is used for assignment for all models (i.e. JEMnR, OSUM, and ABM).

Prior to assignment, the daily demand is processed into period origin-destination (O-D) matrices for the assignment software. Two steps that complete this period O-D process are applying diurnal and directional factors.

Diurnal factors

Time-of-day (diurnal) factors are used to estimate travel by hour of the day by splitting the daily demand into its hourly components. These factors can be specific to one hour (peak) or multi hours (2 or more hours). Separate sets of factors are used for the internal-internal (I-I) trips and external trips. The I-I factors are broken down by trip purpose. These factors are typically estimated from data collected in a Household Survey for I-I trips. The external factors are also applied to the daily external-internal (E-I), internal-external (I-E), and external-external (E-E) trips. External factors are typically estimated based on traffic counts and vary by external station.

Directional Factors

Directional factors are used to convert the hourly I-I trips from production-attraction (P-A) to origin-destination (O-D). These factors are typically estimated from data collected in Travel Behavior Surveys. Directional factors, as such, are not used for the external (E-

I, I-E, and E-E) trips. For the E-I and I-E trips, the daily P-A matrices are converted to O-D matrices by summing them with their transpose and dividing by two.
After those factors have been applied and the O-D matrices by desired periods exist, trip assignment can be completed for each period.

Trip assignment is the process used to estimate paths the trips will take, which ultimately results in traffic flow on the network. Trip assignment is typically based on shortest path by travel time. This iterative process assigns trips to specific routes and establishes volumes on links taking into consideration network characteristics (i.e., speed, capacity, intersection controls, etc.) and as iterative assignments are made, congestion can build effecting travel time and path choice. Trip assignment is the final step in the model process, in which zone-to-zone trips from the trip distribution step are assigned to the auto and transit networks.

The trip assignment model can simulate volumes on the existing system or forecast volumes for alternative future scenarios. This allows evaluation of traffic volumes by time of day, by direction, and by mode on the street network.

OSUM

The Oregon Small Urban area Model (OSUM) is a 3-step trip-based travel demand model developed by ODOT to support smaller cities, which uses local data from eight Oregon rural counties to estimate travel behavior (survey data from 1996-1997). It does not include mode choice capabilities, but it provides an effective and efficient method of modeling where traditional forecasting techniques are not adequate.

A new model is usually triggered for a project, where the size and complexity of the local area (population generally >10,000) outstrips the ability of a cumulative analysis to forecast future volumes (refer to Chapter 6). Small cities in proximity may be grouped together into a single model, e.g. McMinnville-Dayton-Lafayette.

OSUM model updates are needed when the difference from model future year and project future year is more than five years. Significant changes to the network or land use may also trigger a model update: large development, major roadway network changes, and/or urban growth boundary expansion. There may also be a desire to change model type (e.g., conversion to ABM), which would require a new model development.

Some characteristics of the OSUM models are that OSUM uses a different fixed average vehicle occupancy for each of five trip purposes. There is no auto ownership information, no transit, walk or bike modes. The trip generation output reflects only auto trips.

The general structure of the OSUM model follows a process consisting of pre-generation, trip generation, trip distribution, and traffic assignment. Within the pre-generation step, all the necessary inputs for trip generation are produced using a set of household sub-models that stratify households by number of workers, household size, and number of workers by household size. The trip generation model generates average weekday vehicular person trip productions by trip purpose. Within the trip distribution step, a destination choice model is used to distribute I-I trips, while I-E, E-I, and E-E trips are handled with separate procedures. Prior to trip assignment, a special model is used to estimate the percentages of external-internal traffic at each external station as well as a daily through (external-external) trip matrix. Trip assignment is performed using a single-class, equilibrium capacity constrained technique.

Jointly Estimated Model in R (JEMnR)

Starting in the mid-1990s, ODOT and the MPOs coordinated their efforts to conduct household travel surveys and to use those surveys to develop a common structure for metropolitan areas utilizing a 4-step trip-based Travel Demand Model structure. As a result, almost all metropolitan area Travel Demand Models in Oregon developed after this time have similar capabilities and use a similar structure.
This initial structure was referred to as Jointly Estimated Model in R (JEMnR). The trip-based approach, e.g., JEMnR, is in the process of being replaced by activity-based models for ODOT-run MPO models and Portland Metro. ODOT MPO models are updated every five years as part of the federally mandated RTP process.

While the JEMnR model structure is similar among metropolitan areas, each model is developed and calibrated so that it reflects conditions and observed travel behavior present in each individual metropolitan area. Travel behavior is sensitive to household characteristics, land use and transportation system characteristics, travel time, and prices. The Portland Metro, the Salem/Keizer, and the Central Lane MPO models are trip-based and very similar to JEMnR but are slightly different in design and approach. While there are differences, JEMnR will be used generically to refer to all the trip based four-step MPO models.

JEMnR is an enhancement beyond OSUM models in that the code structure allows consideration of mode choice. Each mode requires significant data and effort to calibrate. There are statewide standards for consistency for calibration of the auto mode, but not for other modes. For example, the smaller population MPOs (Albany, Bend, Corvallis, Middle Rogue, and Rogue Valley) may not have the same level of data and calibration for active transportation modes as compared to Metro. Therefore while all JEMnR models have the same capabilities, the information that can be provided about modes varies.

17.2.3 Activity Based Models (ABM)

An ABM does everything the trip-based travel demand model does, but with considerably more behavioral content. It deals with individual persons with a comprehensive set of attributes that influence travel and linked trips or tours (i.e. home to shop to work) instead of groups of households and separate trips (i.e. home to shop and shop to work). The ABM is in a sense a simulation of a travel behavior survey. Thus all the questions that could be asked of a travel survey can be asked of an ABM. As such, it is a much more data intensive tool to develop and operate including data resources, staff training and education.A general comparison of ABM and trip-based models is provided in Exhibit 17-2. The table shows the relative ability of each to answer different types of questions.

Exhibit 17-2 Comparison of Trip-Based and Activity-Based Model Applications[1]

Policy Questions	Trip-Based	Activity-Based
Traditional highway projects	***	***
Transit expansion projects	**	***
Air quality conformity / emissions	***	***
Traffic impact studies (analysis)	**	*
Bike/walk planning	*	***
Land use planning – mixed-use, TODs	*	***
System management and operations – ITS / TSMO	*	**
Highway pricing studies	*	***
Equity analysis	*	***
Peak spreading	*	***
Vehicle miles traveled	**	***

[1]Suitability for Analyzing Topic: *** = Good **=Fair *=Limited

An ABM micro-simulates tours, which are groups of linked trips (i.e. trip chaining), as that is how trips occur. This provides context for trips that do not begin or end at home (e.g., mode for lunch trip may be affected by work commute mode) and allows household interactions for shared vehicle use. ABM models do not allow implausible change in modes during a tour like a non-ABM model can. Microsimulation of households and persons over an entire day of travel enables the evaluation of pricing strategies in the context of a household budget.

ABM output is stochastic rather than deterministic in nature, so multiple runs may be needed for a given scenario and an average taken for analysis results. Due to the stochastic nature of ABM models, they are not an ideal tool for traffic impact studies. For more information refer to Chapter 2 on analysis tools and Chapter 15 on traffic micro-simulation models.

Further guidance on ABM model applications is TBD, including when multiple runs are required, how many, and how the results should be averaged or aggregated and reported out.

The ABMs deployed in Oregon introduce three levels of zones with the typical transportation analysis zone (TAZ) created for auto transportation, the micro-analysis zone (MAZ) used to represent walk and bike travel, and transit access point (TAP) representing transit connections. Non-auto modes are captured better because the smaller MAZ structure better represents short trips which would have been within a larger TAZ. Because of the finer zone representation, the ABM has improved capabilities for active transportation, such as the bus, walk and bike modes; but the model output depends on the quality of the input data and level of effort put into calibration.

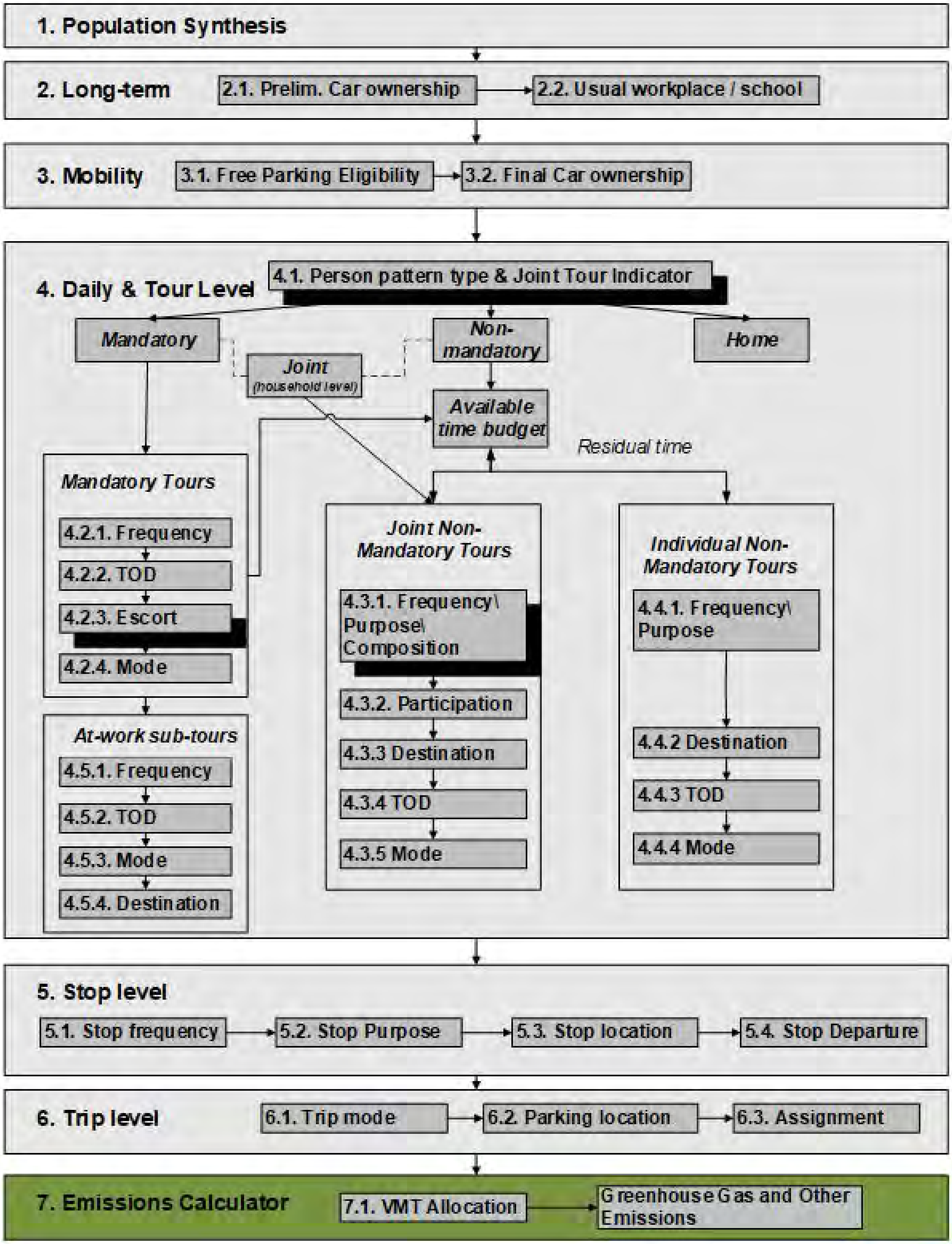
The best ABM application is for providing the required detail for long range regional transportation plans (RTP) required by the Federal Highway Administration (FHWA) and Federal Transit Administration (FTA) for metropolitan areas. The first ABM model in Oregon will be SOABM (Southern Oregon ABM), covering the Middle Rogue and Rogue Valley MPOs.

Activity Based Travel Demand Model Process

Activity-Based Model Components

The current Oregon ABM efforts use OR-RAMP (Oregon Regional Activity-Based Modeling Platform), a series of procedural steps, to run the ABM. The ABM flow chart is shown in Exhibit 17-3. The model starts with long-term and mobility choice models such as auto ownership and usual work and school location choice. Next, the model simulates a typical day of travel for each individual in the region. It starts with day pattern – whether a person stays at home on the simulation day, makes mandatory travel (work and/or school), and/or non-mandatory travel. For each type of travel, it then generates tours and assigns all the required attributes – purpose, destination, time-of-day, and mode. After generating tours, each tour is processed to assign the number of stops along the tour, their purpose, location, and time-of-day. These stops are then processed as trips to assign mode and parking location. Finally, trips are aggregated into demand matrices for assignment at the TAZ and TAP level in Visum. Each model component is described briefly below.

Exhibit 17-3 OR-RAMP Model Flow



17.2.4 SWIM (Oregon Statewide Integrated Model)

SWIM is an integrated tour-based ABM for the entire state. See Chapter 7 for more information. Because SWIM represents economic, land use and transportation behavior in an integrated platform, SWIM has many fundamental differences in comparison to a standard travel demand model. More information can be found here: https://github.com/tlumip/tlumip/wiki .

The following is a brief overview of the model flow in SWIM.

Model design features: The Statewide Model is described as an "integrated" model because the sub-models are interconnected. Information is shared back and forth between the sub-models, mimicking the reactive and interactive behaviors observed in the real world. The model is designed to represent how people and businesses share information and exchange goods and services based on prices and location. The integrated modular design better represents real-world conditions and activities, but requires an immense amount of data, significant development time, powerful computing capabilities and trained staff. For these two reasons, very few states have a statewide economic, land use and transportation model like Oregon's.

The Oregon Statewide Integrated Model consists of specialized sub-models that interconnect with each other:

- **Economic Model**: based on the official state revenue forecast prepared by the Department of Administrative Services, Office of Economic Analysis; provides statewide totals for employment by industry, inflation rate, imports and exports, unemployment rate,
- **Population Synthesizer**: simulates a population with observed Oregon characteristics such as age, household size, household location, income, occupation, worker/non-worker/student status,
- **Production Location Model**: simulates where businesses locate, the commodities they purchase to use as production inputs, number and type of workers hired, the amount of floor space they purchase/lease for their production facility, and production of goods and services sold based on market prices
- **Land Development Model:** identifies land availability based on floor space prices and vacancy rates for firms and households to rent or purchase,
- **Person Travel**: simulates person activities for a typical weekday for the people simulated by the Population Synthesizer, an activity involving travel is assigned a travel mode such as auto, transit, or rail,
- **Commercial Goods Transport**: simulates how commodities are moved as freight by different modes of transport, such as marine, rail, and truck for a typical weekday
- **External Goods Transport**: simulates freight movement for exports, imports and through the state,
- **Transport Model**: assigns trips to a computer network, trips generated in the Person Travel Model, Commercial Goods Transport model, External Goods Transport model

17.2.5 Related Tools

Related tools include: strategic tools (including VisionEval, RSPM/GreenSTEP) – refer to GHG Modeling and Analysis Tools Overview; GIS based land use tools such as PlaceTypes or Envision Tomorrow; land use allocation models such as MetroScope and UrbanSim. There are also many tools which use model output such as the MOtor Vehicle Emission Simulator (MOVES), Models used for system-level analysis are discussed in Chapter 7.

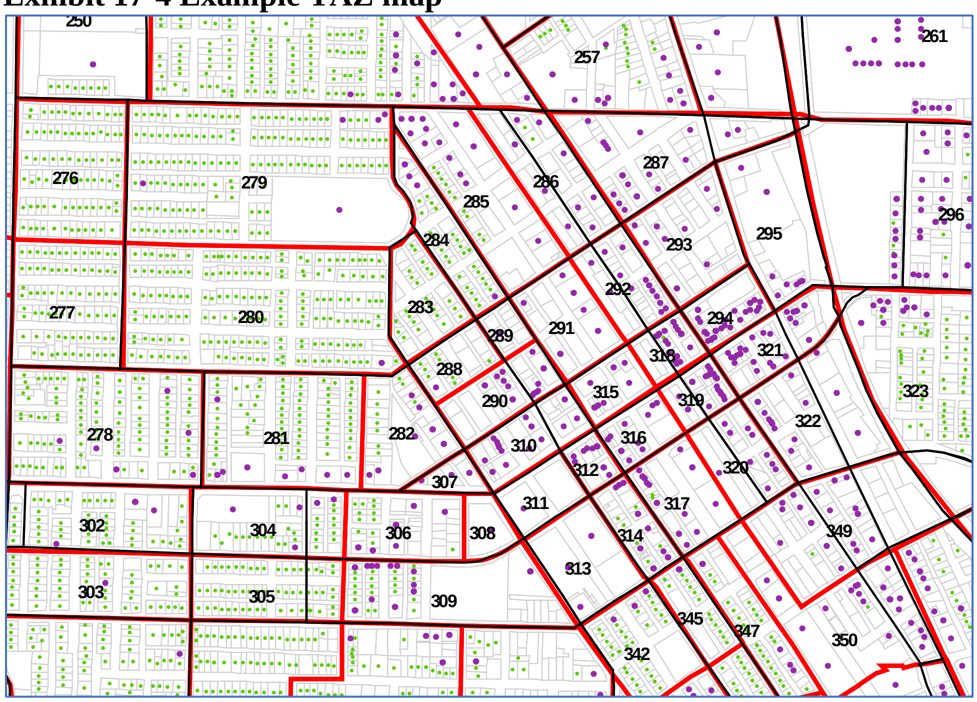
17.3 Model Building Process

17.3.1 Model Building Blocks

Model Zone Structure

The model or study area is broken into homogenous (similar) zones based on land use characteristics. Each zone is identified uniquely. Transportation Analysis Zones (TAZ) are one of the fundamental building blocks of the model. For the newer ABM platform micro analysis zones (MAZs) are another fundamental zone structure. All the land use information (households, employment, enrollment, zonal costs, hotel visitors, greenspaces) are coded to either of these MAZ or TAZ geographies depending on the platform. TAZ/MAZ are where trips originate from and are destined to. An example TAZ map for a portion of a model area is shown in Exhibit 17-4.

Exhibit 17-4 Example TAZ map



For the ABM platform, TAZs are one of the basic spatial units of travel analysis. All the model results can be analyzed at the TAZ level including commuting pattern and mode choice. TAZs are intended to model travel by automobile. Therefore, they can be much larger than MAZs because automobile impedance is less sensitive to small differences in zone size and shape. The size of TAZs can vary with the model application and can be as small as a city block or more than 10 square miles. A typical TAZ system should be specifically designed to fit local planning needs.

TAZ boundaries are typically based on natural and human-made features, such as rivers, canyons, ridges, railroads, and highways. City limits and urban growth boundaries may also determine boundaries.

Model extent is determined by a scoping exercise that depends on several factors such as the use of the model, population density, geography, roadway network and external station considerations.

Recognizing Boundaries

Census Boundaries

Zonal boundaries should follow Census boundaries as faithfully as possible. This is essential as the Census Bureau directly provides socioeconomic data at a variety of geographies including Census Block and Census Tract which can be used as inputs to the model with little or no manipulation. If a TAZ were to split a Tract or a Block, an allocation methodology to distribute the data from the split Census geography needs to be developed.

Physical Boundaries

Natural barriers such as rivers or mountains or man-made barriers such as railway tracks constitute physical boundaries. Physical boundaries restrict free movement and a centroid connector that passes over a barrier to access a road network would be unrealistic. This can be prevented by using the physical barrier as the zone boundary.

Jurisdictional/Political/Planning Boundaries

TAZ boundaries should be consistent with political boundaries such as city, county, MPO and state boundaries. This will help in performing sub-area analysis such as city-city/county-county flows. If the region uses planning districts for performing similar analysis it would be useful to align the TAZs with the planning district boundaries as well.

Zone Centroids represent the center of gravity for the activity in a TAZ. This is commonly not the geometric center of the zone.

Zonal Data

Zonal data or land use from the transportation context usually covers both the physical boundaries on a spatial entity of land (parcel, zones, district, etc.) as well as the demographic, social and economic factors. Land use information is essential in both understanding the present travel patterns and forecasting the future ones.

Travel is related to the population and employment characteristics of a zone. These characteristics are stored in a database used by the model. These data are collected to represent conditions in the model base year. Future land use data are prepared for each TAZ as well, based on input from the local jurisdiction, land use forecasting tools or other manual methods such as buildable land inventories.

Local Agencies are an important partner in developing the base and forecast land use (LU) data necessary for travel demand modeling. The model reflects the local LU data inputs, so it should reflect the local area. The local jurisdictions are asked to develop (or validate) the future LU data at the TAZ level for good reason; it should represent anticipated LU, based on current comprehensive plans and available buildable lands.

For modeling needs, land use information is collected for both population and employment dimensions.

Population

Travel from a zone is related to the population characteristics within the zone. This includes the number of persons residing in the zone as well as their characteristics such as the distribution of ages, income levels, students, and group quarters. Existing zonal population characteristics are obtained primarily from the U.S. Census. Census block group data gives household income, age, size, number of workers.

The make-up of households is integral to understanding the area population demographics. Household attributes such as size, income, auto-ownership, school/college age children, structure type are needed in demand forecasting.

The model uses households as the basic unit for trip generation. Each zone in the model is characterized by its distribution of household attributes, including number of persons, income level, number of autos owned, and type of dwelling unit, which are all factors in trip generation. For example, a zone with mostly high-income households would generate more trips per household than another zone with mostly low-income households, all else being equal.

Households are also classified by type of dwelling unit, such as single-family and multi-family dwellings. The type of dwelling unit is a factor in trip generation. Different types of dwelling units are correlated with different levels of access to automobiles. For example, single family dwellings on average generate more trips than multi-family dwellings because they have a higher level of access to automobiles.

Some additional important considerations related to households are that there can be multiple households in one dwelling unit. Dwelling units that are vacant contain no households. Similarly, group quarters are treated as a type of household with special treatment in the model, such as military barracks, assisted living, or student dormitories.

Employment

Employment is an important factor in defining people's daily activities. Employment tends to be more difficult to estimate and forecast than population. The primary reason might be the fact that employment depends upon local/regional economic forces. In contrast, population tends to grow in a more orderly/predictable fashion. In general, existing employment data used in the model comes from the Oregon Employment Department. Other employment data sources include the Bureau of Labor Statistics (BLS, part of the United States Department of Labor).

For employment data to be useful, they must contain three pieces of information: location of employment, number of employees, and an industrial code for the employer describing the type of work being performed at the job site. Employment data are usually acquired for a specific moment in time.

The model groups employment types into aggregated categories. The most common categories are industrial, service, and retail. Often subcategories are also designated. Some examples are manufacturing vs. non-manufacturing for industrial; trip-intensive vs. non-trip-intensive for retail; or separating out office under the service category. Special employment categories may also be used in the model, such as for hospitals or government.

Oregon models all categorize employment based on the North American Industry Classification System (NAICS). The NAICS "codes" are updated every few years. As an example, employment around server farms and new forms of telecommunications might be different in the 2017 codes than they would have been classified in the 1997 codes. ODOT reviews the employment categorization every time the codes are updated. The latest code set is 2017, which is what the discussion below assumes. In Oregon the employment categories used for the different model types are shown in Exhibits 17-5 through 17-7 below.

Exhibit 17-5 OSUM Employment Categories

Category	NAICS Definition
Retail	440000-459999
Service	520000-569999 and 620000-819999
Other	All other classified employment

Exhibit 17-6 JEMnR Employment Categories

Category	NAICS Definition
Agriculture & Forestry	000000-119999
Mining	210000-219999
Construction	230000-239999
Manufacturing	310000-339999 and 511000-512999
Transportation, Communications & Public Utilities	220000-229999 and 480000-499999
Wholesale	420000-429999 and 491110 (post office)
Retail	440000-459999 and unknown (999999)
Financial, Insurance & Real Estate	520000-531999 and 550000-559999
Service	515000-519999, 532000-549999, and 560000-819999
Government	920000-929999

Exhibit 17-7 ABM Employment Categories

Category	NAICS Definition
Construction	23 series (230000-239999)
Wholesale Trade	42 series
Retail Trade	44 series
Sporting Goods, Hobby, Musical Instruments, Book Stores	45 series
Accommodations and Food Services	72 series
Agriculture, Forestry, Fishing, and Hunting	11 series
Mining, Quarrying, and Oil and Gas Extraction	21 series
Utilities	22 series
Food Manufacturing	31 series
Wood Product Manufacturing	32 series
Primary Metal Manufacturing	33 series
Transportation	48 series
Postal Service	49 series
Information	51 series
Finance and Insurance	52 series
Professional, Scientific, and Technical Services	54 series
Management of Companies and Enterprises	55 series
Administrative and Support and Waste Management	56 series
Education Services	61 series
Health Care and Social Assistance	62 series
Arts, Entertainment, and Recreation	71 series
Other Services (except Public Administration) Religious	81 series
Public Administration	92 series

Certain employment categories have bigger impacts to the results than others. For example, retail/service has more regional impact than agriculture. The total employment must equal the sum of the employment subcategories (government, retail, etc.).

Confidentiality - All model base year employment data are confidential, as provided from the Oregon Employment Department (OED). The reason for confidentiality is so that no party can gain a competitive advantage over another company by knowing how many employees they have. Confidentiality agreements are required on a per project basis for any ODOT, local agency or consultant employees to have access to these data. For ODOT controlled models, TPAU coordinates these confidentiality agreements with OED. Future year data may also fall under the same confidentiality agreements if unchanged from the base year. Further instructions on completing a confidentiality form are found on the "Request for Travel Demand Model Run" form.

Schools and Universities are a special case since they are comprised of both employees and students. A common issue with school employment data is that it can be tied to the school district administration office rather than to the individual schools. The remedy would be to work with the school district to identify the employment for each school. The school district should be able to provide enrollment data as well to estimate student trips generated. Private school student trips if small are generally ignored. Larger private schools are handled on a case-by-case basis. This may also occur with large businesses with satellite offices or field work sites.

Regional employment totals should be compared to household information to ensure a reasonable balance of jobs to workers. In trip-based models this typically means having an employment to household ratio between 0.9 and 1.0. In activity-based models this means comparing total regional employment by category to total regional workers by category. Imbalances should be understood as they may need to be represented in the external model.

OED provides data on employers that have unemployment coverage. Small employers that do not provide this insurance such as agricultural or home businesses are not included. The worker to job balance may help to identify this. Some areas might have significant levels of this in which case region specific treatments may be warranted to properly capture total employment.

Other

There are several other types of land use data required by travel demand models such as parking, hotels, recreation areas, transit service attributes (e.g. percent of zone covered within a reasonable walk time), auto terminal time (time spent at the terminus of the trip, e.g. walking to your vehicle). These will vary depending on the model being used but can be equally important to having a well representative model.

Network

The model replicates the real-world transportation network using interconnected links, nodes and connectors. The network is a geographically scaled model of the transportation system on the ground and includes link lengths and alignments including curvature. Trips are routed along links from one node to the next, for example, from Node 101 to Node 102 to Node 103.

The network is a generalization of what is on the ground, and as such may not include all local facilities in the study area. Generally, only arterial and collector streets are included. Depending on the model, types of networks may include roadway, transit, rail, bike, and walk. Attributes are defined for each link or node on the network, which influence the travel being forecast on the network.

Nodes are representations of intersections, access points, or where link attributes change. Node attributes can include the following:

- **Intersection detail** – Intersection control is an important attribute in a travel demand model to estimate control delay. Some Oregon models only estimate control delay at signalized intersections. ABM models and subarea modeling can estimate delay for other types of controls such as roundabouts, stop control and ramp meters by including additional data such as lane configuration and signal timing such as the green time to cycle length ratio.
- **Turn restrictions** – The model can represent allowed turns including by vehicle classification depending on the model.

The model contains data for each link including length, travel time, and roadway capacity. Links are directional and are defined by a “from” node and a “to” node. Link attributes can include the following:

- Roadway
    - o **Functional classification –** is used in the model as the basis for assigning parameters to links such as capacity or free-flow speed. The common classifications in order of hierarchy are freeway, arterials (principal and minor), collectors and local roads. These are further classified by urban or rural.
    - o **Number of lanes –** Each link in a model is directional. Trip based models contain the number of directional through lanes for each link. For example a two-lane two-way roadway would have two links, with one lane attributed to each directional link. Models with detailed subarea networks also include the number of turn lanes or auxiliary lanes.
    - o **Speeds –** Each link in a model is assigned a speed. The speed represents free-flow speed on the link. ODOT typically uses the posted speed to represent free-flow speed.

- **Traffic counts** – Directional traffic counts on links are used to validate the model's traffic assignment for the base year and to check reasonableness of future year model assignments.
- **Capacities –** Each link in a model is assigned a capacity. The capacity is used in calculating demand to capacity ratios and such as used in volume-delay functions, which are used to estimate delay and speed reductions, which can in turn affect route choice. Capacities are typically expressed as vehicles per lane per hour for the peak hour or peak period, which are then converted to total link capacity. Oregon models use some combination of area type and functional classes to derive the capacities for the model network. In some cases, models may make use of additional variables such as the presence of a median or signal progression factors. Capacity may be adjusted in future model runs to account for the presence of CAVs. Guidance on capacity adjustment factors for CAVs is provided in Appendix 6B and the Final Report for the project *Scenario Guidance for Travel Demand Modeling*, available on the APM website under Supplemental Materials.
- **Volume-delay functions (VDF) –** Volume-delay functions are used in capacity restraint traffic assignments. VDFs reduce travel speed with increased volumes as they approach capacity. VDFs are most often link-based but in some models can also be intersection-based. They vary by facility type and other factors. As the demand to capacity ratio increases, travel times increase, making a link less attractive and potentially shifting trips to other routes or time periods.
- **Link length** – Most new highway networks created for travel demand models are created by importing data from established GIS or linear referencing systems (LRS) datasets. Link lengths need to match the actual travel distance. Some modes may have separate links.

* Allowable Modes (not available for OSUM models) – links can be pedestrian and bike only, or auto only, transit, truck, or any combination of modes. ABM models can include carpool/HOV/HOT lanes. In some cases a managed lane is coded as a separate link.
* Tolls – ABM models can include the amount of toll on a link, which can be by vehicle type (SOV, HOV, truck, bus)
* Median Type – ABM models include a median attribute which assumes an increase in capacity for presence of a physical median barrier
* Progression Factor – an adjustment factor similar to HCM quality of progression
* Transit Routes (not available in OSUM models) – the following are attributes of specific transit routes
    - Fares
    - Frequencies
    - Stops
    - Occupancy

Connectors attach the physical road network to the zone which is represented by a zone centroid. Connectors have unconstrained (infinite) capacity. Connector considerations include:

- Allowable modes
- Connector length/speed (used to calculate travel time)
- Travel time by mode

Special Treatments

OSUM models can have special generators which are locations which have a hard-coded number of ITE trips. A similar treatment also exists in JEMnR models but is applied only to retail employment using a square footage basis. Within a model, certain land uses can be competitive even though in real life they are not (big-box retail store versus a truck stop). Models cannot distinguish the different trips within this commercial zone, so trips can be “stolen” from one attractor to feed another. Special generators are unique to the employer and should have a regional significance both inside and outside the model area. Special generators are typically used for land uses that would not receive the required amount of trips with normal model trip generation like with a regional mall; the model would base trip generation on employment but substantially underestimate the total volume generated from the customers. Adding a special generator requires the model to match an assigned number of trips; for example, if 5,000 trips are coded then 5,000 trips will be sent.

A reason why special generators might be considered is that model employment types are aggregated into broad [North American Industry Classification System](#) (NAICS) categories.
These are the standard categories used by federal statistical agencies in classifying business establishments. For example, the retail employment type does not differentiate between a coffee shop and Home Depot.

In OSUM, special generators are served first (they will always get the daily number of trips coded). The total special generator trips should not be greater than 50% of the total trips from the model. The special generators attract both internal-internal (I-I) and external-internal (E-I) trips so there is a need to determine the total number of trips that they are absorbing. Too many special generators will diminish the sensitivity of the model (forced or directed trips- no destination choice impacts such as congestion or other impedance effects) and they cannot be accommodated by just i-i trips as the external zones need to be adjusted to reflect regional significance.

The number of trip ends should be considered between the base and future year. Competition with other new or existing businesses needs to be considered, otherwise a special generator can inordinately attract trips from other zones, externals, etc. and not leave enough trips to satisfy other uses. It is important that the requestor understands the implications of how special generators affect regional trip generation.

There are instances where a special class of traveler may not be represented in census and city population totals. The following are examples where special treatments may be needed.

- University populations are sometimes captured in city populations (Corvallis as an example), however, travel for the university students differs significantly from the general population; therefore, a special model is required. For example, the OSU sub-model that exclusively deals with university travel within the CALM regional model.
- Some areas have a significant visitor travel component, such as Newport. Visitors are not captured in the general population totals but can be a significant impact on travel within the region. In these cases the visiting population needs to be separately accounted for such using a visitor sub-model. Lodging establishments (hotel/motel, RV parks, rental homes, campgrounds etc.) are equivalent to households for visitors with trips being typically attracted to recreation among other destinations.
- Models typically represent the average time of the year, but travel patterns and magnitudes in some areas in summer or other seasons may differ significantly and require a special treatment. A separate scenario is created by pivoting off the average annual model. The summer scenario would typically include an increased population or a visitor sub-model, increased employment, increased levels of external station volume, and summer special generators.

17.3.2 Base Year construction

Due to data availability and development timelines, it is common for the model base year to be several years prior to the current year. Models can take several years to develop. Some of the input data may lag by a few years by the time of starting the model, such as the employment data. It is important to understand this distinction of the model base year in comparison to the existing year. The model base year data are obtained from many different sources, including census data, state employment data, Origin and Destination trends or data, household travel surveys, traffic counts and field inventory. Collecting this information involves coordination with a variety of data providers and agency partners. Most of these data requirements are described under Model Building Blocks but are summarized here.

- Network
    - Base roadway network
    - Number of lanes
    - Traffic control
    - Posted speeds
    - Functional class
    - Other
- Zone construction
    - Households
    - Employment
    - Other
- Counts

    Traffic counts are needed to compare to model link volumes for the purpose of calibration or validation. Considerations for collecting and use counts include:

- Procedures to attribute counts onto a model network using either ArcGIS or Visum are contained in Appendix 17A. This includes how to use count processor spreadsheet tools found on the Technical Tools page under the Volume Development dropdown to prepare counts for import.
- Most model counts represent average weekday traffic volumes. Use of ATR data requires weekend data to be removed. Refer to Chapter 5 for procedures. These should be 48-hour directional counts. Note: at some locations with high summer travel, a summer day may also need to be developed if travel patterns are significantly different from off season travel
- For external stations, counts need to be at least 48-hour directional classification counts
- In some locations long duration (16-hour intersection counts) may be acceptable, if there are cost savings with other count needs
- After counts are collected, count adjustment factors are applied.
    - Seasonal adjustments are typically needed to estimate annual average volumes from short term counts.
    - Axle factors are used to adjust volume only road tube counts before use. Axle factor procedures are found in Chapter 5.
    - Directional factors are required for combined direction counts unless the road is one-way.
    - Growth factors are used to adjust counts taken in different years to estimate the volume in the model base year. Historical growth factor procedures are found in Chapter 5.

The following datasets are specific to the Base year construction due to the calibration requirements in the following section: Origin destination data from Bluetooth, AirSage, TomTom, etc. If origin-destination (O-D) data are available, it can be used to compare model O-D data. If unavailable, some models may require that O-D data be acquired, for example if visitors represent a significant component of travelers or if needed to validate the O-D patterns in the model.

Speed and/or travel time data sources and tools include Inrix, NPMRDS, and RITIS. These data are not needed for most model macro traffic assignment methods. For more advanced traffic assignment methodologies, speed data may be needed for reasonableness checking or to perform a speed calibration.

Treatment of External Traffic

SWIM is the primary source of external traffic data in most cases. The data are readily available and provides trends for truck and auto flows in Oregon. However, it is important to check the data for reasonableness. There can be areas such as close to borders where the routing trends may not match local data. In those cases there are additional methodologies that can be used to derive traffic patterns flowing into and out of urban areas, such as the Future Volume Tables (Chapter 6) for point volumes, sources of origin/destination data such as from private sources, or other methods from the literature.

Traffic Assignment Considerations

The choice of assignment method can impact the data needs and needs to be considered early in model scoping. There are variations in types of trip assignment methodologies, for more information refer to the Alternative Traffic Assignment Methods Framework Report. For dynamic traffic assignment refer to Chapter 8. Considerations relating to traffic assignment are discussed below.

Volume-delay functions (VDFs) are used in capacity restraint traffic assignments. Existing ODOT trip-based models use BPR volume-delay functions in their macro level traffic assignment treatment. The BPR functions are applied differently for signalized and unsignalized model links. Additionally peak assignment and daily assignment have unique VDFs and treatments:

- Daily- OSUM assigns each hour of day and then sums to generate daily volumes. JEMnR assigns daily (this includes transit) demand in total and PM peak hour only.
- Peak- for OSUM see above. JEMnR has its own hourly peak hour factor that generates demand and then assigns.

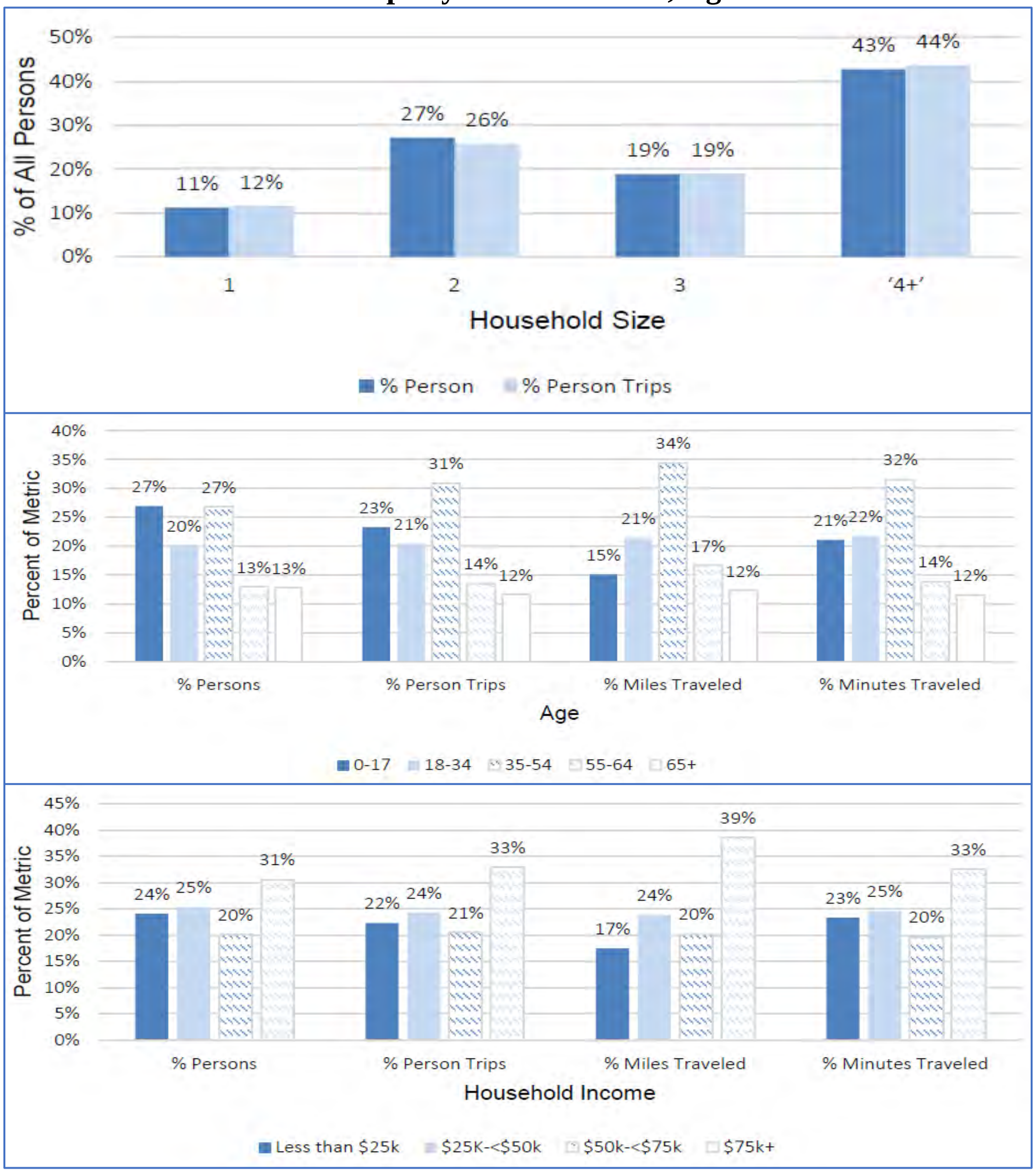
The planned use of the model may require finer detail of traveler segmentation. For example HOV policies require that HOV vehicles be assigned as a separate traveler class, rather than being grouped into a single traveler class.

17.3.3 Model Estimation

In short, travel demand models consist of mathematical relationships between employment, population, and other demographics to replicate trip making. Some of these relationships come from national data such as census or ACS (American Community Survey) or state specific data such as OHAS (Oregon Household Activity Survey).

For trip generation, OHAS is used to determine Oregon values for household size, e.g. number of people per household or trips per household. Some examples of the types of values obtained from OHAS are shown in Exhibit 17-8. Households can also be categorized by age, income, and other characteristics. From this data, basic relationships can be derived, for example how households with higher incomes drive more.

Exhibit 17-8 Persons and Trips by Household Size, Age and Income



Source: Daily Travel in Oregon: A Snapshot of Household Travel Patterns, ODOT, July 2018

The other components of model estimation are much more complex than trip generation. They require a fully functional zone structure and network. All base year inputs need to be assembled at the TAZ level to develop skim tables for the region. A skim table identifies travel times (or distances or costs etc.) between zone pairs. An example skim table is shown in Exhibit 17-9. These are needed to understand choices available to the travelers surveyed in the model regions.

Exhibit 17-9 Example Skim Table

HwyPK.skm-*1 SOVTime

*1 SOVTime | 2 SOVDist | 3 SOVToll | 4 HOVTime | 5 HOVDist | 6 HOVToll | 7 TDTime | 8 TDDist | 9 TrkTime | 10 TrkDist

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	3.42	6.16	10.33	12.41	16.26	11.14	12.57	16.33	18.26	15.14	17.86	20.43	18.87	23.73	22.07
2	6.16	3.17	7.65	7.91	11.88	6.64	9.00	11.83	13.77	10.65	13.38	15.95	14.37	19.24	17.59
3	10.22	8.18	2.16	4.38	6.99	6.50	3.29	11.69	13.62	9.83	9.68	11.33	11.83	15.35	13.89
4	12.43	7.92	4.34	2.57	7.49	6.23	3.79	11.42	13.36	10.24	10.18	11.83	12.33	15.86	14.40
5	15.75	11.41	6.35	6.89	2.66	5.34	4.41	7.08	9.39	5.24	3.65	6.21	5.79	9.50	7.86
6	11.24	6.74	6.04	6.32	5.90	3.30	4.86	7.09	9.02	5.75	7.58	10.16	8.58	13.45	11.80
7	12.66	8.99	3.25	3.80	5.01	4.86	2.13	9.98	11.99	7.85	7.70	9.36	9.85	13.38	11.92
8	16.47	11.96	11.28	11.55	7.77	7.12	9.98	1.96	2.90	3.44	5.88	7.23	4.91	10.44	8.79
9	18.40	13.89	13.21	13.48	10.09	9.06	12.03	2.90	2.30	5.37	8.20	6.95	4.33	8.57	7.27
10	15.27	10.77	9.77	10.32	5.93	5.74	7.84	3.44	5.37	2.80	5.68	8.25	6.67	11.55	9.89
11	18.00	13.50	9.75	10.30	4.36	7.59	7.82	5.88	8.19	5.70	2.40	3.06	4.60	7.12	6.47
12	20.51	16.01	11.25	11.79	6.86	10.10	9.30	7.18	6.94	8.20	3.02	2.15	3.84	5.63	4.98
13	19.00	14.49	11.83	12.38	6.43	8.58	9.89	4.91	4.34	6.68	4.55	3.83	2.55	6.86	5.21
14	23.81	19.31	15.28	15.83	10.16	13.40	13.34	10.44	8.54	11.49	7.08	5.64	6.86	2.09	3.32
15	22.17	17.67	13.91	14.46	8.52	11.76	11.97	8.80	7.26	9.85	6.43	4.98	5.22	3.34	2.62
16	24.43	19.92	17.81	18.36	12.69	14.00	15.88	10.34	8.43	12.11	9.62	8.17	8.14	4.30	4.75
17	26.02	21.52	20.62	21.12	15.50	16.68	18.69	10.52	9.58	12.99	12.43	10.98	10.96	7.12	7.57
18	22.32	17.81	15.90	16.45	10.94	11.90	13.97	7.33	6.30	9.80	8.86	7.40	6.03	6.53	5.22
19	13.13	17.69	11.58	15.39	18.00	17.82	14.31	22.94	24.94	20.84	19.16	20.54	22.02	24.35	23.90
20	7.88	12.44	6.33	10.53	13.14	12.65	9.44	17.84	19.77	15.98	15.83	17.48	17.98	21.50	20.04
21	7.08	12.05	12.33	15.70	19.14	14.44	15.45	19.63	21.57	18.45	21.12	23.48	22.15	26.99	25.34

Source: https://tfresource.org/topics/Skim_Matrix.html

Changes to the highway network must be reflected in the appropriate level-of-service data files (travel time skims) that are input to the model. Level-of-service refers to the characteristics of travel by mode between zone pairs, in the case of OSUM, peak and off-peak travel times between zone pairs.

Reasonableness checks for the level-of-service data (travel times) can be performed by developing a matrix histogram of zone-to-zone impedances or distribution curves using the density function in R. Potential problems may be reflected in unexpectedly large values or concentrations of zone pairs within lower or higher impedance intervals (a formation of an unexpected hump in the curve.

For a more complete understanding of modeling estimation refer to NCHRP Report 716 (Travel Demand Forecasting: Parameters and Techniques).

17.3.4 Calibration and Validation

If estimation was performed for the model region certain sub-models may not need further calibration. If model estimation was borrowed from another region, all sub-models need to be reviewed and calibrated to local data.

Calibration is an evaluation on how well the model replicates observed conditions. Calibration is an iterative process whereby the model parameters are adjusted until model estimates reasonably match the field-measured targets. Calibration requires both software expertise and knowledge of existing travel behavior.

Model validation is the process of testing the performance of the calibrated model using an independent data set (not previously used in the calibration). Validation is an additional check to confirm that a model has been correctly calibrated and closely match the existing conditions.

Calibration is usually performed according to the sequence of the model flow, starting with the higher order sub-models such as trip generation and ending with trip assignment. In a trip-based model the sequence includes trip generation, trip distribution, mode choice, and trip assignment. In an activity-based model the sequence includes long term decisions such as work location and auto ownership, tour generation, trip and stop choice, time of day models, and trip assignment.

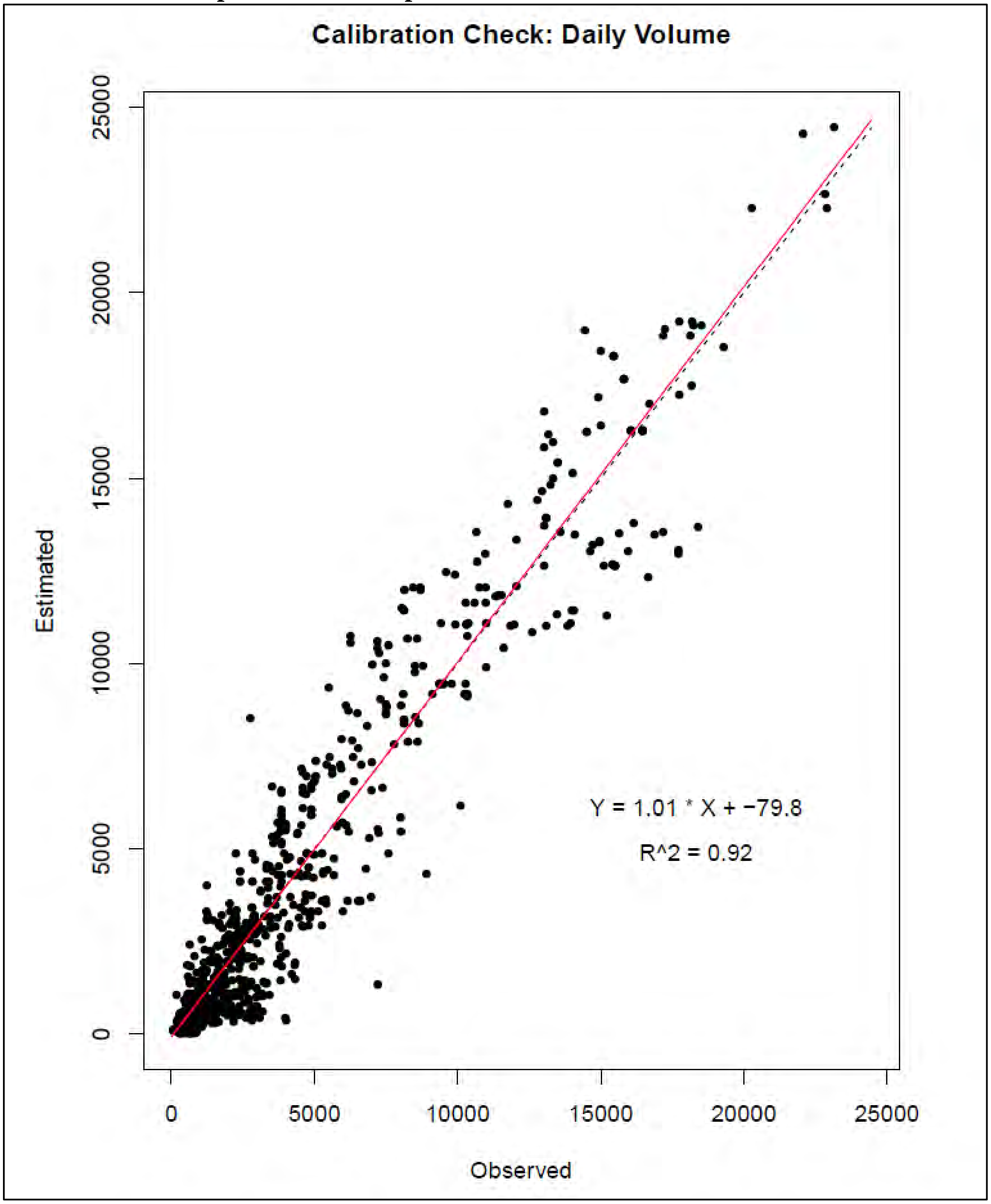
The trip assignment sub-model is common across all travel demand models. Traffic assignment is typically treated as a validation of the calibrated sub-models, which estimate travel behavior. It is common for a model to only be validated to daily and PM peak demand as these are the most common requested periods. Other periods such as mid-day, may be provided with the understanding that the information from these periods may not be validated.

Several measures are used to evaluate the performance of the travel demand model (for more details see model documentation). For traffic assignment validation, three primary types of comparisons are typically used. These are:

- Link scatterplots (by functional class)
- Percent root mean square error
- Screenlines

Link Scatterplots show the results of regressing assigned link traffic volumes on the corresponding link traffic counts. The scatterplot, together with the regression statistics, provide a measure of how well the model replicates overall traffic flows on the network. As shown in Exhibit 17-10, the model performs very well for the daily time period, with the slope of the regression line (1.01) near 1 and an R^2 (coefficient of determination) value of 0.92. A slope close to 1.0 is generally desired, and while there is no national accepted value, most calibration efforts seek to obtain an R^2 of 0.9 or higher. As would be expected, the data points for the lower-volume links generally are more widely dispersed around the regression line than those for the higher-volume links, indicating the larger degree of model error for the lower-volume links.

Exhibit 17-10 Sample Link Scatterplot



Percent root mean square error is a frequently used measure of the differences between values (sample or population values) predicted by a model. An example is shown in Exhibit 17-11, reported out by link functional classification.

Exhibit 17-11 Sample Percent Root Mean Square Error (% RMSE)

Table 27: % RMSE Daily Validation

Link Volume Category	Functional Classification	% RMSE
≥ 16,000 vpd	Freeway/Principal Arterial	11%
8,000 – 15,999 vpd	Principal Arterial	22%
4,000 – 7,999 vpd	Principal/Minor Arterial	43%
2,000 – 3,999 vpd	Minor Arterial/Collector	56%
1 – 1,999 vpd	Collector/Minor Collector	28%
All Links		32%

Screenlines

Screenlines are drawn to evaluate significant regional volume flows. An example would be a screenline between cities or major areas within a region. It is desired that the total amount of trips in the model across the screenline equals the count volume across the screenline. In most cases a model/count ratio within 10% is desired. An example summary of screenline results is shown in Exhibit 17-12.

Exhibit 17-12 Sample Screenline Results

Table 28: Daily Screenline Validation Analysis Results

East-West and North-South Screenlines	Northbound			Southbound			Both Ways		
East-West Screenlines	Counts	Model	Model/ Counts	Counts	Model	Model/ Counts	Counts	Model	Model/ Counts
EW-GP1 (South of Redwood Hwy 199)	29,511	23,915	0.81	29,481	23,506	0.80	58,992	47,421	0.80
EW-GP2 (South of G Street)	37,239	40,799	1.10	42,730	40,186	0.94	79,969	80,985	1.01
EW-GP3 (South of NW Hillcrest Dr)	31,108	26,660	0.86	27,382	27,287	1.00	58,490	53,947	0.92
EW-JCo4 (North of Fish Hatchery Rd/Jaynes Dr)	6,103	7,017	1.15	6,103	6,945	1.14	12,206	13,962	1.14
EW-JCo5 (South of Monument Dr/Three Pines Rd)	16,826	18,163	1.08	17,071	18,332	1.07	33,897	36,495	1.08
2010 DAILY EW-SCREENLINE SUB-TOTAL	**120,787**	**116,554**	**0.96**	**122,767**	**116,256**	**0.95**	**243,554**	**232,810**	**0.96**
North-South Screenlines	Eastbound			Westbound			Both Ways		
NS-GP1 (East of Willow Ln)	11,982	15,332	1.28	16,836	15,334	0.91	28,818	30,666	1.06
NS-GP2 (East of Redwood Hwy 199 NB)	37,566	30,873	0.82	36,815	28,520	0.77	74,381	59,393	0.80
NS-GP3 (West of Murphy-Willaims Hwy 238)	26,344	27,716	1.05	26,503	27,845	1.05	52,847	55,561	1.05
NS-GP4 (East of Beacon Dr) N of River	18,479	16,914	0.92	18,222	18,123	0.99	36,701	35,037	0.95
2010 DAILY NS-SCREENLINE	**94,371**	**90,835**	**0.96**	**98,376**	**89,822**	**0.91**	**192,747**	**180,657**	**0.94**
Total Daily Screenline Vallation Results	**Northbound/Eastbound**			**Southbound/Westbound**			**All Ways**		
	Counts	Model	Model/ Counts	Counts	Model	Model/ Counts	Counts	Model	Model/ Counts
2010 DAILY SCREENLINES	**215,158**	**207,389**	**0.96**	**221,143**	**206,078**	**0.93**	**436,301**	**413,467**	**0.95**

<u>Vehicle Miles Traveled (VMT)</u>

There is usually not observed "on-road" VMT data to measure against, but that measure of VMT is often reported out as an output of a travel model. As a reasonableness check, comparisons could be made, for example VMT summaries by functional class from HPMS if available for the region, or from previous or other travel model VMT estimates. An example comparison of VMT by functional class is shown in Exhibit 17-13.

Exhibit 17-13 Sample VMT Summary

VMT

FC	OBS	EST	DIF	%DIF
FREEWAY	392497	382120	10377	3
PRIN ART	330543	309096	21447	7
MIN ART	151414	125255	26159	21
COLLECTOR	145123	171235	-26112	-15
RAMPS	18541	16718	1823	11
TOTAL	1038118	1004424	33694	3

Oregon Administrative Rules (OAR) 660-012 now requires the calculation of VMT per capita under certain circumstances during the Transportation System Plan (TSP) process. The VMT used in those instances must align with the specific definition in OAR 660-012-0005(64), which is different than how VMT has been reported for HPMS purposes.

The methodology for calculating VMT per capita that aligns the definition and for the purposes contained within OAR 660-012 is provided in Appendix 17B.

17.3.5 Future scenario construction

The value of a travel demand model is not in reproducing historic traffic conditions but to estimate future travel patterns. Once the base year model is calibrated and validated, the next step is to build future scenarios.

Constructing a future scenario involves developing data inputs similar to the base year. In some ways the future is more difficult in that inputs are not observable to be easily tabulated. Even though there are no observable data there are accepted methods of estimating future conditions.

Population

State administrative rules require that Oregon models adhere to population control totals obtained from the Population Research Center (PRC) at Portland State University . These are provided as a coordinated population forecast by county, UGB, age, and gender. The totals are updated on a three-year cycle. Counties allocate population to cities and unincorporated areas to assess future changes. The PRC does not provide population controls by model boundary; the modeling team needs to consider buildable land, expected development, and work with the local government to populate model zones outside of jurisdictional boundaries.

When population is increased, the number of HH's also needs to be increased. When developing the future household totals, the team needs to consider the changing demographics for the region in the future year and how those impact the average household size. It may not be correct to assume the average household size will be the same in the future year as it was in the base year. The group quarters are subtracted from the total future population; this result is then divided by the future average household size to estimate the future number of households. For trip-based models, households are the key parameter rather than population.

Employment

There is no control total for employment. An initial estimate of future employment is needed as a starting point to check with local jurisdictions. The initial estimate can be made in a variety of ways: Oregon Department of Employment projections are available for up to ten years in the future; future employment can be estimated proportional to the increase in households. Running the model can indicate that further adjustments are needed, and employment can continue to be adjusted in an iterative process. The local government is consulted to approve the overall amount of employment by category and zones.

Zone Allocation
Both employment and households need to be allocated at the TAZ level. There are several considerations to keep in mind when doing this:

- Local government may be thinking in terms of planned additional dwelling units. Dwelling units (DU) are not equal to HH's as DUs can be single or multi-family HHs. DUs are converted to HH using a vacancy rate (HH = DU*(1 - Vacancy Rate)).
    - Generally a TAZ is assumed to not lose households or employment, unless there is urban renewal or redevelopment planned, which should be noted to explain the decrease.
    - At the zone level, the average person per HH is usually in the range of 1.0 – 3.0 and the state average is 2.2 – 2.3.
    - Approved development should be included in the future year estimates.
    - Buildable land inventory, given zoning requirements, can be used to estimate additional future households and employment allocation to zones. See Chapter 6 for more information on cumulative model forecasting.

Network
The network also needs to be constructed for the future year. The needs of the model dictate which future networks are required. For example, an MPO model requires an RTP scenario which includes the RTP projects (committed plus financially constrained). Common scenarios that can be constructed are:

- No build – Conditions on the ground at the time of the calibration (base) year
- Committed –Committed network projects (projects that are already funded/programmed at the time of the model build). This does not include the entire financially constrained project list; it is only the projects that are included in the STIP, CIP or TIP at the time of the model build.
- RTP – Regional Transportation Plan, this typically includes both future committed and financially constrained projects.
- TSP – Transportation System Plan, this typically includes the future committed projects, financially constrained, and potentially unconstrained projects
- Interim – some combination of projects above, for chosen interim year, based on discussion with project team

Considerations for the network:

- It's important to remember to include existing conditions that may include projects built since the base year.
- Transit improvements or changes need to be considered for MPO models
- Potential policy changes need to be considered, such as parking, tolling, and technology
- Capacity may change in the future network due to the presence of CAVs

External

The future year external stations' growth and trends are important inputs. At the external stations, future volumes are typically determined from historic growth trends, such as the Future Volume Tables (refer to Chapter 6). SWIM can also be used to help provide information on how external station growth may be impacted by large regional projects, such as the Newberg Dundee bypass. In addition to external station volumes, O-D patterns for volumes coming from external stations are necessary inputs. In recent years SWIM has commonly provided this information, although other sources and methods may be available.

Overall

Local buyoff on population, employment, zone allocation, and network are needed for constructing a future model. Validation is done by running the model, and checking that the results seem reasonable: expected volumes, capacity, d/c ratios, etc.

17.3.6 Model Documentation

After the model is completed, model documentation will be finalized. The model development report documents the model build process including discussion on the following information.

- Model Structure
- Model Network
- Survey Data
- Zonal Data
- External Model
- Sub-Model Calibration
- Assignment Validation
- Future Year Scenario Development

17.4 Travel Demand Model Outputs

Model runs provide most of the basic information needed for a typical plan or project analysis. Some information is readily produced, for example a peak hour volume assignment plot, while other information can be obtained with additional effort, such as an unconstrained run that keeps the demand constant to better understand route choice and latent demand. Some of the primary performance categories that can be addressed by a model include mobility and land use.

A Multi-Criteria Evaluation (MCE) tool is in use by Metro and is being deployed for ABM models to provide a consistent output set, such as the set of measures illustrated in Exhibit 17-14 below.

Exhibit 17-14 Illustrative Set of Measures from Multi-Criteria Evaluation (MCE)

#	Benefit	Category	Type	Quantities	Maturity	Confidence
1	Safety	Safety	Link	Fatal, Injury, Property-Damage Only Crashes	Proven	●●●●○
2	Travel Time	Mobility	OD	Minutes of travel time saved by mode	Proven	●●●●●
3	Travel Time Reliability	Mobility	OD	Decrease in travel time variability (standard deviation of travel time)	Emerging	●●○○○
4	Vehicle Operating Costs	Mobility	Link	Gallons of fuel consumed, VMT-based non-fuel costs	Proven	●●●●○
5	Vehicle Ownership Costs	Mobility	Zone	Number of household vehicles	Emerging	●●●○○
6	Emissions	Environment	Link	Tons of CO_2e, PM2.5, PM10, NOx, VOC	Proven	●●●●●
7	Surface Water	Environment	Link	VMT-based cost of impacts	Emerging	●●○○○
8	Noise	Livability	Link	VMT-based cost of impacts	Emerging	●●○○○
9	Physical Activity	Livability	OD	Quality-adjusted life years (QALYs) saved	Emerging	●●●○○
10	Travel Options / Choices	Accessibility	Zone	Monetary value of additional mode / destination options	Emerging	●●●○○

Equity is a lens or dimension that can be applied to any of these measures. For example, output such as accessibility or mode share can be sorted or filtered by equity factors such as income level, age, or vehicle ownership.

As ODOT moves forward with ABMs, higher resolution travel behavior data become available as the ABM estimates/synthesizes a region's activity survey. The ABM predicts a full daily record of travel expected to occur, with individual records for every trip, for every person. This allows data to be aggregated to answer very specific questions on travel within the model area, for the data fields available. For example, the model could test the effectiveness of free transit passes being provided for people working in a specific area. Some of the more common model outputs are discussed below.

1. Traffic Assignment

Traffic assignment is typically provided as the number of vehicles expected to use each link or segment of the network for a given period. Depending on the study, this may be for ADT or for a peak period. Vehicle assignment can be provided by trip purpose and traveler class.

With an ABM assignment, trip records will have information for any hour(s) of the day. Results are grouped into five bins for early morning, AM peak, mid-day, PM peak (typically one or two hours), and evening; an additional one-hour PM peak can be run if desired. Exhibit 17-15 gives an example of a traffic assignment plot.

Exhibit 17-15 Sample Model Output – Auto Volumes

2. Unconstrained Assignment

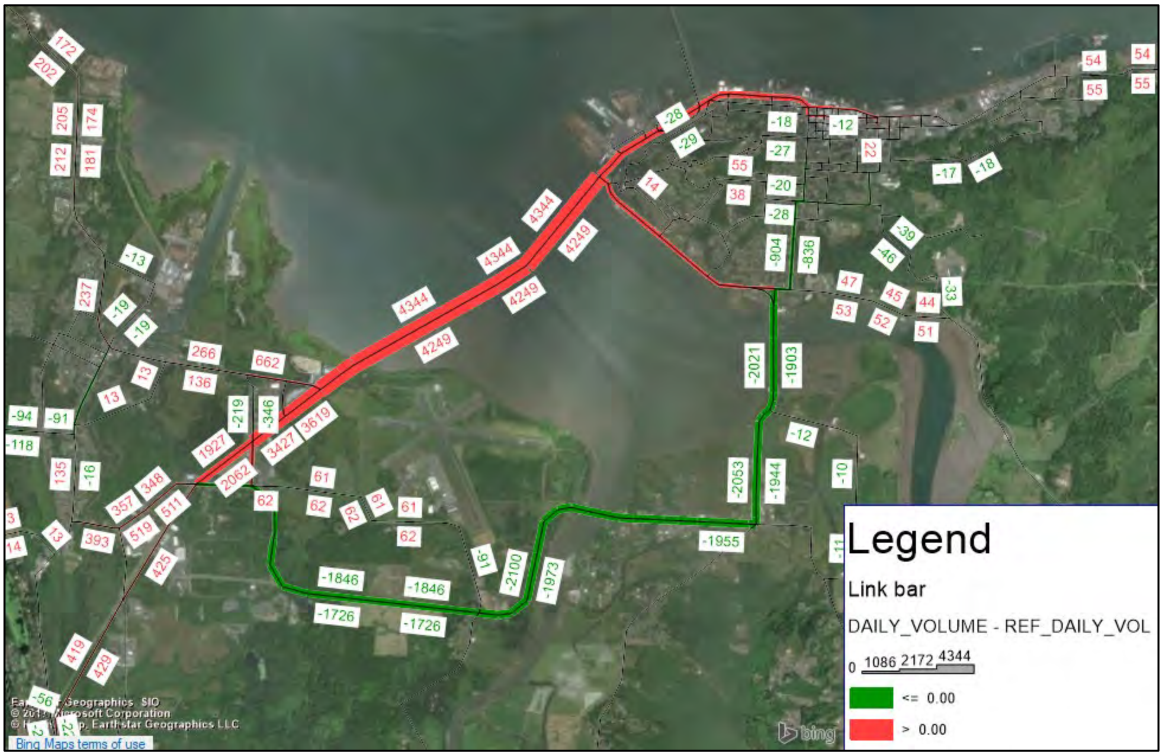
Model scenarios are typically run as capacity constrained. In a constrained model, growth may appear low when compared to growth based on historic trends, which is unconstrained. An unconstrained model run should be considered for any project as it assists with understanding travel patterns within the project area. A difference plot between the unconstrained and constrained scenarios can help identify links with significant capacity constraints. This assignment will show the desired path (where traffic wants to go) if capacity was unlimited. Large differences between constrained and unconstrained model runs points to latent demand concerns (see Section 6.12.2) and may be a flag for special consideration in post-processing.

An unconstrained run can be either using just the assignment model or with a full model run. If running only the assignment model, demand is held constant and trip distribution is not affected. With a full model run, trip distribution is allowed to change based on free-flow speeds instead of congested speeds. Typically unconstrained is run just for the assignment portion of the model because allowing trip distribution to recalculate based on uncongested conditions can be unrealistic. Running unconstrained just for the assignment portion provides an estimate of latent demand for corridors assuming set trip origins and destinations.

3. Scenario volume difference analysis

One scenario is compared with another scenario in terms of the link volume changes for the entire model network. Usually a typical land use or network scenario is chosen to compare with the base reference scenario to see how the action scenario is going to affect the no-action scenario. Exhibit 17-16 is an example of a difference plot. As an example, this can show the wider benefits of a project in terms of reduced volumes on other roadways even when volumes increase on the improved roadway due to latent demand.

Exhibit 17-16 Difference Plot



For initial screening and scenario development and sensitivity testing, running just the assignment portion of the model can save time and result in preliminary difference plots. The model assignment portion is likely to result in most of the link assignment differences between scenarios. The full model run is likely to only refine these results. The full model can be run once the set of scenario changes has been narrowed down. This can be a substantial time savings depending on the full model run time.

4. Demand to Capacity Ratios

A standard model output is a plot showing peak hour demand to capacity ratios on links. This is often used for preliminary screening of capacity issues. For this purpose raw model volumes are used and does not require post-processing. The d/c ratios are typically binned into ranges such at/near/or over capacity to flag potential deficiencies for further analysis. Refer to Chapters 9 and 10 for definitions and applications. Demand to capacity is used as an indicator of congestion when model volumes exceed model link capacities. Actual volumes never exceed actual capacity.

5. Select-Link Analysis

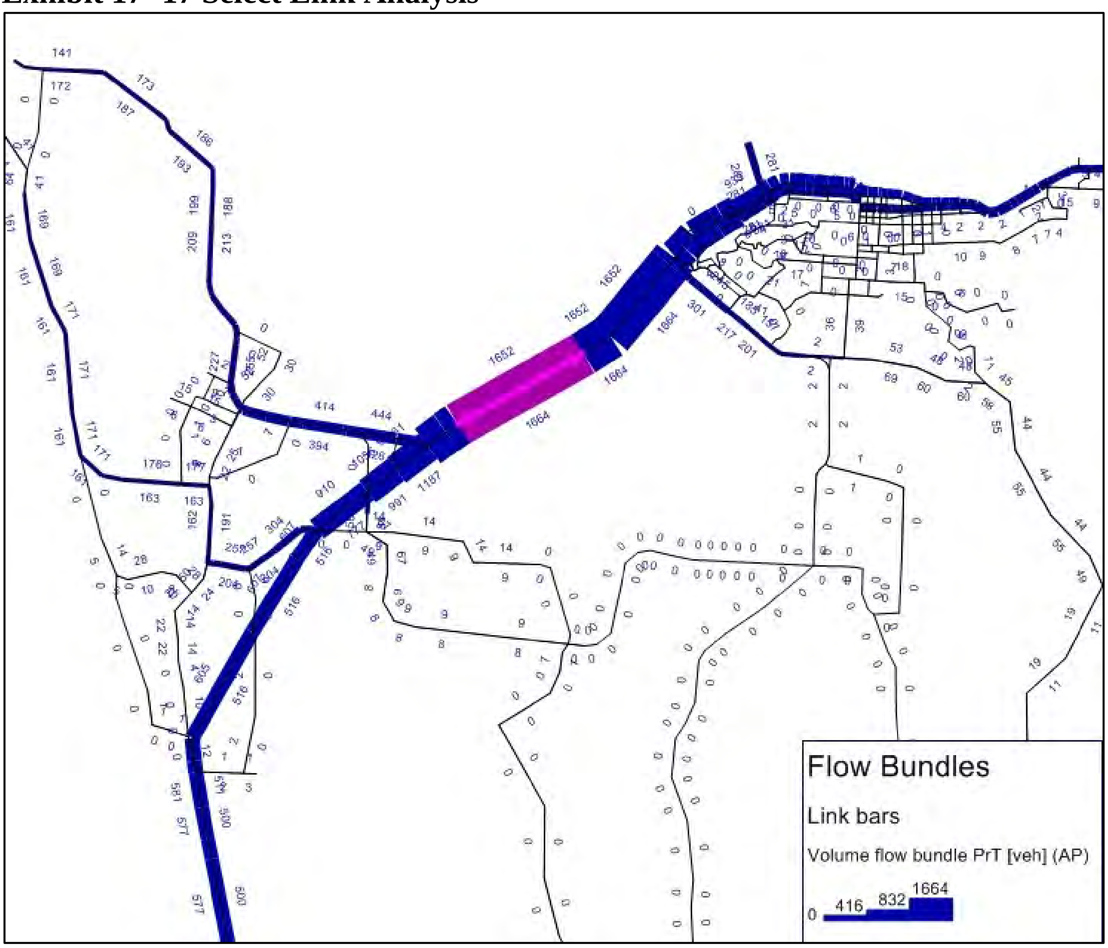
Select link analysis is often requested for supporting intersection turning volume post-processing according to the APM. For a given select link or series of links, a select link plot shows what other roads the trips on the link are traveling on or what zones the trips are traveling to or from. Specific software may have somewhat different terminology for this analysis. Raw model output is provided at the link level, not at the intersection turn movement level. Select-links can also be used to track trip assignments on specific links to assess impacts. Other examples of using select links are:

- Obtaining initial turn movement percentages for use in post-processing. See Chapter 6 for more information.
- Determining percentage of through trips through the project area
- Determine if trip assignments are reasonable after network changes are made.
- Determine weaving section flows if obtained on the mainline and ramp sections leading into and out of the weaving section.

Exhibit 17- 17 is an example of a select link plot. Some example questions that a select-link analysis can answer are:

- Where does a roadway improvement project draw traffic to/from?
- How does a street or bridge connection or closure impact its vicinity traffic flows?
- How does widening a roadway reduce traffic impacts on other parallel facilities?

Exhibit 17- 17 Select Link Analysis

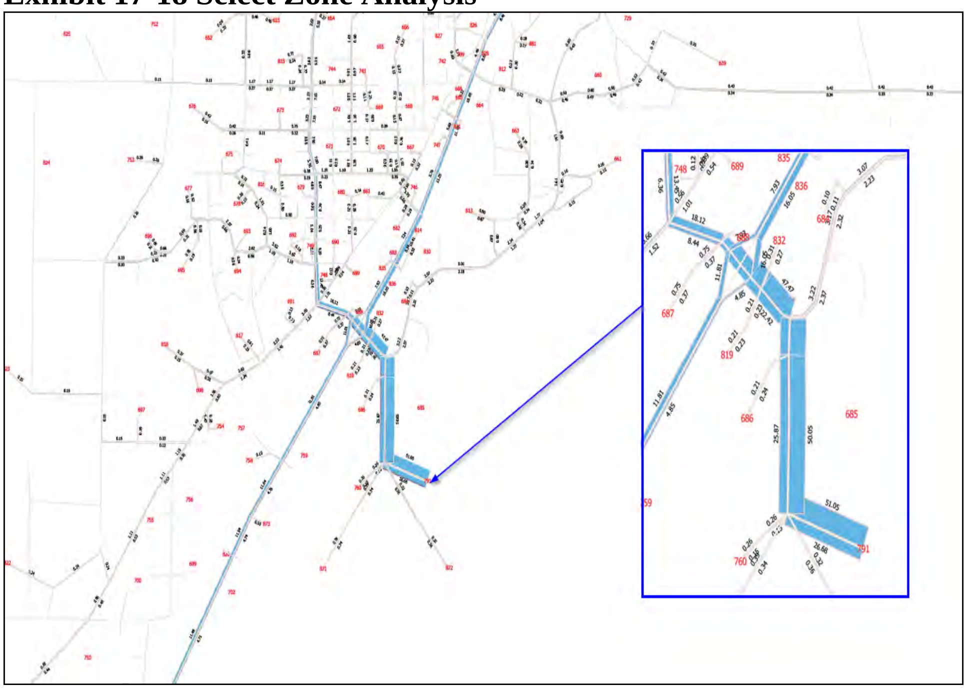


6. Select-Zone analysis

Select-zone analysis is requested when a new land development is placed in a zone or an existing development is relocated to a new zone. A select-zone plot shows the distribution of volume from/to a given zone across the model network. This is useful in determining, for example, volume distribution for a future development not included in the model. The trips to and from the new land use across the entire model network can be tracked through the select-zone volumes. Exhibit 17-18 is an example of select zone analysis. Some examples of use of select zones include:

- Impact area – the extent of the roadway network that a land use change has a significant impact on. Impact area is typically calculated using a threshold for significance, such as where existing AADT is increased by 10% or more.
- Proportional share – the proportion of trips that are contributed by a land use change. For example, where the need for a left turn lane is due to a left turn volume and the land use change contributes 30% of the total left turn volume.
- Impacts to specific facilities

Exhibit 17-18 Select Zone Analysis



7. Transit

The types of information for transit are similar to the auto mode. As an example, a select link/flow analysis can be obtained for transit lines like auto links at a stop or section of the route.

The level of calibration affects the overall accuracy of transit output and type of questions that can be answered. At a minimum calibration is done at the route level. Some models like Metro's calibrate at the level of stops. When requesting transit information a discussion with the regional modeler is needed to determine what information can be extracted and how it should be applied.

Several transit performance measures are possible (not exhaustive):

- Ridership, by route or by stops. Ridership at stops as an output needs to be carefully reviewed if the transit model was not calibrated at the stop level.
- Travel time/delay - running time same as vehicle travel time. Dwell time or stop time are inputs based on defaults in ODOT models.
- Accessibility – can be obtained for both accessibility to transit and by transit

8. Mode share

Only MPO level models include outputs on modes other than motor vehicles. The different modes available include auto, walk, bike, transit, SOV, HOV (carpool), trucks (emerging feature for newer models only). The information available for each mode depends on the level of calibration done for each mode. The design of the model determines the types of questions and outputs that can be modeled. For example, Metro's model was built to model HOV lanes, unlike JEMnR.

In general, model-wide mode share splits are commonly available and can be evaluated. Origin-Destination information by mode (and possibly by trip purpose) is also a common output that can be requested.

The disaggregate nature of ABM MAZs can better represent short trips that may be made by walk or bike modes. There would be less likelihood of needing to split a zone as compared to a trip-based model.

Some restrictions to be aware of include:

- Metro's model and SWIM are currently the only models that are calibrated to truck counts. SWIM can provide commodity flow in terms of dollars and tons moving by corridor.
- In general walk and bike are not assigned to the network, but zonal production information is available.

9. Other Model Outputs

- Vehicle Miles of Travel (VMT) – as part of the TPR reporting requirement for RTPs and TSPs, VMT per capita is reported for a subset of zones representing the UGB or a specific jurisdiction. VMT per capita (or other measurements of VMT) can be helpful to understand the impacts of regional scenario planning or financially-constrained project lists in TSPs. It can also be used as a data input into other tools to calculate measures such as GHG.

- Travel time – Model travel time can be provided at the link level for preliminary screening, for example for relative comparisons of travel time on key routes between alternatives. Model travel times can be particularly useful for links that are outside the project study area.
- Matrices based data – trip data can be provided at the TAZ or district level broken out by trip purpose. These data are typically summarized graphically or statistically using frequency distributions, charts, or maps. Some examples of matrix trip data available include:
  - Demand
  - Travel time
  - Trip length
  - Mode split
- Zonal summary information
  - Household/demographic information
  - Production/attraction information
- Regional information
  - Accessibility – refer to Chapter 9 for more information.
- Travel behavior information - ABM
  - Household level
  - Person level
  - Trip level
  - Tour level

17.5 Travel Demand Model Applications

The following guidance is primarily intended for analysts who are using travel demand models for facility level analysis, rather than regional analysis.

Most travel demand models were developed primarily for use as system planning tools and their traffic volume estimates are not sufficiently accurate to be used as direct inputs into facility-level planning and analysis. It is inappropriate to use raw model outputs as the basis for transportation and land use decisions that require consideration of detailed transportation and land use characteristics such as in analysis software such as Synchro, or in most deterministic, multimodal or microsimulation applications. Therefore, post-processing of model outputs to account for the influence of specific transportation and land use characteristics is mandatory. Methods used for post-processing must conform to procedures found in Chapters 5 and 6.

The relative difference between the raw model output for two scenarios (e.g., current and future conditions) can be used directly such as for the screening of preliminary alternatives.

Seasonal adjustments are directly applied to the base traffic counts to represent average weekday volumes and should not be applied to model output. Refer to Chapters 5 and 6 for more information

If an official travel demand model exists (ODOT or MPO developed), it should be used as the basis for performing forecasts for plans, project development, and development review. Other considerations include:

- There are federal requirements that a travel demand model be used when an MPO is developing an RTP. Criteria and procedures for air quality conformity modeling are found in the Clean Air Act Transportation Conformity Rule [40 Code of Federal Regulations (CFR) Parts 51 and 93].
- NEPA requires that official population forecasts and the latest official financially constrained funded project list are used for land use in the travel demand model. For more information on the NEPA process refer to Chapter 10.
- Metropolitan area (MPO or ODOT developed) travel demand models shall be used for the calculation of VMT per capita to meet TPR-related requirements in OAR 660-012.
- As compared to the use of historical count growth rates; travel demand models can answer questions historical rates can't answer such as changes in policy, network, growth patterns (induced or latent demand), etc. Historical growth rates don't capture changes in trends, such as if UGB is to be expanded.
- Cumulative forecasting is limited to urban areas with generally less than 10,000 in population. Cumulative forecasting is specific to the project it is developed for; it cannot be readily adapted to other projects. Multiple scenarios or multiple purposes require a travel demand model.
- Enhanced zonal cumulative may be used in urban areas with up to 15,000 population. Refer to Chapter 6 for more information.

If the requestor simply needs base/future volume or land use data with no model modifications, previously generated model data are available through the model request process. If no network, land use or zone modifications are needed, previously generated base and future data can quickly be provided. Keep in mind that providing base year land use data (employment specifically) requires a completed confidentiality agreement before the request can be completed.

17.5.1 Typical Model Applications

SWIM model applications are discussed in Section 17.5.4.

1. System Planning: RTP/TSP

In system planning, models can be used to quickly assess the entire MPO/urban planning area which may contain multiple cities and the interactions between them. Use of demand-to-capacity ratios can indicate bottleneck areas or areas that potentially need improvements. Conceptual project scenarios can be added to test impacts on the overall network. These can be bundled into groups of projects for specific objectives (capital projects, multi-modal, mobility, etc.).

Impacts of land use changes can also be tested, such as in a UGB expansion scenario, nodal development, neighborhood urban centers, etc. Transit and other multimodal benefits can be evaluated depending on the detail of the individual networks (i.e. walk, bike and transit) and zone structure. If the model has enough detail, such as economic sensitivities, items like congestion pricing, parking pricing, tolling and travel demand management policies or programs can be evaluated. Scenarios could be developed to test the potential effect of emerging trends and technologies such as CAVs. For example, scenarios could assess the potential effect of CAVs on model outputs with a range of CAV market penetration rates. ABMs typically allow for this additional functionality, depending on the level of effort or calibration.

Models can also be used to create and evaluate accessibility, connectivity, and equity measures. Some operational strategies can be modeled such as TDM or ramp metering. System planning projects that come out of modeling are generally high level such as “Widen to four lanes”, or “Add overcrossing”, etc. which are consistent with the general level of detail available.

Scenarios can be modeled where the population controls are exceeded: number of households (HH) must be increased or decreased accordingly to balance. A variety of different future land use scenarios can be modeled within the system planning process.

ABMs can model GHG more appropriately than trip-based models by being able to represent individuals with specific vehicle types, powertrains, and vehicle ownership.

Metropolitan travel demand models are the tool of choice for calculating VMT per capita, using the methodology in Appendix 17B, to meet Transportation Planning Rule requirements. SWIM is the preferred model for use to determine latent demand to meet requirements in OAR 660-012-0830. For some “Rule 0830 projects”, regional travel demand models, provided land use is modified between scenarios, may be used to forecast latent demand.

2. Air Quality Conformity

Some areas have air quality issues which require them to go through an air quality conformity analysis which requires that improvements on the system not add more emissions than the specific target values. These can be for carbon monoxide (CO) or particulate matter (PM). Trapped PM from woodstoves has been the focus of most Oregon air quality (AQ) issues such as in Grants Pass and Klamath Falls. This application work is an intermediate step (as used as input data to another air quality model) in the overall process and typically requested by DEQ, or a council of governments. TPAU does not do the actual air quality analysis or post-process the speed or travel time data generated.

The overall roadway network, including any improvements, is analyzed by EPA's MOtor Vehicle Emission Simulator (MOVES) emission tool using speed and vehicle miles traveled (VMT) by link as the primary inputs. Project effects need to be balanced to meet the conformity process. Certain projects could lessen VMT and emissions if trips are shortened or mode shifted or allow travel at faster speeds. Conversely, some projects like a new interchange could encourage travel and increase VMT and emissions.

The air quality conformity process typically requires creation of interim year scenarios which requires modifying the network and land use data to:

- Determine compliance with air quality conformity
- Modify regional transportation plans
- Determine impacts of a regional project

3. Facility Plans and Project Development

IAMPs (Interchange Area Management Plans), refinement plans, and modernization projects typically deal with smaller areas or individual facilities or corridors. This type of model application will be generally more specific, rather than across an entire urban area, e.g. adding or modifying roadway connections such as at an existing interchange. Alternative changes to individual facilities can be tested with different link attributes such as speeds or number of lanes or one-way/two-way directions to determine impacts. Land use scenarios with different levels of growth can be evaluated and compared with a baseline scenario for a localized zone or the entire urban area. The use of models for preliminary screening of alternatives is discussed in Chapter 10.

It is important to be aware that when applying an ABM model, the model complexity and the stochastic approach and uncertainty means that individual runs will differ. As with microsimulation, multiple runs will need to be combined and may require post-averaging of run results, adding to the timeline as compared to trip based models.

For congested areas, ABMs allow for some temporal shifting while trip-based models do not, but dynamic traffic assignment (DTA) would be required to fully represent significant congestion (see Chapter 8).

There are a variety of travel demand models that can be used: small city, MPO, or statewide, depending on the project. For example, SWIM could be used to estimate trends and thus impacts in an area that does not have a local travel demand model. Examples include estimating overall truck flow through an area, or highway to highway distribution percentages, such as would be used in long term highway closures.

17.5.2 Model Familiarity and Checking

Model Documentation/Structure

The model documentation and structure should be reviewed by the analyst. The information is primarily available from the model documentation narrative. Based on the information in the narrative, the analyst can obtain the Base and Future Year network and zone plots or GIS layers. More information including how to request model data can be found on the ODOT Planning and Technical Guidance webpage.

The analyst can also work with modeling staff to obtain the data tables containing network and TAZ attributes. Items to review may include:

- Type of model (OSUM, JEMnR, ABM, etc.)
- Existence of multiple model versions to identify the version appropriate for use
- Model Years (Calibrated, Base Year, Future Year, Interim Years)
- Previous scenarios that have been run / tested such as financially constrained or Regional Transportation Plans (RTP).
- Projects included in the scenarios of interest
- Model Calibration Information (Level of detail, peak hour, …)
- Special generators in or near the study area
- Incorporation of large-scale changes in the project area since the base year, either transportation improvements or land uses

Model Representation of Study Area

Travel Demand Models are built and calibrated primarily as system level tools, and generally do not have a high degree of network detail. A given study area may not be represented to the degree of detail needed by the project. Potential study area refinements should be identified and discussed with the modeler. There may be issues within a study area that are significant to the project analysis but were not significant at the regional level the model was built for. Some issues may be adjusted within the model, while others are best handled through post-processing.

The following guidelines are provided to assist the analyst in evaluating the existing model relative to a particular study area and purpose. This should include verifying reasonability and evaluating if the level of representation is sufficiently detailed. The analyst should coordinate with the modeler on any potential refinements identified. Items to review within the project study area are listed as follows:

Network

- Links and Nodes – Review the level of detail of the network components and whether it accurately depicts the road system in the study area. Review for potential need for additional links.
  - The analyst needs to review the network considering what projects are included that may not yet be built. For example, there may be three un-built projects from the TSP included in the future year model. Depending on the year of construction the non-study projects would likely be in the future no-build model runs but the study project would not be included. The study project would only be in a future (build) alternative.
- Centroid connectors – Review the number and connection locations that represent the loading of the trips onto the network. Adjustments or additions may be needed. Additional connectors to different roadways do make a difference. It is undesirable to have multiple connectors to a single roadway section except when windowing or focusing a model. Multiple connectors to the same roadway section split the total volume up and the loading will be dependent on the shortest path.
  - Connectors should be checked visually for the following elements. Connectors represent the local accesses and streets in a model network, so they must represent the loading patterns to the adjacent facilities. Every zone must have a centroid connector, for ingress/egress, connected to the facility or segment. A single segment can have no more than one connector and a zone is limited to a maximum of four connectors. Connectors shall not connect directly to an intersection node as it makes post-processing more complex when dealing with turn movements. Depending on the scenario, connectors can change number and location for a given zone to expedite post-processing.
  - Connectors must have no capacity constraints (Capacity = 9999 in EMME; Visum is automatic default). Connectors must also have applicable modes indicated, directionality (in and out but not always at the same location), and a fixed speed of 25 mph (except at external stations where the speed should represent the facility speed, i.e. 55 mph). Non-transit or non-multimodal networks should only have the auto mode coded. Depending on model approach, transit networks may require transit and walk modes in addition to auto. The walk mode can share or have its own connectors. Connectors must have a realistic link length based on average distance from the center of activity (centroid) to the specific roadway link.
- Nodal Attributes – Review the turn movement restrictions and traffic control type (signalization, stop-controlled, roundabouts, etc.). For ABM models, turn lanes and signal timing parameters should be reviewed and refined if needed.
- Link attributes – Verify facility type, functional class, speed, number of lanes, lane capacity and VDFs. Default link capacities for OSUM and JEMnR models are based on functional class and area type, as shown in Exhibit 17-19.

Exhibit 17-19 Default Model Link Capacity per Lane - Based on Functional Class and Area Type[1,2]

	FC Type	*1 (CBD)*	*2 (CBD Fringe)*	*3 (Urban)*	*4 (Rural)*
Freeway/Interstate	1	1900	1900	1900	1900
Principal Arterial	2	700	800	850	950
Minor Arterial	3	575	625	700	760
Collectors	4	450	500	525	650
Local	5	400	450	500	625
Ramps	30	700	800	850	1000
Centroid Connector & External Stas.	99	Always 9999			

[1]For Oregon Small Urban Models (OSUM) & JEMnR
[2]Similar information for ABM models can be found here.

TAZs – Size and demographics

Local Agencies are responsible for the base and forecast land use data necessary for travel demand modeling. Of these land use data inputs, the most common to focus on are total population, households, and employment by sector (Agricultural, Education, Government, Industrial, Retail, Service and Other).

Zonal data may need to be updated to reflect current conditions within a study area. A discussion with the modeler should determine whether the model is sensitive enough to reflect the differences desired. Items to review include:

- Population and number of households, existing and future. Population is typically constrained by control totals obtained from the PSU Population Research Center. However, scenarios can be modeled where the population controls are exceeded. Population totals are usually provided at city/UGB level and typically not the model level. When population is increased the number of households also needs to be increased. Population by zone is the sum of the average household size by zone multiplied by the number of households by zone and added to the group quarters population for that zone.
- Number and type of employees, existing and future – note that a confidentiality agreement may need to be signed before the existing employment information can be shared with the analyst. There are no specific controls or constraints on changes to employment, but it is important to be aware that certain employment categories have bigger impacts on the results than others. For example, retail/service has more regional impact than agriculture. The total employment must equal the sum of the employment subcategories (government, retail, etc.). Further discussion on the proper way to address significant or large land use changes can be found in the Modeling Procedure Manual for Land Use Changes (MPMLUC).

- Size and detail of zones – some large zones may need to be smaller to better represent travel patterns within a given study area. For example in a large mix-used zone there may be a need to understand trips by all modes between the different uses. By splitting the original TAZ or MAZs, the intra-zonal trips become inter-zonal ones that can be reported. Also, on high planning level studies zones may be aggregated into districts like for travel patterns across an entire community.
- Trips generated and distributed are consistent with the demographics of the zone.

Checking Model Application Output(s)

The results of a model run within the study area should be reviewed by the modeler and the analyst to determine if the model behavior is reasonable and logical. The modeler has a protocol for validation including runs or checks that all steps of the model are performing reasonably. This is to validate that model outputs within the study area makes sense. There may be coding errors or anomalies that show up in the output that should be corrected. For congested study areas testing might be needed to verify that outputs are reasonable. The analyst can assist the modeler with knowledge of the area, project and specific issues.

The analyst should understand how well the Base Year model corresponds to the Existing Year volumes developed from study area traffic counts and whether deviations may be coding errors which can subsequently be corrected. The Base Year model volumes should be projected to the existing year to compare to the count-based Existing Year volumes.

Future year model assignments should also be reviewed based on expected changes and may be compared to historic trend forecasting. Changes not meeting expectations should be examined in terms of land use, socio-economic data and network coding for potential explanations. Any identified disparities should be documented.

Different model application techniques are useful in reviewing model behavior, including select-link and select-zone plots. The analyst should check whether results are reasonable and logical. Any apparent anomalies should be discussed with the modeler and explanations should be documented. The modeler should also be checking the outputs for reasonableness. For example, changes to land uses or the network may have occurred since the model was developed. For some disparities such as where model and actual travel paths are significantly different and unexplained, it may be appropriate to post-process the model assignment by manually re-assigning trips.

17.5.3 Model Modifications

1. Modifying link attributes or adding or removing a link

Modifying links is one of the most basic (in whole or part) model applications in assignment software. This type of application typically includes modifying attributes. Typical examples might be to assess the:

- Impacts of adding or removing a roadway or a travel lane which might require adding and deleting links and/or modifying attributes such as speed or capacity.
- Impacts of converting to or from a couplet (one-way to two two-way links)
- Impacts of a speed zone change
- Impacts of work zones (short/long term effects) through reduced number lanes, total closure, and/or reduced capacity
- Impacts of adding links specific to transit, bike and walk to address new mixed-use development or transit-oriented development (TOD) that might create new pedestrian pathways
- Impacts of adding ramp metering by reducing the capacity attribute at an on-ramp

2. Modifying, adding or deleting nodes

Modifying, adding or deleting nodes is another basic model application in assignment software. This type of application typically includes any kind of link changes as adding new links will require adding or removing links. Additional nodes may be required to properly shape a roadway. Typical examples of modifying nodes are done to assess the:

- Impacts of adding or modifying intersection traffic control
- Impacts of turn restrictions such as one-way or right-in/out
- Impacts of medians (turn restrictions)
- Impacts of additional TAZ connectors (changing the loading location for a TAZ)

3. Modifying connectors or partitioning a zone

Modifying or adding zones may be needed for applications involving new developments, Urban Growth Boundary (UGB) expansions, mixed use subdivisions, Transit Oriented Developments (TODs) that require partitions to keep zones homogeneous. Connectors must be managed anytime changes are made to the TAZ or MAZ structure. Connectors may need to be moved, deleted, or added to depending on the type of change to the TAZ or MAZ. Some typical examples are:

- Zone split for new development to allow quantification of trips between zones (expose the intra-zonal trips)
- Adding zones for a UGB expansion
- Districting (grouping) of TAZ’s to determine large scale origin-destination (OD) flows or trip patterns
- When a new roadway will split/change the travel patterns for a large zone.
- Splitting a transit-oriented development to help show walk trips versus vehicle trips.

4. Modifying land use attributes

Land use attributes are typically compiled in an spreadsheet. This can include changes to household or employment categories. All land use changes must be submitted as part of the completed model application. Typical examples are for assessing the:

- Impacts of zone changes
- Impacts of special generators
- Impacts of different land use scenarios for a local jurisdiction or landowner request e.g. TIAs
- Impacts of parking pricing scenarios

5. Modifying/adding transit services

Transit services are only modeled in MPO areas (JEMnR or ABM models). Transit networks are separate from auto networks and may involve bus routing. When adding transit to a network in JEMnR, the walk attributes (access to transit) must also be addressed. Some typical examples include:

- Impacts of adding or removing transit routes, transit links
- Changes to transit headways
- Network-wide impacts of adding transit centers, park and rides, stops (locations only - not for specific routes)
- Updating transit or master plans
- Quantifying Vehicle Miles Traveled (VMT) reductions and other strategic policies
- Transit signal priority (ABM or Metro model or focused trip-based model)
- ABM modeling of travel demand policies or programs

6. Subarea (windowing and focusing) modeling

Sometimes more detail is needed within a certain area such as a downtown to improve the post-processing of link and turn movement volumes. This may involve creating more zones or adding more details for zone, link and node attributes. A typical example would be a downtown refinement plan involving investigation of couplets, multimodal improvements, intersection traffic control, or modified intersection lane configurations. Complex transportation impact and project analyses may require use of subarea modeling. Mesoscopic modeling including windowing and focusing of travel demand models is addressed primarily in Chapter 8.

There are two different ways this can be done, either through focusing or windowing. If the subarea is done through focusing where detail is added within the model, the subareas can be integrated with the rest of the model. This detail can be smaller zones, additional centroid connectors, modifying VDFs and nodal delay, adding more network links and refining network attributes. The area needs to be large enough to minimize border effects from transitioning the level of detail in adjacent zones/areas (altering the assignment in the area around the focused area). The focused area for this reason should be larger than the study area. This focusing may be done by the modeler or the analyst (if they have the software and expertise).

The windowing process results in cutting out an area from the model. This windowed out area cannot be added back to the original model. The difference from focusing is that trips that cross the windowing cutline are held constant from those in the original scenario. Many of the same details as used in focusing can be used in this technique. Other than physically, “cutting out the area”, this work is typically done by the project analyst rather than the modeler. The model request must include an exhibit indicating the boundaries of the cut area. The analyst would use other forecasting tools like Visum to make the refinements as well as reduce the post-processing.

Some models have bike and walk networks (including off-street paths) which would potentially allow for a windowing or focusing effort to develop a bike trip assignment. Calibration (by mode or by route etc.) would require significant inventorying effort including bike counts. Quality of service could be incorporated by adding attributes such as Level of Traffic Stress (LTS) data. Some models can also capture the trip purpose such as recreation, school or work trips.

7. Creation of Interim, Reference, or Future Year Scenario

Models initially have a single base and future scenario. TSP, air quality conformity, or regional projects/plans planning horizon years often do not line up. Volumes typically can be extrapolated, at most five years beyond the base and future year scenarios. Beyond five years, the creation of new reference or future year scenarios is necessary. This work does not involve calibration or validation, so does not replace the calibrated scenarios or constitute a model update. The requestor should work with the model team to develop all zone, network, and land use data to support the creation of these types of scenarios. These may include different network assumptions (e.g. unpaved roads, important for air quality conformity models) from production model. In summary, these scenarios will be created when:

- Planning horizon is greater than five years beyond the future year scenario or available years do not match desired planning horizon such as a TSP, RTP etc.
- Air quality conformity analyses are being done, which will typically require interim years if not within an MPO area

17.5.4 SWIM Model Applications

Since its original creation in the late 1990’s SWIM has matured beyond the higher-level policy scenarios described in Chapter 7. Over the past two decades, the types of uses for SWIM has broadened and will likely continue to evolve. A current list of all the ways SWIM has been used is listed here: https://github.com/tlumip/tlumip/wiki/Applications.

SWIM applications have started to fall into the following categories:

- Large statewide policy impacts (see chapter 7 for examples)
- Commodity Flow Analysis for Policy or Specific Facilities
- Resilience testing relating to road failure or work zone closures
- Volume growth or distribution estimates for areas that do not have a travel demand model
- Providing flow information in and out of defined areas, such as external models for urban area models.
- Intercity passenger travel (vehicle, rail, transit)
- Forecasting latent demand to meet requirements of OAR 660-012-0830(5)(c)

SWIM has also been used to develop focused area models; an example is the focused model developed for Marion County. SWIM has potential to provide similar focused models for smaller cities where the main questions are on State highways or other higher functional class roadways. This is an option to consider where a full OSUM development might not be worth the time / cost.

17.5.5 ODOT Model Request Process

Model requests are a collaborative process between the requestor and the modeler. It is important for the requestor and modeler to discuss the nature of the request to determine the needed approach. Model requests range from very basic, using available information; to very complicated, requiring extensive calculations and processes. If the request includes obtaining a copy of the base year employment data, completed confidentiality agreement is required before the request will be processed. A map of available transportation models in Oregon is available on the Planning Section website on the Technical Tools webpage.

For application of ODOT models, a "Request for Travel Demand Model Run" form must be submitted. The form serves as a guide to assembling the information needed in order to process the requested model runs. It is highly recommended to discuss the request with modeling staff prior to submittal since the modeler needs to understand the study purpose and objectives to help identify the best model application methodology, the relevance of model variables, or even whether a model application is necessary. The requestor needs to understand the model, its limitations, level of detail, sensitivity to variables, to obtain the appropriate output needed for the project and to properly interpret the model results.

Several items need to be discussed and clearly determined and specified on the request form, such as:

- Name and version of the model
- Model years requested
- Scenarios requested
- Study area
- Projects to include
- Network changes to include

- Land use changes to include
- Project alternatives to be evaluated
- Output requested, such as:
    - Volume plots – daily and/or specific hour (peak)
    - Bandwidth plots
    - Select link or select zone runs
    - Difference plots
    - Demand to capacity ratio (DCR) plot
- Output format (file type)

A map should be attached to the request illustrating locations of changes as described on the request form. Model parameters must be used when specifying changes, such as using "From Node - To Node" for changes to links. New links and nodes need to have attributes specified (speed, number of lanes, functional class, capacity, and traffic control). For most requests, changes should be listed separately for each scenario on attachments.

Changes to land use must include:
- TAZ number
- Desired splitting of zones
- Relocation of centroid connectors.
- Population change
- Change in number of households
- Employment change by industry category

For land use changes, see the Modeling Procedures Manual for Land Use Changes .

Appendix 17A – Network Count Attribution

Appendix 17B – Calculating Vehicle Miles Traveled (VMT) - Overview and Procedures

REFERENCES

1. Horowitz, Alan, et al. "Analytical travel forecasting approaches for project-level planning and design." No. Project 08-83. 2014.
2. Systematics, Cambridge, Inc., "NCHRP Report 716: Travel Demand Forecasting: Parameters and Techniques." Transportation Research Board of the National Academies, Washington, DC (2012).

18 TRANSPORTATION SYSTEMS MANAGEMENT & OPERATIONS

18.1 Purpose

The purpose of this chapter is to provide an overview of transportation system management and operations (TSMO) program elements, methods, strategies and analysis tools. The chapter guides users on integrating established TSMO procedures, analytical tools and data into existing planning processes and project development.

FHWA defines TSMO as “an integrated program to optimize the performance of existing multimodal infrastructure through implementation of systems, services, and projects to preserve capacity and improve the security, safety, and reliability of our transportation system”. This section includes an overview of the content of the chapter and background information on the policy basis and rationale for TSMO.

Technology related to TSMO is changing rapidly and will continue to evolve in the coming decades. With accelerated change as the norm, this chapter represents a snapshot of current conditions and trends that will need to be updated frequently. For more details on many of the methods in this chapter also refer to [FHWA’s Planning for Operations website](#).

18.1.1 Overview of Chapter Sections

This chapter covers a range of TSMO topics:

- Policy Basis and Rationale for TSMO – State and federal policy foundation for TSMO
- TSMO and Data – Considerations and procedures for obtaining and processing TSMO-related data for performance measurement
- Planning and Programming for TSMO – Considerations for objectives-driven, performance-based operations planning, TSMO strategies, programming, multi-modal system performance measures, ITS architecture, systems engineering, and analysis tools
- Corridor Management – Considerations and analytical procedures for incorporating TSMO into corridor management
- System Management – Planning-level sensitivities and considerations for operations strategies related to incident and emergency management, road weather operations, special events management, traveler information, transportation demand management, and connected/automated vehicles

18.1.2 Policy Basis for TSMO

ODOT has established transportation goals that are both supportive of and supported by TSMO. The overarching goal of the 2006 Oregon Transportation Plan (OTP) is “a safe, efficient and sustainable transportation system that enhances Oregon’s quality of life and economic vitality." OTP Goal 2, Management of the System, is specific to TSMO and states "improve efficiency of the transportation system by optimizing the existing infrastructure with improved operations and management." OTP Key Initiatives A and B reflect the desired direction of the OTP to maximize existing system assets and to optimize capacity using TSMO strategies.

Other state transportation policies that are supportive of TSMO include the Oregon Highway Plan (OHP) Policy 1G, Major Improvements, which states "maintain highway performance and improve safety by improving system efficiency and management before adding capacity"; OHP Action 1G.1, which establishes an investment hierarchy that prioritizes strategies that "protect the existing system" above all others; and Policy 2E, Intelligent Transportation Systems, which states "consider a broad range of TSMO services to improve safety and efficiency in a cost-effective manner." Operational Notice PB-03 issued to ODOT personnel provides direction for developing financially feasible ODOT facility plans and local Transportation System Plans consistent with OTP and OHP policies for managing and maintaining ODOT's existing transportation system.

Beyond the ability to advance many key transportation goals in the state, TSMO also provides a platform for implementing a performance-based approach to planning, designing, operating, and maintaining a transportation system. The federal surface transportation legislation, Moving Ahead for Progress in the 21st Century Act (MAP-21), laid the groundwork for a paradigm shift in planning and programming transportation improvements with the establishment of a performance-based program. The need for on-going data to support a performance-based program is acute and TSMO technologies provide a way to automate the collection and archiving of large amounts of operational performance data. The TSMO program also offers an objectives-driven, performance-based approach for planning and programming that effectively applies data in the decision-making process.

18.1.3 Rationale for TSMO

This section is intended as a high-level overview of TSMO. For a more in depth understanding of TSMO refer to FHWA Planning for Operations web page.

TSMO offers a performance-based approach to managing the multimodal transportation system in support of the OTP and OHP goals and policies. The many strategies that fall under the TSMO umbrella address one or more of ODOT's key policy goals of safety, efficiency and sustainability.

Safety

Safe travel is ODOT's highest priority for the transportation system. TSMO can help address system safety for all users through technology and operational strategies that focus on minimizing conflicts. This can take the form of traffic signals with dedicated phasing for different movements and modes; traffic incident management programs that quickly clear incidents to increase safety for responders and reduce the risk of secondary crashes; road weather information systems to notify travelers of adverse weather conditions; or variable speed signs that adjust advisory travel speed based on traffic conditions ahead.

Efficiency

The economic health and prosperity of Oregon and its communities depend on a well-functioning transportation system. TSMO's contribution is considered in two ways: the efficient use of the existing transportation system and efficient use of resources.

With regard to efficient use of the existing transportation system, many TSMO strategies address non-recurring events that cause travel delay in both urban and rural settings such as ineffective traffic control operations, traffic incidents and inclement weather. These strategies support reliable travel for people and goods by actively managing the existing transportation system. The intent is to maximize the function and performance of current transportation networks to reduce delay and improve reliability for all modes. The TSMO strategies applied vary based on the modes, facility types, and land use context. For example, strategies like transit signal priority or pedestrian signal phasing can help keep travelers moving in busy urban environments while in rural locales strategies like smart work zones or incident or event-based traveler information can address those unique needs.

The expected growth in population, freight tonnage, and total vehicle miles traveled will place an enormous burden on the existing transportation infrastructure into the future. As fewer funds are available for adding capacity, optimizing the existing transportation system has become a critical and practical approach. TSMO strategies generate resource efficiency by enhancing system capacity for less money, time, and disruption than traditional approaches. It optimizes resource use by allocating financial and personnel resources to cost-effective programs, such as reducing incident response times or maintaining traffic signal timings that have proven effective in increasing performance of the transportation system.

Another dimension to TSMO's resource efficiency is the opportunity to share resources across agencies. TSMO is most effective when multiple partners coordinate or collaborate to deliver a service like traveler information; or share infrastructure like fiber optic cable network; or establish interagency agreements like joint traffic signal operations and maintenance.

Sustainability

Transportation has an integral role in protecting and preserving livable and sustainable communities. Livability is described by ODOT as "the attributes of a community that affect its suitability for human living". The ODOT Sustainability Act of 2001(ORS 184.421) defines sustainability as "using resources in a manner that enables people to meet their current needs while allowing future generations to meet their needs." Managing how the transportation system operates is a vital aspect of livability and sustainability.

The broad suite of TSMO strategies actively contribute to both goals of meeting community needs today and managing resources for the future. In addition to the livability benefits of transportation safety and efficiency, TSMO can also facilitate multimodal travel choices, optimize on and off-street parking, or provide route options to avoid delay-inducing events. It helps to preserve mobility by implementing operational solutions like bike signals, transit signal priority, and personalized trip planning that support safer and more sustainable travel choices.

A significant environmental benefit of TSMO centers on optimizing the efficiency of vehicles to save fuel and reduce vehicle emissions. Several categories of TSMO strategies such as congestion management (ramp meters) or speed management (variable speed signs) can smooth traffic flow and bring down vehicle speeds. Multiple studies have documented the reductions in fuel use and harmful emissions because of reducing vehicle acceleration and deceleration events. With the growing efforts to address climate change in Oregon, TSMO strategies offer near-term, lower cost, efficiency-focused approaches to transportation-related greenhouse gas reduction and adaptation to a changing climate.

The overall benefits attributed to TSMO strategies typically include:

- Reduced travel delay
- Reduced travel times
- Improved travel time reliability
- Reduced number of crashes
- Reduced instance of secondary crashes
- Reduced fuel consumption
- Improved air quality
- Improved agency operational efficiency

18.2 TSMO and Data

This section provides an overview of the relationship between TSMO and data. While the use of data has long been a key element in the practice of planning, designing and operating the transportation system, the use of technology to actively manage the transportation system has given rise to new data sources and new applications of those data in operating the transportation system.

Data are an integral element of TSMO. The rapid and dynamic changes in transportation technology are delivering a wealth of new data sources that are being generated by both roadside and mobile sensors that register changes in motion, temperature, light, air quality, and the list goes on. These data can be collected and transferred in real-time to end users and it can be captured and stored for later use in evaluation or research. Sensor-based technologies deliver the data necessary to support active operation of the system and objectives-driven, performance-based decision-making for investments, as described in Section 18.3, Planning and Programming for Operations.

While there is an overlap with Chapter 3, Transportation System Inventory, the focus of this section is on sensor-based data and its relationship to managing and operating of the transportation system. The section discusses the following topics:

- Data Management lists considerations for collecting, processing and interpreting sensor-based data.

- Agency TSMO Data describes sources of sensor-based transportation-related data available through ODOT and partner agencies.

- Third-Party TSMO Data describes sources of sensor-based transportation-related data available from private sources.

- Portal Transportation Data Archive provides an overview of the publicly accessible transportation data archive housed at Portland State University.

- TSMO Performance Evaluation and Monitoring discusses the use of before and after analysis and system monitoring in deploying TSMO.

> *ODOT is in the process of preparing a TSM&O Performance Measures Plan. The outcomes of the planning effort will be incorporated in this section.*

18.2.1 Data Management

Data management encompasses the organization and use of data. The first step to good data management is a clear understanding of how data will be used. Clarity in the application of data helps to inform decisions about collecting, processing, and interpreting data. The following list includes common components and considerations for managing data.

- Collection is the systematic process of gathering and measuring data. Traffic data can be collected using either automated or manual methods. Automated data collection methods are designed to continuously record data in discrete time periods. Some examples of automated collection methods are traffic counters, portable traffic recorders, Bluetooth or Wi-Fi recorders, and weigh-in-motion devices. Manual collection refers to visually observing and recording data using tally sheets, counting boards, or manually reported data such as crash data.[1]

[1] https://www.fhwa.dot.gov/policyinformation/tmguide/tmg_fhwa_pl_13_015.pdf

- Data Quality refers to the strength, trustworthiness, and validity of data. A specification can be used to ensure data quality is preserved. There are five key factors to consider when assessing data quality:

    - The *Validity* of the data is the relevance of collected data to the performance measure or goal being studied.
    - The *Completeness* of the data indicates whether there is enough information to draw a reasonable conclusion about the data.
    - *Data Consistency* considers whether data are collected using the same processes and procedures in every instance and by every individual or agency collecting the data.
    - The *Accuracy* of the data refers to whether the data makes sense and are free from significant errors.
    - The *Verifiability* of the data considers whether there are ways to verify the data were reported and collected according to accepted procedures.[2]

- Data Monitoring is the practice of routine checking of data against quality control factors. Monitoring data can assist with design, maintenance, operations, safety, environmental analysis, finance, engineering economics, and performance management.[3] As an example, sensor failures can be detected through data monitoring practices. Data monitoring can also provide feedback to inform planning, such as whether planning goals for implementation and effectiveness of TSMO program investments are being met. For more information see section 18.2.5.

- Calibration is the process of performing a variety of tests on equipment to ensure that it functions as intended and correctly collects, processes, and reports data. The calibration process can identify errors such as failed or improperly set up sensors or incorrect algorithms that can result in collecting, processing, storing, and disseminating inaccurate statistics.[4]

- Storage – Data storage can be a major challenge as the volume of data being collected is increasing drastically. However, the per gigabyte storage cost of data continues to decline as technology advances. There are also many alternate data storage methods available through the private sector, including cloud storage and data warehouses.[5] With the volumes of collected sensor data, established retention policies provide guidelines for how long data are kept and for what purposes.

- Data Fusion is the process of synthesizing data from several different sources to obtain more meaningful information than gained from a single source. The data are gathered, cleaned to remove inconsistencies, and exported to a centralized

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[2] http://www.nationalservice.gov/sites/default/files/resource/Data_Quality_Elements_Performance_Measures_LearningAidFinal7.23.pdf [Document no longer available]
[3] https://www.fhwa.dot.gov/policyinformation/tmguide/tmg_fhwa_pl_13_015.pdf
[4] https://www.fhwa.dot.gov/policyinformation/tmguide/tmg_fhwa_pl_13_015.pdf
[5] https://www.fhwa.dot.gov/asset/dataintegration/if10019/dip06.cfm

database. A detailed analysis of the characteristics of the data is necessary to mitigate issues with merging different sources of data. Fusing highly incompatible data sources can be extremely painstaking work, but some of this work can be minimized with software tools.[6]

- Interoperability is an approach in which a number of databases are linked through a communications network so that they appear to be from a single source. Interoperable databases allow users from multiple agencies to make a query without concern for where the data resides or how they are organized. An interoperable database provides easier access to resources, improved availability, and greater ability to share data than a fused database. However, the interface for an interoperable database is much more difficult to configure and the maintenance needs are much more complex.[7]

- Sharing – Data collected by agencies may be useful to other agencies or the general public through traveler information systems. Considerations for data sharing include establishing procedures for access and dissemination.

- Granularity refers to the level of depth or detail in a set of data. The finest level of granularity is the smallest pieces of information a dataset can be subdivided into. Courser levels of granularity may include more summarized or aggregated representations of data.

- Communication – Traffic systems use communication systems to transmit data between field equipment and traffic management centers. These systems consist of agency-owned cable, leased privately owned cable, or wireless systems. Communication between field devices is standardized based on National Transportation Communications for Intelligent Transportation System Protocol (NTCIP) specifications. Data sharing and storage are important considerations for data communication.

- Security – In the past, transportation agencies relied on security through obscurity for protection of their communication system. Now that more conventional technologies such as Wi-Fi and Ethernet are commonly used in field devices, communications systems are more vulnerable to attack. It is important for agencies to use current best practices and industry standards to improve security. [8] The FHWA has developed a National Institute of Standards and Technology Framework for Improving Critical Infrastructure Cyber Security: https://nvlpubs.nist.gov/nistpubs/CSWP/NIST.CSWP.04162018.pdf.
- Privacy – Agencies must consider any privacy issues that may arise as a result of disclosure of traffic data. Both federal and state laws recognize a certain degree of

[6] https://nvlpubs.nist.gov/nistpubs/CSWP/NIST.CSWP.04162018.pdf
[7] https://nvlpubs.nist.gov/nistpubs/CSWP/NIST.CSWP.04162018.pdf
[8] https://www.fhwa.dot.gov/publications/publicroads/16sepoct/01.cfm [Document no longer available]

privacy with respect to driver information. For this reason, traffic data that a public agency collects or purchases should be anonymous in nature.

- Metadata are data that provides information about other data, often how the data were collected. An example of metadata is the location and timestamp data associated with a photograph. Metadata can also include elements such as the person who collected the data, the title of the dataset, file size, date created, date modified, timeframe, accuracy, collection method, and information about the collection device.

18.2.2 Agency TSMO Data

Agency data services are critical for successfully managing Oregon's transportation system. ODOT manages datasets on both freeways and arterials, including vehicle, pedestrian, bicycle, and transit data. More information about many of these data sources can be found in Chapter 3, Transportation System Inventory.

Freeway Data

ODOT shares live video feeds from the State's traffic cameras as well as weather, incident, closures, delays, and construction data on the Trip Check website – https://www.tripcheck.com/.

On Oregon's freeways, ODOT collects speed, volume, classification, and occupancy data with both loops and radar through Automatic Traffic Recorder (ATR) stations, ramp meters, and temporary counting stations. ODOT also collects travel time data using Bluetooth stations. These data are gathered and managed by the Transportation Data Section (TDS). Volume data are shared through the TDS web site through the Transportation Volume Tables.[9] Volume and classification data are also publicly available through ODOT's TransGIS web site.[10]

ODOT records and logs every message displayed by a Variable Message Sign (VMS) or Variable Advisory Speed (VAS) sign through its data acquisition system, managed by the ITS Unit.

Weigh in Motion (WIM) devices capture and record axle weights and gross vehicle weights as drivers drive over a measurement site. ODOT currently operates 22 WIM stations that pre-clear an average of 4,400 trucks a day.[11] WIM data is managed by TDS.

Multimodal Arterial Data

Many of the same methodologies for data collection on ODOT's freeways can also be applied to arterial corridors. Count stations gather volume, speed, and lane occupancy data. Some traffic signals are also equipped with detectors that can gather vehicle counts by lane and time period. Bluetooth sensors gather travel time and origin-destination data to help measure and optimize arterial performance. Additionally, pedestrian and bike data

[9] https://www.oregon.gov/ODOT/Data/Pages/Traffic-Counting.aspx
[10] https://gis.odot.state.or.us/transgis/
[11] https://www.oregon.gov/odot/mct/pages/green-light-program.aspx

can be collected on arterial corridors. Pedestrian volumes can be estimated using pushbutton actuations at signals and enhanced pedestrian crossings such as Pedestrian Activated Beacons (PABs). Bicycle volumes and travel times can be determined using many of the same methods used to count motor vehicles.

Arterial traffic data are also used to operate adaptive traffic signal control systems. Adaptive signal control systems continuously monitor and evaluate data and adjust signal timings every few minutes to improve travel time and reduce delays.

Transit Data

TriMet, C-TRAN, and many other transit agencies provide real time data for buses and trains in General Transit Feed Specification (GTFS) format. TriMet also publishes monthly ridership and performance statistics.[12,13] ODOT is starting a project to standardize ridership data and formats from transit agencies around the state. The project will seek to develop an ecosystem of open-source software tools around the new data standard.

Incidents

Automated incident detection and management data are available through the Highway Traffic Operations Center System (TOCS). The system collects, analyzes, disseminates and archives data in the transportation areas of operations, traffic, incident, and emergency management. The TOCS will include traffic surveillance, road/weather condition monitoring, incident detection and reporting, signal control, and emergency call taking.

The Highway Travel Conditions Information System (HTCIS) includes the status updates that are shared with the public on the ODOT TripCheck website, through 511, and ODOT's TripCheck Traveler Information Portal (TTIP), which allows external sources to access ODOT's data for redistribution. It includes which lanes are affected and what the expected impact is to travelers (i.e., delay experienced). Each incident usually has multiple status updates throughout the duration of the incident. These databases also include weather related information (atmospheric and the resulting road conditions) at the various weather stations throughout the State. The ITS Unit manages these data sources.

ODOT also collects crash data from individual driver and police crash reports through its Crash Data System (CDS) managed by the Crash Analysis & Reporting Unit.

Weather Data

ODOT has weather stations installed across the state that continuously record temperature, dew point, humidity, wind direction, wind speed, visibility, and precipitation. The public via TripCheck can access weather data.[14] ODOT also has several roadway weather surface state sensors that monitor grip factor (or relative friction) of the roadway. ODOT’s ATM systems currently use these data in adjusting

[12] https://developer.trimet.org/
[13] https://trimet.org/about/performance.htm
[14] https://www.tripcheck.com/textpages/RWISreport.asp?curRegion=0

advisory speeds in variable speed corridors and activating advanced curve-warning systems. The data are captured in the data acquisition system (DAC), managed by ODOT's ITS Unit.

Additional weather data are available from the Oregon Department of Environmental Quality, which monitors air quality and reports an air quality index based on the concentration of pollutants in the air.

ODOT also has a pilot program that collects data from a subset of agency-owned snowplows. Data collected by the pilot program includes road temperature, air temperature, location, speed, vehicle diagnostics, plow position, and details about the surface treatments applied to the road.

18.2.3 Third-Party TSMO Data Sources

Third-party data comprises an increasingly important source of transportation data for ODOT, providing the agency data types, resolution, locations, and date ranges not otherwise available from agency-owned data collection systems. This subsection provides a general overview of the types of data that can be acquired from third-party sources, with a focus on the third-party data providers with whom ODOT has developed agreements for use of real-time data that have been archived and made available for secondary use.

Travel Time, Speed, and Congestion Data

In the absence of strategically installed Bluetooth readers, Wi-Fi sensors, or similar infrastructure elements, travel time data are obtained from GPS and cell-phone probe vehicles runs. The prevalence and low-cost of GPS/cellular-enabled mobile devices makes private crowdsourced travel time, speed, and congestion data collection an attractive offering to third-party developers.

Location referencing for the third-party transportation data is enabled by the Traffic Message Channel (TMC), the commercial industry standard used by HERE and TeleAtlas mapping firms. Use of third-party data by public agencies requires the integration of the TMC referenced road network with the agency road network. The basic data elements provided with third party data are date, timestamp, roadway link identifier, roadway link length, and roadway link travel time or speed.[15]

Major third-party travel time, speed, and congestion data providers include HERE, TomTom, INRIX, and Google-owned Waze. Each data provider utilizes a driver network, comprising vehicles, smartphones, or other GPS-enabled devices, to monitor basic location and speed attributes of the vehicle. The provider often utilizes data analytics capability to derive useful information from these attributes, including travel time and congestion levels. Incorporating a large probe user base with sufficient distribution across a given geographic region, a third-party data provider can generate detailed segment-based speeds and travel times for all the primary roadways in the region.

[15] https://ops.fhwa.dot.gov/publications/fhwahop11029/ch2.htm

ODOT Partnership Highlight – INRIX Travel Time Data

ODOT partnered with INRIX to acquire raw segment speed information for major Oregon roadways for the years 2008 to 2013. As part of this agreement, ODOT received access to RITIS, the INRIX online analytical tool used to perform data visualization, reporting, and analytics on the raw datasets. Note - As of 2016 ODOT does not have a current contract with INRIX and therefore cannot access the RITIS analytics capabilities for the archived raw data.

INRIX data collection does not differentiate between passenger vehicles and freight vehicles, so any freight traffic analysis using this dataset requires making assumptions about the vehicle mixes. The Transportation Planning Analysis Unit (TPAU) oversees access to this dataset.

ODOT Partnership Highlight – National Performance Management Research Dataset (NPMRDS) Travel Time Data

Made available by the FHWA Office of Freight Management and Operations to all state DOTs and MPOs, the NPMRDS contains raw HERE travel time data for all National Highway System routes. This dataset is provided in 5-minute periods and differentiates between auto, truck, and combined travel times. Although freely available, NPMRDS does not include an analytical component. ODOT utilizes data analysis and visualization tools, JMP and Tableau, to analyze, filter, and visualize the data. TPAU oversees access to this dataset.

ODOT Partnership Highlight – HERE

Beginning in 2016, ODOT has access to raw HERE travel time data that expand upon the NPMRDS-provided coverage to include many non-NHS routes. The dataset includes historical data (as far back as 2012) in five-minute increments and current data available in one-minute increments. Like with the NPMRDS, the HERE travel time dataset can be differentiated between passenger and freight vehicles. TPAU oversees access to this dataset.

Incident Data

Most roadway incident data is generated by public agencies, typically by highway patrol or transportation management center systems. However, some crowd-sourced mobile traffic and mapping applications support an active user interaction component in addition to using GPS to report the location of the vehicle. A well-known and well-used example is Waze, which provides users the ability to submit enroute reports on roadway conditions, including incidents, road hazards, traffic jams, and police presence.

Some public agencies are beginning to partner with traffic data companies to acquire their user-generated reports on incidents and other conditions to supplement their own detection and reporting systems. Examples include Florida DOT, which in 2014 entered into a data-sharing agreement with Waze to share its traffic detection data in exchange for Waze's user road conditions reports. In 2015, the city of Los Angeles entered into a partnership with Waze to provide the company information about construction, film shoots, planned closures, and events occurring on L.A. streets. In return, the city receives Waze user-generated incident reports. Additionally, the city can use the Waze application as a broadcast platform to send out hit-and-run and child abduction alerts to Waze users. Public agencies may also purchase data from Waze or other crowd-sourced mobile traffic mapping applications.

Currently, Waze consumes data through the TripCheck Traveler Information Portal (TTIP) on road closures, incidents and construction supplied by ODOT and local agencies using the TripCheck Local Entry (TLE) Tool).

Bicycling and Pedestrian Data

Similar to third-party vehicular travel time data providers, third-party bicycling data providers utilize a GPS device-enabled crowdsourced approach to generate bike trip data that can be purchased by public agencies. Available data types include bicycle volumes, locations (including cut-through pathways), speeds, time of day, direction of travel, route choice for given origins and destinations, and user-generated conditions observations.

Some platforms, like Strava, are geared toward fitness users who use the application to track training rides and compare their results against others in the Strava community. Other app platforms, like Ride Report (https://ride.report/) and Portland State University and ODOT sponsored ORcycle, encourage users to report on their perceived stress level of routes and to provide feedback on crash, safety, or infrastructure issues observed on a ride.

Third-party pedestrian data are a relatively unexplored area of third-party data sources to date. Potential sources include GPS device-enabled fitness tracking apps, similar in function to Strava, in which users run the application in order to track their workouts or monitor their overall movement and activity level. An example is Nike+ Running (https://www.nike.com/us/en_us/c/nike-plus/running-app-gps). Most household travel surveys also incorporate GPS tracking as part of monitoring the daily travel by all modes on a select survey day.

Another emerging approach to obtaining pedestrian movement data is through installed detectors that infer pedestrian activity within the system's field of detection. These systems are most commonly deployed in retail environments to help businesses monitor foot traffic for operational improvements.

<u>ODOT Partnership Highlight – Strava Bicycle Data</u>

In 2014, ODOT became the first public agency in the nation to license bicycling trip data from Strava, a leading app developer and social platform for users to track their bike rides via GPS. Strava data relates to TSMO because it provides a representation of bicycle volumes that can be used to inform both operational and planning decisions. Strava data can also be used to compare one route to another. TPAU oversees access to this dataset.

The motivation for the partnership was to supplement the limited bicycle travel information collected by the agency through travel surveys and strategically located fixed counters. The lack of information on bicycle travel information makes it more difficult to track the success of statewide biking efforts and to target future investments appropriately.

The Strava dataset provides statewide minute-by-minute bike traces (i.e., the presence and direction of travel of a Strava user) mapped to link segments on the Open Streets Map base map. Aggregating these traces reveals not just how many Strava users are using the system, but how, when, and where they are riding.

For the 2014 year, the Strava dataset included travel data for 20,400 Oregon users, who logged 540,000 bike trips (averaging 26 trips per year each), for a total of 5.6 million bike miles traveled.

Comparing Strava data records for a given time period and location against fixed location bicycle counter readings (for example, at Hawthorne Bridge in Portland and various locations in Eugene), ODOT has determined that Strava users represent roughly 1 - 10% of the total bicycle volume. While the share of Strava trips varies by location, the percentage has remained consistent over time at each location. Further analysis by ODOT has indicated that Strava data are representative of which routes are popular with users overall and identify non-auto pathways used by cyclists.

Key use cases proposed by ODOT that make use of Strava data include:

- Identify and validate high-demand areas in which to locate bicycle counters, especially in non-urban areas
- Inform the deployment and placement of highway rumble strips to identify locations where high bicycle volumes may warrant different installation approaches
- Justify and target seasonal maintenance efforts
- Supporting county and other non-ODOT jurisdictions with cycling data to inform bike path plans and designs

Transit Data

Third-party sources of transit data are uncommon since transit providers are typically public agencies. However, there are third-party data analytics companies that specialize in ingesting raw data generated by the transit agency (like fare card data and GPS traces) and performing various analytics functions to visualize travel patterns, identify delays and congestion, platform crowding, and wait times.

NextBus (http://nextbus.cubic.com) is another example of a third-party data analytics company that provides added value to the data generated by transit agency. NextBus uses GPS location information from the transit vehicle and a proprietary algorithm that incorporates historical travel data to track transit vehicles and predict their arrival time.

Weather Data

In addition to the publicly available National Weather Service weather data, several third-party providers offer weather data services. An example is Weather Underground (https://www.wunderground.com/), which provides forecasts generated by a proprietary forecasting system that draws from a network of tens of thousands of neighborhood and personal weather stations across the country. Weather Underground offers an API that provide layers such as radar and satellite and a variety of weather data features, including alerts, conditions, forecasts, and astronomy.

18.2.4 Portal Transportation Data Archive

Portal Transportation Data Archive, formally known as PORTAL the Portland Oregon Regional Transportation Archive Listing (https://portal.its.pdx.edu/home), is a publically funded data archive hosted and maintained by Portland State University. Started in 2004, Portal is the region's archive for many different types of transportation related data for the Portland and southwest Washington metropolitan areas.

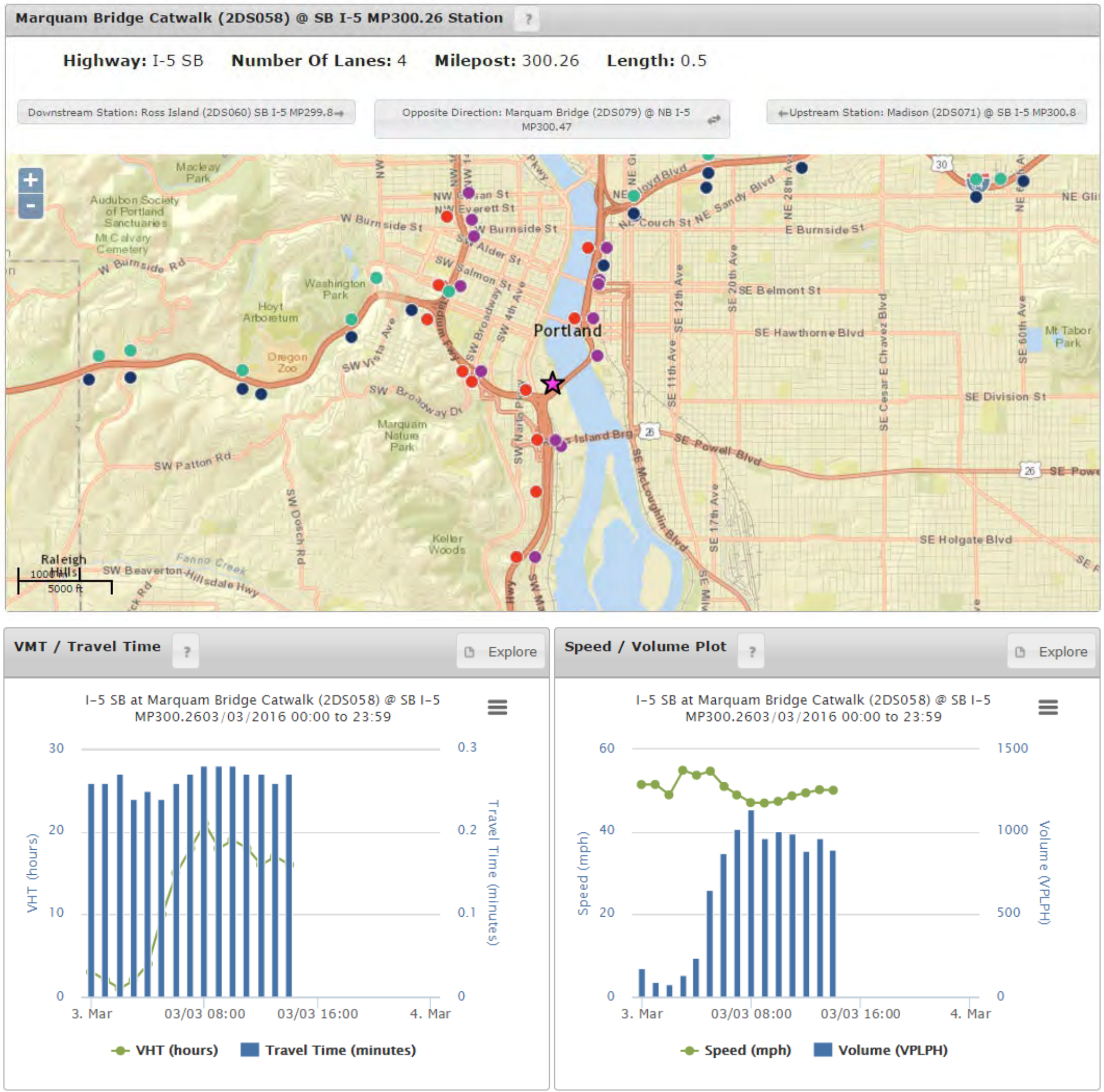
Currently, Portal archives freeway detector data from ODOT and WSDOT, travel time data from ODOT and PBOT, VMS/VAS messages from ODOT, Weigh-in-Motion data from ODOT, arterial detector data from Clark County and PBOT, traffic signal data from PBOT, bike and pedestrian counts from PBOT, automatic vehicle location (AVL) and automated passenger counter (APC) data from TriMet, air quality data from DEQ, and weather data from NOAA.

While Portal archives all the above mentioned data, a graphical user interface is also available for viewing and downloading some of the data described. The Portal website offers maps, graphs, and data downloads related to the following:

- Detector data on OR and WA highways in the Portland and Vancouver areas
- Detector data on Portland and Vancouver Arterials
- Travel time data in Portland
- Transit performance metrics in Portland and Clark County

Examples of some of the Portal visualizations are shown below in Exhibit 18-1.

Exhibit 18-1: Portal Stations Map and Subsequent VMT/Travel Time and Speed/Volume Graphs[16]



In addition, the information available through the Portal website, additional data available upon request. A complete list of data archived is listed below in Exhibit 18-2.

[16] http://portal.its.pdx.edu/stations/view/id/3186/

Exhibit 18-2: Data Archived at Portal including Resolution and Date Ranges

Type of Data	Agency	Detection	Resolution	Date Range(s)	Location	Collection Frequency
Speed Volume Occupancy	ODOT	Loops Radar	20-sec 5-min 15-min Hour	2004- (loops) ~2014- (radar)	Portland freeways, Beltline in Lane Co.	Every 2 minutes
Speed Volume Occupancy	WSDOT	Loops Radar	20-sec 5-min 15-min Hour	~2012	Vancouver freeways	Every 20 seconds
Travel Times	ODOT	Bluetooth	Individual Traversals	~2014-	Portland freeways & selected arterials	Every 2 minutes
VMS/VAS Messages	ODOT	N/A	On sign change	~2014	Where installed	Every 2 minutes
WIM	ODOT	In-road sensors	Hour	~2011-2013	WIM stations in Oregon	Monthly
Travel Time	PBOT	Bluetooth	Individual detections	~2012-	Various locations in Portland	Hourly
Arterial Speed, Volume	Clark County	High Definition Radar	5-min	~2014-	Various locations in Clark County	Daily
Traffic Signal Data	PBOT	TransSuite, MOE & SCATS	5-min	~2014-	City of Portland	Hourly
Passenger Counts, On-time Performance	TriMet	AVL/APC	Quarter	~2012	TriMet service area	Quarterly
Weather	NOAA	NOAA detection	Hour	~2004-	Airports (PDX, Hillsboro, Aurora)	Hourly
DEQ Air Quality	DEQ	Weather and Air Quality data	Hour	~2014	DEQ site near Powell Blvd.	Hourly
Bike counts	PBOT/ LCOG	Loops	Unknown	~2014	Various locations in Portland	Hourly

Type of Data	Agency	Detection	Resolution	Date Range(s)	Location	Collection Frequency
Bike counts	LCOG	Loops, pneumatic tubes, infrared, manual	Short duration	~2012	Various locations in Eugene, Springfield, and elsewhere in Lane County	
Pedestrian Pushbutton Actuations	PBOT/LCOG	Pushbutton	Individual actuations			

18.2.5 TSMO Performance Evaluation and Monitoring

Most TSMO deployment projects involve evaluating, adjusting, and improving the performance of the system. Performance evaluation informs the effectiveness of specific TSMO strategies and can aid decision makers in project selection for future capital investments. Performance evaluation can also identify problems or failures, identify fixes to implement, or determine parameters to adjust to optimize performance. Often, the performance evaluation component of a project takes the form of a before and after study, where baseline "before" transportation data are compared to "after" data collected post-deployment.

Before and after study designs can be used to assess the impacts from TSMO deployments on efficiency, reliability, safety, and travel behavior. Data needs are dictated by intent of the evaluation. For example, a before and after evaluation of impacts to travel time reliability from a TSMO project requires collecting travel time data during the periods before and after deployment. It is important to collect sufficient data ahead of a project implementation and after completion to statistically measure factors like reliability. The cost of performing before and after studies, which has been a barrier, is greatly reduced with technology-based methods.

Evaluation should define in the project planning phase. Depending on the study design, performance evaluation can be set up as a one-time before period vs. after period analysis, or as a comparison of before data to after data collected over time. A study design with multiple after periods allows for observations of trends over time. After periods typically occur at specified intervals (e.g. annually). The initial after period typically does not occur until a duration of time passes beyond project construction (e.g. six months). This is so data is not skewed by abnormal behavior that may occur upon installation of new transportation devices in the system. It is important that the before period and after period occur during the same time of year so seasonal differences in traffic and travel conditions do not skew the results.

Analysis methods can vary depending on the goals of evaluation, data structure, and data availability. In the ideal case, a simple matched pairs comparison[17] can highlight whether significant changes occurred in the data between before and after periods due to a TSMO project. **Examples of some before and after studies completed in Oregon** are shown below in Exhibit 18-3.

Exhibit 18-3: Example Before and After Evaluations

Project Name	Description	Performance Measures	Data Used	Findings
OR 217: Active Traffic Management	Evaluated effectiveness of Active Traffic Management System	Crashes Travel Time Travel Time Reliability	Transportation Data Services Crash Reports Washington County Consolidated Communications Agency Data HERE Data PORTAL	Reduction in travel time, improvement in travel time reliability
Beltline Highway Ramp Metering	Evaluated system performance of Beltline Highway in Eugene, Oregon before and after implementation of ramp meters	Incidents Volumes Speeds Travel Times Reliability	ODOT Incident logs ODOT ATR Data ODOT INRIX™ Analytics Suite	Reduction in incidents, negligible change in speeds, travel times, and travel time reliability
Cornell Road InSync Evaluation	Comparison on operations between TOD signal timing plans and InSync adaptive signal system operation	Travel Time Delay	Bluetooth MAC address data Percent Arrival on Green Traffic Counts	Reduction in travel time during high volume periods, small increase in overall delay
911 CAD Integrated Dispatch Project	Evaluation of 911 dispatch interconnect system between Deschutes County 911 call center, OSP call center, and ODOT incident response system	Notification Time Dispatch Response Time Incident Duration Responder Arrival Time Percentage of Highway-Related Notifications Received by	ODOT Incident Logs OSP Call Center Logs Deschutes County 911 Call Center Logs	Reduction in dispatch response time, incident response time, and incident duration

[17] See Dalgaard, P. (2008). *Introductory Statistics with R*. Springer Science & Business Media, or another introductory statistics text.

Project Name	Description	Performance Measures	Data Used	Findings
		ODOT		
District 8 Incident Response Evaluation	Evaluation of pilot program for a dedicated incident response service patrol	Clearance Time Average Incident Response Time Maintenance Calls	ODOT Highway Traffic Operation Center System (HTOCS) Highway Travel Conditions Information System (HTICS)	Reduction in response time, incident duration, clearance time, and maintenance calls

Key considerations for evaluations include collecting quality baseline (before) data, accounting for external factors that may affect data collected in the future, and developing analysis tools for reproducibility.

Collecting quality baseline (before) data

Issues with baseline before-period data are frequent in before and after studies, and often include incomplete or missing data, structures and formats that don't correspond to after data, and inadequate sample sizes for comparisons. These issues are difficult to overcome because once project deployment occurs, one cannot simply go back and collect more data for the before period. Before data should match the after data in structure and therefore should be collected by the same or similar methods. Before period data should be examined and verified for quality and accuracy prior to project deployment. This can be done through testing and creating a baseline report. Statistical tools like power analysis can help analysts determine minimum sample sizes needed to make appropriate comparisons.

Accounting for external factors

After-period data are subject to influence by external factors beyond the TSMO deployment. Examples include a reduction in vehicle miles traveled due to a spike in fuel costs, a downturn in the economy, or weather conditions that change travel patterns. Therefore, performance evaluations should anticipate normalizing or scaling after period data by also collecting traffic volumes on nearby facilities or other variables. In the case of controlling for traffic volumes, annual ATR data could be used as a potential data source. The nature of the study can inform the types of external variables that may be of interest.

Developing analysis tools for reproducibility

Before and after performance evaluations can benefit greatly by employing reproducible analysis tools. Reproducible tools allow analysts to reach the same results given a set of data and include clear documentation. Reproducible tools can expedite the analysis task in cases when after data are collected at specified intervals. Employing traditional analysis tools may lead to more opportunities for problems in the future. For example, complex spreadsheets in Microsoft Excel may be hard to reproduce when an extended period of time has passed between reporting periods or when employee turnover occurs.

Many open-source scripting languages like R and Python are well suited for creating reproducible analysis tools.

Performance monitoring differs from performance before and after evaluation in that it is the periodic measurement of progress in meeting operational objectives. Monitoring includes a set of agreed-upon measures that are reported out at regular intervals. For TSMO application, performance monitoring can be used to support day-to-day operational decision making, such as winter road operations or incident management. It also applies to the planning for operations process as described in Section 18.3.

18.3 Planning and Programming for TSMO

The section introduces the concept of planning and programming for TSMO. It begins with an overview of the objectives-driven, performance-based approach that supports integration of TSMO into ODOT's planning and programming processes. Key elements of the objectives-driven, performance-based approach are described in this section including stakeholder collaboration, operational objectives and performance measures, TSMO strategies, programming, intelligent transportation system (ITS) architecture and system engineering requirements, and analysis tools.

The intent of this section is to provide guidance for developing and incorporating TSMO into ODOT planning processes including transportation system plans, corridor plans, and modal plans. As noted in Section 18.1, the OTP and the OHP have established the policy basis for TSMO in planning, designing, and operating Oregon's transportation system. Additionally, ODOT has completed a statewide ITS plan and numerous regional ITS plans that serve as resources for planning processes.

18.3.1 Objectives-Driven, Performance-Based TSMO Planning

The basic tenet of the objectives-driven, performance-based approach is to maximize the performance of the existing transportation system. To accomplish this, the approach relies on a collaborative process to establish measurable operational objectives that are supported by system and demand management strategies focused on improving near-term travel conditions for all modes. For comparison, the traditional approach to transportation planning focuses on addressing issues and problems with a set of infrastructure projects that extend into the distant future. The difference between approaches is the focus on near-term measurable outcomes and solutions. While the objectives derive from issues and problems, the solutions are matched to future-focused objectives and monitored for performance over time. The objectives-driven, performance-based approach is a cyclical process of objectives setting, strategy development, implementation, and performance measurement to manage the transportation system.

Exhibit 18-4 demonstrates the process flow for the objectives-driven, performance-based approach. It is important to note that this approach has points of integration with the broader community planning processes to ensure TSMO is incorporated. Integral to the process is the coordination and collaboration with a broad range of stakeholders that occurs at every step. The implementation and operation of TSMO strategies often requires partnership among multiple business units within an agency and across modes and jurisdictions in a region. Applying an objectives-driven, performance-based approach to operations planning further broadens the circle of collaboration to include transportation planners and non-transportation entities such as public safety officials, special interest groups, key attractions, and major employers. Coordination and collaboration among planners and operators is necessary across all steps in the approach and is particularly important in defining operations objectives that feed into long-range transportation plans.

Exhibit 18-4: Objectives-driven, Performance-based Approach

Source: *Advancing Metropolitan Planning for Operations: An Objectives-Driven, Performance-Based Approach – A Guidebook*, FHWA

Establishing TSMO goal(s) focused on the safe, efficient and reliable management and operation of the transportation system is where the process begins. This is a point of integration with broader corridor, regional or statewide planning processes. A transportation plan may identify a single overarching TSMO goal or a set of goals that are broad but address different aspects of TSMO, such as reliability, efficiency, quality of service, and travel options.[18] As noted in Section 18.1.2, ODOT has established TSMO goals and objectives in the OTP and the OHP. It is important to note that TSMO and ITS plans will often include a vision for TSMO. However, when TSMO is integrated into broader planning processes including RTPs and TSPs, TSMO should be reflected in the vision created for these plans.

Operations objectives are the bridge between goals and actions and help determine the selection of and investment in TSMO strategies. These objectives focus on the desired operational performance of the transportation system and address issues related to system

[18] Regional Goals, https://ops.fhwa.dot.gov/plan4ops/focus_areas/integrating/regional_goals.htm, FHWA

reliability, congestion, safety, incidents, weather, work zones, and major events. The process for setting operations objectives relies heavily on stakeholder participation and an understanding of the needs and interests of the traveling public. Section 18.3.2 provides further detail on developing operations objectives and associated performance measures.

The process for identifying and selecting TSMO strategies begins with an analysis of needs in a study area. Here, operations objectives and baseline performance measurement provide the criteria for determining operational needs. Stakeholder input is also critical to ascertaining and verifying needs. Potential TSMO strategies are matched to operations objectives they help to achieve. Strategy evaluation assesses the benefits and costs for each strategy and identifies any co-benefits to other goals and operations objectives. Analysis tools, such as sketch planning tools, travel demand forecasting model post-processors, and simulation modeling may be used to help forecast system needs and analyze the potential benefits of operations strategies.[19] Section 18.3.3 describes common TSMO strategies and considerations for their application. Section 18.3.6 provides an overview of analysis tools commonly used in assessment of operational needs and benefit-cost analysis.

With the selection of operations objectives, associated performance measures, and supportive TSMO strategies, the process shifts to integrating these elements into long-range transportation plans and funding programs, such as the OTP or ODOT's Statewide Transportation Improvement Program (STIP). It is important to note that TSMO strategies can be implemented as standalone projects or programs, or they can be incorporated into the implementation of other projects types like preservation, modernization, and safety. The benefit of on-going collaboration across a diverse set of stakeholders is the opportunity to partner on implementation of TSMO strategies. Section 18.3.4 delves into more detail on programming for TSMO.

Given the diversity of geography that ODOT serves, it is important to note that the objectives-driven, performance-based process is highly scalable to the planning context and can evolve over time. Large urban regions may develop a robust plan with several TSMO-related goals and supporting operations objectives that address a variety of issues. In contrast, smaller rural regions can apply the same process that results in a single TSMO goal with a few operations objectives that address critical system performance issues. The cyclical nature of the process means that initial TSMO planning can be refined and expanded upon over time.

[19] Advancing Metropolitan Planning for Operations: The Building Blocks of a Model Transportation Plan Incorporating Operations – A Desk Reference, FHWA 2010

18.3.2 Multimodal System Performance Measures

> *To learn more about establishing operations objectives and performance measures including examples refer to* Advancing Metropolitan Planning for Operations: The Building Blocks of a Model Transportation Plan Incorporating Operation - A Desk Reference.

Operations objectives are the bridge between goals and actions and help determine the selection of and investment in TSMO strategies. These objectives focus on the desired operational performance of the transportation system and address issues related to system reliability, congestion, safety, incidents, weather, work zones, and special events. The process for setting operations objectives relies heavily on stakeholder participation and an understanding of the needs and interests of the traveling public.

Operations objectives should exhibit S-M-A-R-T characteristics:

- **Specific** The objective provides enough detail (e.g., decrease travel time delay) to identify actions that will achieve the objective.
  Criteria: What is the target issue? What will be accomplished?

- **Measurable** The objective is quantifiable (e.g., by 10 percent) defining how many or how much should be accomplished. Tracking progress against the objective enables an assessment of the effectiveness of an action or set of actions.
  Criteria: Is the objective quantifiable? Can it be measured? How much change is expected?

- **Agreed Upon** Stakeholders come to a consensus on a common objective. This is most effective when the planning process involves a wide range of stakeholders to facilitate regional collaboration and coordination.
  Criteria: Does the objective address an issue of importance to stakeholders? Does it have broad support from stakeholders?

- **Realistic** The objective can reasonably be accomplished within the limitations of resources and other considerations. The objective may require substantial coordination, collaboration, and investment to achieve. To be achievable, it may need to be re-evaluated and adjusted once strategies and costs are defined.
  Can the objective be accomplished with available resources and support?

- **Time-bound** The objective identifies an achievable timeframe (e.g., within five years). As with the Realistic characteristic, the timeframe may need to be adjusted once strategies and costs are defined.
  Can the objective be accomplished within the proposed timeframe?

A basic sentence structure for operations objectives looks like this:

(Action verb) + (the target subject) + (descriptor) by X (unit of measure) by (year).

- Common Action verbs show directionality and include words like *increase, achieve, reduce,* and *decrease*.
- The Target Subject is the activity being measured such as *mode share* or *delay.*
- Descriptors provide details about the subject such as time period of interest (e.g. peak hour, average weekday), geographic focus (e.g. region, neighborhood), mode (e.g. transit, trucks) facility type (e.g. bus route, corridor), and user type (e.g. freight carriers, single-occupancy vehicle drivers).
- Unit of measure defines the standard for a quantity such as *percent* or *miles.*
- Year is the target timeframe such as by *2040*.

Below are some examples of S-M-A-R-T operations objectives:

- *Reduce the number of freeway miles at level of service F in the PM peak by 5% by 2040.*
- *Increase the percentage of major employers actively participating in transportation demand management programs by 20% within 10 years.*

There are two general categories of operations objectives: *outcome-based* and *output-based*. Outcome-based operations objectives address whether strategies resulted in overall improvement. These objectives are high-level, crosscutting and user focused. These characteristics mean that plans will include fewer outcome-based measures. Examples of outcome-based objectives are delay reduction, travel time reliability, and increases in non-single occupancy vehicle mode share.

Output-based operations objectives are focused on the quantity or magnitude of output. They are focused on the operational activities of organizations and support desired system performance outcomes. Output-based objectives tend to be more abundant in plans because organizations more readily monitor outputs and have data to support measurement of the objective. Examples of output-based operations objectives include the frequency of signal re-timing, average incident response time, and share of transit stops with real-time traveler information.

Performance measures quantify operations objectives. They are indicators of the extent to which the transportation system is achieving desired operations objectives. Performance measures also help identify transportation system issues and needs; assess potential impacts of TSMO strategies; communicate progress in achieving goals and objectives; and provide accountability. Exhibit 18-5 provides a table of common operations objectives, associated performance measures, and relationship to modes.

Exhibit 18-5: Example Operations Objectives and Performance Measures Reference Table[20]

Operations Objectives	Performance Measures	Modes Potentially Affected by Objectives and Performance Measures Auto = A; Transit = T; Freight = F; Bike = B; Pedestrian = P				
		A	T	F	B	P
System Efficiency						
Extent of Congestion	• Percent of intersections operating at LOS F or V/C > 1.0 • Rate of increase in facility miles operating at LOS F or V/C > 1.0	●	●	●		
Duration of Congestion	• Hours per day at LOS F or V/C > 1.0 (or other threshold)	●	●	●		
Vehicle Miles Traveled	• Average VMT per capita per day, per week, or per year	●	●	●	●	●
Safety	• Traffic fatalities and injuries per 100 million vehicle miles traveled • Traffic incident clearance time	●	●	●	●	●
Delay	• Hours of delay per capita or per driver	●	●	●	●	●
Energy Consumption	• Total fuel consumed per capita for transportation	●	●	●	●	●
System Reliability						

[20] Source: Adapted from Table 3.2.1: *Cross-Reference Table in* *Advancing Metropolitan Planning for Operations: The Building Blocks of A Model Transportation Plan Incorporating Operation - A Desk Reference*, FHWA

Operations Objectives	Performance Measures	Modes Potentially Affected by Objectives and Performance Measures Auto = A; Transit = T; Freight = F; Bike = B; Pedestrian = P				
		A	T	F	B	P
Non-recurring Delay	• Travel time delay per capita during scheduled and/or unscheduled disruptions to travel	●	●	●	●	●
Travel Time	• Average travel time during peak periods (minutes)	●	●	●	●	●
Transit On-time Performance	• Arrival and departure times (if different) from a select number of stops on transit facilities of interest		●			
System Options						
Mode Share	• Share of trips by each mode of travel	●	●	●	●	●
Modal Options for Individuals with Disabilities	• The percent of intersections with ADA provisions • The percent of individuals with disabilities that can access transit		●			●
Bicycle and Pedestrian Mobility	• Average delay for pedestrians and bicyclists on primary routes • Average pedestrian or bicyclist comfort level measured by survey				●	●

There are limitations that must be considered when developing operations objectives and performance measures. The availability of data needed for performance measures and the difficulty in agreeing upon an appropriate target or timeframe for achievement are obstacles to the agency's use of this approach. Developing operations objectives requires data on baseline conditions and often requires information on historical conditions and forecasts of future conditions to determine a reasonable target and timeframe. These limitations should be factored into consideration and selection of operations objectives and performance measures. It is advisable to begin with a select number of agreed upon objectives and performance measures and develop a process for data management, evaluation, reporting, and decision-making. The cyclical nature of the planning process provides ample opportunity for refinement over time.

Chapter 9 includes more information about performance measures.

18.3.3 TSMO Strategies

Numerous TSMO strategies can be applied to support operations objectives and should be considered and evaluated as part of any planning effort. Appendix 18A provides an extensive list of strategies with a description, key benefits, order of magnitude cost, geographic application, influencing factors, data needs, and level of staffing associated with each one. The strategies are grouped by category:

- Regional Traffic Control
- Traveler Information
- Maintenance and Construction
- Road Weather Operations
- Incident Management
- Emergency Management
- Public Transportation
- Freight
- Archived Data Management
- Transportation Demand Management
- Complementary Strategies
- Strategies that Require Political and Policy Changes
- Emerging Strategies

Exhibit 18-6 provides a list of TSMO strategies to consider based on operations objectives.

Exhibit 18-6: TSMO Strategies to Consider Based on Operations Objectives

<table>
<tr><th>High-Level
Operations Objectives*</th><th colspan="2">TSMO Strategies to Consider</th></tr>
<tr><td colspan="3">System Efficiency</td></tr>
<tr><td>Extent and Duration of Congestion
• Reduce recurring congestion (extent and duration)
• Improve intersection LOS</td><td>(see right)</td><td rowspan="8">Applicable to all system efficiency objectives:
• Traffic network surveillance
• Transportation operations centers
• Enhanced traffic signal operations
• Transit signal priority
• Incident management
• Travel demand strategies that encourage shifts in travel mode, time, or route
• Congestion pricing strategies that encourage shifts to off-peak periods
• Roadside truck electronic screening/clearance
• Traveler information
• Archived data management
• Connected/autonomous vehicles
• Ramp metering
• Freeway/arterial integrated corridor management (ICM)</td></tr>
<tr><td>Intensity of Congestion (Travel Time Index)
• Reduce travel time index</td><td>(see right)</td></tr>
<tr><td>Travel Time
• Improve travel time</td><td>(see right)</td></tr>
<tr><td>Delay
• Reduce hours of delay
• Reduce control delay at traffic signals
• Improve roadway LOS</td><td>• Access management
• Managed lanes
• Bottleneck removal
• Ramp closures</td></tr>
<tr><td>Energy Consumption
• Reduce energy consumption
• Reduce fuel consumption</td><td>(see right)</td></tr>
<tr><td>Cost of Congestion
• Reduce cost of congestion</td><td>(see right)</td></tr>
<tr><td>Vehicle Miles Travel
• Reduce vehicle miles traveled</td><td>• Transportation demand management: trip elimination, trip chaining, mode shifts, increasing vehicle occupancy, land use strategies
• Improve transit travel times and reliability</td></tr>
<tr><td>Trip Connectivity
• Reduce door-to-door trip time
• Reduce cost of transfer fees</td><td>• Transportation demand management for end users: minimize trip transfers, fare integration</td></tr>
<tr><td colspan="3">System Reliability</td></tr>
<tr><td>Non-Recurring Delay
• Reduce person hours of delay caused by scheduled events, work zones, system maintenance, unscheduled disruptions, and traffic incidents</td><td>(see right)</td><td rowspan="5">Applicable to all system reliability objectives:
• Traffic network surveillance
• Transportation operations centers
• Active traffic management
• Freeway/arterial integrated corridor management
• Safety applications
• Incident management
• Emergency management
• Special event management
• Work zone management
• Maintenance and construction activity coordination
• Road weather operations
• Traveler information
• Archived data management
• Connected/autonomous vehicles</td></tr>
<tr><td>Travel Time Buffer Index
• Reduce travel time buffer index</td><td>• Managed lanes
• Freight only lanes</td></tr>
<tr><td>Planning Time Index
• Reduce average planning time index</td><td>• Ramp metering
• Managed lanes
• Public transportation improvements</td></tr>
<tr><td>Travel Time 95th/90th Percentile
• Reduce the 90th (or 95th) percentile travel time</td><td>(see right)</td></tr>
<tr><td>Variability
• Reduce travel time variability</td><td>(see right)</td></tr>
</table>

High-Level Operations Objectives*	TSMO Strategies to Consider
On-Time Performance • Improve average on-time performance for transit and freight	• Advanced transit operations management • Transit signal priority • Truck traffic signal priority • Managed lanes (including transit and freight options) • Electronic fare collection • Roadside truck electronic screening/clearance
System Options	
Mode Share • Reduce per capita single occupancy vehicle commute trip rate • Increase alternative (non-SOV) mode share • Increase active (bicycle/pedestrian) mode share	• Travel demand management • Parking management • Traveler information • Congestion pricing (with electronic toll collection and automated enforcement)
Transit Use • Increase transit mode share • Increase average transit load factor • Increase passenger miles traveled	• Advanced transit operations management • Electronic fare collection • Transit surveillance and security • Traveler information • Travel demand management: marketing, rider incentive programs, transit ease of use
Travel Time – Transit Compared to Auto • Reduce the travel time differential between transit and auto	• High performance transit • Queue jump lanes at signalized intersections • Transit lanes/managed lanes • Transit signal priority
Bicycle and Pedestrian Accessibility and Efficiency • Decrease average delay for pedestrians and bicyclists • Increase the number of intersections with pedestrian features	• Bicycle and pedestrian operations and safety (e.g. pedestrian countdown signals, bicycle detection, timing)

* For numerous SMART objectives, see *Advancing Metropolitan Planning for Operations: The Building Blocks of A Model Transportation Plan Incorporating Operation - A Desk Reference*, FHWA.

18.3.4 TSMO Programming

Programming is the process of selecting projects for funding (based on long-range planning efforts and a prioritization process), identifying funding resources, and scheduling project implementation. This section provides an overview of long-range planning, project selection, TSMO funding sources, and additional TSMO programming resources.

Long-Range Planning

The Oregon Transportation Plan (OTP) and the six associated mode/topic plans (e.g. Oregon Highway Plan) provide the foundation for long-range planning for regional and local transportation system plans in Oregon. The OTP is a 25-year plan that sets statewide goals for mobility and accessibility, management of the system, economic vitality, sustainability, safety and security, funding the transportation system, and coordination/communication/cooperation. The Transportation Planning Rule (OAR 660-012) requires regional and local transportation system plans to be consistent with the OTP.

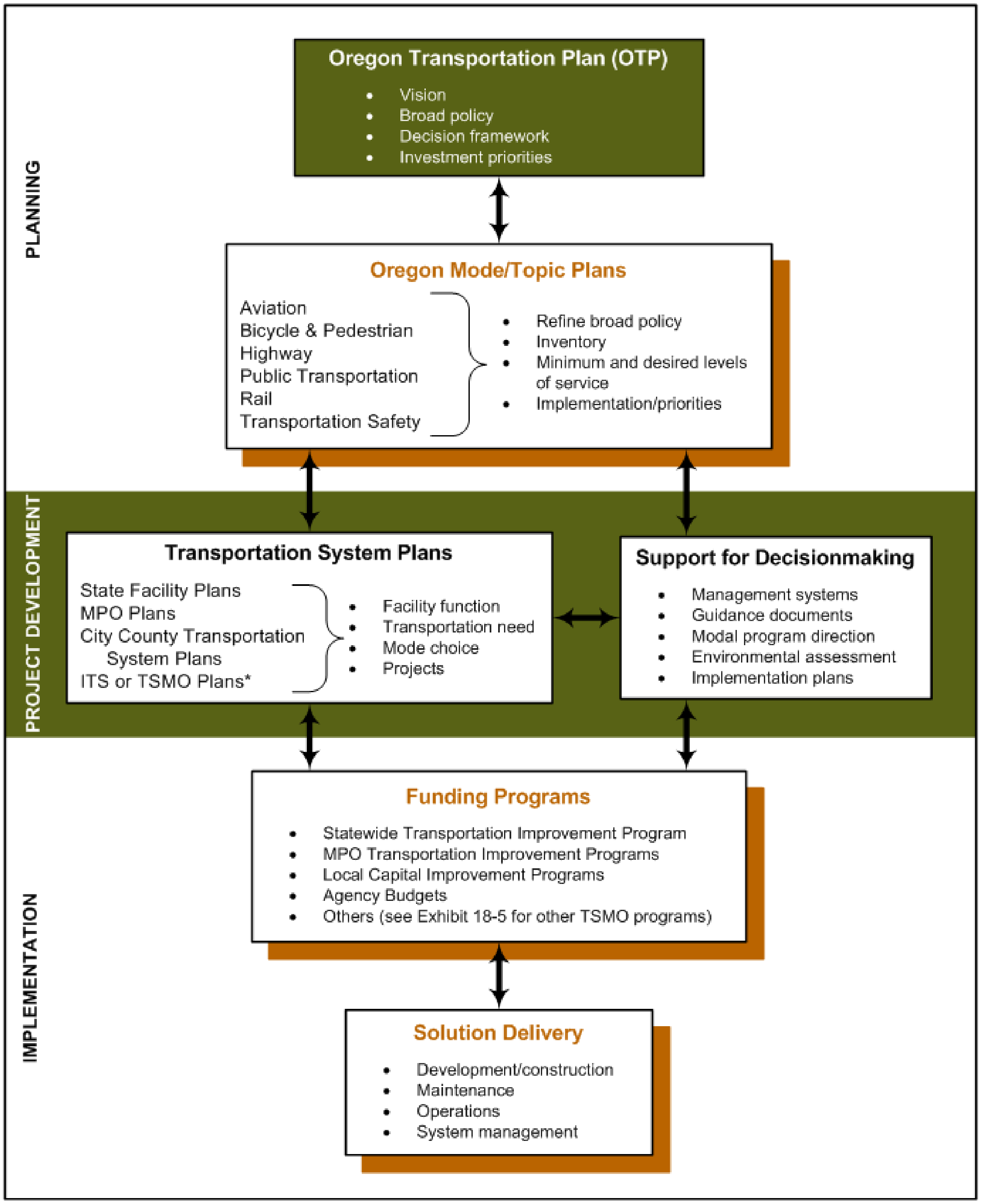
Exhibit 18-7 illustrates the relationship between the OTP, statewide mode/topic plans, and regional and local transportation system plans. The project development and funding programs shown in the exhibit are discussed in the following sections.

TSMO Project Selection for Programming

Projects are typically identified as part of the development of regional or local transportation system plans. These plans use the OTP and state mode/topic plans as policy guidance and develop a list of long-range projects to meet the objectives and goals of the transportation system plan. In the context of the objectives-driven, performance-based approach to planning for operations, measurable operations objectives should be used to support funding decisions as well as to monitor and evaluate projects so that prioritizations can be modified as necessary. Other measures are also used in the project selection process such as benefit/cost analysis or ODOT's least cost planning method.

To date many ITS or TSMO plans have been developed separately from regional or local transportation system plans. [Note: TSMO is broader than ITS as previously defined at the beginning of this chapter. Until recently most agencies focused on ITS plans instead of TSMO plans.] However, the ITS or TSMO plans are often referenced in the applicable transportation system plan or adopted as an appendix. ITS or TSMO plans typically include a phased implementation plan of projects that often compete for funding with projects from the transportation system plans. More work is needed to bridge the gap between traditional planning and operations planning.

Exhibit 18-7: Integrated Transportation Planning and Programming



Source: *Oregon Transportation Plan*, 2006, Figure 7

* Regional or local ITS or TSMO plans are often referenced in or adopted as an appendix to MPO, city, or county transportation plans.

TSMO Funding Sources

Successfully funding TSMO projects or a TSMO program depends on a combination of capital, operations, and maintenance investments to support active management and operations of the transportation system. The funding sources identified in Exhibit 18-8 primarily identify funds that could be used for capital investments; however, funding is also critical for ongoing operations and maintenance. Often operations and maintenance budgets are funded through each agency's own budget.

Exhibit 18-8: Potential TSMO Funding Sources and Application Cycles

Potential Funding Program	Applicable Project Types	Application Cycle
Oregon Statewide Transportation Improvement Program (STIP) *(Note: Prior to the current programs, most TSMO projects received funding through the STIP Operations Program)*	Enhance: Activities that enhance, expand, or improve the transportation system Fix-It: Activities that fix or preserve the transportation system; currently each region receives an allocation of funds specifically for ITS as a subcategory under Operations.	Process begins in odd-numbered years and results in a 4-year funding program
ConnectOregon	Non-highway freight, transit projects, and active transportation projects	Roughly every 2 years
U.S. DOT Transportation Investment Generating Economic Recovery (TIGER) Discretionary Grants	Projects that generate economic development and improve transportation safety, reliability, and affordability	Yearly (typically May)
FTA State of Good Repair (SGR) Grant Program	Transit projects	Varies; part of a 4-year program
U.S. DOE Energy Efficiency and Conservation Block (EECBG) Program	Projects that improve energy efficiency (e.g. signal timing)	Yearly
Homeland Security Funding	Traffic surveillance, CAD integration with 911 centers	Varies
Metropolitan Transportation Improvement Program (MTIP)	Any regional or local TSMO project within an MPO	Varies by MPO and results in a 4-year program
Local Funding (e.g. gas tax, property tax, system development charges)	Any TSMO project; Local match for other funding programs	Varies

Agencies have developed creative solutions for obtaining TSMO funding or reducing costs through practices such as these:

- System sharing (e.g. central signal system shared by multiple agencies)
- Communications infrastructure sharing with other departments within an agency, other transportation agencies, emergency management agencies, or the private sector (e.g. PGE)
- Installing conduit underground as part of capital improvement projects that have been identified as future fiber optic communications corridors
- Staff sharing between agencies for TSMO activities
- Local flexible fund for TSMO that can be used as the local match for STIP, MTIP, or other funding sources
- Purchase of spare parts or back-ups as part of capital construction projects

TSMO Programming Resources

Additional information about programming for TSMO projects is available at:

- FHWA Integrating Operations into Planning and Programming
- FHWA Performance-Based Planning and Programming Guidebook
- FHWA Programming for Operations: MPO Examples of Prioritizing and Funding Transportation System Management & Operations Strategies (includes a Portland case study)
- NCHRP 20-07/345 Program Planning and Development for TSM&O in State Departments of Transportation
- SRHP2/TRB Guide to Incorporating Reliability Performance Measures into the Transportation Planning and Programming Processes
- Oregon Statewide Transportation Improvement Program
- Metro Regional 2021 TSMO Strategy, Appendix F: Finance Report

18.3.5 ITS Architecture and Systems Engineering

This section describes how the ITS architecture and a systems engineering approach can be used in the planning process.

ITS Architecture

An ITS architecture provides a common framework for planning, defining, and integrating ITS within a region. The U.S. Department of Transportation developed the National ITS Architecture so that intelligent transportation systems deployed around the country, and within a region, can communicate with one another and share information to maximize the return on investment for ITS. For example, if a transportation agency wants to clear incidents faster, the architecture defines a function to monitor roadways and identifies the interconnection and information flows between roadway devices, the traffic operations center, and the emergency management center needed to provide responders with incident information. The architecture provides the framework for the process but does not define the technology or management techniques used to provide the information flows.

The FHWA published a Final Rule policy (FHWA Docket No. FHWA-99-5899) that requires all agencies using federal highway trust funds for ITS projects develop a regional or statewide architecture that is compliant with the National ITS Architecture. The FTA published a similar policy (FTA Docket No. FTA-99-6147) that applies to federal funding from the mass transit account of the highway trust fund. For more details on the Final Rule refer to https://ops.fhwa.dot.gov/its_arch_imp/archrule_final_1.htm.

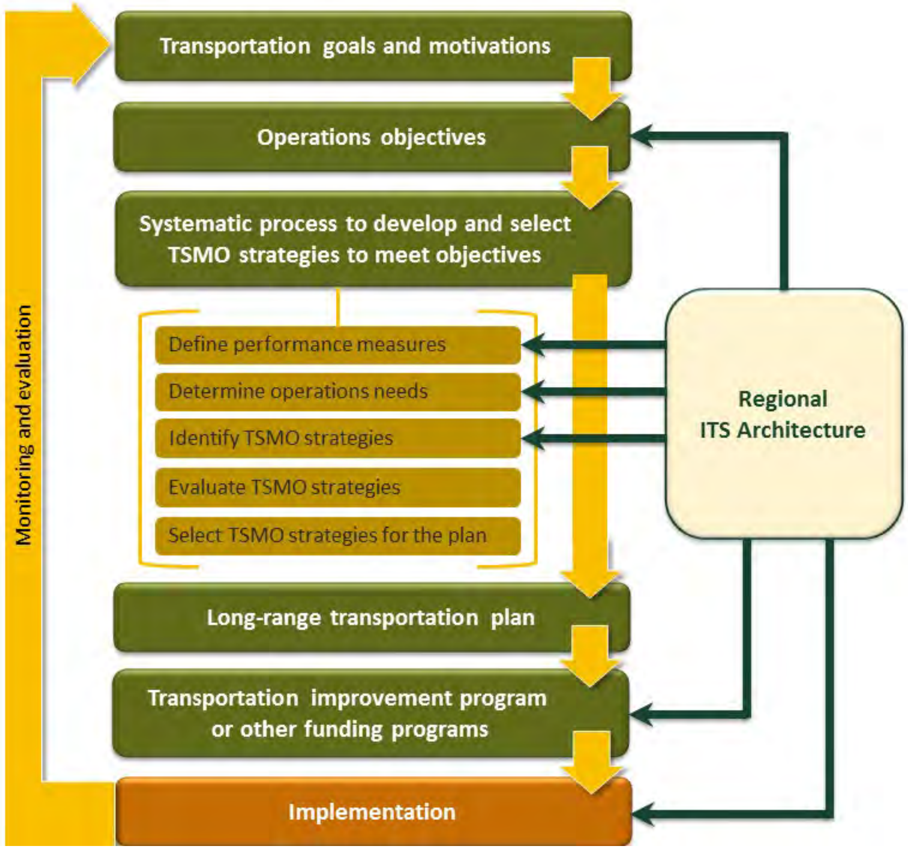
In Oregon, ITS architectures have been developed for the following regions (documentation at https://www.oregon.gov/odot/Maintenance/Pages/Plans,-Architectures-&-Reports.aspx):

- Oregon Statewide
- TransPort (Portland area)
- Salem-Keizer
- Central Willamette Valley
- Eugene-Springfield
- Rogue Valley
- Deschutes County

Exhibit 18-9 shows how a regional ITS architecture can provide support for an objectives-driven, performance-based approach to planning for operations. The regional ITS architecture helps answer important questions such as:

- What existing or planned TSMO strategies may be available to help achieve operations objectives?
- What stakeholders and collaborative relationships can be leveraged as part of the planning process?
- What data are available to monitor transportation system performance and track progress toward operations objectives?
- What parts of the architecture's operational concepts, functional requirements, or other contents can be used to support project development?

Exhibit 18-9: Regional ITS Architecture Use in Planning for Operations



Source: *Applying a Regional ITS Architecture to Support Planning for Operations: A Primer*, FHWA

Systems Engineering

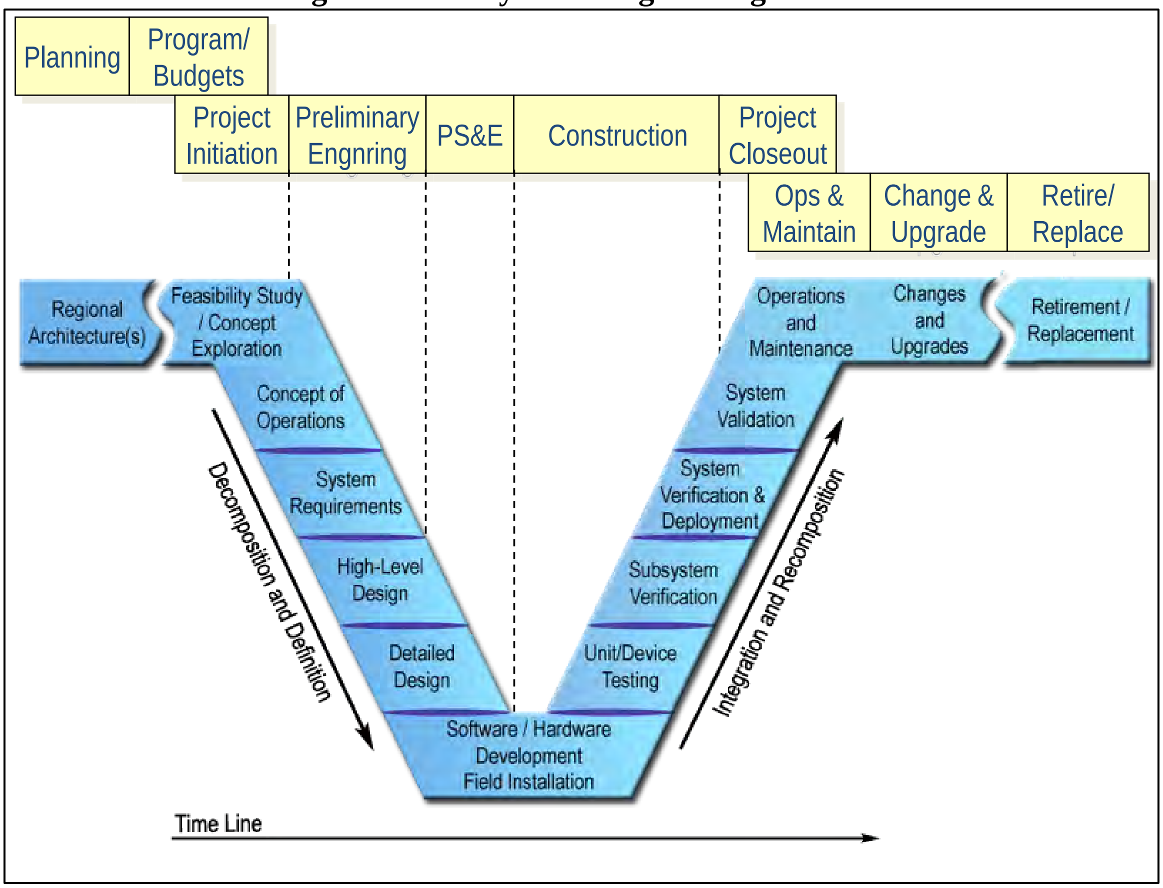
Systems engineering is an organized approach intended to improve the success rate of systems projects by clearly communicating user needs and providing a logical organization that ties all aspects of the implementation cycle to those user needs and requirements. The approach can be applied to any of the TSMO strategies related to systems that are described in this manual. Systems engineering analysis is required for all ITS projects using federal funds per the Final Rule described in the ITS Architecture section above. The Final Rule requires a systems engineering analysis that includes, at a minimum:

- Identification of portions of the regional ITS architecture being implemented (or if a regional ITS architecture does not exist, the applicable portions of the National ITS Architecture)
- Identification of participating agencies' roles and responsibilities
- Requirements definitions
- Analysis of alternative system configurations and technology options to meet requirements
- Procurement options

- Identification of applicable ITS standards and testing procedures
- Procedures and resources necessary for operations and management of the system

Although there are many ways to represent the systems engineering process, the winged "V" (or "Vee") model diagram shown in Exhibit 18-10 has been broadly adopted in the transportation industry. The left wing of the "V process shows the regional ITS architecture, feasibility studies, and concept exploration that support initial identification and planning for a project. Transportation planning fits within the left wing. The operations objectives and performance measures identified during the planning phases should be applied throughout the systems engineering process and actually validated once the project reaches the deployment phase in the right wing of the "V" diagram. This approach provides a systematic method to plan and design systems to achieve the desired operations objectives.

Exhibit 18-10: Planning within the Systems Engineering "V" Model



Source: *Systems Engineering Guidebook for ITS*, ver. 3.0, FHWA CA Division

ITS Systems Engineering and Architecture Compliance Checklist

The ODOT ITS Unit developed a systems engineering and architecture compliance checklist, which essentially forms a Systems Engineering Plan for an ITS project or a project with ITS/TSMO elements. The checklist can be used to define how a project will comply with federal requirements and identify which systems engineering tasks need to be completed as part of the project scope of work. This checklist is required for all federally funded ITS projects in Oregon but is also useful for ITS projects funded through other sources. The checklist is available at https://www.oregon.gov/odot/Maintenance/Pages/Plans,-Architectures-&-Reports.aspx.

18.3.6 Analysis Tools

This section describes the scoping and selection of TSMO planning-level analysis tools and methods, which include a full range of analytical level of detail from sketch models through simulation. See Chapter 2, Section 2.4 for additional information on planning tools.

There is a wide selection of tools and methods that can be used during the various stages of the TSMO planning process. The analysis tools are generally broken out into these categories:

- Sketch-planning tools
- Deterministic tools
- Traffic signal optimization tools
- Simulation tools

Sometimes classified as tools and methods, these items are also used as inputs to many of the before mentioned tools:

- Travel demand models
- Crash modification factors
- Archived operations data

These tools, methods, and inputs range from easy to use to very complex. Exhibit 18-11 lists basic advantages and challenges of each tool or method.

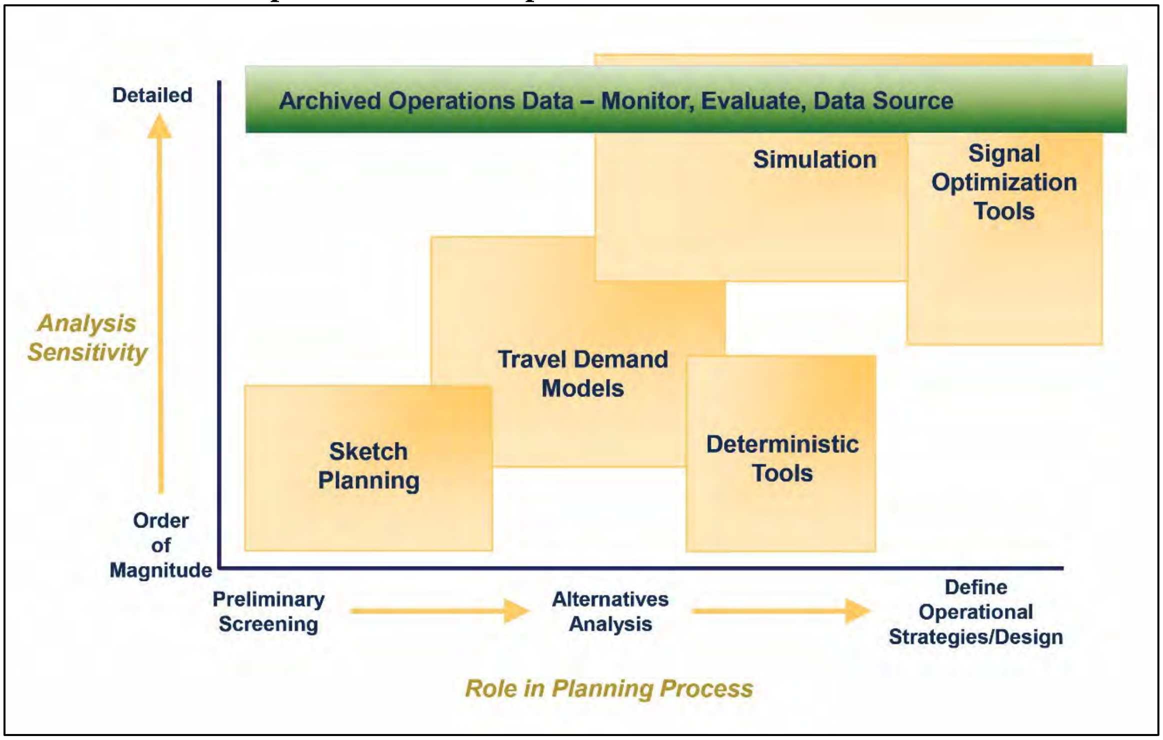
Exhibit 18-11: Advantages and Challenges of TSMO Analysis Tools/Methods

Tool/Method	Advantages	Challenges
Primary Tools and Methods		
Sketch-planning tools *Used to produce general order of magnitude estimates of impacts*	• Low cost • Fast analysis times • Limited data requirements • View of the "big picture" • Efficient initial screening	• Limited in scope, robustness, and presentation capabilities
Deterministic tools *Used to predict impacts based on inputs*	• Quickly predict impacts for an isolated area • Widely accepted	• Limited ability to analyze broader network impacts • Limited performance measures
Traffic signal optimization tools *Used to develop optimal signal phasing and timing plans*	• Effective tool for testing plans prior to field implementation • Proven operational benefits	• Calibration process can be time consuming
Simulation tools *Used to observe the effects of inputs in a modeled environment*	• Detailed results, particularly microsimulation • Dynamic analysis of incidents and real-time diversion patterns • Visual presentation opportunities	• Demanding data and computing requirements, particularly microsimulation • Research requirements may limit network size and number of analysis scenarios • Requires substantial experience working with models
Supplementary Tools and Methods		
Travel demand models	• Validated models available for most metro areas • Evaluation of the regional impacts • Consistent with current planning practices	• Limited ability to analyze operational strategies • Typically does not capture nonrecurring delay
Crash modification factors	• Low cost • Fast analysis times • Limited data requirements	• Quality of factors can be low • Factors not present for many countermeasures
Archived operations data	• Quick data collection • Current/up-to-date data • Provides detailed response to public officials based on real-world data	• Limited availability of quality data • Requires access to data/privacy concerns

Adapted from Table 2 in *Applying Analysis Tools in Planning for Operations,* FHWA

The detail and complexity of a tool can vary between quick and easy (sketch-planning) to time intensive and complex (simulation). Tools also have different uses depending on the stage of the planning process. Preliminary screening would typically involve sketch-planning tools while simulation would start during the alternatives analysis process and potentially continue through design. Exhibit 18-12 shows the comparison of tool capabilities as they relate to the TSMO planning process and their respective analysis sensitivity.

Exhibit 18-12: Comparison of Tool Capabilities



Source: *Applying Analysis Tools in Planning for Operations Participant Workbook*, FHWA Workshop Series

Throughout the planning process, archived operations data and crash modification factors/countermeasures may be used as inputs for a variety of tools. For example, historical vehicle volumes may be used to calibrate travel demand models or simulations while a crash modification factor might be used in a benefit/cost analysis.

There are many tools and methods that can be used to evaluate TSMO strategies throughout the planning process. This section describes some of these tools in more detail and Exhibit 18-13 provides an overview of what category applies to each tool or method and the level of effort to apply it regarding data needs, staffing, and time. See Section 2.4.2 for detailed descriptions of the High (H), Medium (M), and Low (L) rankings.

Some of the tools previously developed by FHWA for operational analysis have been phased out for newer models. Screening Analysis for ITS (SCRITS), Surface Transportation Efficiency Analysis Model (STEAM), and IMPACTS are no longer available online.

Exhibit 18-13: TSMO Analysis Tools/Methods

Tools/Methods	Sketch-Planning	Deterministic	Traffic Signal Optimization	Simulation	Input to Other Tools	Data Needs	Staffing	Time
	Category					Applications		
Tool for Operations Benefit Cost Analysis (TOPS-BC)	X					L	M	L
SHRP2 Analysis Tools	X					L	M	L
ITS Benefits/Cost Database and Before-After Studies	X				X	L	L	L
Mesoscopic/Dynamic Traffic Assignment				X		M	M	M
Travel Demand Model		X			X	M	M	M
Crash Modification Factors (CMFs)/Countermeasures					X	L	L	L
Regional Strategic Planning Model (RSPM)					X	H	H	H
Traffic Signal Optimization			X			M	M	M
Microscopic Simulation				X		H	H	H

Tool for Operations Benefit Cost Analysis (TOPS-BC)
(https://ops.fhwa.dot.gov/plan4ops/topsbctool/index.htm)
TOPS-BC is a sketch-planning tool that was developed in parallel with the FHWA Operations Benefit/Cost Analysis Desk Reference. This spreadsheet-based tool has four key capabilities:

1. Allows users to look up the expected range of TSMO strategy impacts based on a database of observed impacts in other areas
2. Provides guidance and a selection tool for users to identify appropriate benefit/cost methods and tools based on the input needs of their analysis
3. Provides the ability to estimate life-cycle costs of a wide range of TSMO strategies
4. Allows for the estimation of benefits using a spreadsheet-based sketch-planning approach and the comparison with estimated strategy costs.

In addition to the four key capabilities, local weather station data can be used in evaluation of non-recurring events and to calibrate to local conditions. TOPS-BC can also be used to evaluate emissions for Congestion Mitigation and Air Quality (CMAQ) funds assuming that there is a local emissions model/tool that can also be used. The most common tool used for emissions modeling is MOVES, which was developed by the US

EPA.

SHRP2 Analysis Tools

(https://www.fhwa.dot.gov/goshrp2/Solutions/Operations/List)
The SHRP2 analysis tools are a repository of different solutions designed to meet various challenges. These tools are the result of over 100 research projects that address various transportation challenges. The SHRP2 solutions are broken out into the following disciplines:

- Planning and Environment
- Safety
- Design
- Risk Management
- System Management and Operations
- Bridges
- Utilities and Railroads
- Pavement and Materials
- Construction

Under the discipline of system management and operations, the focus areas include capacity, reliability, and renewal. Some examples of solutions include advanced travel analysis tools for integrated travel demand modeling, freight demand modeling and data improvement, validation of urban freeway models, and incorporating travel-time reliability into the highway capacity manual.

Incorporating travel-time reliability into the highway capacity manual is one tool that is part of a suite of tools to help transportation planners and engineers improve monitoring and analysis of data to achieve more consistent, predictable highway travel. In order to implement the methodology of the freeway reliability analysis, a Java-based tool with a graphical user interface was created call FREEVAL-RL (FREeway EVALuation ReLiability).

The FREEVAL-RL tool analyzes freeway segments for both undersaturated and oversaturated conditions. For oversaturated conditions, demand, volume, and vehicle queuing are tracked over time and space. Using outputs from the FREEVAL-RL tool, reliability can be estimated over various scenarios.

Other Sketch-Planning Tools

Additional sketch-planning tools include:

- ITS Benefits/Cost Database (https://www.itskrs.its.dot.gov/costs)
    - This website-based tool developed by the US DOT provides a database of previous studies for numerous TSMO strategies. The user must sort through data to find similar benefits or costs to apply, which can be time consuming.

- Results from other before and after studies
    - Results from other studies with similar TSMO strategies or project elements can be useful in a quick evaluation of potential benefits and costs.

These methods are limited to available data and studies and may be more time intensive to sort through. However, these methods may be more cost-effective for smaller projects or for projects evaluating emerging TSMO strategies or TSMO strategies not available for analysis in other tools.

Mesoscopic/Dynamic Traffic Assignment

Mesoscopic/dynamic traffic assignment falls under the category of a simulation tool. This tool is one of the more complex tools and requires a higher cost and more staff training to use; however, it provides a higher level of resolution than the other tools discussed. Per Chapter 8 (Mesoscopic Analysis), a mesoscopic model is defined as:

> *A hybrid model that includes combinations or approximations of elements from both macroscopic and microscopic models. Mesoscopic models may include a routable network similar to a macroscopic model (with a supplementary origin-destination matrix), while also incorporating more detailed operational elements of the transportation network to better estimate travel time based on traffic operations similar to a microscopic model. Elements from either the macroscopic or microscopic models may be generalized or simplified.*

One piece of mesoscopic modeling is Dynamic Traffic Assignment (DTA). DTA considers traffic routes and travel times that can vary by time of day whereas the more traditional travel demand models use a static assignment of routes during a time interval. The advantage of DTA models is they come close to achieving the same detail that microsimulation models achieve while maintaining elements of macro-simulation models. DTA is a further level of effort and refinement of model assignment between the typical travel demand model and micro-simulation. Common DTA software tools used are DynusT and Dynameq.

More details on DTA including general concepts, scoping, and tool selection can be found in Chapter 8, Section 8.5.

Travel Demand Model

The Travel Demand Model is typically a system-level tool that is generally limited to the facility level. Details include basic characteristics such as number of lanes and speed limits. Travel demand models are used to compare scenarios or help screen alternatives by using origin and destination patterns, demand to capacity ratios, differences in percent volumes, and rough estimates of travel times. They are also used as input to other tools when design hour volumes are needed.

While travel demand models typically do not include the level of detail (e.g., intersection performance) or specific components/modules (e.g., intelligent transportation systems) to directly assess TSMO strategies, there may be methods for manipulating the travel demand model tool in an indirect manner that serves as a proxy for TSMO strategies. For example, the analyst could consider increasing the capacity of a corridor in order to emulate the potential corridor benefits related to improved access management or signal control system enhancements.

Additional information regarding TDM can be found in Chapter 17 (not yet written).

Crash Modification Factors (CMFs)/Countermeasures

Crash Modification Factors and countermeasures can be used as a simple tool on its own but more often than not, is used as an input to other more detailed tools. A CMF is a multiplicative factor, that when used in conjunction with a countermeasure, provides the long-term expected number of crashes. In a benefit/cost analysis that incorporates safety, there are two sources that can be used to estimate the impact of a particular countermeasure: the CMF Clearinghouse and the All Roads Transportation Safety (ARTS) program. If available, the list through the ARTS program should be used prior to other sources because ARTS was developed for Oregon through collaboration with ODOT, the League of Oregon Cities (LOC), and the Association of Oregon Counties (AOC). Funding for crash countermeasures is also available through the ARTS program.

Chapter 4 (Safety) provides additional information regarding crash analysis. In particular, Section 4.4 provides detail on predictive crash analysis with a thorough explanation of crash modification factors in Section 4.4.2.

Regional Strategic Planning Model (RSPM)

The Regional Strategic Planning Model (RSPM, formally known as GreenSTEP) is used for strategic planning preceding an RTP or TSP goal-setting process. As such, it can be helpful in setting a vision, as it quantifies the regional benefit of several TSMO strategies, against competing non-TSMO policy actions/investments. RSPM was initially developed for state and regional greenhouse gas emissions reduction planning and helped establish MPO emissions targets. It has one of the most detailed understandings of future vehicle types and fuels, per collaboration with sister state agencies Department of Environmental Quality (DEQ) and Department of Energy (DOE). RSPM has no network to shorten run-times, but uses demand to capacity relationships and congestion approach by functional class to evaluate the impact of various freeway and arterial ITS (ramp metering, incident response, signalization, and access management) and other speed smoothing policies. Other policies addressed include Transportation Options programs (e.g., workplace TDM and individualized marketing programs). Recent applications have used a broader set of transportation and land use outcomes such as household travel costs, walk and bike travel, and delays. Estimations are done at the household level for a metropolitan planning area or larger. RSPM is being implemented in Oregon MPOs and typically reports at a region or jurisdiction level.

Chapter 7 includes more information about scenario planning assessment and RSPM.

Traffic Signal Optimization Tools

Traffic signal optimization tools are primarily used to develop optimal signal timings for corridors, networks, and individual intersections. These tools can incorporate items such as intersection capacity, cycle lengths, splits, signal coordination and offset. The most common traffic signal optimization tool used in Oregon is Synchro.

Microscopic Simulation

Microscopic Simulation tools can simulate the individual movement of vehicles and pedestrians. These tools are typically complex and require the highest cost and staff training; however, they provide a high level of confidence. An example of a microscopic simulation tool is Vissim.

Vissim is a tool that involves microscopic traffic simulation for multimodal traffic and can handle congested conditions under different traffic scenarios. It is capable of handling dynamic route choice and realistic differences in driver behavior. Vissim can analyze capacity, traffic control, signal systems and re-timing, and public transit. Vissim can handle adaptive traffic control (such as SCATS). It has the capabilities to handle multimodal networks, including pedestrians, bicyclists, and transit. It can be linked to various emissions models as well.

18.3.7 Incorporating TSMO in Planning Statements of Work

ODOT conducts many different types of planning activities: regional and local transportation system plans, corridor/facility plans, and topical or area specific refinement plans. Exhibit 18-14 lists the typical tasks performed under a planning work scope and identifies considerations for incorporating TSMO.

Exhibit 18-14: TSMO Considerations in Planning Work Scopes

Planning Tasks	TSMO Considerations
Stakeholder/Public Involvement	Broaden stakeholders to include internal and external operations staff on the project advisory committee. If a standing operations committee exists in the area, notify them of the planning process and seek their input at key points in the planning process.
Vision, Goals, and Objectives	TSMO is an element supporting the broader planning vision. Include a TSMO-related goal and supporting SMART objectives and performance measures.
Evaluation Criteria	Operational performance measures provide useful information on how well the existing transportation system is functioning. Carry forward performance measures for system efficiency, system reliability, and system options into the evaluation criteria.
Existing Plans, Policies, Regulations, and Standards	A number of regions along with the state of Oregon have completed ITS and/or TSMO plans and architectures. The OTP and OHP include TSMO policies. Incorporate these references when documenting supporting plans and policies.
Current Conditions, Deficiencies, and Transportation Needs	Capture ITS infrastructure and TSMO program activities such as TIM and TDM as part of the transportation system inventory. If available, existing ITS/TSMO plans are a resource for this information.
Solution Alternatives	The OHP 1G1 Policy requires ODOT to maintain highway performance and improve system efficiency and management before adding capacity. Include TSMO strategies as part of the solution alternatives. See Appendix18A for list of TSMO strategies.

18.4 Corridor Management

This section provides considerations and analytical procedures for incorporating TSMO into planning for corridor management. A corridor is typically considered a linear system of multimodal facilities where an existing roadway or transit facility serves as the spine. Some corridors may also include a broader travel shed with parallel facilities or connections to major activity centers or logical destinations. Corridor limits can range from a few miles long in urban areas to tens or hundreds of miles long for state or multi-state corridors. Corridor planning typically includes a combination of modes and the modal focus can vary by corridor (e.g. freight route, high-capacity transit route, limited access highway).

TSMO strategies used for corridor management are grouped into four categories in this document:

- **Arterial Management** is the use of strategies on arterial-type facilities to improve the efficiency, safety, and capacity of the facility, without increasing its size.[21]Arterial management generally applies to roadways that have signalized intersections and carry a combination of vehicles (autos, transit, and freight), pedestrians, and bicyclists.
- **Freeway Management** is the management of freeway-type facilities for safety and mobility improvement. Mobility improvement addresses congestion (recurrent and non-recurrent) in response to prevailing and predicted traffic conditions. Freeway management generally applies to roadways that have uninterrupted flow and only carry vehicles (autos, transit, and freight).
- **Demand Management** is the dynamic management of travel demand to influence traveler behavior in real-time to achieve operational objectives.
- **Integrated Corridor Management (ICM)** is an approach to managing the transportation network that encourages multi-agency coordination and combines arterial and freeway strategies to balance and manage travel demand across networks (freeway, arterial, transit, and parking). ICM plays an important role during events such as incidents, planned special events, inclement weather, and through work zones.

The first three groups are all elements of Active Transportation and Demand Management (ATDM). FHWA defines ATDM as "the dynamic management, control, and influence of travel demand, traffic demand, and traffic flow on transportation facilities."[22] FHWA's definition of ATDM includes three components: 1) Active Traffic Management, 2) Active Demand Management, and 3) Active Parking Management. In this document the active traffic management element is divided into arterial and freeway management categories, and the demand management category encompasses both the active demand management and active parking management components.

Appendix 18A includes a more detailed list of TSMO strategies, benefits, cost magnitude, and implementation factors.

Exhibit 18-15 provides a list of corridor TSMO strategies to consider based on facility type and surrounding area. In some cases, there is a clear distinction between the strategies applied to an arterial versus a freeway. However, there are many strategies that may apply to both types of facilities, such as dynamic warning signs and variable speeds. The following subsections provide an overview of each of the four categories of corridor TSMO management as well as analysis considerations and procedures for evaluation as appropriate.

[21] Oregon Transportation Planning Rule, Oregon Administrative Rule (OAR) 660-012-0005
[22] Definition according to FHWA Office of Operations https://www.its.dot.gov/index.htm

Exhibit 18-15: TSMO Strategies to consider based on Facility Type and Area

Corridor TSMO Strategies	Facility Type*					Area*		
	Interstate	Freeways & Expressways	Ramps	Principal Arterial	Minor Arterial & Major Collector	Urbanized	Small Urban	Rural
Arterial Management (Section 18.4.1)								
Traffic signal enhancements (updates, phasing, advanced)				●	●	●	●	●
Dynamic lane use			●	●	●	●	●	●
Queue jumps (ramp meter or signal)			●	●	●	●		
Transit signal priority			●	●	●	●	❍	
Truck signal priority			●	●	●	●	❍	●
Bicycle signals				●	●	●	●	●
Automated enforcement			●	●	●	●	●	●
Added capacity for critical movements (Section 10.3.2)			●	●	●	●	●	●
Improved traffic control schemes (Section 10.3.2)			●	●	●	●	●	●
Freeway Management (Section 18.4.2)								
Variable speed control	●	●		●		●	●	●
Automated speed enforcement	●	●	●	●	●	●	●	●
Dynamic lane use control	●	●		●		●	❍	
Dynamic lane reversal	●	●		●		●	❍	❍
Managed lanes (HOT/HOV)	●	●	❍	❍		❍	❍	
Adaptive ramp metering			●			❍		
Dynamic merge control	●	●				●	●	●
Dynamic warning	●	●	●	●	●	●	●	●
Hard shoulder running	●	●				●	●	❍
Dynamic re-routing	●	●	●	●	●	●	❍	●
Dynamic truck restrictions	●	●	●	●	●	●	●	●
Demand Management Strategies (Section 18.4.3)								
Corridor monitoring	●	●	●	●	●	●	●	●
Corridor specific traveler information	●	●	●	●	●	●	●	●
Active transit management	●	●	●	●	●	●	❍	❍
Active parking management						●	❍	
Congestion pricing	●	●		●		●		
Gamification	●	●	●	●	●	●	●	●
Shared-use mobility	●	●	●	●	●	●	❍	❍
Employer based transportation demand management	●	●	●	●	●	●	❍	❍
Neighborhood based transportation demand management	●	●	●	●	●	●	❍	❍
Integrated Corridor Management (Section 18.4.4)								
Corridor management strategies above	●	●	●	●	●	●	●	●
System Management (Section 18.5)	●	●	●	●	●	●	●	●
● Strategy applies to facility								

Corridor TSMO Strategies	Facility Type*					Area*		
	Interstate	Freeways & Expressways	Ramps	Principal Arterial	Minor Arterial & Major Collector	Urbanized	Small Urban	Rural
❍ Strategy MAY apply to facility								

* For facility type and area by highway and milepost, see: https://www.oregon.gov/ODOT/Data/Documents/FC_NHS_State_Highway_List.pdf

18.4.1 Arterial Management

The arterial management strategies are part of a broader category of Active Traffic Management (ATM) strategies. Active Traffic Management (ATM) is "the ability to dynamically manage recurrent and non-recurrent congestion based on prevailing and predicted traffic conditions."[23] Additional information is available on FHWA's Active Traffic Management website.[24]

This document divides active traffic management into two categories: arterial management and freeway management. Arterial management focuses on facilities that have traffic signals, interrupted flow, and carry a combination of vehicles, pedestrians and bicyclists. On the other hand, freeway management focuses on facilities that mainly carry vehicles (a combination of autos, transit, and freight) and have limited access points. As shown in Exhibit 18-15, some of the freeway strategies can also apply to arterials, yet arterial strategies generally do not apply to freeways.

The arterial management strategies discussed in this section aim to improve the efficiency, safety, capacity of an arterial-type transportation facility without increasing its size.

This section focuses on seven types of arterial strategies:

- Traffic signal enhancements
- Dynamic lane use
- Queue jumps
- Transit signal priority
- Truck priority
- Bicycle signals
- Automated enforcement

Traffic Signal Enhancements

These strategies can be implemented by making improvements to traffic signal timing. This may include updating the base signal timings to accommodate multi-modal users, modifying the phasing to improve efficiency, or implementing advanced signal control

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[23] FHWA Office of Operations, https://ops.fhwa.dot.gov/atdm/approaches/atm.htm
[24] FHWA website: https://ops.fhwa.dot.gov/atdm/approaches/atm.htm

strategies such as adaptive signal control. The strategies also include upgrading the existing signal system infrastructure to facilitate implementation of the operational improvements.

- Base Signal Timing – Reviewing and updating the basic signal timings, such as minimum and maximum green, walk, flashing don't walk, yellow and red clearance intervals and vehicle detector settings can improve the safety and efficiency of the intersection operation. It is not uncommon for the basic signal timings to remain unchanged from the first day a signal is turned on. It is good practice to review and update the base timings on a routine basis as standards for clearance intervals and detector settings may change.

- Traffic Signal Phasing – Modifying the traffic signal phasing, such as adding a protected left turn phase, converting a protected left turn phase to protected/permissive left turn phase or installing a right turn overlap can improve the safety and efficiency of the intersection operation. Depending on the traffic volumes, changes to the phase sequence (lagging a left turn phase) may improve the efficiency of an intersection. Changes to the traffic signal phasing may require minor equipment changes to the traffic signal indications.

- Advanced Features – Implementing advanced features of the traffic signal software, including phase re-service, dynamic phase length, or ped-friendly flashing yellow arrow (FYA) can improve the safety and efficiency of an intersection. Some of the signal timing parameters may work better in free (not coordinated) operation and others may work better in coordinated operation. The advanced features should be explored if there is a unique operating condition, and not implemented everywhere, since some of the features may work against others.

- Coordinated Timings – Coordinated signal timings require a group of signals to operate on a common cycle length. Coordinated signal timings are best used on corridors where the intersections are regularly, closely spaced (within one half mile), or where platoons of vehicles do not disperse too much from one signal to the next and where there is a predominate movement that needs to be progressed between multiple intersections. A well-timed corridor (or group of signals) can serve traffic efficiently using Time-of-Day coordination without the need for advanced signal timing. To get the most effective operation, the timings must meet the operational goals of the corridor (favor main street flow, balance delays on all approaches, promote transit, etc.). Typically, signals operate coordinated timing plans based on the time of day for set time periods (am peak, midday and pm peak), without much attention paid to the weekend timing plans. A simple way to improve the operation of a corridor is to evaluate the weekend operations of the signals and implement weekend timings if needed. A common rule of thumb is for agencies to update the coordinated timings every three to five years, however using performance measurement data as a trigger allows agencies to use their resources more efficiently. Doing this will require the agency to monitor operations on a more on-going basis to see trends in the performance measures that need to be addressed.

- Event/Incident Timings – Event and incident timings are special timing plans that are developed to serve traffic associated with an event (sporting event/concert or adverse weather) or an incident (crash, either on the arterial being controlled or on parallel facility). The timings are developed based on assumptions from previous events and can include operating special coordinated timing plans or operating one or more intersections in free mode. Typically, the event/incident timings are manually turned on and off using a command from the central signal system, but they can also be enabled using a trigger, like volumes from a system detector or weather sensor.

- Traffic Responsive Signal Timing – Traffic Responsive (TR) signal timing operates similar to traditional coordinated timings, but instead of using the time of day, it uses volume and occupancy as the trigger to change the previously developed timing plans. For example, under traditional time-of-day operation, the p.m. peak plan starts at 4:00 p.m., but under traffic responsive, it may start at 3:30 p.m. (ex. school or work shift ends) or not at all (ex. holiday). It is best used on corridors with predictable and distinct traffic patterns (ex. heavy inbound in the a.m. and heavy outbound in the p.m.) where adjusting the start times of the existing plans is needed. Traffic responsive plan selection normally occurs in a field master or a central signal system. It uses the detection data to calculate the most appropriate timing plan and then sends a command to the local signal controllers to tell them which plan to operate. TR requires additional system detectors to be installed along the corridor and can be time consuming and complex to set up and monitor.

- Traffic Adaptive Signal Control – Traffic Adaptive signal control uses special algorithms to optimize the signal timings along a corridor. Typical adaptive systems adjust the cycle, split, and offsets every cycle to match the fluctuation in traffic. Adaptive systems show benefit at locations where the traffic conditions are unpredictable and/or volumes are high, as adaptive systems can delay the onset of oversaturation and can recover more quickly from a saturated condition. The adaptive algorithms are typically calculated in a central system and the cycle, splits and offsets for each intersection are commanded from the central system. However, some systems provide the processing power at a field "master" location along the corridor. Adaptive systems require robust vehicle detection and reliable communications between the field controllers and the master/central system. Each corridor/group of signals should be evaluated before deploying an adaptive signal system to ensure it is the best operational/management choice.

- Traffic Signal Controllers – In the state of Oregon, agencies operate two types of traffic signal controllers (with various firmware), including the Model 170 and the Model 2070. In recent years, agencies have been replacing the Model 170 controllers with Model 2070 controllers, as they provide additional operational features, extra computing power and more data storage. ODOT has recently decided to install ATC controllers at some intersections and is expected to make the ATC controller a standard in the near future. These new controllers provide even more computing power, collect more performance data, and are compatible with future connected vehicle technology. Upgrading the traffic signal controllers to the current standard 2070 or to the ATC will improve the intersection operation because of the additional operating features and computing power available.

- Central Signal System – The central signal system manages the local traffic signal timing databases, provides remote access to the local controllers, and collects/disseminates performance measurement data. A central system allows an agency to monitor the operations of their traffic signals and adjust if necessary, such as during an incident. Multiple agencies within a region can benefit by sharing a single central system.

- Communications – Communications between traffic signals provides the ability for the signals to talk to each other and is most beneficial for keeping the time clocks synchronized. Communications between the traffic signals and a central system allows an agency to remotely manage and monitor traffic signal operations. Reliable communications is very important when operating traffic responsive or adaptive operations, as the computing is done by the central system or by a field master with the commands being sent in real-time from the central or master location to the local controller.

- Detection – A traffic signal system needs robust detection in order to operate in the most efficient manner. Detection is necessary for all signal operations, except fixed-time control, yet it is the one thing that is often not maintained as well as it should be. Traditional vehicle detection uses inductive loops cut into the pavement. They are a very reliable form of detection, but they are susceptible to damage by poor pavement conditions or accidentally being cut. Once they are installed, there is no flexibility to move them to a different location and it is sometimes challenging to detect motorcycles and bicycles with them. Video detection is a non-intrusive form of detection as it uses cameras mounted above the pavement. Video detection can detect vehicles and bicycles and allow agencies to adjust the zones if needed. This form of detection is susceptible to false calls due to fog and shadows and can sometimes miss vehicles because of occlusion. It also has a limited range of how far it can see in advance of the intersection. Video detection requires more frequent maintenance than loop detection. Radar detection is a non-intrusive form of detection as it uses a radar detector mounted above the pavement. Radar detection can detect vehicles and bicycles and allow agencies to adjust the zones if needed. This form of detection is not susceptible to false calls due to fog and shadows but requires different units for advance and stop bar detection. The choice of type of detection at an intersection should be evaluated considering the physical condition of the site and the operational objectives, with the most appropriate type installed.

- Closed Circuit Television (CCTV) Cameras – CCTV cameras, in conjunction with a central traffic signal system, help an agency assess operational issues quickly from a remote location. The central signal system tells the agency what the traffic signals are doing and a CCTV camera allows the agency to see what may be causing a problem (crash, power outage, etc.). CCTV cameras help the agency to monitor signal operations if timings are changed. For example, if additional green time is added to a left turn phase, the agency can watch how the other phases operate with less green time. CCTV cameras should be installed at most major intersections to allow the agency to view traffic operations along the major corridors. CCTV cameras are also used for identifying incidents and dispatching responders.

Dynamic Lane Use

Modifying intersection approach lane uses may be done with dynamic lane use signing. For example, an intersection may have two approach lanes (left/through and through/right), but during the p.m. peak, the left turn movement is restricted. The restriction can be conveyed to the driver by using a blank-out sign that uses the time of day or a volume threshold to turn on/off. The appropriate lane assignments should be based on a traffic analysis to ensure that the intersection works well throughout the various peak periods. This type of operation requires additional signing to be installed on the signal mast arm and in advance of the intersection along with special logic programmed in the controller.

Queue Jumps

A queue jump is a special phase for an intersection approach that is only used under certain conditions. One common application of a queue jump is for transit, where, if a bus is detected at a certain location, the traffic signal will provide a special indication for the bus so it can proceed through the intersection ahead of the adjacent through traffic. A queue jump can also be used for bicycle traffic. It is an effective solution if there is an existing source of delay at the intersection. A queue jump requires that the bus has a dedicated lane and specific detection. It can work during Free and Coordinated operation; however queue jumps may not be beneficial where there are high volumes of right turning vehicles. A queue jump may also be employed at a ramp meter where transit or other high occupancy vehicles are provided with a special lane to avoid the queue on the ramp. This may require additional right-of-way to implement.

Transit Signal Priority

Transit signal priority (TSP) provides additional green time to the approaches at an intersection that serve buses. TSP can be implemented on any corridor that has transit service, with more benefit going to corridors with higher numbers of buses and corridors that are not over capacity. Most TSP systems require that the bus be equipped with an emitter and the controller cabinet be equipped with a detector unit. This can be done using infrared, wireless communication (such as Wi-Fi) and GPS, or connected vehicle technology. Once a bus is detected, the traffic signal will determine if and how much extra time the bus approach can be given. The time can either be a green extension (hold bus phase green longer) or an early green (bringing up the bus phase earlier) depending on which phase of the cycle the traffic signal is when the priority request is detected. Typically, TSP provides an additional 2-10 seconds for a bus approach, which is taken from other approaches at the intersection.

Truck Priority

Truck priority is like transit signal priority in that it provides additional green time to an approach where a heavy vehicle is detected. The additional green time is only provided as green extension, since the goal of truck priority is to minimize the stops (and subsequent start-ups) for heavy vehicles. Truck priority works well on corridors with a high percentage of heavy vehicles, speeds above 35 mph, on roads with no major curves, and where the traffic signals operate in free mode. The traffic signal needs to receive an input that indicates the presence and speed of a heavy vehicle. This can be accomplished via inductive loop detectors in the pavement or connected vehicle technology on the truck. Based on the data, the traffic signal will provide additional green time when a heavy vehicle is detected, and it does not have enough time to enter the intersection before it turns yellow. Typically, truck priority provides an additional 5-10 seconds to the truck approach.

Bicycle Signals

Most traffic signals can provide priority to multi-modal traffic including bicycles, buses, and heavy vehicles. For example, bicycles can be detected using specific detectors and the bicycle rider can be shown a special bicycle phase, separate from cars.

Automated Enforcement

Automated red-light enforcement and automated speed enforcement are two somewhat controversial techniques to improve safety. Both have been shown to be effective in improving compliance with red lights or speed limits. In the correct location, where there is a history of crashes related to red-light running or speed, the technologies can be effective at reducing the crashes related to poor driver behavior. The use of these devices should be carefully weighed and used only when the results would yield a reduction in crashes. These technologies require statutory authority to implement and participation from law enforcement.

18.4.2 Freeway Management

With a focus on trip reliability, freeway management works to maximize the efficiency and effectiveness of facilities by using automation to quickly respond to conditions to influence travel behavior with respect to lane and facility choices. This section describes freeway management strategies and how they may be applied to facilities in Oregon.

Variable Speed Control

Variable speed control (aka variable speed limits, variable advisory speeds, speed harmonization, or dynamic speed limits) is a management strategy to actively manage the posted or advisory speeds at a location or throughout a corridor. On multilane facilities, speeds can also be managed for individual lanes. The speeds are adjusted in real-time to respond to roadway traffic and weather conditions and can be regulatory speed limits or advisory speeds. Variable speed control can be used virtually anywhere including rural or urban settings and can be applied to different modes of travel (autos, bikes, buses, trucks).

The common goals and objectives of agencies considering the development and deployment of variable speed control is improved safety, increased mobility, and reduced environmental impacts. Speed harmonization is a term that is used synonymously with variable speeds in urban areas with recurring congestion. Speed harmonization refers to bringing everyone's speed into "harmony" or reducing the variability between the high and low speeds. Reducing this variability improves safety by reducing starting and stopping congested traffic and the potential number of rear end collisions. By improving safety and harmonizing speed, mobility in a corridor can be increased through higher average speeds and increased capacity. With less congestion and less starting and stopping, environmental impacts can also be reduced.

Variable speed control is typically applied to interstates, freeways, expressways, or principal arterial highways and is most used to address congestion, inclement weather, or construction. Congestion applications should be considered for corridors where there is frequent queuing at bottleneck locations or a history of rear-end crashes. While congestion applications are more commonly applied to urban corridors, they can also be applied to rural corridors to address seasonal congestion. Inclement weather applications should be considered for corridors with a history of crash issues related to inclement weather (e.g. ice, snow, rain, high winds). Work zone applications can be considered for any work zone, particularly for larger projects with a longer duration where work is being done in proximity of the travel lanes. For ODOT facilities, coordinate with the ITS Unit, who developed an *Oregon Statewide Variable Speed System Concept of Operations* and can help with planning.

Input from the State Traffic Engineer, law enforcement, local jurisdictions, and elected officials should be considered in the planning process.

Automated Speed Enforcement

This strategy requires statutory authority to use and participation of law enforcement. The strategy may involve fixed mounted cameras or cameras that can be stationed at different places along the roadway. The cameras are often installed at locations where there has been a history of speeding violations and/or crashes associated with speeding. The camera records violators and issues citations that are reviewed by law enforcement officials. While this strategy can be applied on any facility, it has been found particularly effective to support variable speed control and in work zones.

Dynamic Lane Use Control

Lane use control is the dynamic control of lanes typically through opening or closing them as conditions warrant. Lanes will typically be closed due to incidents or work zones with the goal of safely merging traffic into adjoining lanes. By closing lanes due to an incident or work zone, the goal is to reduce rear end and secondary crashes. Dynamic lane control is typically applied to urban interstates, freeways, expressways, or principal arterials.

Dynamic Lane Reversal

This strategy dynamically reverses the travel direction for one or more lanes to better allocate capacity based on traffic demand. This strategy should only be considered where lane reversal can be done safely, such as a lane separated by barrier on a high-speed facility or a clearly marked lane on a low-speed facility. Dynamic lane reversal may also be considered for facilities adjacent to or near event centers where there is a large imbalance in travel directions, such as a large ingress pre-event and a large egress post-event and minimal traffic in the opposing direction.

Managed Lanes (HOT/HOV)

Managed lanes refer to travel lanes dedicated to a particular use, most commonly to high occupancy vehicle (HOV), high occupancy toll (HOT), and electronic toll lanes. HOV lanes require passenger vehicles to have a minimum number of occupants to use a specific lane, while HOT lanes allow single occupant vehicles to pay a toll to drive in a specific lane, while HOVs travel free or at a discount. Electronic toll lanes may require all vehicles using the lane to pay a toll. The pricing of HOT lanes and electronic toll lanes may vary depending on the demand (higher price when demand is higher).

HOV and HOT lanes both improve the air quality by reducing the congestion on a facility. HOT lanes have proven to be more efficient than traditional HOV lanes because more vehicles are able to use the reserved lane, while continuing to encourage drivers to use transit or carpool. HOT lanes produce revenue that can be used to supplement the cost of construction, maintenance and enforcement.

Adaptive Ramp Metering

Adaptive ramp metering is the dynamic turn on, turn off, and adjustment of ramp metering rates that respond to real-time traffic conditions. Ramp metering uses signals at ramps to control the rate at which vehicles are allowed to enter the facility (typically an urban interstate, freeway, or expressway). Adaptive ramp metering considers current roadway conditions and varies the metering rate depending on those conditions. Other features of adaptive ramp metering may include bottleneck identification, incident detection, and integration into neighboring traffic signals.

The objective of adaptive ramp metering is to prevent a high-speed, low-access network or corridor from being inundated with vehicles and causing a heavily congested network. An adaptive ramp metering system can assist in keeping all vehicles moving, by regulating the number and frequency of vehicles entering a facility.

Considerations for adaptive ramp metering include:

- Corridor wide implementation
- Density of detection
- Length of ramps

In order for adaptive ramp metering to be successful, all entrances to a corridor should be regulated by a ramp meter signal. If some entrances are not controlled by ramp meters, users may try to use those entrances instead and the system would have a harder time regulating the density of vehicles on the facility.

Having reliable detection throughout the corridor is necessary for the successful deployment of adaptive ramp metering. At a minimum, detection should be present near each on-ramp with additional detection desired near off-ramps and in sections between ramps. Detection is also needed at the ramp meters themselves and further up the ramp.

The length of a ramp can also dictate how fast or slow a ramp metering rate might be. If a ramp is not sufficiently long and a queue builds up, there is a potential for the queue to spill onto nearby local streets. If a ramp is long and the ramp metering rate is very slow, it can take a vehicle a significant amount of time to enter the facility. Balancing these two items can be a challenge.

Although dynamic ramp metering has been implemented in several corridors in Oregon, it has been controversial with the public. Implementation of this strategy needs to include a public education component to facilitate understanding of its benefits and acceptance.

Dynamic Merge Control

Dynamic merge control is also known as dynamic lane merge or dynamic early merge. Using a dynamic message sign, information on an upcoming merge and expected behavior can be displayed during congestion conditions. Dynamic merge typically closes an upstream lane that neighbors a downstream merge point. This can improve the speed in which the downstream traffic is able to merge while maintaining a higher vehicle throughout.

Typical installations are in places where there is recurrent congestion and significant merging vehicles, particularly for urban interstates, freeways, or expressways. The mainline should have sufficient capacity upstream of the merge point and may be better suited for locations where peak periods of the mainline and merging roadway are at different times.

Dynamic Warning

Dynamic warning signs notify drivers of a specific condition ahead based on real-time traffic conditions. Dynamic queue warning automatically notifies drivers of a queue of stopped or slowed vehicles ahead. Additional information such as distance to the queue and queuing lane(s) is desirable.

The objectives of dynamic queue warning are to reduce rear-end crashes and smooth traffic flow. By notifying drivers of slowed or stopped vehicles ahead, drivers will be less likely to be surprised by the end of a queue. In addition, if a queue is caused by a crash, secondary crashes can also be reduced. Queues are determined by comparing the speed, volume, and occupancy from one detection location to another.

Dynamic queue warnings can be used in spot locations or throughout a corridor. A spot location installation may be applicable where there is recurrent queuing or throughout a corridor where recurrent congestion occurs at multiple locations.

Dynamic warning signs may also warn drivers of adverse pavement conditions based on weather sensors or about wildlife close to the roadway based on animals in the detection zone. The signs would advise drivers of the condition and recommend an action, such as a slower speed. Dynamic warning signs may be applicable on both freeways and arterials.

Hard Shoulder Running

Hard shoulder running, also known as dynamic shoulder lanes, is the use of shoulders as a travel lane or lanes. To be considered an "active" traffic management option, this strategy needs to be implemented, when conditions warrant, with the use of real-time information. Hard shoulder running can be set up for all travelers or be for special purpose only (e.g. transit only). The goal and objective of hard shoulder running is to increase capacity, improve speeds, and improve volume of a segment when conditions warrant.

If the hard shoulder is on the right side, corridors with a higher number of ingresses and egresses may not be a good candidate for hard shoulder running unless ramp tapers can provide smooth merging operations. Hard shoulder running also requires a sufficiently wide shoulder to adequately function as a lane of traffic.

Dynamic hard shoulder running is often implemented with the use of variable speed and lane use control. The lane-use control signs over the shoulder provide frequent real-time reminders of the status of the hard shoulder (open or closed). The variable speed displays can reinforce any reduced speeds needed in the corridor, particularly if the shoulder lane requires a lower speed. Like with lane use control, a typical distance between over lane signs is generally in the half-mile range in the U.S.

Currently, an interpretation of Oregon statutes has determined that hard shoulder running is not legal and would require enabling legislation to apply it. For more information on use of freeway shoulders see Use of Freeway Shoulders for Travel — Guide for Planning, Evaluating, and Designing Part-Time Shoulder Use as a Traffic Management Strategy, FHWA, February, 2016.

Dynamic Re-Routing

This strategy provides alternate route information to travelers in response to increasing congestion on a corridor at bottlenecks or due to events such as incidents (on the corridor or a parallel facility). This is an emerging strategy that will likely advance further as connected vehicles advance with broadcast traveler information and dynamic mapping features. In the meantime, traveler information systems can be used to provide dynamic re-routing information that travelers can check pre-trip or passengers can check en-route.

Dynamic Truck Restrictions

This strategy limits trucks to certain ramps, lanes, or routes based on traffic demand so that at least one lane of traffic is available exclusively for passenger traffic. The goal of this strategy is to allow passenger traffic to flow more freely without having to brake or slow down for less agile truck traffic. In turn this helps improve safety, support uniform speed, and improve traffic flow. Dynamic truck restrictions should be considered for highways with high truck volumes and major arterials and collectors that serve both freight and passenger traffic. This strategy requires enabling legislation as well as political support and support from the freight community.

18.4.3 Demand Management

Demand Management strategies dynamically manage travel demand to influence traveler behavior in real-time to achieve operational objectives such as preventing or delaying bottleneck conditions, improving safety, promoting sustainable travel modes, reducing emissions, and maximizing system efficiency. Specific demand management strategies and how they may be applied to facilities within Oregon are discussed herein. Additional information on Demand Management strategies can be found on FHWA's websites for Active Demand Management and Active Parking Management.

Corridor Monitoring

Continuous multimodal monitoring of corridors is a fundamental requirement for deploying other multimodal strategies because it provides a current snapshot of transportation conditions and can be used with archived data and predictive methods to maintain or achieve system performance. While monitoring is often done at Traffic Operations Centers for larger agencies like ODOT, it can also be done from single workstations at smaller agencies or at ODOT district or maintenance offices. The most common forms of monitoring include:

- Camera monitoring systems provide a visual means for observing traffic operations and identifying unplanned events. For interstates, freeways, and expressways, cameras are often installed at major interchanges, at regularly spaced intervals in urbanized areas, or at known trouble spots (e.g. winter conditions, geometric curvature). For arterials and collectors, cameras are often installed at traffic signals due to the availability of power, communications, and available mounting locations. When additional coverage is needed in urbanized areas, street light poles or separate camera poles are also used.
- Detection systems (e.g. radar, video, inductive loops, Bluetooth) collect a variety of data such as volume, speed, occupancy (vehicle density), and travel times. Data collected by the detection system(s) is used to produce performance measurement reports. Detection systems should be considered for all facilities. Extensive coverage is typically needed in urbanized areas. Pedestrian and bicycle specific detection should be considered in urbanized and small urban areas.
- Transit Operations Data can be used to inform system operators about transit ridership, service routes, service frequency, current transit vehicle availability, and other data to improve the efficiency of the system. This type of data may be especially important to coordinate integrated corridor management when transit is brought into the equation.

- Third party data (e.g. INRIX, Waze) is also a good source of information for monitoring corridors. This type of data is often captured by tracking mobile phones or from crowd-sourced information.
- Automated system performance or alerts should be used and considered when acquiring new transportation management systems. For example, central signal systems can provide alerts when traffic signals go into flash or a transit management system can provide alerts when a bus route is running behind schedule.
- Predictive algorithms can be developed using archived data to generate corridor travel forecasts based on real-time data.

Corridor Specific Traveler Information

Although providing traveler information is often a system wide effort (see Section 18.5), traveler information can also be tailored to specific corridors. This is done by providing travelers with existing or predictive travel times for parallel corridors through roadside dissemination (e.g. dynamic message signs) or broadcast through other systems (e.g. mobile apps, multimodal trip planners, websites). This can help travelers decide which travel route and mode to take. For example, if both a parallel freeway and arterial are congested, the parallel bus rapid transit or light rail line may be a much faster option.

Active Transit Management

Several strategies are available for actively managing demand for transit services:

- Dynamic fare reduction can be used to reduce the fare for the transit system in a particular corridor as congestion or delay increases to encourage travelers to select transit.
- Dynamic transit capacity assignment involves reorganizing schedules or reassigning assets (e.g. buses) based on real-time and anticipated demand patterns on a particular corridor.
- On-demand transit allows travelers to make real-time requests for transit services with flexible routes and schedules.
- Transfer connection protection works to improve the reliability of transfers between a high-frequency transit service (e.g. light rail) to a low-frequency transit service (e.g. bus). For example, a bus departure at a transfer point may be delayed allowing passengers to make the connection from a light rail train that is running late.

Active Parking Management

Active parking management involves actively optimizing performance and utilizing parking facilities to affect travel by influencing trip timing choices, mode choice, and parking facility choice at trip ends. Active parking management strategies include:

- Dynamic overflow transit parking uses overflow parking facilities (e.g. large retail parking lots) near transit stations or park-and-ride facilities when existing parking facilities are at or near capacity. Agreements are often needed with other entities for this strategy. This strategy helps encourage transit use on corridors over automobiles.

- Dynamic parking reservation allows travelers to reserve a parking space at their destination facility on demand to ensure availability. In a corridor context, the availability of parking at a traveler's final destination may influence them to travel by automobile (alone or with other occupants) or the availability of parking at a popular transit park-and-ride near the beginning or mid-point of their trip may encourage them to take transit along the corridor instead.
- Dynamic wayfinding provides real-time parking information about space availability and location to optimize the use of parking facilities and minimize the time spent searching for parking. This strategy helps travelers choose which corridor to use.
- Dynamically priced parking uses variable parking fees based on demand and availability to influence trip timing choice and parking facility choice in an effort to balance parking supply and demand. This strategy has an impact on mode choice, which corridors travelers select, and when they travel.

These strategies are typically applied in urbanized or small urban areas but can also be used in rural areas where there is high parking demand at special event facilities such as fairgrounds or amphitheaters.

Congestion Pricing

Although not currently used in Oregon, congestion pricing is a strategy that can be used to manage demand on congested facilities by time of day or day of week. For corridors, congestion pricing is typically applied to the entire corridor, a segment of the corridor, or by certain lane(s) within the corridor. The use of variable pricing by lane, segment, time of day, or day of week influences travel demand by encouraging travelers to consider alternate corridors or travel modes during peak demand periods. Pricing can be pre-set by time of day or can be updated dynamically based on current travel conditions. In the US, congestion pricing is typically used on interstates, freeways, or expressways in urbanized areas. Internationally, many cities have designated a "cordon" around the central business district that charge for the entry of private vehicles on a time-of-day basis. Often, variable pricing is used for specific lanes during peak periods so that traffic in those lanes continues to flow for high occupancy vehicles, transit, emergency services, and those willing to pay the higher prices. In other parts of the world, such as downtown London, congestion pricing is also extended to arterials and collectors in urbanized areas. For more information on congestion pricing, see https://ops.fhwa.dot.gov/congestionpricing/.

Caveat: The use of congestion pricing in Oregon requires legal authority and integration with ODOT's road user charging initiatives. Also, it typically requires political and public support.

Gamification

There are numerous applications (apps) that travelers can use to earn points or even monetary rewards for delaying travel or using a route that avoids peak congested areas.

This gamification process manages demand by rewarding travelers to use routes and travel times to make the transportation system as a whole more efficient.

Shared-Use Mobility

Shared-use mobility includes any transportation services that are shared among users such as traditional public transit, ridesharing, carsharing, ridesourcing, or bikesharing. Shared-use mobility services can be provided by public agencies, private entities, or through public-private partnerships. Although typically a broader system wide transportation demand management effort (see Section 18.5), shared-use mobility can also be applied to corridors. Ridesharing services can be targeted to provide carpool or vanpool options along particular corridors (typically interstates, freeways, expressways but also for principal arterials) to provide high occupancy travel between outlying areas of a city to employment centers or between urban areas (e.g. service on I-5 between Wilsonville and Salem). In urbanized areas, shared-use mobility can be achieved on arterial and collector corridors by providing travel options such as bikesharing or parking spaces for carsharing.

Employer Based Transportation Demand Management

Although typically a broader system wide transportation demand management effort (see Section 18.5), employers can help influence travel demand on corridors by:

- Adjusting shift times to start and end during non-peak periods of the adjacent corridor
- Providing incentives to use modes such as transit, bike, or walking
- Providing shuttle services to provide employee transportation between the nearest transit center and the employment center

These strategies are often used in urbanized areas but can also be applied in rural areas. For example, when a large employment center is located at an interstate interchange, adjustments to shift time may reduce delay for employees and travelers at the nearby interchange.

Neighborhood Based Transportation Demand Management

Another TDM strategy may target specific neighborhoods with incentives to travel via carpools, car-shares, walking, biking, or other non-single occupancy vehicle modes.

18.4.4 Integrated Corridor Management

The vision of Integrated Corridor Management (ICM) is to optimize the movement of people and goods within the corridor, balancing travel demand across all networks. ICM requires proactive management and collaboration between partner agencies, allowing for real-time operational decisions that benefit the corridor as a whole. ICM strategies focus on balancing travel demand across networks (freeway, arterial, transit, and parking) and providing multi-agency management of events such as incidents, special events, inclement weather, and work zones[25].

[25] Note that ODOT's Mobility Procedures Manual (April 2015) provides detailed information about

The ICM approach is based on three key concepts:

1. Corridor-level focus on operations
2. Agency integration
3. Active management of corridor assets and facilities

The first concept is a corridor-level focus on operations. As described earlier in Section 18.4, a corridor is a system of multimodal facilities that link travelers to a variety of destinations and services. A corridor also includes cross-network connections that allow travelers to connect to other corridors. ICM focuses on optimizing operations within the corridor to balance travel demand as much as possible.

Integration is the second key concept of ICM and includes three levels: institutional, operational, and technical. The overarching theme of integration is sharing information and infrastructure between agencies. The path to full integration can progress in steps. Initial steps may be sharing data and information with other agencies, and eventually pave the way to sharing full communication infrastructure and central system critical to managing the transportation system.

- Institutional integration involves collaboration between agencies within and across corridor boundaries and is necessary for successful implementation of ICM. The collaboration efforts might include sharing responsibilities and traffic operation functions, dissemination of traveler information, or sharing analysis information.

- Operational integration involves implementing multi-agency management strategies across the corridor. The multi-agency management strategies often require real-time information sharing to maximize corridor capacity. For operational integration to work smoothly, there are three key factors that need to be considered:

    - Assets – all of the transportation facilities and capabilities along the corridor
    - Availability – whether or not the assets are available
    - Scale of Response – whether the response needs to be conservative, medium, or aggressive

- Technical integration provides the means, such as communication links and system interfaces for agencies to collaborate and see the impact of operational decisions in real-time.

Active management of the corridor assets and facilities is the third concept of ICM. With active management, an action is implemented, the results are monitored and assessed,

coordinating with the freight industry for a variety of corridor work zone conditions.

then additional actions are evaluated and recommended, and (if necessary) another action is implemented. This active management cycle is continuous. This process can be fully automated or developed to include human input at key decision points.

Once all of these concepts are adopted and in place, corridor management can be optimized during planned and unplanned events. To understand how all these pieces might come together, take an example of a major freeway blocking event. With this scenario ICM could be used to inform drivers via variable message signs or personal devices to shift to a different route. Active traffic management strategies can be implemented or modified and ramp metering rates can be adjusted to facilitate traffic movement to avoid the incident. ICM could be used to help inform emergency responders how best to reach the scene, as well as adjusting signal timing on arterials handling an unexpected spike in traffic. If the event is severe enough, drivers can even be advised to use transit, and transit partners can increase transit service temporarily.

18.4.5 Analysis Procedures

This section provides analysis procedures for evaluating TSMO strategies at the corridor level using:

- Tool for Operations Benefit Cost Analysis (TOPS-BC)
- Crash Modification Factors (CMFs) – also see Chapter 4 in APM Version 2
- Dynamic Traffic Assignment (DTA)

Each tool includes discussion on steps, input parameters and settings, output, results interpretation and reporting, sensitivities, and caveats and cautions.

In addition to the tools discussed in depth in this chapter, there are several additional analysis tools that can be used to evaluate TSMO strategies. Synchro and SimTraffic are most appropriate for optimizing traffic signals. Vissim is a powerful microsimulation tools that can be used to evaluate many types of TSMO strategies including but not limited to adaptive signal systems, ramp meters, transit signal priority, variable speed limits, and hard shoulder running.

Tool for Operations Benefit Cost Analysis

The Tool for Operations Benefit Cost Analysis (TOPS-BC) was created by the FHWA Office of Operations as a sketch-planning analysis tool for TSMO related strategies. It is meant to provide a preliminary screening and initial prioritization of TSMO strategies and replaces the ITS Deployment Analysis System (IDAS). As described in Section 18.3 the TOPS-BC tool provides four key functions:

1. Allows users to look up the expected range of TSMO strategy impacts based on a database of observed impacts in other areas nationally and internationally
2. Provides guidance and a selection tool for users to identify appropriate benefit/cost methods and tools based on the input needs of their analysis
3. Provides the ability to estimate life-cycle costs of a wide range of TSMO strategies

4. Allows for the estimation of benefits using a spreadsheet-based sketch-planning approach and comparison with estimated strategy costs.

As a sketch planning level tool, TOPS-BC allows users to quickly understand typical benefits for a range of TSMO strategies and then estimate the benefit-cost ratio of those TSMO strategies. The tool also provides a suggested list of analysis tools for TSMO strategies depending on user-selected criteria such as: the level of confidence required, which TSMO strategies are being investigated, key measures of effectiveness, and a few other filters.

The tool exists as a Microsoft Excel spreadsheet with several tabs and color-coded cells that clearly identify where the user needs to provide information. Before getting into a detailed description of TOPS-BC, it is worth noting that since the tool is an Excel spreadsheet, the user does have the ability to modify it as needed. The Excel file for TOPS-BC can be found here: https://ops.fhwa.dot.gov/plan4ops/topsbctool/index.htm

Exhibit 18-16 provides a list of the TSMO strategies included in the TOPS-BC tool, and whether typical impacts/benefits are provided for the strategy, as well as more detailed tabs to calculate costs and benefits.

Exhibit 18-16: TSMO Strategies Included in TOPS-BC

TSMO Category	TSMO Strategy	Typical Impacts	Cost	Benefit
Traveler Information	Dynamic Message Signs (DMS) Highway Advisory Radio (HAR) Pre-Trip Traveler Information Web/Internet Multi-Modal Traveler Info In Vehicle – traveler info or route guidance	Yes Yes Yes Yes Yes	Yes Yes Yes No No	Yes Yes Yes No No
Traffic Signal Coordination Systems	Preset Timing Traffic Actuated Central Control Transit Signal Priority Emergency Signal Priority	Yes Yes Yes Yes Yes	Yes Yes Yes Yes No	Yes Yes Yes No No
Ramp Metering Systems	Central Control Traffic Actuated Preset Timing	Yes Yes Yes	Yes Yes Yes	Yes Yes Yes
Incident Management	Traffic Incident Management (general) Incident Detection/Verification Incident Response/Management Incident Detection and Response Freeway Service Patrols	Yes Yes Yes Yes Yes	Yes Yes Yes Yes Yes	Yes No No No No
Advanced Traffic Demand Management (ATDM)	Speed Harmonization Employer Based Traveler Demand Management Hard Shoulder Running High Occupancy Toll (HOT) Lanes Road Weather Management Work Zone Advanced Public Transit Systems Congestion Pricing (HOT lanes and variable tolls)	No No Yes Yes Yes Yes Yes Yes	Yes Yes Yes Yes Yes Yes No No	Yes No Yes Yes Yes Yes No No
Public Transit	Fixed Route Transit (various features) Paratransit (various features)	Yes Yes	No No	No No
Supporting Strategies	Traffic Management Center Loop Detection CCTV Cameras	No No No	Yes Yes Yes	No No No

The cost and benefit components of TOPS-BC each calculate costs and benefits on an annual basis and provide an overall benefit/cost ratio for the strategy.

TOPS-BC calculates an annualized cost for each strategy by incorporating the useful life of the equipment, the replacement cost, and the annual operations and maintenance cost. The net present value of implementing the strategy is also provided, and while default values for the discount rate and time horizon are provided, the user can change those values.

The cost components for each strategy are broken down into two categories: the one-time costs to create the backbone structure for the strategy such as software and system integration; and the incremental cost of each additional installation such as additional loops, weather stations, and dynamic message signs. Default costs are included in the spreadsheet for all of the cost components. The user simply needs to enter the number of infrastructure deployments (typically one), and the number of incremental deployments.

If better cost data are available, the user can easily modify the spreadsheet with project specific costs or add components to the spreadsheet. One cost component that is not captured in the cost estimating tool is right-of-way acquisition, which should not be ignored.

Exhibit 18-17 provides an example of how TOPS-BC calculates the cost of dynamic message signs. Again, the tool provides default unit costs for capital equipment and operations and maintenance annual costs, as well as the useful life of the equipment. These values can all be changed if the user has more precise information relevant to the specific project. However, for a quick planning level cost analysis, the user simply needs to provide information in the green cells:

- The number of infrastructure deployments (typically one as is the case in this example)
- The number of incremental deployments, shown as five in this example
- The year of deployment.

Based on this example, the average annual cost to implement and maintain this strategy over a 25-year lifespan is $88,400.

Exhibit 18-17: Example Cost Calculation of Dynamic Message Signs

FHWA Tool for Operations Benefit/Cost (TOPS-BC): Version 1.1

PURPOSE: Estimate Lifecycle Costs of TSM&O Strategies

WORK AREA 1 - ESTIMATE AVERAGE ANNUAL COST

Traveler Information: DMS

Equipment	Useful Life	Capital / Replacement Costs (Total)	O&M Costs (Annual)	Annualized Costs
Basic Infrastructure Equipment				
TMC Hardware for Information Dissemination	5	$ 7,500	$ 375	$ 1,875
TMC Software for Information Dissemination	5	$ 20,000	$ 1,000	$ 5,000
TMC System Integration	20	$ 100,000	$ 5,000	$ 10,000
TOTAL Infrastructure Cost		$ 127,500	$ 6,375	$ 16,875
Incremental Deployment Equipment (Per Sign Location)				
Communication Line	25	$ 750	$ 900	$ 930
Variable Message Sign	25	$ 92,500	$ 4,400	$ 8,100
Variable Message Sign Tower	25	$ 125,000	$ 275	$ 5,275
TOTAL Incremental Cost		$ 218,250	$ 5,575	$ 14,305
INPUT Enter Number of Infrastructure Deployments	1			**$ 16,875**
INPUT Enter Number of Incremental Deployments	5			**$ 71,525**
INPUT Enter Year of Deployment	2016			
Average Annual Cost				**$ 88,400**

The annual benefits calculated by TOPS-BC focus on travel time savings for both recurring and non-recurring congestion, energy/fuel savings, and savings due to reduced crashes. A tab marked “Parameters” includes all of the assumptions used to calculate the benefits such as the cost of fuel, fuel economy of autos and trucks, the cost of hourly travel for autos and trucks, the typical percent of trucks on the facility, the cost of crashes by severity, typical emissions, typical crash rates by facilities, speed-flow relationships, and typical incident delay factors. Exhibit 18-18 shows a section of the parameters tab. If local information is known, the percent of trucks on the facility and crash rates on the facility by severity, those should be modified to provide a more accurate benefits assessment. However, if local factors are not known, the default values make the tool ready to use for an approximation of the benefit for the desired TSMO countermeasure.

Exhibit 18-18: Parameter Tab in TOPS-BC

Benefit Estimation Parameters

General Parameters

Year of Dollars Displayed	
Year of Dollar Display	2014
Inflation Rate	3%
Adjustment Factor	1.13
Annualization Factor	
Number of Periods per Year	250
Net Present Value Calculation	
Default Time Horizon (Years)	20
Traffic Mix	
Percentage Trucks	10%
Percentage "On-the-Clock" Travel Purpose (Autos)	20%
Average Auto Occupancy	1.67
Discount Rate	
Discount Rate (for 20 year analysis)	7.0%
Analysis Time Horizon	
Years	20

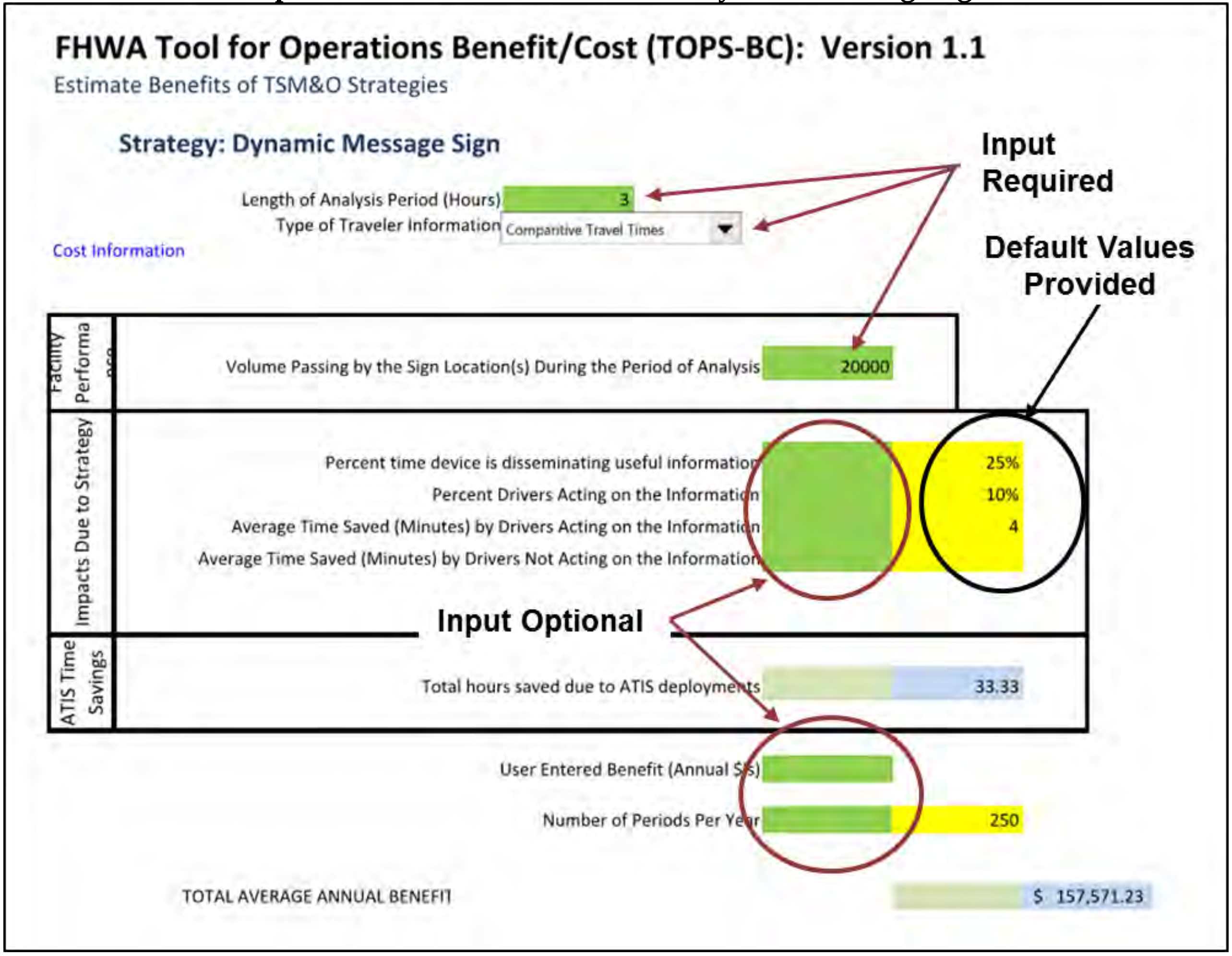
Benefit Valuations

Recurring Travel Time (per hour)		
"On the Clock" Travel Time	$	31.51
Other Auto Travel Time	$	15.76
Truck Travel Time	$	31.51
Non-Recurring Travel Time (per hour)		
"On the Clock" Travel Time	$	31.51
Other Auto Travel Time	$	15.76
Truck Travel Time	$	31.51
Crashes (per occurrence)		
Fatality	$	10,129,579
Injury	$	75,409
Property Damage Only (PDO)	$	2,589
Fuel Use		
Per Gallon (Excluding Taxes)	$	4.13
Non-fuel Operating Costs (per VMT)		
Auto		0.25
Truck		0.37
Emission Cost (per ton)		
CO	$	79
CO2	$	42
NOx	$	18,346
PM10	$	148,342
VOC	$	1,283
Noise (per VMT)		
Auto	$	0.0012
Truck	$	0.0371

Similar to the cost component in this tool, each strategy has its own tab to calculate benefits. Again, color coded cells are used to easily identify where input is required. Green cells indicate user defined input; however, not all green cells require user input. Often a default value will be used, as is the case when a yellow (default) cell is adjacent to a green one. In the Exhibit 18-19, there is also a green cell at the bottom that can be used to add additional benefits not represented by the TOPS-BC sheet, but input there is not required. For the dynamic message sign shown in Exhibit 18-16, the user is required to provide three pieces of information:

- Length of the analysis period
- Type of traveler information (three options are provided on a pull-down menu). In the example shown below the "Comparative Travel Times" option is selected. The other two options include "Congestion Warning" and "Alternative Route Recommendation"
- Traffic volume passing the sign location(s) during the period of analysis

Exhibit 18-19: Example of the benefits calculated for dynamic message signs

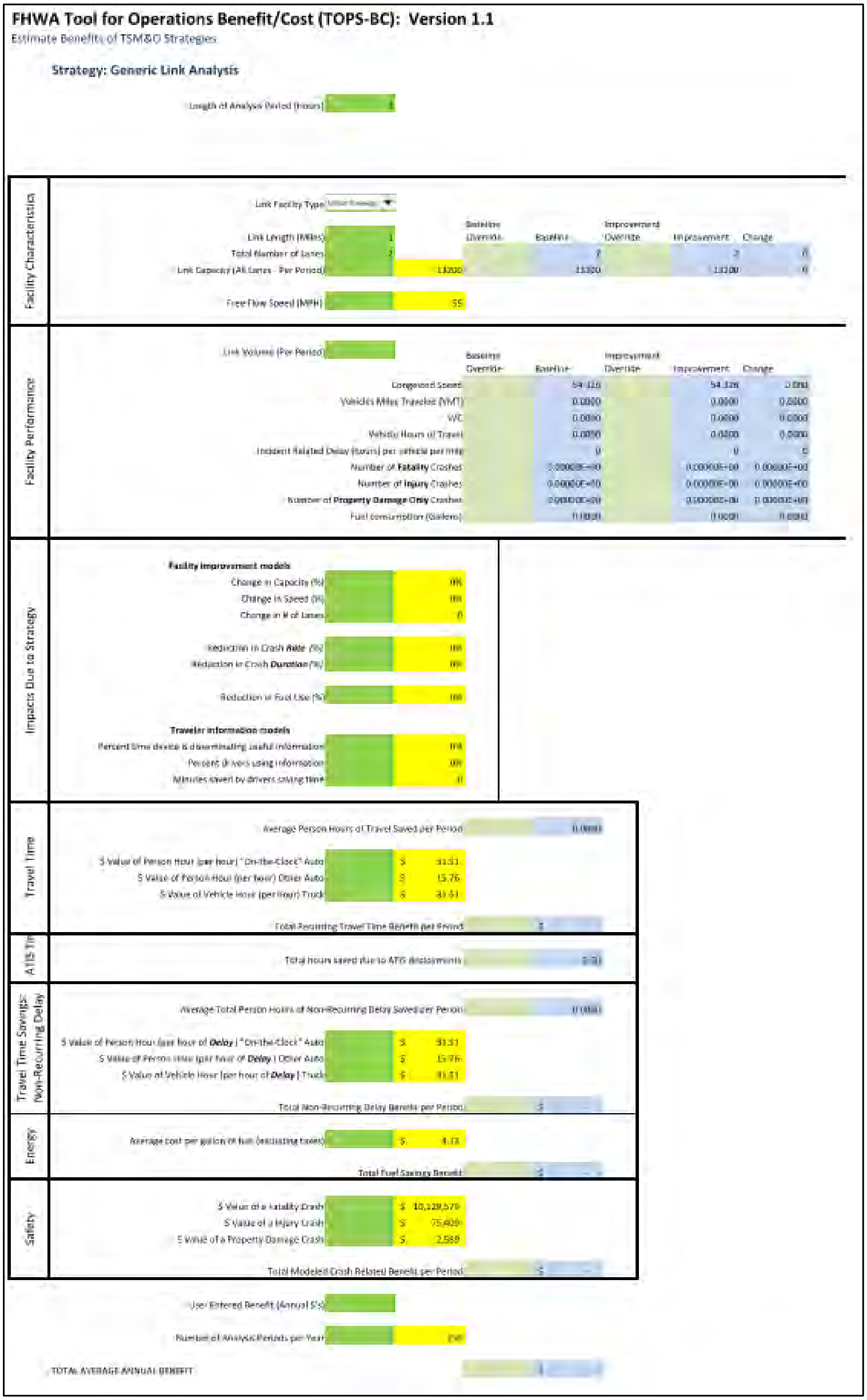


There are additional categories where default values are provided, or the user can override those values when location specific information is known. For example, in Exhibit 18-19, default values are provided in the yellow cells for four items. The user can input their own values in the adjacent green cells if more precise information is known or opt to use the default values.

Using the information from the benefit and cost tabs, the user can easily create a benefit-cost ratio since both elements are provided in terms of annual cost and annual benefit. Based on the DMS example, the benefit-cost ratio is 1.78.

In addition to the benefit tabs for each strategy, the tool provides a "generic link based" analysis tab that can be used to enter benefits related to a TSMO strategy not specifically identified, as shown in Exhibit 18-20. This tab allows the user to customize a benefits calculation that may not fit precisely into any of the other categories. In this tab, the user must input information about the facility characteristics and expected impacts of the strategy such as changes in capacity, crash rates, delay, speed, and energy.

Exhibit 18-20: Link Based Benefits Tab



The TOPS-BC tool also provides a summary tab that allows the user to select some or all of the strategies for a total benefit-cost summary.

For more information and access to the TOPS BC tool, refer to FHWA TOPS BC website.[26]

Crash Modification Factors

Crash modification factors (CMFs) are used to determine the effect a specific countermeasure will have on future crashes at a given location or along a corridor. In terms of TSMO related strategies, CMFs apply directly to the objectives-driven, performance-based approach, by providing the likely quantifiable change in crashes. Section 18.3 introduces CMFs and Chapter 4 also goes into detail on using CMFs, especially related to predictive crash analysis.

In most cases the CMF targets a specific type of crash, severity, or type of facility. When calculating benefits for a strategy at a specific location, it is important to ensure that the CMF is applied to those same types of crashes and severity categories. For example, if a countermeasure has a CMF that reduces fatal crashes by 20 percent, and the location where it is being applied had an annual average of five fatal crashes and 15 non-fatal injury crashes, the benefit from that CMF would be the reduction of one fatal crash per year. That CMF would not apply to the non-fatal crashes.

In general when two countermeasures are being applied to a location, the CMFs are not additive. For example, if both variable speeds and ramp metering are installed along a freeway segment, you should use either the CMF related to variable speeds or the CMF related to ramp meters, but not both. The only time you might use two (or more) CMFs for combined countermeasures are when the CMFs each target different types of crashes or severity of crashes at a given location. However, even then, one should use judgement to minimize any sort of exaggerated additive benefit from using more than one CMF.

Dynamic Traffic Assignment

While this section provides an overview of DTA relative to corridor management, APM Chapter 8.5 provides a broader overview of DTA concepts and tools, as well as publication by the Transportation Research Board[27].

DTA models can provide a tool for measuring corridor management within the context of the broader (subarea or regional) travel network. Many attributes or variables in DTA models provide an enhanced ability to measure operational impacts at the corridor level. While such impacts could also be assessed in microsimulation models, such models do not provide the routable nature of “trips” present in a DTA network (which are typically replaced with pre-defined intersection-level turn movements). Further, microsimulation models typically require more effort to prepare than DTA models (for a given size

[26] TOPS-BC website: https://ops.fhwa.dot.gov/plan4ops/topsbctool/index.htm

[27] *TRB Transportation Research E-Circular E-C153: Dynamic Traffic Assignment: A Primer,* Transportation Research Board, Washington DC, June, 2011.

network).

While there are many differences between different DTA platforms, the following general characteristics are common:

- Time-dependent paths where the path through the network is influenced by travel times that vary depending on when travelers arrive at a given network link, as opposed to assuming that travel times are constant throughout the period being simulated.
- Travel routes typically change at shorter intervals than static models (typically minutes instead of hours).
- Network congestion estimates, which are often based on traffic operations, are typically more detailed than a static model to account for travel time difference between routes.
- Vehicles queue on the network and are not forced through over-capacity conditions due to a timer interval constraint. Thus some demand may remain unserved during the analysis period.
- Individual vehicle "simulation" (whether visualized or not) is generally present to account for vehicle interaction and operational impacts.
- Like static assignment, multiclass assignment (including trucks) can be captured in DTA tools, which allow for different path sets and attributes among classes.
- Transit elements are included in varying degrees based on each DTA tool, but may include the ability to include service information including routes, stop locations, and schedules. Next generation DTA tools are starting to model individual transit persons as well.

A key differentiator among DTA models is the overall fidelity and detail for which traffic flows are captured along a road. The two types of models are referred to as "link-based" and "lane-based:"

- <u>Link-Based Models</u> – Traffic flow along a roadway is analyzed macroscopically, where the total number of lanes is considered as an overall link capacity. Differences among individual lanes (including the amount of traffic demand for an individual lane), interactions among individual vehicles, and friction related to movement and lane changing are not directly modeled. Examples: Visum Dynamic User Equilibrium (DUE), DTALite/NeXTA, and DynusT.

- <u>Lane-Based Models</u> – Traffic flow along a roadway is analyzed for each lane, using car-following algorithms that account for interactions among vehicles and flow differences in each lane. Storage of vehicles related to turn bay lengths and other differences between lanes along a given link may influence operations at the adjacent node/junctions/intersections. Examples: Dynameq and Vissim DTA.

In general, lane-based DTA platforms will provide additional details needed to evaluate corridor operations, including the following considerations:

- Intersection control (approach geometry and signal timing)

- Transit stop treatment

- Lane-changing friction caused by weaving between intersections or access density
- Time of day lane management (such as reversible lanes or closures)
- Bottleneck impacts can be better captured with the additional operational detail

18.5 System Management

This section provides an overview, considerations, and analytical procedures for a collection of TSMO strategies that are programmatic and applied systemically. Whereas corridor management focuses on TSMO strategies applied to a specific facility or set of parallel facilities, system management addresses programs that are implemented across the transportation system in a given geography (i.e. city, county, regional or state). This section discusses seven system management program areas including:

- **Traffic Incident Management** is the practice of coordinating resources across partner agencies and the private sector to quickly detect, respond to, and clear traffic incidents to reduce impacts on safety and congestion.
- **Emergency Operations** is the planned, coordinated response to man-made or natural events causing or threatening injury or loss of life, property damage, human suffering, or financial loss.
- **Road Weather Operations** is the use of strategies to minimize or eliminate the impacts of weather events such as rain, snow, high winds, or flooding on safe and reliable travel.
- **Work Zone Management** is the practice of managing traffic impacts during construction to maximize traveler and worker safety, minimize traffic delays, maintain access for adjacent land uses, and support timely completion of construction.
- **Planned Special Event Management** is the advanced planning and coordination to manage travel before, during and after an event.
- **Traveler Information** includes strategies to deliver pre-trip and en-route information about travel options and conditions.
- **Transportation Demand Management** is the policies and strategies aimed at enhancing travel opportunities and choices that make more efficient use of the transportation system.
- **Connected and Automated Vehicles** is the emerging technology in vehicles that will enable the communication between vehicles, infrastructure and mobile devices to make travel safer and more efficient.

18.5.1 Traffic Incident Management

Traffic Incident Management (TIM) is the practice of planned and coordinated detection, response, and clearance of traffic incidents. TIM is a multi-disciplinary effort among incident responders to manage and clear traffic incidents as quickly as possible while maximizing responder safety and minimizing traffic impacts during the incident. Exhibit 18-21 lists several examples of hazardous traffic incident events. Traffic incidents are said to account for up to one-quarter of congestion on US roadways[28]. Swift clearance of

[28] http://ntimc.transportation.org/Documents/Benefits11-07-06.pdf

incidents has two main benefits: safety and system efficiency. Clearing an incident as quickly as possible limits the amount of time incident responders and travelers are exposed to dangerous conditions, decreases the chances of a secondary crash, and more quickly restores the roadway to normal capacity.

Exhibit 18-21: Examples of Hazardous Traffic Incident Events

Abandoned Vehicle – Hazard	Downed Power Lines	Pedestrian in Roadway
Animal on Roadway	Fatal Crash	Pothole
Animal Struck – Hazard	Hazard Tow (minor)	Rock Fall
Closure	Hazardous Debris	RR Crossing Equipment Failure
Crash	Hazardous Tree/Vegetation	Signal Not Working
Crash Investigation	Hazmat Spill (minor)	Spilled Load
Damaged ODOT Property	High Water	Vehicle Fire
Disabled Vehicle – Hazard	Obstruction on Roadway	

Oregon adopted its first TIM Strategic Plan in 2011 and recently completed a 2015 update[29]. Oregon's TIM Strategic Plan identifies and prioritizes actions to implement statewide over the next five years. The updated plan identifies over sixty actions in the areas of technology integration, agency and stakeholder collaboration, public outreach, policy and regulatory, system evaluation, and responder training.

Successful TIM programs and strategies rely on cooperation and coordination across a broad range of partners. The Oregon TIM Strategic Plan identifies over twenty stakeholders. A multi-agency approach ensures that all response agencies are acting toward a common goal while understanding each partner's unique roles and responsibilities.

Traffic incidents can severely reduce the capacity of roadways, even when incidents occur in the shoulder area. Exhibit 18-22 demonstrates the loss of freeway capacity due to lane blocking incidents. For example, on a two-lane freeway if one lane is blocked, the freeway is limited to just 35% of its normal capacity (not 50% as one might linearly assume).

[29] Oregon Traffic Incident Management Strategic Plan. Prepared by DKS Associates, in cooperation with ODOT and Oregon State Police (OSP). 2015. Web link: *https://www.oregon.gov/ODOT/Maintenance/Pages/Traffic-Incident-Management.aspx*

Exhibit 18-22: Proportion of Freeway Segment Capacity Available Under Incident Conditions

Number of Lanes (One Direction)	Shoulder Disablement	Shoulder Accident	One Lane Blocked	Two Lanes Blocked	Three Lanes Blocked
2	95%	81%	35%	0%	N/A
3	99%	83%	49%	17%	0%
4	99%	85%	58%	25%	13%
5	99%	87%	65%	40%	20%

Source: *Highway Capacity Manual 2010*, Exhibit 10-17

When considering which strategies are most appropriate for a given area, there are numerous factors to consider. The following list documents a number of the key factors including:

- Crash data (see Chapter 4 for safety and crash analysis procedures)
- Traffic volumes
- Geographic context
- Facility type
- Existing capital infrastructure
- Existing ITS infrastructure
- Proximity to detour routes
- Available resources and resource sharing between agencies
- Status of integrated corridor management

Common Traffic Incident Management Strategies

The following list includes common traffic incident management strategies currently in place in Oregon. For a complete list of TIM Strategies Oregon is pursuing, Oregon's TIM Strategic Plan should be referenced.

- Safety Service Patrols arrive at the scene and provide preliminary traffic control, assist the motorist to the side of the road, or other safety measures to improve safety. Oregon has a total of 23 Dedicated Incident Responders that cover key state roadways in regions 1, 2, 3, and 4. The service patrol hours of operation vary by district and may also vary seasonally.

- Driver Removal Laws require drivers to remove their vehicles out of travel lanes after a traffic incident if there are no serious injuries or fatalities. Drivers can then exchange information in a safe location and wait for law enforcement assistance. Oregon's Move It Law requires drivers involved in an incident to remove their vehicle from the travel lanes as long as there are no serious injuries.

- Authority Removal Laws allow a pre-designated set of public agencies to remove damage or disables vehicles (and/or spilled cargo) from the roadway that is safety concern for travelers. Typically, these laws include liability protection for the designated agencies. Oregon's Authority Removal Law allows law enforcement to remove vehicles or debris so the incident is not blocking traffic.

- Move Over Laws require drivers approaching a scene where incident responders are present to either change lanes or reduce speeds. The exact requirements and penalties vary from state to state. In Oregon, the current law requires drivers to either move to a lane not adjacent to that of the response vehicle, or, if changing lanes is not possible, reduce the speed to five miles per hour under the speed limit. Note that in Oregon's TIM Strategic Plan, there is an action to determine whether the Oregon law should be stricter (requiring a larger speed reduction), and if so, to begin the required legislative process.

- Shared Quick Clearance Goals sets a specific clearance time target that the incident must be cleared from the roadway. Oregon legislature established a 90-minute quick clearance goal for all incidents in 2013.

- Pre-established Towing Service Agreements set the requirements for tow companies to meet to be on a tow rotation list. When a tow is needed for a traffic incident, these agreements allow the agency to easily contact the appropriate tow company using a central point of contact.

  In Oregon, the Oregon State Police (OSP) set the requirements tow companies must meet to be on the tow rotation list. When an incident occurs, either ODOT or OSP will initiate a tow request if necessary. The tow company called to the scene depends on which agency initiates the request.

  The Portland region is unique in regard to towing regulations in the state. In the Portland region, several agencies coordinate to allow for a central point of contact when an incident requires a tow. Those agencies include Portland Police, Multnomah County, City of Portland, TriMet, and ODOT. The vetting to get on the Portland tow list is different than the statewide list, so the list of tow companies is different, although some tow companies may be on both lists. Creating a formal process that unites all transportation agencies under a single Tow Board in the Portland area allows for a forum to present and implement new towing strategies much easier than other areas in the state.

- Incentivized Towing offers a financial incentive for tow companies to clear an incident in an allotted time period, and can institute a penalty if the tow company takes longer than that time period. Oregon does not currently have any incentivized towing contracts in place.

- Instant Towing or Staged Towing Contracts initiate a tow request as soon as an incident is reported. If it turns out the tow is not necessary, the tow company receives a “no service” fee. Staged towing strategically stations tow trucks near areas of frequent accidents, and tow companies are paid for the full time they are stationed, regardless of whether they are actually called to a tow incident.

  In Oregon, the Portland region participates in Instant Towing year round, and during winter events the Portland region has Staged Towing contracts where tow companies locate trucks at specific pre-determined locations and are ready as incidents occur. An action in Oregon’s TIM Strategic Plan is to identify if there are other regions in the state that could benefit from tow contract plans.

- Dispatch Co-location can improve communication between response agencies and transportation agencies.

- TIM Task-Force is an organization of first responder agencies that convene on a regular basis to discuss TIM challenges, procedures, resources, and other TIM areas to improve upon. In Oregon, there is a statewide TIM Task Force, and TIM teams that meet regularly in the Portland area, Rogue Valley area, and Central Oregon area, as well as an annual statewide winter operations meeting. Oregon’s TIM Strategic Plan identifies other regions along the I-84 and I-5 corridors where additional TIM Teams may be beneficial.

- Strategic Highway Research Program (SHRP2) Training created a coordinated, multi-disciplinary training program for all emergency responders, which helps put all incident responders on the same page, leading to a safer, faster, integrated responder team.

- After Action Reviews are incident debriefing sessions where all involved response agencies discuss the incident and determine what worked well and what can be improved.

After TIM projects are selected and implemented, monitoring TIM performance measures is critical for agencies to demonstrate accountability and program effectiveness in order to support future TIM planning.

FHWA identified three national TIM performance measures:

- Roadway clearance time – the amount of time between the first recordable awareness of the incident by a responsible agency to the time that all lanes are available for traffic flow

- Incident Clearance time – the amount of time between the first recordable awareness of the incident by a responsible agency to the time that all responders have left the scene
- Number of secondary incidents **–** the number of unplanned incidents beginning with the time of detection of the primary incident where an incident occurs as a result of the original incident either within the incident scene or within the queue in either direction.

Oregon currently collects and monitors the first two performance measures, as well as roadway closure time and the number of responders trained in the National TIM training program. Oregon legislature set the current goal for incident clearance – clear 100 percent of all lane-blocking crashes within 90 minutes. Tracking the number of secondary incidents is marked as a high priority action in the 2015 Oregon TIM Strategic Plan. Developing a method to track incident responder fatalities and struck-bys is also noted in the plan.

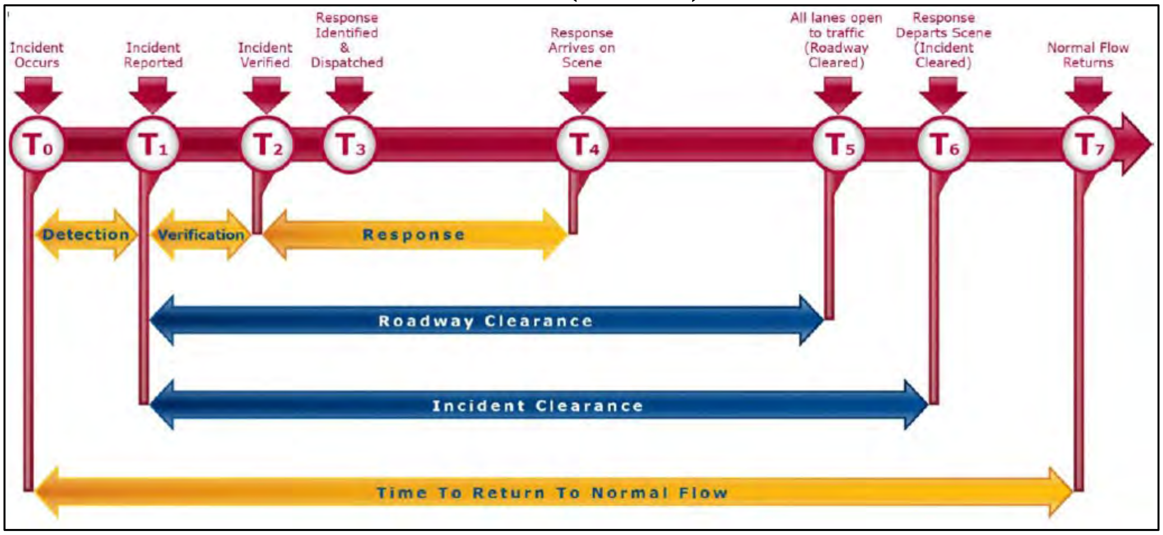
The National Cooperative Highway Research Program (NCHRP) advanced the TIM performance measure element and created a website to help guide performance measurement related to TIM: https://data.transportationops.org/nchrp-07-20-guidance-implementation-tim-performance-measurement. The purpose of the website is to provide guidance on consistent use of TIM performance measures and to support the overall efforts of TIM program assessment. NCHRP website goes beyond the three national TIM performance measures by recommending other key performance measures, listing common sources of TIM data, and identifying challenges in collecting and analyzing TIM data. The NCHRP website also provides recommendations for developing TIM databases and model scripts.

As shown in Exhibit 18-23, NCHRP recommends tracking six key time intervals during incident response:

- Detection time
- Verification time
- Response time
- Roadway clearance time (also recommended by FHWA)
- Incident clearance time (also recommended by FHWA)
- Time to return flow to normal

A seventh time interval not captured by the NCHRP timeline but worth considering, is the time for upstream platoons caused by the incident to dissipate. In some cases, the flow at the scene of the incident may return to normal while long platoons are still present upstream.

Exhibit 18-23: View of Incident Timeline (NCHRP)



NCHRP highlights other performance measures to consider including: number of secondary incidents involving first responders, percentage of fatal crashes that are secondary, queue lengths, travel delay, service patrol statistics, and several others. Oregon already tracks incident data involving the crash location, severity, fatalities, type, involved vehicles, and others information captured in the crash database.

For incidents, the following three databases provide information: Transportation Data Section (TDS) Crash Reports, Highway Traffic Operations Center System (HTOCS), and Highway Travel Conditions Information System (HTCIS). The information within these systems includes:

- TDS Crash Reports – This database is compiled from individual driver and police crash reports submitted to ODOT. The reports in this crash database include information about the crash (such as location, time, type, and severity), information about the vehicle (such as ownership, type of vehicle, and vehicle movement), and information about the participants (such as age, sex, injury severity, and drug or alcohol use).
- HTOCS Events and Unit Status – This database includes most of the information about each incident such as time, location, primary unit in charge, description of incident, and response result. It also contains status timestamps for each unit (i.e., person) as they are notified and respond to incidents.
- HTCIS Traveler Information – This database includes the status updates that are shared with the public on the ODOT TripCheck website, through 511, and ODOT's Traveler Information Portal (TTIP), which allows external sources to access ODOT's data for redistribution. It includes which lanes are affected and what the expected impact is to travelers (i.e., delay experienced). Each incident usually has multiple status updates throughout the duration of the incident. This

database also includes weather related information when available (atmospheric and the resulting road conditions).

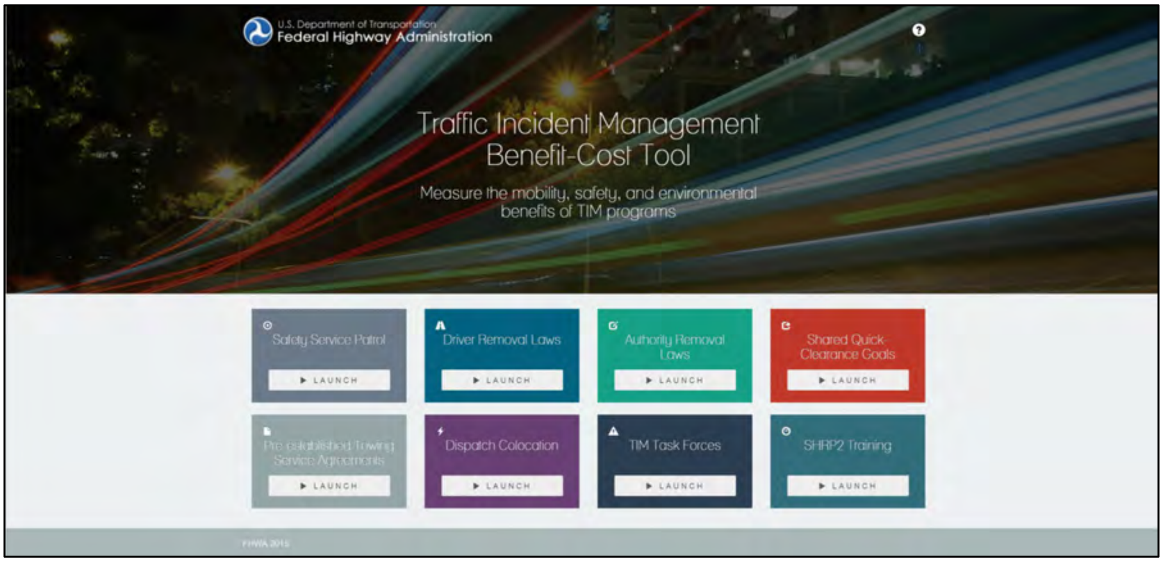
Traffic Incident Management Analysis Tools

The USDOT ITS Knowledge Resources website, described in Section 18.3.6, is a useful tool for identifying benefits and costs for a wide variety of TIM strategies under the categories of surveillance and detection; mobilization and response; information dissemination; and clearance and recovery. The website compiles evaluation findings about TIM-related projects from across the globe and documents benefits, costs, and lessons learned. The benefit and cost sections can be easily searched by application, specific goals, and geographic location. The USDOT ITS Knowledge Resources website can be found at https://www.itskrs.its.dot.gov.

FHWA’s Office of Operations and Research Development recently developed a tool, the Traffic Incident Management Benefit-Cost (TIM-BC) tool, which can be used to evaluate the benefits of TIM strategies as shown in Exhibit 18-24. It is a web-based software tool (https://www.fhwa.dot.gov/software/research/operations/timbc/) that evaluates eight of the most common TIM strategies:

- Safety Service Patrols
- Driver Removal Laws
- Authority Removal Laws
- Shared Quick Clearance Goals
- Pre-established Towing Service Agreements
- Dispatch Co-location
- TIM Task-Force
- Strategic Highway Research Program (SHRP2) Training

Exhibit 18-24: FHWA's TIM-BC Tool



FHWA's Incident Management Benefit Cost Tool allows the user to customize project information and obtain benefit cost information. For optimal results the tool requires the user to have a considerable amount of data for each roadway segment the strategy targets. For example, the user needs to enter the average incident duration and number of incidents categorized as shoulder blockage or one lane blockage, as well as the percentage of estimated secondary incidents for each roadway segment. In some cases, default values are available. There is also an element of subjectivity in the data entry area for "Project Savings", requiring the user to input the proportion of incidents in which clearance times would decrease by implementing the strategy.

Once all of the information is entered, the program calculates savings for delay, fuel, secondary accidents, and emissions. The tool also produces a formatted pdf report of the analysis.

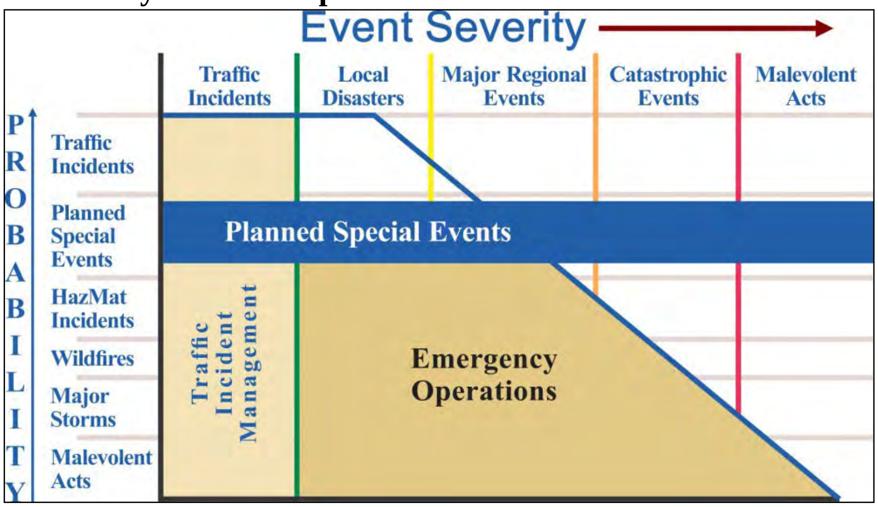
18.5.2 Emergency Operations

Emergency operations is broadly defined as the response and management of the transportation system during a wide range of non-recurring events including major traffic incidents, special planned events, and disasters such as flooding or evacuations. For the context of this sub section, emergency operations refer to less probable and more severe types of non-recurring events such as natural disasters, evacuations, and large-scale hazardous materials events. Exhibit 18-25 lists examples of emergency events and Exhibit 18-26 illustrates the relation of event probability with event severity, with the shaded region indicating the type of event addressed in this section. Both traffic incident management and planned special events are covered in other sections (18.5.1 and 18.5.5 respectively).

Exhibit 18-25: Examples of Emergency Operation Events

Air Crash	Explosion	Other Disaster
Avalanche	Evacuation	Rail Crash
Bomb	Flood	Road Surface Collapse
Earthquake	Landslide	Tsunami
Erosion	Major Hazmat	Wildfire

Exhibit 18-26: FHWA's Emergency Transportation Operations – Severity and Probability Relationship



The federal government provides extensive guidelines and recommendations for emergency operations through FHWA's Emergency Transportation Operations website,[30] and through FEMA's National Incident Management System (NIMS) website.[31] Additionally, Oregon has its own Emergency Operations Plan[32] that provides the framework to respond and recover from major emergencies and disasters. Oregon's plan documents the roles and responsibilities of state agencies while managing emergency response efforts.

Successful emergency operations rely heavily on TSMO strategies and multi-agency coordination to ensure mobility is maintained and that real-time data are accurately communicated to both the public and transportation operators.

[30] FHWA website accessed on December 9, 2015: https://ops.fhwa.dot.gov/tim/about/index.htm
[31] FEMA website for NIMS access on December 10, 2015: https://www.fema.gov/national-incident-management-system
[32] State of Oregon Emergency Operations Plan. Oregon Office of Emergency Management. Publication date: July 2010; Latest Revision Date: March 2014.

FHWA describes three phases of emergency response, with most of the TSMO related strategies linked to the first two phases.

1) Preparation and Activation. Emphasized TSMO strategies include ITS infrastructure investments and creating transportation management centers.
2) Response. Emphasized TSMO strategies include communication systems, information sharing, and resource management.
3) Re-entry and Return to Readiness. This phase is more about debriefing and documentation, and does not have much emphasis on TSMO strategies.

Overview of TSMO Emergency Operation Strategies

Key TSMO strategies to successfully implement Emergency Operations include:

- Integrated Corridor Management. When emergencies close transportation facilities, require a detour, or even a mass evacuation, integrated corridor management can ensure that all transportation facilities within and across corridor boundaries are being used at maximum efficiency. For example, if a segment of freeway is closed, integrated corridor management can be used to modify traffic signal timing on surface streets to accommodate a sudden spike in traffic or unexpected direction of flow. See Section 18.4.4 for more information about Integrated Corridor Management.
- Real-Time Traveler Information. During an emergency it is critical to share current and accurate information with the public. Agencies need to be able to connect to a traveler information system and quickly update the information as it changes. See Section 18.5.6 for more information about Traveler Information strategies.
- Communication between Agencies. In the case of emergency operations, the scale is likely large enough that multiple transportation facilities and services will be impacted. For optimum mobility, it is critical that agencies communicate plans before implementing. For example, if a freeway segment needs to be closed, it is critical to notify the agencies whose facilities will be impacted. Notifying the appropriate agencies ensures that available resources can be optimized (detour routes, variable message signs, signal timing adjustments, etc.).
- Integration of ITS into Emergency Response Plans. On the planning spectrum, agencies should integrate ITS/TSMO strategies into emergency response plans. See Sections 18.4.1 and 18.4.2 for Arterial and Freeway Management strategies.
- Statewide Communication Interoperability Plan (SCIP)[33]. This plan establishes protocol to communicate with agencies across the state and enables a unified approach to enhance interoperable communications.

Considerations for Effective Emergency Operations

Effective emergency operations are closely related to integrated corridor management requiring institutional, operational, and technical integration. Institutionally, there needs to be collaboration between agencies. Operationally, agencies need to share assets and

[33] Website: *https://www.oregon.gov/siec/Pages/SCIP.aspx*

communicate asset availability to all involved agencies. Technically, the means to communicate and interface with agencies and traffic devices needs to be available.

Benefits of Fully Integrated Emergency Operations

When Emergency Operations are fully integrated, there are several benefits to the transportation system and the general public:

- Improved mobility during large scale emergencies
- Improved safety of the public and responders
- Ability for the public to make well informed decisions

Emergency Operations Analysis Tools

The USDOT ITS Knowledge Resources website, described in Section 18.3.6, has a limited database of studies that identify benefits and costs for Emergency Operations strategies such as freeway lane reversal and hazmat technologies. The USDOT ITS Knowledge Resources website can be found at https://www.itskrs.its.dot.gov.

18.5.3 Road Weather Operations

Adverse weather can have many negative impacts on travel in the areas of safety, mobility, productivity, and operational decisions. FHWA, through their Road Weather Management Program, summarizes the most common impacts weather can have on roads, traffic, and operational decisions as shown in Exhibit 18-27.[34]

[34] https://ops.fhwa.dot.gov/weather/q1_roadimpact.htm

Exhibit 18-27: Weather Impacts on Roads, Traffic and Operation Decisions

Road Weather Variables	Roadway Impacts	Traffic Flow Impacts	Operational Impacts
Air temperature and humidity	N/A	N/A	• Road treatment strategy (e.g., snow and ice control) • Construction planning (e.g., paving and striping)
Wind speed	• Visibility distance (due to blowing snow or dust) • Lane obstruction (due to wind-blown snow or debris)	• Traffic speed • Travel time delay • Crash risk	• Vehicle performance (e.g., stability) • Access control (e.g., restrict vehicle type, close road) • Evacuation decision support
Precipitation (type, rate, start/end times)	• Visibility distance • Pavement friction • Lane obstruction	• Roadway capacity • Traffic speed • Travel time delay • Crash risk	• Vehicle performance (e.g., traction) • Driver capabilities/behavior • Road treatment strategy • Traffic signal timing • Speed limit control • Evacuation decision support • Institutional coordination
Fog (smoke and dust also limit visibility in some parts of Oregon)	• Visibility distance	• Traffic speed • Speed variance • Travel time delay • Crash risk	• Driver capabilities/behavior • Road treatment strategy • Access control • Speed limit control
Pavement temperature	• Infrastructure design	N/A	• Road treatment strategy
Pavement condition	• Pavement friction • Infrastructure damage	• Roadway capacity • Traffic speed • Travel time delay • Crash risk	• Vehicle performance • Driver capabilities/behavior (e.g., route choice) • Road treatment strategy • Traffic signal timing • Speed limit control
Water level	• Lane submersion	• Traffic speed • Travel time delay • Crash risk	• Access control • Evacuation decision support • Institutional coordination

The first step in setting up a road weather operations strategy is to determine the adverse weather situation that needs to be addressed. A corresponding strategy can be selected to reduce or eliminate the impacts of the adverse road weather condition.

Road weather operations strategies are typically separated into three main categories:

- Advisory strategies aim to inform users and managers of a facility of current and future road weather conditions. These strategies may be directed to users of a system by way of radio, variable message signs, social media, or the Internet.
- Control strategies actively manage a facility to restrict, permit, or regulate operations based on weather conditions. Control strategies can be in the form of variable speeds, highway closures, chain requirements (chain-up laws, chain-up areas, and checkpoints, or altered traffic signal timing.
- Treatment strategies aim to reduce the impact on weather events to users or maintainers of a roadway or facility. In a winter weather situation, this might be by plowing the roadway, applying sand or salt to a highway, or applying liquid deicer during icy conditions. It may also include asset tracking such as snow plow tracking that may also be used to provide traveler information under advisory strategies.

FHWA has compiled a list of road weather management best practices used throughout various states called the Road Weather Management (RWM) Best Practice Library (BPL). Version 3 of the BPL was released in 2012 that provides 27 innovative mitigation strategies and practices for varying adverse weather situations. The most recent version of the RWM BPL can be found at https://ops.fhwa.dot.gov/weather/mitigating_impacts/best_practices.htm.

Some strategies, like variable speed control, variable message signs, and altered traffic signal timing require new or existing infrastructure to be in place to support their operation. Having reliable power and communications infrastructure, especially during inclement weather, is necessary for their operation. Other strategies require a support structure to facilitate their operations. This can be in the form of social media like Twitter or updated traveler information via the Internet. ODOT operates TripCheck (https://tripcheck.com) that provides up-to-date information and camera images of road weather travel conditions in Oregon.

Road Weather Operations Analysis Tools

The analysis tools described in Section 18.3.6 may be considered for analyzing the potential effectiveness of road weather operations strategies. In particular, the TOPS-BC tool provides sketch level planning support for road weather operations and the crash modifications factors clearinghouse also provides guidance. Crash analysis (see Chapter 4) can also be used to help identify crash rates where there is a correlation to the weather. Analysis of maintenance logs and other agency asset tracking tools can be done to analyze how and where maintenance resources are allocated to help determine locations or activities that may benefit from road weather strategies.

Factors to consider when evaluating the applicability of road weather operations strategies:

- Known locations that impact travel during adverse weather conditions (e.g. mountain passes, low water crossings, bridges with high winds)
- Crash data with a correlation to weather conditions
- Weather conditions (weather service information, data from road weather information systems)
- Maintenance resource allocation (e.g. locations, activity types, frequency)

18.5.4 Work Zone Management

Work zone management includes all policies and practices related to minimizing travel delays and maintaining the safety of all travelers and workers from construction or maintenance work zones and associated detour routes. This section provides an overview of TSMO strategies for work zone management, evaluation considerations, and a summary of tools for work zone analysis. FHWA's Work Zone Mobility and Safety Program website provides a wealth of information and resources to support work zone management: https://ops.fhwa.dot.gov/wz/index.asp. ODOT also provides work zone management resources:

- Work Zone Safety: https://www.oregon.gov/ODOT/Safety/Pages/Work-Zone.aspx
- Traffic Control Plans Unit: https://www.oregon.gov/ODOT/Engineering/Pages/Work-Zone.aspx

The most comprehensive set of work zone management strategies is included in FHWA's Work Zone Operations Best Practices Guidebook, 3rd edition, at https://ops.fhwa.dot.gov/wz/practices/best/bestpractices.htm, which covers approximately 40 topics and numerous subtopics grouped in these overarching categories:

Policy and Procedures

- Public Relations, Education, and Outreach (Program-Level)
- Modeling and Impact Analysis
- Planning and Programming
- Project Development and Design
- Contracting and Bidding Procedures
- Construction/Maintenance Materials, Methods, Practices, and Specifications
- Traveler and Traffic Information (Project Related)
- Enforcement
- ITS and Innovative Technology
- Evaluation and Feedback

This guidebook includes 12 best practices from Oregon such as 20-minute maximum delay specifications, media and public outreach, and work zone incident management.

Some of the most commonly used TSMO strategies for work zone management include closure policies, stakeholder coordination and planning, public outreach and traveler information, work zone incident management, dynamic warning systems, and traffic control as described in the following sub-sections.

Closure Policies

Minimizing closures of lanes, ramps, and roadways during peak travel times goes a long way towards maintaining travel mobility. All travel modes should be considered when developing closure policies. For example, freeway work zone projects should consider Interstate design vehicles, over-dimension vehicles, and bus routes. Alternatively, an urban work zone projects should consider accommodations for pedestrians, bicycles, transit, passenger vehicles, and the appropriate types of freight vehicles.

ODOT has a standing policy for all freight routes that any lane closure that is expected to cause traffic delays triggers a mandatory coordination process with mobility stakeholders. For this purpose, delay is expected when the hourly volumes during construction exceed an established free-flow threshold. These thresholds are defined in the corridor-specific Traffic Management Plans that were established during the OTIA-III bridge rehabilitation efforts.

The work zone analysis tools discussed later in this subsection provide input on procedures for analyzing closures.

Stakeholder Coordination and Planning

Planning efforts for work zone management should include a variety of stakeholders such as transportation agencies, construction management divisions within agencies, contractors, transit agencies, traffic management centers, 911 centers, law enforcement, fire and rescue agencies, emergency medical agencies, utility agencies/companies, area businesses, freight community, freight distribution centers, event centers, school districts/school bus operators, and neighborhood associations. Transportation Management Plans (TMPs) can be used to identify strategies for work zone management. TMPs are required for significant federally funded projects but can also be scaled to any size project. ODOT uses three levels of TMPs: program-level (overarching statewide safety and mobility policies), corridor-level (targeted for high-volume freight and passenger travel routes), and project-level (as needed for individual projects). See the FHWA and ODOT TMP websites for more information: https://ops.fhwa.dot.gov/wz/resources/tmp_factsheet.htm and https://www.oregon.gov/odot/Engineering/Pages/Work-Zone.aspx.

ODOT's TripCheck Local Entry (TLE) feature allows transportation agencies within Oregon to share information about construction and maintenance projects between one another. This system also feeds pertinent traveler information to the media and the public. ODOT strongly recommends all transportation agencies in Oregon actively use TLE. Contact ODOT to get started: https://www.tripcheck.com/Pages/ATCU.asp.

Public Outreach and Traveler Information

Key components of successful work zone management include public outreach during design, public outreach immediately preceding work zone activities, and traveler information while construction is underway. Public outreach can be done through many forums such as workshops, websites, social media, and partnerships with the media. Traveler information systems (see Section 18.5.6) can be used to provide work zone schedules, travel impacts, real-time or predictive travel times through work zones or alternate routes, and estimated delay.

Work Zone Traffic Incident Management

Traffic incident management (TIM) strategies (see Section 18.5.1) can be applied to specific work zones, particularly for longer-duration construction projects. This may include TIM teams for the work zone as well as incident detection sensors and camera surveillance to quickly identify incidents so a coordinated response can be initiated. Although TIM teams have not specifically been established for Oregon construction projects to date, ODOT does work closely with construction teams and is available to provide incident response as needed.

Some ODOT projects include a full-time traffic control supervisor to periodically patrol the work zone (mostly to supervise the temporary traffic control procedures) and who can also be on-call 24 hours per day. This traffic control supervisor coordinates with ODOT incident responders as needed. Some ODOT regions pre-position tow vehicles in or near work zones to support incident management and have maintenance personnel work overtime to support incident management.

Some states, such as Utah, are experimenting with performance incentives for keeping traffic moving safely and efficiently. This includes incentives for quick clearance of incidents.

Dynamic Warning Systems

A variety of dynamic warning systems can be used to detect existing traffic, vehicle, or road weather conditions and provide warnings via roadside dynamic message signs or emerging connected vehicle technologies:

- Queue or congestion warning
- Over dimension vehicles

- Workspace intrusion
- Construction vehicle activities (e.g. merging, crossing, exiting)
- Hazardous road weather conditions (e.g. ice/water on road, visibility)
- Geometric conditions (e.g. curves)

Traffic Control
Beyond traditional work zone traffic control measures, these strategies may also be considered:

- Speed management (e.g. driver feedback speed advisory signs, variable speed control, automated speed enforcement)
- Dynamic merge control (see Section 18.4.2)
- Temporary ramp metering
- Adaptive or traffic responsive temporary traffic signals

ODOT has developed a mobile speed management system for stationary and moving maintenance activities that uses driver feedback speed advisory signs. Future applications aimed at improving worker safety include applications such as automated truck-mounted impact attenuators.

A variety of factors should be considered when evaluating what strategies to apply to a work zone:

- Planned construction/maintenance activities and duration:
    - For the work zone under evaluation
    - For any nearby projects that impact area mobility
    - For any projects on the same corridor that impact regional or statewide mobility
- Transportation conditions of the work zone roadways and areas of influence (e.g. parallel routes where travelers may detour):
    - Average daily traffic (including freight presence)
    - Seasonal traffic variations/factors
    - Posted speed
    - Peak period travel speeds
    - Travel time reliability
    - Average daily hours of congestion
    - Intersection traffic control devices (e.g. stop signs, traffic signals)
- Existing systematic or local TSMO strategies in place (e.g. TIM program, roadside traveler information, dynamic speed or warning systems)

Work Zone Analysis Tools

This section provides a summary of work zone analysis tools that can be used at the program level (SWIM2, WISE, CA4PRS, dynamic traffic assignment) or when analyzing specific projects (ODOT Work Zone Analysis Tool, Highway Segment Analysis, QuickZone).

Statewide Integrated Model, 2nd Generation (SWIM2)

SWIM2 is an Oregon model that allows testing of regional and statewide policies and potential projects to inform decision makers on the complex interactions between land use, the transportation network, and the economy. The SWIM2 model can be used for a bigger picture evaluation of the impacts of significant work zones and other closures. A work zone example is provided in Chapter 7 along with more details about the SWIM2 model. Additional SWIM2 information is available at https://www.oregon.gov/ODOT/Planning/Pages/Technical-Tools.aspx#SWIM. Contact TPAU for SWIM2 analysis applicability and requests.

Work Zone Impact and Strategy Estimator (WISE)

WISE was created by FHWA to support work zone planning and scheduling at the regional program level to minimize delays to the traveling public and costs to the agency. The WISE tool builds on existing traffic simulation software used by transportation agencies and MPOs. It uses basic network geometry (link/node and number of lanes) and basic traffic volume information along with a user-defined library of demand-based and duration-based strategies to determine project sequencing based on user and agency costs. Optimized project sequencing can be developed using the Operation Module, which uses TransModeler from the DynusT dynamic traffic assignment model. More information on WISE is available at https://www.fhwa.dot.gov/goshrp2/Solutions/Reliability/R11/WISE_Work_Zone_Impacts_and_Strategies_Estimator_Software

Construction Analysis for Pavement Rehabilitation Strategies (CA4PRS)

Caltrans developed CA4PRS to help state transportation agencies and paving contractors develop construction schedules that minimize traffic delay, extend pavement service life, and reduce agency costs. The tool takes into consideration project alternatives for different pavement design, construction logistics, and traffic operations options. More information on CA4PRS is available at https://trid.trb.org/view/1302459

Dynamic Traffic Assignment

Some dynamic traffic assignment programs (see Sections 9.5 and 18.4.5) can be used to support decision making for regional work zone management.

ODOT Work Zone Analysis Tool

ODOT has developed a Work Zone Traffic Analysis Tool and User's Guide to determine lane closure restriction recommendations and construction delay estimates for ODOT roadways. It also includes a section for analyzing non-ODOT facilities. This tool determines when highway segment lane closures should be allowed based on traffic volumes (see Chapter 5 for developing design volumes) and free flow thresholds. Lane closures are discouraged when traffic volumes exceed free flow thresholds. Note- this tool is not designed to analyze intersection operations. The User's Guide is available at https://www.oregon.gov/ODOT/Engineering/Pages/Work-Zone.aspx

The tool can also be used to estimate delay, or the average additional travel time as a result of the work zone activities. These estimates are based on traffic microsimulation tools and regression analysis. The estimated delays can be used to help determine work zone staging and detours. ODOT has a maximum delay standard of 20 minutes. Exceptions are sometimes granted for exceeding the delay threshold.

Highway Segment Analysis

Chapter 6 includes analysis procedures for highway segments, ramps/ramp junctions, merging, diverging, weaving, and passing and climbing lanes that can be used for analysis of work zones as applicable.

QuickZone

QuickZone is a spreadsheet-based traffic analysis tool that compares the traffic impacts of work zone management strategies (e.g. project phasing, diversions, travel demand measures) for both urban and rural work zones. It looks at each strategy's impact on traffic delays, potential queues, and costs. QuickZone is available at https://store.mctrans.ce.ufl.edu/quickzone.

18.5.5 Planned Special Event Management

Planned special events can have major impacts on travel based on when and where the event is held, how many people are in attendance, and what travel modes are available. This section describes special event management, the TSMO strategies that may be applied, and the evaluation considerations.

The list of special events held in Oregon is a long one but typically includes happenings such as sporting events, concerts, conventions, fairs, festivals, motorcades/parades, public/political events, or heavy shopping days (e.g. Thanksgiving through Christmas). Venues can be as varied as arenas, stadiums, theaters, fairgrounds, recreational facilities, public open spaces, and public roadways closed to vehicular traffic. Event frequency also varies. Some events are one-off events (e.g. political rallies) while others occur annually (e.g. state and county fairs), seasonally (e.g. sporting events), or frequently (e.g. venues that hold a variety of events year-round).

Special event management is the application of coordinated operations strategies to inform the traveling public about travel conditions, monitor changing travel conditions, and manage travel demand associated with the planned special event. The level of effort from both a staffing and investment standpoint varies and can be scaled to the level of traffic impacts. For instance, a venue with year-round events may require more capital investments in parking systems, transit infrastructure, and traffic signal operations whereas smaller, less frequent events may have more of a focus on traveler information and smaller day-of-event traffic control (e.g. police on hand directing traffic).

The evaluation of special event management is similar to analyzing TSMO strategies for a sub-region with the primary difference being the origin-destination pairs and the time-of-day/day-of-week. A typical sub-region analysis uses home-employment origin-destination pairs whereas special events have a venue as the destination and the origin depends on when the event is held. Special event analysis also varies by both time-of-day and day-of-week whereas sub-region analysis focuses on the weekday AM and PM peak commute periods. Any of the tools in Section 18.3.6 used for TSMO evaluation can be applied to special event management. Often existing models (e.g. travel demand, simulation, traffic signal optimization) have already been developed for the area surrounding a venue for weekday commute conditions and these models can be tailored to reflect special event conditions.

The evaluation of special event management should take into consideration:

- Venue size and typical attendance
- Time-of-day and day-of-week of planned special event
- Mode split of available travel options
- Capacity impacts to roadways and transit systems for event ingress and egress
- Capacity impacts to venue parking facilities and nearby parking facilities (on-street, public facilities, and privately operated facilities)
- Impacts to traffic signals on event travel routes
- Availability of existing TSMO strategies (e.g. traveler information, surveillance systems, traffic signal systems, dynamic traffic control)
- Staff availability for events during non-standard work hours

TSMO strategies that may be considered for special event management include the following:

Stakeholder Coordination

Coordination between all applicable stakeholders can greatly improve the management of special event travel demand. Stakeholders may include transportation agencies, transit agencies, law enforcement, 911 centers, special event promoters, venue management teams, parking facility operators, media, and any sectors of the public affected by event traffic. Face-to-face meetings to coordinate in advance of the event are key and a post-event meeting to review best practices and areas for improvement is helpful for events held with some regularity. Staffing considerations are also important for all affected agencies since many events are held outside non-standard hours and special accommodations may need to be made to allow for staff support.

Temporary Traffic Control

Several temporary traffic control strategies may be considered:

- Temporary traffic control devices (e.g. temporary traffic signals, portable VMS, tubular markers, barricades) may be used to modify roadway traffic control near an event venue.
- Flaggers or law enforcement personnel may be used to help direct traffic at bottlenecks.
- Detours around the event venue may be used for commercial vehicles and other non-event-related traffic.
- The standard ODOT special provisions boilerplates provide a place for construction projects to list special events that will prohibit the use of lane closures or sidewalk closures.

Surveillance

Existing and temporary systems may be used to monitor travel conditions before, during, and after a planned special event:

- Vehicle detection systems
- CCTV cameras
- Event service patrol- by foot, vehicle, or aerial surveillance

Consider making surveillance video and data available in a central location for all stakeholders.

Traffic Management Center (TMC)

An existing TMC, a satellite TMC (e.g. room at the venue), or portable TMC (e.g. traveling vehicle set up with TMC systems) may be used to manage operations for an event. This may require additional staff or the use of existing staff after hours.

Traffic Signal Operations

Many of the traffic signal strategies discussed in Section 18.4.1 may be considered for special events. Special timing plans may be developed to handle event traffic that may be turned on manually from a central signal system or enabled using a trigger, such as volume thresholds from a system detector.

Dynamic Traffic Control

A variety of dynamic traffic control may be considered:

- Reversible lanes or changeable lane assignment on corridors with a dominant travel direction during event ingress or egress
- Dynamic trailblazer signs to route travelers to and from an event

- Existing dynamic traffic control already in place near the event such as variable speed control, adaptive ramp metering, dynamic merge control, dynamic warning systems, and dynamic re-routing

Transit Management

For less frequent events at venues without transit infrastructure, transit shuttle service may be considered. This service could use existing park-and-ride lots or new ones could be negotiated at public or private parking facilities, including retailers with large, under-utilized parking lots. Shuttle stops or dedicated transit lanes should be considered at the venue. For venues with year-round events, permanent transit infrastructure (e.g. bus routes and stops) may be considered. Service frequency may need to be increased before and after an event to handle demand.

Parking Management

On-site and off-site parking capacity should be evaluated to determine what is needed to meet parking demand for motor vehicles and bicycles. Parking traveler information and electronic payment systems help make parking operations more efficient. Active parking management strategies (see Section 18.4.3) may also be considered: dynamic overflow transit parking, dynamic parking reservation, dynamic wayfinding, and dynamically priced parking.

Traveler Information

All available traveler information systems (see Section 18.5.6), including permanent and portable roadside VMS, should be used to provide travel impacts in advance of special events as well as to provide real-time travel conditions on the day of the event. This also includes close coordination with the media.

18.5.6 Traveler Information

Traveler information systems provide relevant information on travel options and conditions before and during travel. The intent of these services is to give travelers actionable information that can change their behavior in ways that improve the efficiency of the transportation system.[35] Traveler information strategies support many of the TSMO strategies described in this chapter, particularly the transportation demand management strategies (also called transportation options) discussed in Section 18.5.7. An overview of the types of information delivered to travelers and the various media used to do so is presented below. A discussion of considerations for implementation, and summaries of safety benefits and operational impacts of traveler information systems

[35] Chorus, C.G., Molin, E.J. and Van Wee, B., 2006. Use and effects of Advanced Traveler Information Services (ATIS): a review of the literature. *Transport Reviews*, *26*(2), pp.127-149.

follows the overview. Additional information on traveler information systems is available at:

- FHWA Real-Time Travel Information website: https://ops.fhwa.dot.gov/travelinfo/
- NCHRP Web-only Document 192: https://www.trb.org/Publications/Blurbs/168370.aspx
- NCHRP Synthesis 399: https://www.trb.org/Publications/Blurbs/161865.aspx

Overview of Information Types and Media

Common types of traveler information include:

- Alternate routes can aid travelers in making real-time route choices. Alternate routes go together with construction zones, special events, traffic incidents, abnormal travel times, and adverse weather.
- Congestion information can notify travelers of levels of congestion relative to normal traffic, the lanes affected, and congested roadway segments.
- Live traffic cameras record traffic conditions visually at specific camera locations. They allow travelers to see roadway conditions and make decisions accordingly.
- Parking availability information allows drivers to save time and fuel by knowing where parking options are available. It is useful primarily in dense urban areas and parking garages.
- Public safety information can include Amber alerts, Silver alerts, and evacuation information. These types of information prompt travelers to attend to or do something, for example look for a certain type of vehicle from an Amber alert.
- Road work/construction zones allow travelers to change plans by avoiding a particular area or allowing for extra travel time. Examples include locations of tree trimming, bridge closures, lane closures, and construction activities.
- Special events can disrupt normal traffic. Large events can cause road closures, traffic diversions, traffic volume changes, and changes to parking options. Recurring events like sports games can have similar effects as well. Information pre-trip and during trips can help travelers make according travel decisions.
- Traffic incident information includes incident type, times of occurrence, location, number of lanes affected, and when lanes will reopen.
- Travel times aid travelers in route, departure time, and mode decisions prior to a trip. During a trip, information on travel times to certain points can help travelers with route choices.
- Weather information includes severity of weather conditions, road closures, alternate routes, evacuation routes, and advisories to travelers (e.g. snow chains required).

Traveler information is disseminated in many ways. Some information sources are more established and readily available (e.g. radio, television) while others are newer and less widely available (e.g. smartphone apps). Sources vary in their ability to reach travelers as well. Sources like radio and television may provide information passively; travelers receive information relevant to their trip while not actively seeking it. Other sources, like websites and apps, require the user to actively seek out information. Technology innovations continue to expand the sources and flow of information. Many common information sources are described below.

- 511 phone systems allow users to simply dial 5-1-1 on the phone to hear travel information. In Oregon, most of this information is the same as displayed on TripCheck and includes roadway conditions by highway, roadway conditions for mountain passes, roadway conditions for major cities, commercial vehicle restrictions, and chain requirements.
- Mobile smartphone apps can provide a wide range of information to users to help trip planning and decision making. There is a trend toward agencies allowing private developers to access real time data to develop apps and solutions. ODOT offers the TripCheck Traveler Information Portal (TTIP) for this purpose. Example applications include Waze providing incident and detour information, Google Maps providing travel times and trip planning options, and Pango providing parking information and reservations. Smartphone technology continues to evolve: hardware improvements enhance their capabilities, and software improvements in non-transportation apps like calendars and scheduling create opportunities for traveler information to integrate with these products.
- On-board devices like TomTom, Garmin, and OnStar provide information such as alternate routes and congestion. Some of these services require a paid subscription.
- Radio is available in almost every vehicle and in some areas has more reliable coverage than cellular networks. Many radio stations broadcast travel information like closures, delays, emergencies, and adverse weather as part of news content. ODOT also utilizes Highway Advisory Radio (HAR) to broadcast warnings, advisories, directions, and other non-commercial information of importance to motorists on dedicated radio stations. For example, HAR messages are intended to be under one minute in duration and repeated continuously so that travelers can hear the message at least twice while passing through a station's coverage area.
- Social media provides agencies with a venue to interact with travelers directly. The ODOT twitter feed (@OregonDOT) posts updates on advisories, delays, incidents, weather, road closures, and photos. There are also several TripCheck Twitter accounts that provide similar information for certain corridors (e.g. @TripCheckUS97A). Social media also allows user-to-user exchanges of traveler information.
- Subscription-based email and short message service (SMS) messaging services allow users to receive email or text messages about weather and roadway

conditions. The Amber alert and Silver alert systems use SMS messaging to disseminate information.

- Television is similar to radio in that it is widely accessible and travel information is broadcast as part of regular news coverage. But, it is limited to providing pre-trip information.
- Websites, from both public agencies and private entities, can provide every type of travel information listed above. ODOT's award-winning TripCheck website (https://www.tripcheck.com) provides summarized information by roadway and by category, real-time photos from CCTV cameras, and offers an interactive map-based platform for users to retrieve data. The TripCheck Local Entry (TLE) tool allows local agencies to input information about their roadway network to the TripCheck system. TripCheck Mobile (https://www.tripcheck.com/mobile) also provides a platform optimized for low-bandwidth and mobile phone users. TripCheck TV (https://www.tripcheck.com/tv/) provides camera images, alerts, and delays and is designed for display on televisions at public locations where travelers are waiting such as lobby areas and transit stops. Data collected on roadways are sent to the TripCheck system, and is then shared via TTIP with other information outlets like 511, the ODOT Twitter feed, and smartphone apps.
- Variable message signs (VMS) are traffic control devices along roadways that display dynamic messages containing traveler information to motorists. VMSs can be temporary (located on trucks or trailers parked along the side of roadways) or permanent (affixed to permanent supports or bridge structures). Along with variable advisory speed signs and travel time signs, permanent VMSs comprise the ODOT Real Time system. VMSs should be placed strategically at locations that allow motorists to change travel plans depending on the message, where there is sufficient sight distance, and where access to power and communications is available. ODOT provides recommendations for VMSs in the Guidelines for the Operation of Permanent Variable Message Signs document: https://www.oregon.gov/ODOT/Engineering/Docs_TrafficEng/PCMS-Handbook.pdf Connected vehicle technology is an emerging area for traveler information. As vehicle-to-vehicle and vehicle-to-infrastructure technology matures, opportunities for in-dashboard and in-vehicle travel information alerts will arise.

Considerations for Effective Deployment

NCHRP Web-only Document 192 (available at http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_w192.pdf) presents results of a study on the use and deployment of traveler information systems across six regions of the US. The study provides many recommendations for successful implementation of traveler information systems. Primarily, traveler information "should be targeted, easy to use, relevant, clear, trustworthy, reliable, and accurate." The report emphasizes four main features for successful deployment:

1. Provision of information on non-recurring events in real time. Non-recurring events like traffic incidents and special events disrupt the transportation system

substantially, and providing relevant information can lessen their negative impacts.

2. Dissemination across a wide array of media. Travelers have varying levels of access to information and needs, so providing many options to obtain information is beneficial. A central agency need not control all information streams, and information sharing with private entities and third parties can aid in circulation. ODOT's TripCheck Traveler Information Portal (TTIP) is a prime example of information sharing in action.

3. Aligning information with the needs and wants of the traveling public. Information providers must be aware that different travelers have different needs and wants. Trip purposes (e.g. commuting, business travel to a meeting, leisure, etc.), departure/arrival time constraints (e.g. the need to travel during peak hours), familiarity with the geographic area of travel, mode options, and market penetration of smartphones are all examples of factors that affect wants and needs for information of travelers.[36]

4. Evaluation. Traveler information programs should be evaluated to ensure they are producing desired and intended results. Data collection efforts like surveys, usage statistics, and focus groups can inform agencies about effectiveness of traveler information systems. However, a critical first step is to define performance measures that can assess whether traveler information systems meet goals.

Traveler Information Analysis Tools

The benefits of deployment of traveler information systems are well documented. The sketch planning tools described in Section 18.3.6 may be used to screen traveler information strategies. In particular, the USDOT ITS Knowledge Resources website is a repository of benefit summaries from traveler information systems. Case studies are organized by types of information sources and whether information is related to pre-trip information, en-route information, tourism, and special events. The TOPS-BC tool also provides analysis of pre-trip and en-route traveler information.

18.5.7 Transportation Demand Management

Transportation demand management (TDM), also called transportation options, includes actions or programs aimed at reducing the motor vehicle demand for transportation infrastructure, which often improves the efficiency of transportation systems. Example TDM strategies include rideshare programs, transit fare discount programs, telecommuting, and individualized trip planning. TDM strategies have long been a part of transportation planning in Oregon. Goal 2 of the *Oregon Transportation Plan* is "to improve the efficiency of the transportation system by optimizing the existing transportation infrastructure capacity with improved operations and management." The target strategies for achieving this goal come from TDM and transportation system management (TSM). The Oregon Transportation Planning Rule (OAR 660-012) requires urban areas with a population greater than 25,000 people to evaluate TDM and TSM

[36] Chorus, C.G., Molin, E.J. and Van Wee, B., 2006. Use and effects of Advanced Traveler Information Services (ATIS): a review of the literature. *Transport Reviews*, *26*(2), pp.127-149.

strategies as part of their transportation system planning efforts. It also encourages urban fringe areas to consider low-cost TDM and TSM strategies as applicable. The Oregon Transportation Options Plan provides an overview of existing transportation options programs in use in Oregon, challenges/trends/opportunities, and policies and strategies for 10 goals to support the state's vision for transportation options. It also includes an implementation section with guidance for integrating transportation options into the planning process.

Additional information and a comprehensive description of the many available TDM strategies may be found at:

- Oregon Transportation Options Plan: https://www.oregon.gov/ODOT/Planning/Pages/Plans.aspx#OTOP
- FHWA Integrating Demand Management into the Transportation Planning Process: A Desk Reference: https://ops.fhwa.dot.gov/plan4ops/trans_demand.htm
- Victoria Transport Policy Institute's Online TDM Encyclopedia: http://www.vtpi.org/tdm/

The full menu of available TDM strategies is too long to list here. TDM strategies can generally be grouped as:

- Physical measures (e.g. transit or parking infrastructure)
- Operational measures (e.g. active transportation and demand management, see Section 18.4)
- Institutional measures (e.g. sustainable travel planning)
- Financial/pricing measures (e.g. congestion or parking pricing).

TDM strategies can also be grouped by type of travel choice: mode, time, location, and route. Exhibit 18-28 provides an overview of the effectiveness of the most common TDM strategies in addressing key policy objectives (mobility, congestion relief, air quality, economic development, land use interaction, goods movement, and livability). Appendix 18A includes some of the high-level TDM strategies most commonly used in Oregon. Marketing and outreach to individual travelers, businesses, and transportation agencies are all typically part of successful TDM programs.

Exhibit 18-28: TDM Strategies and Their Relative Effectiveness in Addressing Key Policy Objectives

Strategies	Mobility	Congestion Relief	Air Quality	Economic Development	Land Use Interaction	Goods Movement	Livability
Traditional TDM							
HOV/HOT Managed Lanes	◐	○	○	○	◔	◔	◔
Employer Trip Reduction Programs	◐	◔	◐	x	-	◔	◔

Strategies	Mobility	Congestion Relief	Air Quality	Economic Development	Land Use Interaction	Goods Movement	Livability
Alternative Work Arrangements	◐	◔	◐	◔	◔	◔	●
Individualized Marketing*	◐	◐	◔	◔	-	◔	◔
School-Based Trip Reduction	◐	◔	◐	○	○	-	●
Event-Based Trip Reduction	◐	◔	◔	○	○	◔	◔
Recreation-Based Trip Reduction	◐	◔	◔	○	○	◔	◔
Car-Sharing	◐	x	◐	◔	◔	-	◐
Ridesharing*	◐	x	◐	◔	◔	-	◐
Vanpool Programs	◔	◔	●	◔	○	◔	◔
Land Use/Active Transportation							
Developer Trip Reduction	◔	◔	◔	x	◐	-	◔
Land Use Strategies	◐	◔	◔	◔	◐	-	●
Access Management*	◐	x	x	◔	◐	◔	◔
Car-Free or Access-Restricted Zones	◔	◐	◐	x	◐	◔	●
Bicycle Facilities and Programs	◐	◔	◐	◔	◐	○	●
Pedestrian Facilities and Continuity	◐	◔	◐	◔	◐	○	●
Transit							
Transit Service Improvements	◔	◔	◐	○	◔	○	◔
Transit Prioritization/ BRT	◐	◔	◐	◔	◔	x	◔
Transit Fare Discounts	◔	◔	◐	◔	◔	x	◔
Parking							
Parking Information	◐	◔	◔	◔	◔	◔	◔
Parking Supply Management	x	◐	◐	x	◐	-	◐
Parking Pricing	x	◔	◐	x	◐	◔	◐
Pricing							

Strategies	Mobility	Congestion Relief	Air Quality	Economic Development	Land Use Interaction	Goods Movement	Livability
Cordon Pricing	x	●	●	○	◔	◐	○
Congestion Pricing	x	●	●	○	◔	◐	○
General Financial Incentives	●	◔	●	◐	◔	◐	◔
Vehicle Miles Traveled (VMT) Tax	x	●	●	x	◐	◐	○
Systems Management							
Traffic Signal Optimization*	◐	◐	◔	x	-	◔	x
Ramp Metering	x	◐	x	-	-	◔	x
Traffic Incident Management*	◐	◐	◔	-	-	◔	x
Integrated Corridor Management	◔	◔	○	○	-	◐	-
Traveler Information	◐	◐	◔	◔	-	◔	◔
Eco-Driving	-	x	◔	x	x	◐	◐

Key: ● = highly effective; ◐ = moderately effective; ◔ = nominally effective; ○ = likely effective (but undocumented);

x = minimal to no impacts; - = not applicable

Source: *Integrating Demand Management into the Transportation Planning Process: A Desk Reference*. FHWA-HOP-12-035. Table 10.6

* Strategy was not evaluated in the FHWA Desk Reference but is commonly used in Oregon.

The evaluation of TDM strategies should take into consideration:

- Mode split
- Person throughput on key facilities or corridors
- Ratio of travel times on all travel options to one another
- Vehicle miles traveled and vehicle trip reduction
- Emission reduction
- Transit ridership
- Rideshare program usage
- Parking utilization
- Breakdown of employment by location
- Enrollment in employee incentive programs
- Demand estimates based on parking or tolling costs
- Satisfaction with travel options and incentives to use them

TDM Analysis Tools

The typical tools for analyzing TSMO strategies discussed in Section 18.3.6 have limited ability to evaluate TDM strategies. Emerging TDM strategies, such as rideshare programs, may have a big impact on transportation demand but the extent of that impact are still unknown.

Tools that may be used to evaluate TDM:

- Travel Demand Models: Some planners in Oregon apply trip reduction factors in travel demand models based on data from the Oregon Department of Environmental Quality's *Guidance for Estimating Trip Reductions from Commute Options* (1996) to represent TDM strategies.
- Trip Generation Rates: Section 6.6 of this APM talks about incorporating TDM into trip generation rates. Sections 9.4.3 and 10.3.2 discuss considering TDM strategies as a part of the future transportation alternatives analysis.
- Regional Strategic Planning Model (RSPM) (See Sections 7.6 and 18.3.6): The RSPM may be used as a screening tool to understand the tradeoffs and first order average day impacts of non-auto modes.
- Worksite Trip Reduction Model (WTRM) (https://rosap.ntl.bts.gov/view/dot/34140): This sketch-planning tool uses either an online worksheet or look-up tables to predict the changes in vehicle trip rates or parking rates for TDM strategies.
- EPA COMMUTER Model: This basic computer model is 20 years old and calculates the transportation and emissions benefits based on TDM incentives (e.g. public transportation service improvements) and disincentives (e.g. parking charges). This program relies heavily on professional judgment.
- Trip Reduction Impacts of Mobility Management Strategies (TRIMMS) (http://trimms.com/): This tool is a visual basic application spreadsheet that estimates the impacts of TDM strategies, including their benefit-cost ratios.

18.5.8 Connected and Automated Vehicles

Rapidly emerging connected and automated vehicle technologies are posed to disrupt long-standing practices in planning, designing, maintaining and operating the transportation system. While ODOT is taking initial steps to understand and prepare for the coming changes, the state of the practice for analysis procedures related to this new transportation paradigm is thin. This sub-section provides an overview of Connected and Automated Vehicles (CAV) as currently understood. See Appendix 6B for more information.

Connected vehicles include on-board technology that enables communication with their environment using a variety of different wireless communication technologies as appropriate to the level of communication needed. Depending on the application, dedicated short range communication (DSRC), cellular, or Wi-Fi communications may be appropriate. Connected vehicles can "talk" to:

- Other vehicles on the road, termed Vehicle to Vehicle (V2V)

- Roadside infrastructure, like traffic signals, termed Vehicle to Infrastructure (V2I)
- Everything else, like mobile devices, termed Vehicle to Everything (V2X)

CAV aim to benefit transportation in three areas:

- Safety: CAV may reduce crashes and injuries by providing tools to drivers and vehicles to better anticipate and prevent a crash, or if a crash is imminent, the technology could provide adjustments to minimize injuries.
- Mobility: CAV technology can provide information to transportation system users and operators that allow them to make informed choices and reduce travel delay. Automated vehicles may open more mobility options for disabled and aging populations, and expand last mile connections for transit. The automated parking feature may reduce the number of vehicles circulating in search of parking and potentially decrease the allocation of space for vehicle parking.
- Environment: CAV technology can enable drivers (and vehicles) to use real time information to operate more efficiently to reduce energy use and emissions.

These are developing technologies, and although the systems are not yet mature, agencies should begin preparing for the impacts of CAV technologies on the transportation system. Ways to prepare include but are not limited to:

- Assessing the impacts on planning practices and processes:
    - How will travel demand models consider CAV effects?
    - How will longer term road capacity needs be re-assessed?
    - What are the effects to funding and programming?
    - What are the equity impacts of CAVs?
    - How will vehicle ownership patterns change?
- Assessing impacts on infrastructure
    - What are the impacts on signing and striping?
    - Do lane widths change?
    - What are the effects on traffic control systems?
    - What roadside infrastructure, including communications, is needed in the future?
    - How is the design and use of curb space impacted?
    - How are transit, bicycles and pedestrian facilities impacted?
- Assessing the impacts on agency operations:
    - What workforce skills are needed?
    - How does this change licensing, regulations, and enforcement policies and procedures?
    - How are procurement processes updated to support new public-private partnerships?

Connected Vehicle Analysis Tools

USDOT is supporting research and developing guidance to help agencies consider and plan for connected and automated vehicles. The Connected Vehicle Reference Implementation Architecture (CVRIA) tool is directed at incorporating CAV into ITS architectures.[37] The tool includes information about the many CAV applications being developed such as eco-traffic signal timing, eco-multimodal real-time traveler information, and eco-smart parking. It identifies the various categories of information flows; standards necessary for implementation; and other resources to support integration into ITS architectures. USDOT has also supported the development of a software tool, Systems Engineering Tool for Intelligent Transportation (SET-IT)[38], which links drawing and database tools with the CVRIA to visualize the architecture elements and information flows.

Appendix 18A – Summary of TSMO Strategies

[37] https://local.iteris.com/cvria/html/about/about.html
[38] https://local.iteris.com/cvria/html/resources/tools.html

19 TRAFFIC ANALYSIS REPORTS

19.1 Purpose

Traffic analysis reports are a comprehensive explanation and accounting of the existing and future conditions or final recommendations and the decision-making processes for a project or plan. These reports can range from a technical memorandum describing conditions for a specific period or a single topic such as for micro-simulation calibration to a full traffic analysis narrative report on the entire analysis for a project. This chapter presents an overview of the basic elements that document the assumptions, methods, findings and recommendations of traffic analyses, report types, and reviewing analysis documentation of others. Topics covered include:

- Background
- Technical Memorandum
- Traffic Analysis Narrative Report
- Reviewing Analysis Documentation

19.2 Background

In many cases, the report text and associated diagrams are developed incrementally during the study process in the form of Technical Memorandums, and then circulated for review and discussion at key milestone points during the project review. Any revisions to the Technical Memorandums or new directions in the study analysis are carried forward and then compiled into a full Traffic Analysis Narrative Report (TANR) at the end stages of the study. The Final TANR serves as the legacy document for the study and must be comprehensive enough to explain and support the final recommendations and the decision-making process that led up to it.

19.2.1 Technical Writing Tips

Presentation of technical information in a clear, concise, and readily understandable way can be challenging in many regards. This section is not intended to fully answer those challenges, but to highlight several important tips that help to make a technical document achieve these goals. The document author is encouraged to avail themselves of training materials or mentors that could help them become proficient technical writers. A few basic tips to suggest in preparing any technical document include the following:

- **Target Audience**: The intended audience for the document will help to determine the appropriate level of assumed technical knowledge about the subject at hand, and their assumed understanding of the review, adoption, and implementation processes for a particular project. In general, most traffic reports will be developed for the review and implementation by staff within, or contracted by, ODOT. In general, these team members have minimal background in the technical traffic issues, but significant experience with the overall process involved. To this end, the technical aspects and outcomes of the project need to be clearly explained with a minimum of technical detail necessary to support and explain the traffic analysis. This is very important because writing at the wrong level can generate unintended questions. More extensive technical calculations, findings, software input/output reports, and other reference materials should be attached to the document as appendices.

In most cases a document is normally circulated to the general public, the press, or other outside agency. In these cases, many of these more fundamental assumptions and process steps should be clearly detailed in the document. t Creating an executive summary written in simpler or plainer language that can also be a standalone document makes it easier to facilitate the public consumption of the information (e.g. used as a handout for a public open house). Presentations to project stakeholder groups are generally handled like any general public group, with the focus on overall process, criteria, outcomes, recommendations and next steps, with a bare minimum of technical content.

- **Tone and Style**: It is recommended that the document, regardless of purpose or scope, in all cases, remain objective, impartial, and impersonal so that the results and conclusions are untainted by any biases. It should be recognized that any internal ODOT document may be released for public review outside of the designated committee groups. This typically occurs by informal sharing in the interest of coordination or, more formally, through a public records request. All documents should be treated as if the general public and press will review them, even though many only circulate to the immediate committee members.

- **Readability and Document Structure**: The following sections of this chapter have suggestions about the narrative (i.e. “storytelling”) general layout of the document, but these need to be tailored, as appropriate, to address individual study scopes and objectives. One of the keys for rapidly understanding materials is to divide the document into a logical, easy-to-follow flow of narrative text, summary tables and illustrations that are grouped according to key topics. In a TANR, for example, they would be grouped by chapter, or by sub-topic in a lengthier chapter. This basic structure provides a convenient framework for presenting and referencing a wide range of materials.

- **A Word About Acronyms**: A comprehensive list of acronyms used in transportation evaluations are assembled in the List of Abbreviations and Acronyms of this manual for reference purposes. Limit the number of acronyms, except for the most common ones that appear repeatedly throughout a particular document. The most common examples include: ODOT, v/c ratio, OHP and HDM. Excessive use of acronyms generally degrades the readability of the document, even when the reader understands their meaning. It is standard practice to introduce any acronym in the narrative when it is first used by defining it. In longer reports, it is also useful to attach a short list of all the acronyms used as a quick reference guide.

19.2.2 Diagrams and Illustrations (Figures)

Technical diagrams are a powerful resource for quickly explaining analysis assumptions, findings, and recommendations. One measure of a high-quality report allows readers to scan through the study tables and figures, and then be able to glean the general conclusions without reading any of the narrative text. For the purposes of traffic analysis reports, the technical diagrams include the following list of typical illustrations:

- Study area map
- Local street and highway system
- Traffic volumes on links or turning movements at intersections or junctions
- Intersection performance measures (e.g. v/c ratio)
- Segment performance measures (e.g. congestion heat map)
- Trip patterns or trip distribution routes
- Lane diagrams of existing or proposed intersection approaches
- Queuing diagrams
- Existing or proposed circulation routes within the study area
- Existing and proposed street or ramp centerline alignments
- Roadway cross-sections
- Alternative street improvement scenarios
- Land use and zoning maps

The best report graphics clearly label key reference streets, maintain a reasonable 10 to 12-point (minimum 8-point only if absolutely necessary) font size, and avoid trying to illustrate many layers of new information at one time. All diagrams need to have a legend clearly defining each symbol or line color/type used. A good rule-of-thumb is to limit the number of new layers to three or less for any diagram. Examples of different information layers are streets, peak hour volumes and functional street class. Complex diagrams can be developed in stages, explaining each new set of layers.

In general, street project alternatives are illustrated on separate diagrams. Depending on the overall layout of the project, a landscape orientation is generally better than portrait. Consider use of 11x17 paper format (i.e. foldout) to show larger areas or to show side by side groupings of alternatives instead of creating a larger number of smaller diagrams.

All documents need to be legible and usable in black and white. This can be an unavoidable issue with land use and zoning maps as these are typically created by outside parties and copied into a document. Unless many patterns are used, it is difficult to distinguish separate colors especially for individuals with differing levels of color-blindness.

For best results, it is recommended that diagrams be pasted into Microsoft Word documents via “Paste Special” and the “Enhanced Metafile” format. This will automatically allow for proper insertion without overwriting adjacent text, keeps diagrams intact (i.e. occasionally an issue with layered PowerPoint slides), and minimizes overall file size. Diagrams/figures created in PowerPoint ideally are grouped into a single object before copying to avoid accidentally leaving parts behind or moving them out of position.

19.2.3 Tables

Tables offer a quick way to show and summarize analysis results and other repetitive common information across multiple periods or alternatives. Tables should deal with just a single subject to avoid excessive size and clarity issues. Typical table types include:

- Traffic count location, type, date/time, and duration
- Roadway inventory/characteristics
- Applicable operational state targets and local standards
- Intersection/segment operations (e.g. v/c’s, LOS, delay, queues)
- Assumed project lists to be included in committed or financially constrained scenarios
- Historic crash analysis/characteristic/crash listing summaries
- Crash analysis results
- Multimodal analysis results
- Alternative comparison summaries

The preferred software to build tables in documents is MS Word as opposed to MS Excel, due to formatting issues, although MS Excel is acceptable for appendices especially with calculated values. Conditions exceeding a noted target/standard/threshold should be denoted with bolded white text on a black background. Column headers need to be understandable without excessive abbreviation, include units where appropriate, and be set off from the table body contents with shading.. Abbreviation definitions and meanings of cell shadings need to be footnoted at bottom of the table. Exhibit 19-1 shows a sample table showing the overall layout, header/cell shading, and footnotes.

Exhibit 19-1: Sample Table Layout

Segment	Side	From-To	LOS[1]
Main St	South	W Project Limit – Helman St	**E**
		Helman St – Oak St	C
		Oak St – E Main St	**E**
Siskiyou Blvd	South	E Main St – E Project Limit	C
	North	E Project Limit – E Main St	C
E Main St	South	E Main St –Third St	B
Main St	North	Third St – Oak St	**E**
		Oak St – Church St	C
		Church St – Helman St	**E**

[1]Black-shaded cells indicate that the multimodal LOS D analysis threshold has been exceeded.

Consider inserting landscape-oriented sections to show wider tables with more clarity instead of having too-narrow columns. Typically the maximum is six or seven columns on a portrait-oriented page. Table breaks across pages should be avoided, but ones that do, the headers need to be repeated on the next page. It is recommended that multi--page tables or a series of them are placed in an appendix to avoid creating disruptions for the reader.

19.3 Technical Memorandum

19.3.1 Purpose

A technical memorandum (TM) typically addresses one major stage of the project evaluation process, and presents the analysis, findings, and any potential next steps for that stage. Subsequent technical study stages build on the information presented in the previous memorandums, and allow for an incremental process to assess, refine, and build consensus on the preferred project. These technical memorandums are also described in Chapter 2 as part of the scoping considerations.

19.3.2 Products

The focus of a technical memorandum can vary widely, but, in general, they include the following technical materials, in a typical three-stage study development process. Smaller projects normally have a series of discrete memorandums while larger projects combine these memos into a TANR. Small projects (e.g. a single intersection) may have all the work combined into a single memorandum as multiple memos would likely be too much.

The overall study context/scope will determine how many memorandums will be necessary but as every project is unique with its own set of issues, the actual number of memorandums will differ. Certain memos can be combined, or an additional memorandum is needed to explain a certain issue or a new alternative option. Small projects could have the entire analysis documentation summarized in a single memorandum (almost a “mini” TANR). Regardless of how many memorandums there are, it is important to capture the noted elements below within each consistent with the overall project context and level-of-detail (e.g. a scoping-level intersection project will likely have less detail and reporting requirements than a multi-intersection congested project using micro-simulation). Any final draft technical memorandum that contains professional-level traffic analysis needs to be stamped by an Oregon-registered professional civil or traffic engineer.

TM#0 – Methodology & Assumptions: All ODOT traffic analyses must have a discussion on methodologies and assumptions used. Next to the scope of work, this is the most important documentation to have as it tells how the analysis work in the scope will be completed from what guidelines are to be used, to the data being collected, assumptions made, tools used, and reports produced. It is better practice to put more detail in this document rather than adding lots into the (contract) scope of work. This document is critical for reviewers as it is supposed to give assurance to the reader that the work was done according to the agreed upon processes, avoids more questions later (i.e. are the existing seasonal factors correct or the proper tools used for the future volume projections, etc.), and generally shortens review times. Reviews can fall back to this document if the work does not follow it as the scope of work is normally not as detailed.

Normally, this a separate memorandum, but could be an appendix to another (e.g. existing conditions). Alternatively, if the analysis is relatively simple such as for high-level scoping, the discussion on overall methodology and assumptions can be a paragraph or two. This memorandum is based on the overall scope document and its task requirements (see Chapter 2) as it tells how the described tools and data are used to achieve the project outcomes.

This memorandum details out the methodologies and assumptions that are to be used in the existing conditions, the no-build future conditions, and the alternative for any volume development and analyses. Generally, the range of analysis methodologies and proposed tools from identifying count locations through simulation, including any safety and multimodal analyses needs to be included. Appendices can be included initially or added later in a revised memorandum to cover micro-simulation needs such as calibration data, methodologies, and results of calibration. This memorandum should be provided to and approved by ODOT Region Traffic (and the Transportation Planning Analysis Unit as necessary) before any analysis work is conducted. This helps to significantly reduce the amount of review by ODOT and potential re-work by the Contractor. Appendix 19A contains an annotated example methodology memorandum. This example does not necessarily include all methodologies that are applicable in each context.

TM #1 - Existing/No-Build System Analysis: This memo presents the key system inventory features and performance deficiencies (e.g. safety, geometrics, mobility, reliability, access spacing) that will shape development of study alternatives. This memorandum is important it establishes the foundation for the analysis of existing or no-build future conditions (e.g. operations, safety, accessibility). This allows direct comparisons of the benefits and impacts of an alternative to the current or future conditions. Otherwise, it is difficult to tell if an alternative is having the desired impact on noted existing or future issues.

Most analyses such as planning projects, grant applications, environmental/operational/safety analyses, or micro-simulation applications will require this documentation. Sometimes if a study effort is only for analysis of current conditions (e.g. operations) or is completely in the future (e.g. planning analysis using travel demand model scenarios) then there may be only documentation of existing or future conditions instead of typically both. An analysis without an existing and/or future comparison condition is incomplete and generally will not be able to answer all of the questions asked.

Depending on project type and size there may be more than one memorandum covering this stage. While not typical, the inventory gathering, and volume development could be in separate memos. Larger, or more complex projects may have the existing conditions may be in a separate memorandum from the future no-build. Frequently, the methodology and assumptions are included as an appendix to this memorandum.

The memorandum typically includes statements on the project purpose and need, study area background, inventory data collected, and existing and future volume development. Discussed results generally include the historical and predictive (where applicable) crash analysis and any safety issues documented should be tied back to proven safety countermeasures (e.g. referencing the ARTS crash reduction factor/countermeasure listing) that can be identified to potentially improve safety performance. Other included discussion items are preliminary signal warrants, multimodal and reliability evaluations, access or spacing issues, noticeable operational issues, the intersection volume-to-capacity ratios, LOS, or other performance measures as appropriate, and the 95th percentile queues. Narrative text typically includes the positive or negative impacts to multimodal users, impacts to freight operations and truck routes, transit facilities, etc.

Comparisons should be made back to existing standards and thresholds (e.g. OHP interchange spacing standards, or HDM pedestrian crosswalk spacing) to identify all the applicable deficiencies (e.g. not just v/c and queues). These include operations, safety, multimodal, and geometric design. Many deficiencies will come from tool outputs, but many are field observed (e.g. vehicle consistently turning into the wrong lane because of too-short access spacing). A summary of the deficiencies by type for the existing conditions and future no-build conditions can also be included for easy reference.
The memorandum needs to include a set of appendices that support the results shown in the main body. These include:

- Volume development including seasonal and other adjustment factors, peak hour documentation, trip generation/distribution/assignment data, travel demand model assumptions/screenshots (Note that volume development spreadsheets typically do not fit well in a report format, so just a statement that this file is available upon request is sufficient). Raw counts are not normally included as the data is part of the volume development spreadsheet but can be if desired.
- Crash data – crash listings, HSM predictive calculations, SPIS lists, etc.
- Analysis output – Software tool output on lane configurations, intersection control, volumes, performance measures; preliminary signal warrant worksheets, multimodal analysis worksheets, performance measure calculations such as for intersection v/c, queuing, vehicle-miles traveled, etc.
- Micro-simulation or analysis tool calibration – advanced tools usually require some sort of calibration so reported conditions match the existing. This appendix is important as it gives documentation what was done to match to existing conditions and gives assurances to how the tool will properly reflect the future no-build and build conditions.

TM #2 - Preliminary Alternatives Screening: This memorandum presents the screening criteria, the initial roster of project alternatives and related options and the table-based scoring of how well the preliminary alternative or option matched up with the screening criteria. The alternatives and options shown in this memorandum need to address the deficiencies shown in the existing/future no-build memorandums. Unless there are a lot of scenarios, alternatives, and/or options, many times this memorandum is optional. Generally, this memorandum is only necessary for medium to larger projects where there are several potential solutions identified or where multiple levels of analysis are needed to objectively consider all the context.

To help with screening of potential alternatives/scenarios the memorandum should also have a summary review of deficiencies, impacts, and project assumptions in earlier overarching planning documents such as Transportation System Plans (TSP) or Interchange Area Management Plans (IAMP). Many times the evaluation of plan projects is used as the basis for a higher-level screening for a greater-detailed analysis. These documents will have discussions on deficiencies (which are normally consistent with the subject plan if not corrected), impacts, and any project list assumptions.

Screening criteria are more general indicators of performance. There should be at least one level of screening criteria shown. However, depending on project size there may be multiple iterations each with their own set of screening criteria. For example, a fatal-flaw screening comparing against minimum acceptable standards followed by a goal/objective-based screening (e.g. environmental impact, impacts to the built environment).

Screening measures generally include key volume-to-capacity ratios or LOS's, model-based results (travel times, speeds, v/c ratios, or relative comparisons), predicted crash reductions, and high-level multimodal or reliability values. The reasons why alternatives/options were dropped, and any alternative naming and overall naming convention changes need to be recorded as this will be needed for the final narrative report and included in an appendix. Detailed screening criteria, scoring methodologies, and evaluation tables with related calculations are typically shown in tables in an appendix.

TM #3 - Future Alternatives Analysis: This memorandum presents the detailed evaluations of all scenarios, alternatives, or options that progressed through the screening process. This memorandum runs in parallel with the future no-build analysis and completes the analysis of future conditions. Any analysis that covers the future no-build either as part of TM#1 as noted above or in a separate memorandum will also need documented analysis of build alternative conditions. It is important to note that the future no-build is a viable alternative as sometimes it is the preferred solution. It also can include the impacts of new developments, financially constrained projects, or operational improvements (i.e. a new enhanced crossing, updated signal phasing, etc.) outside of the subject project that will occur beyond the existing condition timeline. The future alternative analysis compares build alternatives with each other and the future no-build.

Depending on review or outreach comments and related required changes, there could be an additional memorandum on refined, hybrid, or preferred alternatives. These alternatives have full performance assessments and any other related evaluations (preliminary environmental, compliance with standards, etc.) as defined in the study goals, objectives, and evaluation criteria. Additional volume development sections will also likely be required as future build traffic volumes are usually different from the future no-build versions.

Narrative text should be included regarding the positive or negative impacts to safety for all users, on modes, impacts to freight operations and truck routes, transit facilities, etc. Specific impacts of the alternatives that need to be discussed include impacts of latent demand which can cause traffic re-distribution, peak spreading across time, or shifts across modes. . These can have substantial impact on the parallel road and multimodal networks for the better or worse. For alternatives that are on urban fringes or increase capacity or are in areas under economic stressors (e.g. cost of living) the memorandum should also discuss any potential impact of induced demand.

Detailed results typically include predicted crashes, multimodal analysis, preliminary signal warrants, turn lane criteria, volume-to-capacity ratio, LOS, predicted 95th percentile queues and required storage lengths, intersection/access/crosswalk spacing, and other operational performance measures (e.g., travel-time, average speed, reliability),

The memorandum needs to include a set of appendices that support the results shown in the main body. These include:

- Volume development for the alternatives, trip generation/distribution/assignment data, travel demand model assumptions/screenshots (Note that volume development spreadsheets typically do not fit well in a report format, so just a statement that this file is available upon request is sufficient).
- Crash data – HSM predictive calculations
- Analysis output – Software tool output on lane configurations, intersection control, volumes, performance measures; preliminary signal warrant worksheets, multimodal analysis worksheets, performance measure calculations such as for intersection v/c, queuing, vehicle-miles traveled, etc.
- Alternatives considered but dismissed – Documentation of alternatives/scenarios/options considered but screened or dropped out. At a minimum this needs to be a short description and reason for dropping. Figures are optional but very helpful to include.

19.3.3 Distribution

The technical memorandums should be distributed to the project team or at least to the project leader/manager/planner for review and comment. Sometimes a smaller internal working group will review these memorandums first for preliminary comments before distribution to other ODOT units and project stakeholders. Depending on the study context, others should be included in the distribution (e.g. ODOT region traffic manager/engineer, modeling staff, lead workers/manager, etc.).

19.4 Traffic Analysis Narrative Report

19.4.1 Purpose

Most of the traffic analysis will be completed by the point that the Draft Traffic Analysis Narrative Report (TANR) is developed. The purpose of this report is to present the final solution(s) selected from the study alternatives. This includes other documentation created as described in other portions of this manual (e.g. microsimulation calibration memorandum) as part of the full product.

19.4.2 Product

The Draft Traffic Analysis Narrative Report (TANR) presents the full study process and outcomes incorporating the interim Technical Memorandums, feedback from team committees, public involvement comments, any new information, recent decisions, or any scenarios/alternatives/options not captured in earlier memorandums. The major step to be completed with the TANR is to provide conclusions on the function of alternatives from a traffic analysis standpoint.

These can vary in length as it is really a function of geographic scope, the type of analyses included, the level(s) of detail considered, and the total number of scenarios, alternatives, or options that were analyzed in full detail. If the context and detail level of the project leads toward not requiring or needing a single summary report at the end, then consider doing a series of technical memorandums as described in Section 19.3. Larger, more complex, or longer duration efforts will likely result in substantial content to summarize which is best done in a single document. A TANR only has a few optional sections, so most of the following detail will be needed to be considered complete.

Projects that result in environmental documents (e.g. EA or EIS) require a final technical report for transportation which the TANR will cover the need. Refinement studies (especially any with planning-environmental linkages) should use a TANR to summarize the project as these typically have multiple levels of alternative analysis detail (e.g. high-level scenarios in a travel demand model, preliminary alternative screening using sketch -level tools, and followed by full operational analysis).

Ideally, the TANR is developed from merging previous technical memorandums to save on effort and time. The TANR shall be descriptive with necessary explanations of why certain conditions or results exist or why they do not (e.g. traffic diversion from latent demand). This is more than a simple reporting of performance measures as the analyst needs to tell the “story” (i.e., the narrative) of what the conditions are now, projected to be in the future, and the outcomes of the future alternatives that address the earlier identified needs. The selection process for a preferred alternative overall uses the analytical evaluation outcomes, relative scoring evaluations to reduce the total alternatives to a single, a few at most, or a creation of a hybrid alternative that combines several alternatives that best meet the study objectives. The narrative discussion needs to be seamless through the alternative development process, so that the reader knows why each alternative was created, why it remains, or why it was dropped. This is necessarily a collaborative process with established team members, stakeholders, local jurisdictions, and affected ODOT technical units.

The report itself is generally developed consistent with the following standard outline below. Project context, detail and scope will determine the degree that each item is needed. Larger, more complex projects will generally need more detail.

Sample Outline

- **Cover Sheet**
  - Agency/Company Title, Division, Unit, City, State (in header, footer or along bound edge)
  - "Project Title Traffic Analysis Narrative Report" (to clarify that this is just the traffic analysis)
  - City (if applicable) and County
  - Highway Name, Number and Route Number
  - Milepoint Range
  - Month and Year report published
- **Title Page**
  - "Project Title Traffic Analysis Narrative Report" (to clarify that this is just the traffic analysis)
  - Highway Name, Number and Route Number
  - Milepoint Range
  - Full Mailing Address
  - Prepared by and reviewed by (including stamp by preparing PE or reviewing PE if preparer is not registered; requires signature of non-registered preparer)
- **Table of Contents, List of Figures, List of Tables, List of Appendices**
- **Executive Summary**: Summary of report including purpose, need, background, scope of alternatives, high-level summary of results, alternative screening/evaluation, and re-statement of conclusions. These range from a couple to a half-dozen pages depending on the number of alternatives. Study area and alternative figures can also be included to increase understanding. The takeaways for the reader should be the same as if they had read the entire report. For complex efforts, writing the executive summary in a way that it can be standalone may help in the digestion of the material especially for non-technical audiences.
- **Background Information**: Contains an overview of the study area including vicinity and study area maps, affected facilities and jurisdictions, a table of operational targets and standards for the applicable jurisdictions, past project or planning decisions that generally influence outcomes, a general problem statement, and objectives for the study.
- **Existing Conditions**: Contains discussion of inventory and analysis of base year facility and operating conditions. This includes five-year historical crash summaries, any applicable Highway Safety Manual-based Part B screening and predictive (i.e. expected crashes) Part C crash analyses, volume development, facility-level roadway and multimodal results, and comparison with applicable targets, standards, and thresholds. Normally, discussion includes any constraints

or impacts to freight routes, multimodal facilities, and potential of traffic diversion. Tables of analysis results and/or figures are necessary to summarize information and provide understanding. An optional list of existing deficiencies is a good way of summarizing issues across multiple subjects.

- **Future Year Forecasts and Needs (No-Build)**: Discussion of future year volume development including summary of travel demand model scenarios, horizon (design) year traffic forecasts, HSM predictive crash analysis, and performance assessment on the existing street system with no project improvements across all applicable modes. Discussion typically includes any constraints or impacts to freight routes, multimodal facilities, potential of traffic diversion, and latent and induced demand (see Section 6.12.2). Tables of analysis results and/or figures are necessary to summarize information and provide understanding. An optional list of future deficiencies is a good way of summarizing issues across multiple subjects to help ensure that these are addressed by the build alternatives. Previously agreed upon network assumptions need to be documented here which includes committed (i.e. funded for construction STIP & CIP projects), planned but financially constrained projects in a TSP or RTP, and private development land use projects. See Chapter 9 for more details.
- **Preliminary Alternatives Screening**: Optional, as it depends on if there were enough scenarios or preliminary alternatives to require screening. Includes screening/evaluation criteria and process, concept alternative descriptions to address outstanding needs, and preliminary screening of alternatives along with reasons for dropping alternatives from further evaluation (see Chapter 10). The evaluation matrix and related details should be placed in an appendix.

- **Alternative (Build) Results**: Discussion of performance results for each analyzed alternative for the build (i.e. year of opening), interim (if applicable) and design years for the same comparisons across targets, standards, and thresholds for all applicable modes as done for the existing and future no-build conditions. Crash analysis also includes specifics on potential countermeasures to address safety issue locations in the existing and future no-build conditions. Typically, discussion includes constraints or impacts to freight routes, multimodal facilities, potential of traffic diversion, and latent and induced demand. Tables of analysis results and/or figures are necessary to summarize information and provide understanding.
- **Alternative Summary**: The alternatives are compared against each other, including a summary table, according to appropriate performance measures. There should be no new material introduced in this section as it is intended to summarize the build results. The future no-build alternative is normally also included for comparison as this also is a viable alternative (i.e. to do nothing). The summary table represents each performance measure (or family of measures) at a higher level. For example use, “Number of intersections exceeding targets” instead of showing individual v/c’s. Exhibit 19-2 shows an example summary table.

Exhibit 19-2: Example Alternative Summary Table

Measure	No-build	Alt 1	Alt 2	Alt 3
Number of intersections over capacity	3	3	3	2
Number of intersections over LOS D	9	7	5	6
Total Main St SB approach delay (s)	69	73	68	114
Number of queue blocked intersections	9	7	8	8
Average percentage of segments at BLTS 1 or 2	87	95	95	98
Average percentage of segments at PLTS 1 or 2	58	67	67	67
Average of unsignalized Main St crosswalk delay (s) for options	696	240	60	157

- **Conclusions**: The analyst needs to be careful to make conclusions based on the traffic analysis results, rather than recommendations on a preferred alternative, as the best alternative from a pure traffic standpoint is unlikely to be the best overall given complete context and considerations (e.g. impacted environment, pedestrian safety). The conclusions are essentially a summary of the main points coming out of the Alternative Summary section. These should also be re-stated as part of the Executive Summary.
- **Further/Future Areas of Study/Next Steps**: Optional; formatted in a bullet list or short paragraphs
- **Appendices**

Appendices normally include the following subjects with the items listed for each below. Depending on the project scope and size, some of these are normally combined or split apart for easier reference.

  - Crash History: Detailed historic yearly crash summary listing for each roadway in study area, HSM Part B screening-level, and HSM Part C predictive crash analysis. Background on selected countermeasures (e.g. ARTS Crash Reduction Factor information, CMF Clearinghouse details) can also be included.
  - Inventory: Spreadsheet-type data listings of roadway, bicycle, and pedestrian facilities for segments and intersection locations commensurate with level of detail required by methodology and tools used.
  - Record of Calibration (required if micro-simulation was performed or calibratable tools like SIDRA were used): The calibration record will vary in detail level and length by project and specific tools used, but the record needs to address the following items:

    - <u>List of key calibration locations</u>
    - <u>Calibration data gathered for the key locations</u>
    - <u>Measures of effectiveness (MOE) needed to meet calibration thresholds</u>
    - A table or list citing all changes that were made to the inputs or model modules to achieve calibration, beyond the standard changes that occur after collecting field inventory (see Section 3.3). This list or table should include:
      - the issue that was occurring before the change was made,
      - the goal of the change, and
      - how the change improved the calibration.
    - For each Measure of Effectiveness (MOE) of the calibration, include a table that shows the before and after results for each MOE. Before results have all standard inputs, but no changes beyond the standard adjustments. After results have all changes to achieve calibration were included in the model.
    - The record needs to indicate that the key calibration locations met the calibration standards.
  - <u>Volume Development</u>: Count locations/type/duration/dates, text explanation (along with figures/tables as needed) of base and future volume development across all applicable modes including any seasonal/historical adjustments, trip generation/distributions, trip patterns via select-zones/links, model scenario descriptions, and model post-processing including any significant manual assignment adjustments. A list of network assumptions including committed and financially constrained projects needs to be included along with any related land use and zoning maps. Appendix header page should include a note that volume development spreadsheets are available upon request since most of these are not print or online document friendly.
  - <u>Existing Year Volumes</u>: Peak hour(s) volume and lane configuration diagrams and daily roadway segment volume diagrams for the existing (base) year. If available, also include bicycle and pedestrian facility segment and intersection volumes.
  - <u>Existing Year Analysis Inputs & Outputs:</u> Analysis software inputs and formatted output reports (e.g. v/c, LOS, queuing, multimodal, reliability). A spreadsheet of critical intersection v/c ratio calculations including phase timing and critical pair identification should be included for any signalized intersections.
  - <u>Future No-Build Volumes</u>: Peak hour(s) volume and lane configuration diagrams and daily roadway segment volume diagrams for the future no-build years. This can include the year of opening/build year, interim years (i.e. 10 years beyond the build year) and the future horizon/design year (i.e. 20 years beyond the build year). If available, also include bicycle and pedestrian facility segment and intersection future volumes.
  - <u>Future No-build Analysis Inputs & Outputs:</u> Analysis software inputs and formatted output reports (e.g. v/c, LOS, queuing, multimodal, reliability).

A spreadsheet of critical intersection v/c ratio calculations including phase timing and critical pair identification should be included for any signalized intersections.

- Alternatives Considered but Dismissed: Short description of each dismissed alternative including why it was dropped listed in chronological order along with any optional figures for further-developed alternatives. This appendix is important as it has been generally found over time that alternative disposition is not well documented as it is a source of project questions especially when an "old" idea is re-introduced such as in a public meeting or comment letter.
- Alternative (Build) Volumes: Peak hour(s) volume and lane configuration and daily roadway segment volume diagrams for each alternative. Each build, interim (if applicable) and horizon/design year for each alternative can be a separate appendix or logically combined depending on the project. If available, also include bicycle and pedestrian facility segment and intersection future volumes.
- Alternative Analysis Inputs & Outputs: Analysis software inputs and formatted output reports (e.g. v/c, LOS, queuing, multimodal, reliability). A spreadsheet of critical intersection v/c ratio calculations including phase timing and critical pair identification should be included for any signalized intersections. This is a critical inclusion for any reviewers.
- Analysis Methodologies: Final methodology memorandum in entirety (i.e. including any appendices). Also, include any analysis methodology that was created or updated later after the memo (e.g. documentation of a screening-level or a reliability analysis added in later).
- Environmental Traffic Data (required if noise, air quality or greenhouse gas (GHG) modeling was performed): For No-Build and Build alternatives, including link diagrams and tabular traffic data for noise, air quality as required, and GHG for applicable years and roadway segments.

In addition, electronic-only documents should be assembled and packaged (i.e. in a zip file) to be provided to reviewers. These include:

- Volume development (with or without model post-processing) spreadsheet workbooks
- Deterministic (i.e. HCM-based) analysis software files
- Critical intersection v/c spreadsheets
- Final micro-simulation/animation runs
- Other documentation that did not translate well in the report format

The narrative report appendices, and other related materials (e.g. volume development spreadsheets, micro-simulation files) may also be copied to a USB flash drive or other storage device/location for a backup copy. Flash drives and other physical storage media should be retained in the physical project file. A shortcut (or a direct location link) should be documented for any online backup/archive storage locations.

A draft of the narrative needs to be sent, as a minimum, to the project leader/lead planner and the corresponding region traffic engineer and/or manager in the Region Traffic office. Depending on the context (more are needed for a NEPA project versus a planning project) other groups are the Traffic-Roadway Section, Environmental Section, active transportation and mobility liaisons, roadway design lead, corresponding consulting staff, local jurisdiction engineering staff, and any others who might be affected, for review and comment. Allow 3-4 weeks for a thorough technical review of the narrative, appendices and related on-line only materials. Generally, this review is completed within two weeks for smaller projects.

Consider attaching a blank comment log to capture substantial comments and later responses. The traditional track change and comment form can be used for smaller projects or as a second round of editorial review for larger ones. About two weeks need to be allowed for responding to comments.

19.4.3 Distribution

Upon incorporation of comments received on the draft, any draft watermarks, "Draft" language on the header/title/headers/footers is removed and the TANR is signed and stamped on the title sheet by the responsible professional engineer.. The document and the appendices bundle (i.e. save as a group) needs to be saved as separate pdf files for distribution to prevent accidental changes and corruption. The pdf version should be sent to the project leader/planner, the consultant project leader, and main project contacts in the appropriate Region Traffic office, other ODOT sections, and local jurisdictions.

19.4.4 Document Close-out

After distribution of the TANR, and while the project analysis work is still fresh, this is a good time to do a documentation clean-up in any paper or electronic files. This is important as while the TANR (or final technical memorandum for a smaller project) represents conclusion of the analysis work, it also represents the start of the next phase such as environmental documents or design. Also, while things make sense "today", but after months or years have gone by, direct recollections fade.

During this future work it is common that questions will be asked on the preferred alternative, or clarifications on analysis assumptions (e.g. TSP financially constrained project inclusions or did the future volumes include the impact of latent and induced demand) or whether this past work is still valid in cases of a future phase or activation of a "shelf" project. These might show up in correspondence, project meetings, environmental document comments or even in public hearings that will require figuring out what was done. In some cases this will involve new analysis or corrections/modifications that will require additional documentation through a new technical memorandum(s), or an updated TANR. Anything that can be done today to improve understanding and save time in the future by the analyst, or their successors is important.

Old or obsolete TANR or technical memo drafts should be deleted to clean up project files and to avoid confusion in the future, so it is clear what was used to develop the existing, future, and alternative conditions and results. The existing conditions, future no-build conditions, and each alternative need to be in separate folders if they are not already. Final analysis files should be in their own separate folders and noted with "_FINAL." Date stamps in folder and file names are also helpful. Readme text files should be added for any future reference to folder contents (although the best time to create these as the work proceeds) or to back up assumptions or decisions made in specific files. Spreadsheets can have cell notes added to clarify sources or calculations.

19.5 Reviewing Analysis Documentation

Often an analyst will be required to review work conducted by others, whether it was performed within the Department by a peer or by a consultant. All traffic analysis work (either done internally by ODOT staff or by a consultant) must be reviewed by an Oregon-registered civil or traffic engineer. At a minimum, this is a peer review if both the analyst and reviewer are Oregon-registered civil or traffic engineers. The reviewer should be the analyst's lead worker/supervisor as they should be involved in the flow of the work. If the analyst is not registered, then the reviewer must be the lead worker/supervisor who is registered as they must be familiar with the work as the professional responsibility falls under them. Work performed by a non-registered consultant analyst must be reviewed by their registered lead worker/supervisor prior to submission to ODOT for review.

The review parameters such as who the reviewers are, what is going to be reviewed, and what level of detail the review will be documented at should be done before the review starts. Ideally, two weeks is the desired review turnaround time to allow for other workloads, time off/emergencies, etc. especially when full reports and memorandums including appendices and electronic files are to be reviewed. Normally, more time is needed to review a TANR and appendices as noted in the previous section.

The following section provides general guidance for reviewing traffic analysis that is applied to any type of analysis project. Specific guidance for the review of Traffic Impact Analyses/Statements (TIA/TIS) is found in ODOT's Development Review Guidelines.

19.5.1 Purpose of the Review

The reviewer should generally know what the purpose/scope of the review is as this establishes what the review needs to cover. Some considerations include:

- Audience – is this only for internal technical staff or is this to be included as part of a public document (i.e. guidelines) or public-facing document for a web page or handout?
- Completeness – is this a rough draft to start a process or a discussion or is this to report analysis, discussions, or decisions?
- Full Documentation – are there short-cuts taken like referring to other past memos/reports/projects? Note that report readers might not have access to the

other reports. Reports ideally are stand alone, so past memos need to be provided separately or as appendices. For example, just stating that "US 101 in the project has an OHP mobility standard v/c ratio of 0.85 in Tillamook", leaves out the missing classification information which includes items like what is the highway classification, expressway, freight route, or Special Transportation Area designations, etc.

- Accuracy - need to verify things like OHP/HDM targets and standards (including any alternative targets and performance measures), speeds, lane configurations and traffic control
- Consistency - thinking "outside the box" may be good in some cases, but it should not carry through freely to documentation as readers are looking for specific types of information and having to sort through pages of data can be difficult.

19.5.2 Organization and General Format

The report should be set up for the specific (target) audience, using words and sentences (word size and sentence length) appropriately with acronyms defined and used minimally. Most readers need some sort of organization, so thoughts are grouped linearly (i.e. time, location, or process) or grouped by topic (i.e. safety, configuration/geometrics, policies/standards, procedures, and findings/conclusions).

The report needs to have good readability. Color does not always copy well as graphics usually just turn into multiple impossible to differentiate shades of gray. Generally, use patterns in graphics to distinguish different features as much as possible. Typeface size for general text needs be at least a 12 point (or larger) and limited to standard fonts with few "extras" (i.e. Times New Roman, Georgia, Arial, Verdana). Footnotes in text, tables or figures need not to be any smaller than 10 point. Remember that clarity dissolves with copying and not all programs use all fonts. All pages are numbered as review comments typically are tied back to a reference page/paragraph/line. Line numbers are optional and are more common for larger reports or for significant efforts.

White space throughout the document should be evident instead of continuous lines of text. Paragraphs after paragraphs of text lead to low readability. Tables are better but tend to be complex especially if multiple variables are involved. Tables make data easy to read by making comparisons. Failing/exceeding threshold values should be pointed out with emphasis (i.e. bolded, shaded, etc.). Large tables that continue across multiple pages or are larger format (i.e. 11x17 landscape orientation) are best placed in separate appendices to avoid disruption to the reader.

Drawings and figures are better than long-winded description paragraphs, but some text is still needed to point out or explain to the reader the important features, issues, key locations, etc. Numbers are typically noted and balanced appropriately (e.g. future volumes rounded and not to the exact amount; or not losing traffic between ramps on a freeway). Information needs to be limited to about three layers on a single diagram (e.g. street names, classification, and volumes).

19.5.3 Checking Information

A reviewer does not have to check every fact and figure but normally covers major sections and areas that affect the results of the work. Some of these areas are:

Study Area

When reviewing analysis conducted by others, knowledge of the study area is typically beneficial. The reviewer should first examine all study area mapping and use available aerial/street-level images available. If practical and if the effort is large or important enough a physical visit can be performed.

Roadway Classification and Jurisdiction

This establishes what type of road it is and who controls it. This will determine what overall performance measure to use (state or local) and the specific value. This also includes special designations such as for level of importance, freight routes, expressways, and Special Transportation Areas which also affect the performance measure values.

Analysis Methods and Processes Expectations

The document should state its purpose and need as that establishes the level of detail needed in the analysis, the answers that are needed and what the expected results are. Review the methodology memorandum to make sure that the assumptions, parameters, and tools to be used in the existing conditions, future conditions and the alternatives is appropriate for the level of detail of the work and consistent with the overall scope of work.

If the work follows the memorandum and any corresponding scope of work (See Chapter 2), then the rest of the review is streamlined for both sides. Any disagreements need to be taken care of before analysis work is started to minimize rework and issues later. Keep in mind assumptions made by the analyst performing the work normally have a significant effect on the analysis results, even if specific analysis procedures are followed correctly.

Each of the major analysis areas to be covered in the project as noted in the scope also need to be included in the methodology memorandum (e.g. travel demand modeling, safety analysis, multimodal analysis, reliability, microsimulation). For example, if the project is within an area represented by a travel demand model such as in a MPO area, then the model must be used to develop future volumes and alternatives. The specifics relating to model name, version, years, scenario assumptions should be stated in the methodology memorandum or at least in the volume development section or appendix of the future no-build and build alternative memorandums.

Safety analyses (see Chapter 4) should have some sort of historical analysis of trends and summaries. If there are more than a half-dozen intersections, then some sort of screening methodology using crash rates, or applicable HSM Part B methodologies is generally expected. The number of locations needing predictive HSM part C analysis performed and crash mitigations ideally are minimized. Any countermeasures used need to come from the CMF Clearing house (with three stars or better, similar volumes) or the ODOT ARTS CRF hot-spot and systemic crash listings.

Multimodal analyses (see Chapter 14) need to be at the appropriate level for the project type. At a minimum this will be qualitative multimodal assessments (QMA) or a level of traffic stress (LTS) analysis for most planning projects. Detailed refinement planning and projects are ideally using multi-modal level of service methodologies. Any modifications such as additions or subtractions from index-level methods should be documented (e.g. adding grades to LTS). Subjective methods like QMA need documentation on what is good-fair-poor for each factor used.

Certain analysis tools (e.g. SIDRA) and microsimulation tools (e.g. SimTraffic/Vissim) require calibration for existing conditions to ensure that reported results are correct. See Appendix 12/13A and Chapter 15. Calibration data, key locations, thresholds, modifications done, and result comparison with thresholds need to be provided in a methodology memo, specific memo, report section or appendix. Reviews should not be deemed complete until this is available and reviewed.

Other specific analysis methodologies need to have assumption sources documented (e.g. free-flow speed basis, congestion thresholds for reliability, use of private "big data/information" sources, checking for latent & induced demand effects). Electronic-only files such as volume development spreadsheets, Synchro/Vistro/SIDRA/HCS scenario files should be provided initially as attachments.

Data

With any type of technical analysis, the proper collection and processing of data is critical to obtaining accurate results. Before reviewing the analysis itself, verify the data used is appropriate for the analysis conducted. Consider things such when was the data collected, type of data used, and whether any processing of data (e.g., volume balancing) was conducted correctly. Does the inventory data collected adequately support the desired tools (e.g. AADTs for HSM analyses, sidewalk width for pedestrian analyses, private information origin-destination data for weaving sections)?

Appropriate Factors

This means checking that the count data was correctly obtained and correctly seasonally adjusted to the 30th highest hour (or another applicable alternative standard) and future year. Are seasonal adjustments less than 30%? Are all the counts adjusted to a common base year? Are other analysis parameters correct such as the peak hour factor, heavy vehicle factors, and saturation flow rates? Is the appropriate future methodology followed (e.g. historic, cumulative or travel demand model)? See Chapters 5 and 6 for more information.

Spot Checks

Typically, the reviewer performs a few quick checks by pulling the cited data and verifying correctness. With the given data, can the reviewer reproduce the seasonal and growth adjustments? Or can they follow the methodology used in a volume development or future post-processing spreadsheet? Do the volumes balance between intersections where there are no driveways or uncounted locations (e.g. between interchange ramp terminals)? If there is an extended distance between study intersections, is the volume increase/decrease consistent with the land uses?

The calculations performed in the analysis should be checked for computational errors, and procedures used should be appropriate for the given situation and in compliance with accepted ODOT practices. Knowledge of the study area, prevailing traffic conditions and accepted ODOT analysis procedures will aid the reviewer in determining which assumptions are appropriate, and which are not.

Correct Processes

Make sure that what is reported was analyzed with the correct program or tool. Listed below are generally the expected tool or program for each analysis type. Note that alternative tools and processes are allowed that are not listed below, but there must be documentation in a reviewed and approved (Region Traffic and/or TPAU as appropriate) methodology memorandum or other correspondence that explains the reasons for using the alternative tool/process. The methodologies that use the tools and processes below in the analysis should be consistent with the final d scope of work, workplan, or methodology memorandum.

- Existing volume development seasonal adjustments – ATR On-site, Characteristic Table or Seasonal Trend Table (see Chapter 3) documented in a spreadsheet
- Axle factoring for roadway tube non-classification counts – OTMS (see Chapter 3)
- Future volume development –
    - Historic, cumulative, travel demand model (see Chapter 6, 8, and 17), or Statewide Integrated Model (see Chapter 7). Note that for MPO areas, use of the regional travel demand model is required. Where small urban travel demand models exist, they should be used (see Travel Demand Model Map). For all methods, there should be spreadsheet workbook available illustrating the calculation steps with proper headers and callout/cell notes as needed.
    - Alternative mobility standards (see Chapter 9) – Documented in a volume development spreadsheet showing steps
    - Peak hour spreading (see Chapter 8) – Region 1 Hours of Congestion tool, spreadsheet documentation
    - Latent & induced demand (see Section 6.12.2) – Documented with travel demand model/SWIM plots/output as required by study area geography

- Safety analysis (see Chapter 4):
  - Historical – ODOT crash reports/summaries/rate tables with documented work in a spreadsheet workbook
  - HSM Part B screening – ODOT calculator spreadsheet tools for Critical Crash Rate and Excess Prohibition of Specific Crash Types or equivalents
  - HSM Part C predictive analysis – ODOT HSM spreadsheets, ODOT ARTS CRF list and/or CMF Clearinghouse, ISATE freeway/interchange tool, or commercial equivalent
  - Geometric design screening – Spreadsheet documentation of functional area, sight distance, access spacing, and conflict points
- Intersection analysis (see Chapters 10, 12 & 13) – Synchro, Vistro, HCS, or SIDRA. Signalized intersections need to have a supporting spreadsheet workbook for v/c calculations for all programs except Vistro.
- Future signalization (see Chapter 12) - ODOT Preliminary Signal Warrant form or equivalent of MUTCD Warrant 1
- Roundabouts (see Chapter 12) – Spreadsheet tools, HCS, Vistro, or SIDRA (preferred)
- Queuing (non-congested conditions; see Chapter 12 & 13) – ODOT unsignalized queuing equations, Synchro, Vistro, HCS, or SIDRA
- Queuing (congested conditions; see Chapter 15 & Appendix 12/13A) – SIDRA, SimTraffic, or Vissim with calibration documentation
- Rural mainline turn lanes (see Chapter 12) – Documented ODOT turn lane criteria
- Segment/facility analysis (see Chapters 10 & 11) – HCS, ODOT two-lane highway follower-density method, or related spreadsheet tools
- Reliability analysis (see Chapter 11) – HCS, FREEVAL, RITIS (existing only), HERS (future only)
- Multimodal analysis (see Chapter 14) – Qualitative MMLOS, Bike/Pedestrian LTS, Simplified/streamlined MMLOS spreadsheet tools, NCHRP 562 screening tool or equivalents
- Environmental traffic data (see Chapter 16) – Applicable spreadsheet workbooks for noise, air quality, and GHG traffic data as appropriate

Correct Targets/Standards

Once the adequacy of the analysis has been verified, compare the results to ODOT's and any local jurisdiction's adopted performance measures (see Chapter 9) including any alternative targets/standards/measures. If alternative targets/standards or performance measures are proposed, check to see if the analysis steps establishing the target/standard (see Sections 9.2.3 and 9.2.4) are documented. Check any proposed mitigation against ODOT's (or local jurisdiction as appropriate) design standards. Often the review process will require coordination with other units within ODOT or other governmental bodies that have specific expertise in, or authority over, certain elements of the design or approval of the mitigation proposed.

Reasonableness

In addition to technical accuracy, the results of the analysis should be evaluated using a "reasonableness" test. The reviewer should compare the subject data, such as the traffic volume counts, lane configurations and traffic controls, and determine whether the conclusions and recommendations of the study are reasonable. This can be done by checking the operational results (e.g. queuing impacts, corridor travel times, average speeds, lane utilization, and lane changing/merging/diverging/weaving behaviors). For example, if the results are showing congested conditions or volumes are shown to be high at a particular location, it would be expected to see substantial spatial queuing and slower speeds. This type of test often helps pinpoint sources of error in analysis and might reveal questions likely to arise when the project is presented to the public. It does help if the reviewer is familiar with the study area in question or can ask others who are especially about any operational issues that occur (e.g. poor lane utilization for a dual left turn caused by an immediate downstream right turn).

Addressing Errors

When sources of error are detected in the analysis, the reviewer typically notes not only just the error itself, but acknowledge the significance of the error to the results of the analysis. There will be times when correcting the error requires a substantial amount of work, but the results of the corrected analysis would not be significantly different, and the recommendations of the study would remain unchanged. If the documentation of the process is important to avoid questioning/legal challenges in the future, then it is generally best to fix the error. Noting the significance of the error ahead of time will enable ODOT to determine whether correction is necessary or cost-effective.

19.5.4 Documentation

- Typically, report documentation includes the following: Study area map
- Methods and assumptions (ideally in a separate section/memo in an appendix)
- Applicable polices, standards, background conditions
- Local street and highway system including (freight routes, pedestrian, bicycle, and transit modes)
- Data and inventory summary as well as source(s) of the information
- Traffic volumes (segments and/or intersections)
- Volume development – raw counts, system peak hour, adjustment factors, unbalanced volumes, base year, build (opening) year, future years, model versions/scenarios used, evidence of consideration and or analysis for induced and latent demand
- Trip patterns/distributions
- Lane configurations
- Land use and zoning maps
- Circulation routes
- Existing or proposed scenarios/concepts
- Existing or proposed alignments/alternatives
- Existing & future no-build and build alternative analysis
- Summaries as appropriate, including any evaluation criteria, screening matrices, cost estimates, benefit-cost studies
- Conclusions
- Technical data included in appendices with electronic files available upon request

Missing sections or other errors/issues found are normally addressed in a comment log or memorandum or email, so the reviewer's comments can be documented as well. Pages, sections/tables/figures/exhibits, and/or line numbers need to be identified for easy reference. Many times the team/project or planning lead will be consolidating comments from several reviews and reconciling any conflicts between reviewers.

Appendix 19A – Sample Methodology Memorandum

Glossary

The purpose of this glossary is to define terms as used for ODOT-specific analysis. It is written in plain language as much as possible to enhance understandability. It is not intended to change definitions in other engineering publications.

30th Highest Hour (30 HV) – Also see Design Hour Volume. This is the 30th highest hour of the year, which is typically the design hour for ODOT plans and projects. It is typically the average p.m. peak hour for most urbanized areas. Hours higher than the 30th are typically holidays and other high-traffic days of the year. The concept of using the 30^{th} highest hour is that it would not be appropriate to design for the highest hours of the year as the design may be overbuilt.

95^{th} Percentile Queue Length – Queue length (in vehicles) that has a 5% probability of being exceeded during the analysis period. This is typically defined as the design queue length.

Accessibility – degree to which the system is usable to as many individuals as possible.

Access Management – The proactive management of vehicular access points to land parcels adjacent to all manner of roadways.

Access Spacing – The practice of increasing the distance between intersections to improve the flow of traffic on major arterials, reduce congestion, and improve air quality within heavily traveled corridors.

Active Traffic Management (ATM) – The ability to dynamically manage recurrent and non-recurrent congestion based on prevailing and predicted traffic conditions.

Active Transportation and Demand Management (ATDM) – The dynamic management, control, and influence of travel demand, traffic demand, and traffic flow on transportation facilities utilizing the following three components: Active Traffic Management, Active Demand Management, and Active Parking Management.

Adaptive Signal Control Systems – Traffic signal systems that self-adjust to traffic conditions, demand, and capacity.

Advanced Public Transportation System (APTS) - Applies advanced technologies to the operations, maintenance, customer information, planning, and management functions for the transit agency. APTS includes advanced communications between the transit departments and the public, personnel, and other operating entities such as emergency response services, and traffic management systems; automatic vehicle locator (AVL); traffic signal priority; transit operations software; advanced transit scheduling systems (ATSS); transit security; and fleet maintenance.

Advanced Traveler Information System (ATIS) - Ranges from simply providing fixed transit schedule information to multimodal traveler information, including real-time traffic conditions and transit schedules, and information to support mode and route selection.

All Roads Transportation Program (ARTS) – A safety program addressing safety needs on all public roads in Oregon (formerly known as Jurisdictionally Blind Safety Program).

Annual Average Daily Traffic (AADT) - The total traffic for the year divided by 365 (or 366 in a leap year). Can either be actual values from automatic traffic recorders or estimated with seasonal factors.

Annual Average Weekday Traffic (AAWDT) – AADT considering only Monday-Thursday volumes.

Average Daily Traffic (ADT) – The total traffic volume during a given period (1-365 days) divided by the number of days in that period.

Average Weekday Daily Traffic (AWD or AWDT) – ADT considering only Monday-Thursday volumes.

Air and Noise (Traffic) Data – An assembly of hourly and daily volumes, speeds, and level-of-service for peak vehicle and truck hours broken down by directional links by analysis year for a build or no-build alternative. These are input into specific models by environmental specialists to determine impacts of air quality and noise.

Air Quality Conformity – a method to ensure that Federal funding and approval goes to those transportation activities that are consistent with air quality goals. Conformity applies to transportation plans, transportation improvement programs (TIPs), and projects funded or approved by the Federal Highway Administration (FHWA) or the Federal Transit Administration (FTA) in areas that do not meet or previously have not met air quality standards for ozone, carbon monoxide, particulate matter, or nitrogen dioxide.

Alternate Mobility Standards – Adopted by the Oregon Transportation Commission and developed in accordance with Action 1.F.1 in the Oregon Highway Plan, these modify the mobility targets for a specified corridor/area/intersection via an adopted plan. These can be volume-to-capacity ratios or based on other substituted performance measures.

Aggregate Probabilistic Limiting Velocity Model (APLVM) – HERS-ST model used to calculate free-flow speed based on curve geometry, pavement roughness and posted speed.

Area Type (Urban/Rural) - The area type is determined from the FHWA's functional class for the highway segment.

Arterial – A roadway with primary function being mobility rather than property access.

Arterial Management - Applies State and local planning, capital, and regulatory and management tools to enhance and/or preserve the transportation functions of the arterial roadway using surveillance devices, advanced signal algorithms, and coordination.

Automatic Traffic Recorder (ATR) – Electronic counting site on a roadway that counts vehicles continuously.

Assignment (Model) – The placement of travel demand model volumes on the model roadway network typically by shortest path by time.

Automatic Vehicle Classifier (AVC) – Similar to an ATR, but these new installations also record the 13 FHWA vehicle types including passenger cars, buses, and trucks.

Advanced Vehicle Control Safety System (AVCSS) - Includes vehicle safety systems such as vehicle or driver safety monitoring; longitudinal, lateral, or intersection warning control or collision avoidance; pre-crash restraint; and automated highway systems.

Auxiliary Lane (Aux Lane) – An extra lane that restores mainline throughput and improves safety diminished by local operations.

Auxiliary Through Lane (ATL) – A limited through lane added midblock upstream and dropped downstream of a signalized intersection.

Back of Queue – Refers to how far back it is to the last car lined up at a traffic signal. Maximum extent of the queue relative to the stop line during a signal cycle. The last queued vehicle that joins the back of queue is the last vehicle that departs at the end of the saturated part of green interval or the available gap interval.

Benefit Cost Ratio (B/C or BCR) – The relative monetized (can be tangible or non-tangible) benefits divided by the cost, typically expressed as a decimal. Accepted ratios are 1.0 or greater.

Bicycle Level of Stress (BLTS) – Multimodal methodology that breaks roadway segments into four classifications for measuring the effects of proximity to motorized traffic on bicycle riders.

Blocking Percentage – The proportion of time expressed as a percentage of the peak hour that a queue obstructs an upstream turn lane or intersection or other significant point (i.e. railroad crossing).

Bottleneck - A specific location where roadway performance is reduced due to a physical or temporary constraint, which when activated reduces the throughput of the roadway segment.

Broad Brush Level Analysis - A high-level roadway capacity analysis requiring minimal amounts of data and incorporating several assumptions or default values.

Buffer Index – The percentage additional travel time (or time cushion) users must add to their average travel time to ensure on-time arrival 95 percent of the time.

Build Volume – The volume used for a build alternative. Also known as the design hour volume (DHV).

Built Environment – refers to the human-made surroundings that provide the setting for human activity, ranging in scale from buildings and parks (green space) to neighborhoods and cities. This includes supporting infrastructure such as water supply, and energy networks.

Bus Lane - Highway or street lane reserved primarily for buses during specified periods (may be used by other traffic under certain circumstances, such as for making a right or left turn, or by taxis, motorcycles, or carpools that meet the requirements of the jurisdiction's traffic laws).

Bypass Lane – A lane that allows through vehicle to pass a slowing or stopped left-turning vehicle.

Calibration – Calibration refers to comparing the output from travel demand models run using data on existing population, employment, and travel patterns with current traffic counts. Adjustments are made to the model when inconsistencies are identified between the models and actual counts. Calibration also applies to micro-simulation models.

Capacity – The maximum sustainable flow rate at which vehicles or persons can reasonably be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions; usually expressed as vph, pcph, or pph. In other words, capacity is the maximum number of cars per hour that can travel on a particular stretch of roadway, with consideration given to the number of lanes, lane width, traffic signals, speed limit and other features.

Car Following Model - Driver behavior model that controls how a following vehicle adjusts its speed in relation to the leading vehicle.

Centroid Connectors – Links that connect centroid nodes with the model network. These can represent local streets and accesses not included in the model network. Centroid Connectors provide the linkage between the trips associated with the TAZ land uses and the roadway segments (or links).

Centroids – They represent the center of activity for a transportation analysis zone (TAZ). This is not the geometric center of the zone.

Channelized Intersection – intersection with restricted turn movements either by signs, pavement markings, medians, or other types of traffic control.

Climbing Lane – (or truck lane) A type of auxiliary lane that allows heavy or underpowered vehicles to ascend a steep grade without slowing other traffic. These do not affect the overall system capacity of a roadway.

CMF Clearing House – A database produced by the FHWA that contains over 2500 crash modification factors for over 700 safety countermeasures as part of the Highway Safety Manual's crash analysis methodologies.

Collision Diagram – See Crash Diagram

Commercial Goods Transport - Simulates how commodities are moved as freight by different modes of transport, such as marine, rail, and truck for a typical weekday.

Commercial Vehicle Operations (CVO) - Performs advanced functions that support commercial vehicle operations, including communications between drivers, fleet managers, and roadside officials; automates identification and safety processing at mainline speeds; and timely and accurately collects HazMat cargo information after a vehicle incident.

Comprehensive Plan – A generalized, coordinated land use map and policy document of a local government that interrelates all functional and natural systems and activities relating to the use of lands, including but not limited to sewer and water systems, transportation systems, educational facilities, recreational facilities, natural resources and air and water quality management programs.

Conflict Points – The crossing, merging, or diverging of two vehicular, bicycle, or pedestrian traffic movements on a roadway. These are the points where collisions are likely to occur.

Congestion – A condition on road networks that occurs with increased traffic volumes, and is characterized by slower speeds, longer trip times and increased queueing.

Congestion Management System (CMS) – A systematic process which provides information on transportation system performance and alternative strategies to alleviate congestion and enhance the mobility of persons and goods.

Congestion Pricing – The policy of charging drivers a fee that varies with the level of traffic on a congested roadway. Congestion pricing is designed to allocate roadway space more efficiently. Congestion pricing is also known as relief tolling, variable pricing, and road pricing.

Context Sensitive Solutions (CSS) – a collaborative interdisciplinary approach involving all shareholders to provide a transportation facility that integrates well within the physical environment and preserved the local scenic, aesthetic, historic and environmental resources while maintaining safety and mobility.

Coordinated (Signals) – Signals that are adjusted or connected so that they provide for continuous flow of traffic between intersections at a given speed. Coordinated signals all have the same speed. Coordinated signals can be timed, interconnected, or controlled from a central operations center.

Cordon – An imaginary boundary (non-linear) strategically drawn across an area. The volumes on the links crossing the cordon are typically summed to understand the number of trips entering and exiting an area.

Corridor Plan – A transportation plan that addresses a specific segment of the transportation system. Can address a single roadway or more likely, parallel multimodal facilities.

Corridor/Small Network - Expanded study area that typically includes one major corridor with one or two parallel arterials and their connecting cross-streets, typically less than 200 square miles (mi^2).

Cost Effectiveness Index Analysis Spreadsheet – adaptation of the HSM spreadsheets to analyze countermeasures for bicycle and pedestrian crashes on urban and suburban arterials.

Crash Coding Manual – A publication produced by ODOT that compiles data from reported motor vehicle traffic crashes occurring on city streets, county roads, and state highways.

Crash Diagram – A graphical illustration of historical crashes at a location including position, time of day, number of injuries and other conditions present.

Crash Decoder Tool – a macro-based spreadsheet tool that converts information from the PRC crash listing. This tool eliminates the need to use the ODOT Crash Code Manual.

Crash Modification Factor (CMF) – A multiplicative factor used to compute the expected number of crashes after implementing a given countermeasure at a specific site.

Crash Reduction Factor (CRF) – As used in ARTS, the inverse of CMF.

Critical Crash Rate – a crash rate that has been statistically adjusted based on other roads with similar characteristics to remove random chance elements.

Critical Hour Listing – Typically the top 100- 500 hours at a location used to determine when the 30th highest hour or other chosen design hour occurs.

Cycle Length – The time it takes for a signalized intersection to go through all movements and indications.

Critical Movement Analysis – A planning-level analysis methodology to estimate capacity of a signalized intersection with existing or forecasted volumes.

D-Factor - Percent of traffic in a single direction

Delay – The additional travel time experienced by a vehicle, bicycle, or pedestrian with reference to a base travel time, e.g., the free-flow travel time.

Detailed Level Analysis - A low-level analysis in which all or nearly all input data are known, and the analysis results will be used to make final decisions about roadway design elements, traffic control, and/or project approval.

Demand to Capacity Ratios (D/C) – Similar to the volume-to-capacity ratio (See Volume to Capacity Ratio) but is allowed to exceed and be reported at levels greater than 1.0. Typically used in transportation demand models in preliminary screening exercises as link volumes may exceed capacity.

Design - This project phase includes approved and funded projects that are going through analysis of the alternatives or preliminary design to determine the best option for implementation. This phase also includes the analysis of roadway features needed to operate at a desired level of service (LOS). The final design phase (e.g., horizontal/vertical alignments, pavement design, etc.) are not included in this category.

Design Hour Volume – The design hour volume is the amount of traffic that a new facility is designed to accommodate. The 30th highest hour traffic is generally used as the design hour for most highway facilities; however it also could be the average weekday volume, or average summer weekday, etc. as decided upon by each application.

Design Life – The number of years into the future that a project element operates satisfactorily considering increases in traffic demand volumes.

Design Speed – The maximum safe speed that can be maintained over a specified section of highway. The design speed of a roadway dictates which geometric design standards are used, such as stopping sight distance, radius of curves, and banking (super-elevation) of road surfaces. This differs from posted speed.

Desirable Condition – The ideal maneuver and PIEV distance used for calculating intersection functional area.

Detector – A device by which vehicle or pedestrian traffic registers its presence. The most common detectors are the inductive loop detectors in the pavement for vehicles and the push-button detectors for pedestrians. The most common use of detectors is at intersections where they can be used to manage the traffic and pedestrian signals. However, detectors are also used on freeways and freeway ramps to provide information such as speed and volumes for freeway traffic.

Deterministic – A mathematical model that is not subject to randomness. For a given set of inputs, the result from the model is the same with each application.

Diurnal Factors – Time of day factors used to estimate travel by hour of the day by splitting the daily demand into hourly components. They can be shown as peak (1 hour) or multi-hour.

Diverge – A movement in which a single stream of traffic splits into two separate streams without the aid of traffic control devices.

Division 51 – General reference to OAR 734-051, which pertains to Highway Approaches, Access Control, Spacing Standards and Medians.

Downstream Functional Area – Functional area for vehicles leaving an intersection.

Downtown Plan – A type of area plan focusing on the central business district usually evaluating pedestrian/bicycle/vehicle safety and operations and parking.

Dynamic Traffic Assignment (DTA) - A process for assigning vehicle routes in a simulation model based on network conditions. It is an iterative process that converges to a path assignment based on vehicle travel time and delay between origin and destination (O-D) points in the network. While sometimes used in practice to refer to the macro-or mesoscopic traffic assignment in a travel demand model such as Visum, for the purposes of this document, DTA refers to the microscopic dynamic traffic assignment within Vissim.

Economic Model – Model based on the state revenue forecast

Electronic Payment System - Allows travelers to pay for transportation services by electronic means, including tolls, transit fares, and parking.

Emergency Management - Represents public safety and other agency systems that support coordinated emergency response, including police, fire, emergency medical services, hazardous materials (HazMat) response teams, Mayday service providers, and security/surveillance services that improve traveler security in public areas.

Enhanced Interchange Safety Analysis Tool (ISATe) – tool from the HSM used to predict the frequency and characteristics of crashes on freeways and interchanges.

Environmental Justice – Process that ensures that highway projects do not disproportionately impact one segment of the population, e.g., low-income or minorities.

Exponential (compound) – Compound growth is an accelerating growth curve typically associated with brand new growth in an area that has plenty of land and road capacity. Application is typically limited to five years or less as use over a prolonged period can substantially overestimate future growth.

Expressway – An expressway is a divided highway facility usually having two or more lanes for the exclusive use of traffic in each direction and incorporating partial control of access.

External Goods Transport – Simulates freight movement for exports, imports and through the state.

External Station – A location where a roadway crosses the outside boundary of a travel demand model or zonal cumulative analysis.

Facility Analysis – Used to evaluate performance measures such as travel time, travel speed, vehicle hours of delay, and measures of congestion. Roadway operations are also evaluated when demand exceeds capacity (e.g. identifying bottlenecks and queue extents).

Facility Plan – A study that focuses on a specific facility. A facility can be any roadway, bikeway, or pedestrian path made up of multiple segments.

Fatal Flaw – A flaw in the design that would ultimately keep the facility from functioning as intended and/or violates certain policy or minimum design standards (where design exceptions would be unsafe).

Floating Car – A probe vehicle traveling with the traffic flow for the purpose of recording travel times, where the car is driven such that the number of vehicles that pass the "floating car" is equal to the number of vehicles that the "floating car" passes. This is how the floating car approximates the average travel time of the given section.

Flyover – A directional ramp structure that is typically used to remove a left turn movement out of an at-grade intersection to improve operations.

Focusing Model – A model with additional refinement and detail within a subarea. The additional resolution may be added to the transportation network or the zone structure.

Follower Density - the number of followers in a directional traffic stream over a unit length of highway.

Free-Flow Speed – Speed at which vehicles travel unimpeded by effects of other vehicles. Typically taken as five mph over the posted speed for planning and preliminary engineering applications.

Freeway – Divided highway with a minimum of two lanes for exclusive use of traffic in each direction, with grade-separated connections, and with full access control.

Freeway Management - Controls, guides, and warns traffic to improve the flow of people and goods on limited-access facilities. Examples of freeway management include the integration of surveillance information with freeway road geometry; vehicle control, such as ramp metering; dynamic message signs (DMS); and highway advisory radio (HAR).

Freight Route – A highway that has been recognized for its overall importance in intra and interstate commerce. May have specific mobility and design considerations applied.

Frontage Road – A roadway that parallels a major transportation facility, such as a freeway, and provides access to residents and businesses.

Functional Classification – FHWA (federal) classification of the urban and rural roadways based on the type of service the road provides.

Functional Area – (See: Influence Area) The area in which an intersection affects vehicle paths such as influencing driver decisions, vehicle movements, and vehicle queues.

Future Volume Table – Table that shows future AADT volumes for state highway segments based on historical traffic counts or travel demand model-based growth trends.

Gap - The time or distance between the back end of a leading vehicle and the front end of the following vehicle.

General Transit Feed Specification (GTFS) – Defines a common format for public transportation schedules and associated geographic information.

Goals and Objectives (G & O) – Primary desired outcomes on a project or plan. Evaluation criteria are based on these.

Grade – The slope (ratio of change in elevation to change in distance) of a roadway typically given in percent. For example, a 2% grade represents a 2-foot elevation change over a 100-foot distance.

Grade Separation – A vertical separation between intersecting roads or railroad tracks. One facility travels over the other via an overpass or other structure.

Gravity-Based Distribution – Trip distribution based on the gravity model which illustrates the distance, time and cost relationship between activities and their respective locations.

Growth Factor – A percentage increase applied to current traffic demands to estimate future demands. Expressed as 1 + the decimal percentage of the change (i.e. 1.34).

Growth Rate – This is the rate at which traffic volume is expected to increase annually on a specific facility.

Headway – The time between two successive vehicles as they pass a point on the roadway, measured from the same common feature of both vehicles (for example, the front axle or the front bumper), expressed in seconds.

Heavy Vehicle Percentage – Percentage of heavy vehicles within the traffic count (FHWA classifications 6 – 13).

Highway – High-speed roadway connecting major areas or arterials, with little or no traffic signal interruption (e.g., two-lane highway, expressway).

Highway Classification (Per OHP) – Classification based on FHWA (federal) Functional Class.

Highway Economic Requirements System (HERS-ST) - HERS-ST is a high-level planning analysis tool used for statewide, regional, and corridor planning studies.

Highway Safety Improvement Program (HSIP) – FHWA program that provides federal aid for safety projects on all public roads.

Historical Trends – Long-term trends identified from analysis of historical data

Hot/Cold Start Percentages – These are calculations used in air quality analysis. They provide an estimate of the amount of time vehicles have been running when they enter a section of roadway.

HOV Bypass Lane – Exclusive on-ramp lane for vehicles with a defined minimum number of occupants (more than one), including buses, taxis, carpools, for specified time periods.

HOV Lane – An exclusive road or traffic lane limited to buses, vanpools, carpools, emergency vehicles, and, in some cases, single occupant motorcycles. HOV lanes typically have higher operating speeds and lower traffic volumes than adjacent general-purpose lanes.

Incident – An event or condition on a roadway that impedes the normal flow of traffic.

Incident Management - Manages unexpected incidents so that the impact on the transportation network and traveler safety is minimized. Includes incident detection capabilities through roadway surveillance devices and incident response through coordination with freeway service patrols and emergency response agencies.

Induced Demand – A long-term economic response that typically occurs outside of a particular study area based on improvements or lack of to the transportation system infrastructure.

Influence Area – The overall length of a segment controlled by the operation of a geometric or other traffic control feature. This is much longer than the feature itself. Typically involves merge/diverge points or traffic signals.

Integrated Corridor Management (ICM) – An approach to managing the transportation network that encourages multi-agency coordination and combines arterial and freeway strategies to balance and manage travel demand across networks (freeway, arterial, transit, and parking).

Intelligent Transportation Systems (ITS) – Transportation technology that allows drivers, vehicles, devices, and system operators to gather and use real-time information to improve vehicle navigation, roadway system operations, or both.

Interchange Area Management Plan (IAMP) – A plan to determine transportation solutions or land use/policy actions needed in an interchange area and how best to balance and manage transportation and land use issues over time.

Interim Year – A forecast year between the base/existing year and the design or horizon year.

Isolated Intersections – Single crossing point between two or more roadway facilities with typical greater than a two-mile spacing to adjacent intersections.

J-turn – An intersection design to facilitate a minor street left turn onto a major street where a non-traversable median is present. This design accommodates all vehicles including trucks via a right turn followed by a larger radius U-turn.

Jam Density – Queue forming upstream of the bottleneck (maximum density)

K-Factor - Percent of ADT in the peak hour

KABCO – A scale of crash injury severities.

Latent Demand – A short-term driver-based response to added or removed constraints typically within a particular study area. This can be shown as change of route, time, or mode.

Land Development Model – Identifies land availability based on floor space prices and vacancy rates to rent or purchase.

Least Cost Planning – The process of comparing direct and indirect costs of demand and supply options to meet transportation goals, policies, or both, where the intent of the process is to identify the mix of options with the best value.

Light-Rail Line - Electric-powered railway system operating single cars or short trains on a variety of alignment types on a partially controlled right-of-way.

Link Diagram – A link and node representation of an intersection or transportation facility.

Links – A length of roadway between two nodes or points.

Logarithmic – A decelerating growth curve which tapers off as land approaches built-out status and capacity of roadways. Future growth is mainly contributed by growth in background (through) traffic.

LOS (Level of Service) – A quantitative measure describing operational conditions within a traffic stream and motorists' perceptions of those conditions. For example, LOS A represents free flow - almost complete freedom to maneuver within the traffic stream. LOS F represents forced flow - more vehicles are attempting to use the highway than can be served, resulting in stop-and-go traffic.

LOS C Volume – Term used in noise analysis. LOS C represents the level of congestion where speeds begin to reduce in a meaningful way. Therefore, LOS C represents the maximum volume at the maximum speed that produces the maximum noise.

Macroscopic Model – an aggregate model with a high-level view of the transportation system, which does not include many transportation network details. Macroscopic models are generally large and focus on the general flow of travel and route/mode choice from one area to another. System details usually approximates, or averages including number of lanes, free-flow speed, and vehicle capacity.

Managed Lanes - A lane that is restricted or controlled for a particular purpose (e.g., HOV lanes, bus only lanes, and regular and high-capacity toll lanes/HOT).

Merge– A movement in which two separate streams of traffic combine to form a single stream without the aid of traffic signals or other right-of-way controls.

Mesoscopic Model – A hybrid model that includes combinations or approximations of elements from both macroscopic and microscopic models. May include a routable network similar to a macroscopic model, while also incorporation more detailed operation elements of the transportation network to better estimate travel time based on traffic operation similar to a microscopic model. Accounts for queuing on each link but not at the individual vehicle level.

Methodology & Assumptions Memorandum – A memorandum describing all the volume development and analysis assumptions for the existing and future no-build and build conditions. Submitted for approval before analysis tasks begin.

Microscopic Model – A calibrated highly detailed model simulating individual vehicles and driver behaviors on a transportation network requiring a high degree of detail.

Microsimulation - Modeling of individual vehicle movements on a second or sub-second basis for the purpose of assessing the traffic performance of a transportation network.

Mitigation –An action that avoids, addresses, or modifies a negative impact. Typically used in environmental analysis and development review.

Mobility – The ability of the transportation system to facilitate the movement of people, goods and services to and from desired destinations.

Mobility Target – An Oregon Highway Plan volume-to-capacity ratio that indicates a desirable level of performance on a facility.

Mode Choice – The process used to determine the modeled choice in which a user will reach their intended destination (i.e. Car, bus, bike, walk...etc.).

Model Area - The total area to be modeled to accurately analyze the study area (an area equal to or greater than the study area).

Modernization – The process of updating current infrastructure for the purpose of increasing safety or functionality of the system.

Metropolitan Planning Organization (MPO) – An association of local agencies established by federal law to coordinate transportation planning and development activities within a metropolitan region.

Multi-Criteria Evaluation (MCE) – A tool used by Metro and ABM models to provide consistent output sets.

Multi-Modal – Multiple modes of transportation consisting of but not limited to automobile, bus, bicycle, and pedestrian travel.

Multi-Resolution – An integrated series of models, each built or scaled for the appropriate level of detail given the context of the project application and need.

National Transportation Communications for Intelligent Transportation System Protocol (NTCIP) – A family of standards designed to achieve interoperability and interchangeability between computers and electronic traffic control equipment from different manufacturers.

National Electrical Manufacturers Association (NEMA) - Vissim's default emulator for standard signal controller logic (prior to Vissim version 5.0). NEMA was developed internally by PTV America to replicate the common features of a signal controller.

No-build Volume – Can refer to existing conditions or more commonly to a set of future conditions without any of the subject plan/project improvements in place. The no-build will usually include other projects in the area that might be in a funded capital improvement and/or financially constrained plan. In the context of a travel demand model, no-build is thought of as a "do nothing" with no other project improvements assumed.

Nodes – Indicates the intersections of links.

Nomograph – a graph containing three parallel scales graduated for different variables so that when a straight line connects values of any two, the related value may be read directly from the third at the intersection point.

Non-Attainment Area – An area where air pollution levels persistently exceed the nation ambient air quality standards.

OHP Mobility (V/C) Target – Thresholds set by the Oregon Highway Plan for the volume to capacity ratio performance measure for each specific facility classification.

Operational Analysis – An application of a methodology where the user supplies all or nearly all required inputs to the procedure instead of using defaults. Should not be confused with the analysis of operations which could occur at any detail level.

Operations – Strategies and solutions that optimize or preserve the existing transportation system for mobility and safety through the process of improving the flow of the existing system.

Operations/Construction - These projects share many similar characteristics with design projects but are performed to determine the best approach for optimizing or evaluating existing systems.

Oregon Highway Plan (OHP) – A plan which establishes long-range policies and investment strategies for the State Highway System.

Oregon Transportation Plan (OTP) – Oregon's long-range multimodal transportation plan with the overarching goal of providing a safe efficient and sustainable transportation system that enhances Oregon's quality of life and economic vitality.

Origin – Destination (O-D) Study- Conducted as a part of an overall regional transportation study to identify travel patterns between the starting (origin) and ending (destination) points of trips within the region.

ORS 366.215 – Oregon Revised Statute written to restrict permanent reductions of vehicle-carrying capacity of over-height and oversize trucks along identified (i.e. Reduction Review Routes) freight routes.

Overlay Zone – Area established with special regulations that address specific subjects in addition to and used to modify the regulations of the base zone. Overlay Zones can exist in multiple types such as Buffer Zones, Environmental Zones…etc.

Park-and-Ride – Park-and-Ride lots are designed for automobile parking at outlying locations along transit routes.

Passing Lane - A type of auxiliary lane that allows vehicles to overtake slower ones. These do not impact the system capacity of the overall roadway. Passing lanes in steep terrain are also termed as a climbing lane.

Pedestrian Hybrid Beacon (PHB) – A user-actuated traffic control device with a "red" indication designed to help pedestrians safely cross busy roadways at midblock crossing and uncontrolled intersections.

Pedestrian Level of Traffic Stress (PLTS) – Multimodal methodology to classify pedestrian facilities according to their condition and proximity to motorized traffic on users.

Peer Review – An evaluation of work performed by one or more individuals of similar field and competence to the producers of the original work. Peer review is used as a method to maintain standards of quality, improve accuracy, and provide credibility.

Performance Measure – An individual quantitative or qualitative value that identifies the degree that a facility/strategy/action meets a certain goal, objective, or policy.

Person Travel – Person activities for a typical weekday simulated by a population synthesizer in an activity or tour-based travel demand model.

Phase – The part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals. A phase can include green, yellow change, red clearance, pedestrian, and bicycle intervals.

Phase Split (Length) – Duration of an individual interval in a signal cycle.

Phase Sequence (or Phase Rotation) - the order in which the various signal phases are served.

Planning - This phase includes short- or long-term studies or other State, regional, or local transportation plans (e.g., master plans, congestion management plans, ITS strategic plans, etc.).

Planning Analysis – An application of a methodology where most or all required inputs are defaulted.

PLANSAFE – A regional scale safety analysis tool.

Point Capacity – Capacity of a roadway section at a specific location typically within an auxiliary lane section.

Posted Speed – The posted speed is a regulatory sign identifying the legal speed on a roadway. It is based on a statistical sampling of existing traffic speeds, safety issues, etc., and is typically lower than design speed.

Population Synthesizer – simulates the population with observed Oregon characteristics.

Post-processing – Refers to additional processing of data after it's been collected to enhance the data or make the original data easier to understand. In the context of the APM this refers to the future volume development process merging traffic counts with relative changes between different travel demand model scenarios.

Practical Design – A design philosophy that is used to conserve resources while meeting system needs, balancing cost with system value, and following business practices.

Pre-breakdown Capacity – Segment capacity determined through application of CAFs relative to the freeway segment's base (ideal) capacity at the bottleneck

Predictive Method – A detailed Highway Safety Manual Part C methodology that calculates future crash frequency.

Preservation – Projects that maintain facilities but do not add significant safety or capacity improvements. Typically these are pavement, shoulder, curb/gutter/sidewalk and striping/signing projects.

Probe Data Analysis Tools – The range of tools available to evaluate existing or historical travel time and speed-based measures (e.g. travel time reliability) from commercial sources of probe data.

Production Location Model – Simulates where businesses (i.e. employment) are located.

Progression Analysis – Study conducted to optimize speed and delay in the traffic flow along a signalized corridor.

Project Development Stage – Final traffic analysis decisions are made using detailed operational methods (i.e. HCM, HSM, and others) about roadway design features, multimodal aspects, and traffic control.

Project Limits – the physical boundaries of a project usually defined by milepoints.

Project Prospectus – document that defines the major features of a project and includes enough detail to fairly scope the project.

Purpose and Need (P & N) – Explanation of what the project intends to address (purpose) and why it is necessary (need).

QCEW (Quarterly Census of Employment and Wages) Data – A quarterly report on employment and wages by industry, provided by the Oregon Employment Department.

Qualitative Multimodal Assessment (QMA) – A methodology that uses roadway characteristics and applies a context-based subjective "excellent/good/fair/poor" rating. This method applied when comparing different alternatives side-by-side or applied to a single scenario to compare the proposed improvement to existing conditions and to applicable standards.

Queue – A line of vehicles or pedestrians waiting to proceed through an intersection or bottleneck. Slow-moving vehicles or pedestrians joining the back of the queue are usually considered part of the queue.

Queue Discharge Rate – The average flow rate during oversaturated conditions

Queue Spillback – When traffic queues at an intersection or bottleneck build up to the point that they block turn lanes, driveways, or even upstream intersections. See Blocking Percentage.

Rail Grade Crossing Monitors - Manages traffic at highway-rail intersections where operational requirements demand advanced features. Includes the capabilities from the Standard Rail Crossing equipment package and augments these with additional safety features, including positive barrier systems and wayside interface equipment that detects or communicates with the approaching train.

Ramp – Short segment of roadway connecting two grade-separated roadway facilities often with mixed traffic characteristics of both.

Ramp Acceleration/Deceleration Lane Lengths – The distance from the ramp gore point to the ending/starting point of the taper.

Ramp Metering – (or ramp signal/metering light) is a device, usually a basic traffic light or a two-section signal light together with a signal controller, that regulates the flow of traffic entering freeways according to current traffic conditions.

Random Seed - A micro-simulation parameter in Vissim and other software that initializes a random number generator.

Rectangular Rapid Flashing Beacons (RRFB) – User-actuated amber LEDs that supplement warning signs at unsignalized intersections or mid-block crosswalks.

Reference Phase – Coordinated phases for an actuated signal

Refinement Plan – Level of transportation plan that focuses on a specific topic, feature, mode, or highway segment in a sub-area usually at a high level of detail.

Region (Regional) – Citywide or countrywide study area involving all freeway corridors and major arterial.

Regional Transportation Plan (RTP) - Identifies the long-term (20-year or longer horizon) transportation needs of a metropolitan area exceeding 50,000 population, incorporating projects from local TSPs, developing actions to address those needs, and prioritizing recommended projects in a financially constrained plan.

Reversible Lane – Roadway lane that changes directions during different hours of the day. These are typically used to help alleviate congestion by accommodating the peak direction of traffic.

Ring-Barrier Controller (RBC) - Vissim's default emulator for standard signal controller logic. RBC is a direct implementation of an actual real world signal controller firmware (D4) and includes more advanced features than the NEMA emulator.

Riparian – relating to or situated on the banks of a waterway.

Road Diet – A reduction in through-lanes for a given roadway; occurs within a "complete street" process that optimizes the available pavement width across all modes. Typically occurs with the conversion of a four-lane street down to two travel lanes and a two-way left turn lane. It is also known as roadway reconfiguration.

Roadway Functional Class – Federal classification of a roadway according to a jurisdiction by typical use and volume. Major categories typically include Interstates, other freeways, arterials, collectors, or locals.

Roundabout – Unsignalized intersection with a circulatory roadway surrounding a central island with all entering vehicles yielding to circulating traffic.

Rural – Areas with less than a population of 5,000 outside of established urban growth or metropolitan planning boundaries.

Safety Priority Index System (SPIS) – A method developed by ODOT to flag safety issues on state highways.

Safe Routes to School – A set or programs sustained by community leaders, local, state and federal governments to improve the health and well-being of children by enabling and encouraging them to walk and bicycle to school.

Saturation Flow Rate – The maximum departure (queue discharge) flow rate achieved by vehicles departing from the queue during the green period at traffic signals.

Screening/Screening Level Analysis – The process of evaluating and reducing the potential number of alternatives.

Screenlines – Imaginary lines that are strategically drawn across network links. The volumes on the links crossed by the screenlines are summed. Typical use of a screenline is to compare the volume of traffic entering and leaving the study area for each alternative.

Scoping – the process of identifying approach, tools and efforts based on analysis need prior to beginning a project.

Seasonal Adjustment – the process to adjust traffic volumes to reflect a specific time of year. This is typically the summer peak or the average weekday conditions.

Seasonal Factor – the calculated value used for adjusting traffic volumes to specific time of year.

Seasonal Trend Table – An ODOT-produced table of factors calculated from yearly patterns of automatic traffic recorder data used to estimate seasonal traffic count adjustments.

Seeding Period - The time between the start of the micro-simulation and when the network has the necessary number of vehicles in the system for the representative time period.

Segment – Typically a distance between two features (e.g. speed limit change, intersection, or off-ramp, etc.)

Segment-Based Analysis – Facility analysis performed between intersections or ramp junctions.

Sight Distance – A distance a vehicular driver needs to be able to see to have adequate room to stop or otherwise avoid an obstacle or collision.

Signal Progression – The timing of signals such that a group or platoon of cars arrives at a succession of green lights and proceeds through multiple intersections without stopping.

SimTraffic – performs micro simulation and animation of vehicle traffic, modeling travel through signalized and unsignalized intersections and arterial networks, as well as freeway sections, with cars, trucks, pedestrians, and buses. SimTraffic includes the vehicle and driver performance characteristics developed by the Federal Highway Administration for use in traffic modeling.

Special Events – Management of planned events so that the impact on the transportation network and traveler safety is minimized through coordination with other traffic management, maintenance and construction management, and emergency management centers, and event promoters.

STIP (State Transportation Improvement Program) – A multi-year, statewide, multi-modal program of transportation projects. The STIP must be consistent with the Oregon Highway Plan, Oregon Transportation Plan and regional and local transportation system plans.

Stochastic – Describes an outcome derived from random probability distribution that may be analyzed statistically but may not be predicted precisely (repeated attempts result in different results).

Storage Length – The available lane distance for holding queued vehicles.

Straight-Line Growth – Steady (linear) growth over time.

Study Area – The geographical area selected for analysis.

Sub-Area Modeling – Process for increasing the detail level in an existing travel demand model. The two major methods are Focusing and Windowing.

System Capacity – The general capacity of a roadway section outside of (i.e. upstream and downstream) of an auxiliary lane section.

System (Systemic) Level – Consideration of all transportation facilities and modes in a particular region.

System Peak Hour – The predominant peak hour used for all locations within a study area.

Terrain Class (Specific Grade) – Using ODOT's Vertical Grade Report for any segment that contains a grade that is either (1) between 2–3% and longer than ½ mile, or (2) steeper than 3% and longer than ¼ mile, should be analyzed as a specific grade (where the slope and grade length are required) rather than as a general terrain class.

TFlowFuzzy - A matrix estimation utility used to adjust a given O-D matrix in such a way that the result of the assignment closely matches desired volumes at points within the network.

Traffic Analysis Narrative Report (TANR) – Comprehensive traffic analysis report compiled at the end of a project serving as a legacy document supporting the final recommendations for the project.

Traffic Incident Management (TIM) – A multi-disciplinary effort to practice planned and coordinated detection, response, and clearance of traffic incidents.

Traffic Message Channel (TMC) –Refers to the ID number of predefined road segments that probe vehicle data is referenced to, used by HERE and other third-party transportation data providers as a commercial industry standard.

Transportation Analysis Zone (TAZ) – A geographic unit used in travel demand models. These contain data population, employment, and household characteristics, as well as other land use attributes.

Transportation Demand Management (TDM) – Actions or programs that encourage people to travel at alternative times or with fewer vehicles, e.g., rideshare/carpool programs, transit fare discount programs, and flextime.

Transportation System Plans (TSPs) – The long-range (20-yr) transportation plan for a city or county which contains (ideally) a financially constrained project list covering all applicable modes as the transportation element of the jurisdiction's comprehensive plan.

Travel Time – The time taken to travel between two points.

Transportation Strategy – A general approach to solving a transportation problem which can consist of policies, plans, and /or physical projects.

Transportation System Management (TSM) – Operation-based actions (e.g., ramp metering) that control or improve the flow and safety on the roadway system.

Transportation Systems Management & Operations (TSMO) – An integrated program to optimize the performance of existing multimodal infrastructure through implementation of systems, services, and projects to preserve capacity and improve the security, safety, and reliability of our transportation system.

Transportation System Plan (TSP) – A plan required by the Transportation Planning Rule (TPR) establishing a system of transportation facilities and services to meet state, regional and local needs.

Travel Demand Management (TDM) - TDM strategies are designed to maximize person throughput or influence the need for or time of travel. They are typically implemented in urban areas to reduce traffic congestion and air pollution, and to increase the efficiency of the transportation system. TDM strategies include employer trip reduction programs, vanpool programs, the construction of park-and-ride lots, and alternative work schedules.

Travel Demand Models – Computerized model that represents travel decisions that are consistent with the actual travel trends and patterns.

Travel Time Reliability - measures the extent of this unexpected delay or the consistency or dependability in travel times, as measured from day-to-day and/or across different times of the day.

TripCheck Local Entry (TLE) – A TripCheck feature that allows transportation agencies within Oregon to share information about construction and maintenance projects between one another.

TripCheck Traveler Information Portal (TTIP) – A portal which provides incident and road and weather data, in Extensible Markup Language format at no cost to over 175 public and non-public subscribers.

Trip Distribution – The number of trips that occur between each origin zone and each destination zone.

Trip Generation – The number of trips created from a specific type of land use. The ITE Trip Generation Manual provides the accepted source for estimates of vehicular traffic generation for various land use types. It is also the first step in the transportation forecasting process for travel demand models that estimate person trips based on housing and employment data.

Truck Lane – A designated lane for commercial vehicles, but not for public transit vehicles.

Unconstrained Assignment – A model run that has no link capacity constraints and uses free-flow speeds instead of congested speeds.

Upstream Functional Area – functional area for vehicles approaching an intersection.

Urban – Areas with moderate to high densities of development and population.

Volume-to-Capacity Ratio (v/c) – The ratio of the traffic flow rate to the capacity of the road. Volume to capacity ratios are limited to a maximum of 1.0. A v/c ratio reported over 1.0 is actually a demand-to-capacity ratio. See Demand to Capacity Ratio.

Validation –Testing a calibrated model under different conditions for reasonability.

Variable Advisory Speed (VAS) – Speed limits determined with a two-stage speed reduction scheme. It is intended to advise motorists to slow down because there is slowed or stopped traffic on the road ahead.

Variable Messaging – practice of using variable message signs with the capability of displaying a different message based on system needs or changing conditions.

Vehicle- Carrying Capacity – the horizontal and vertical distance above the road (a.k.a. "the hole in the air") for oversize trucks.

Vehicle Hours of Travel (VHT) – the sum of the travel times incurred by all motor vehicles in a specified system of highways for a given distance. VHT is calculated by multiplying the AADT value for each section/segment of road by the section/segment travel time (in hours) and summing all sections to obtain the VHT for the complete route.

Vehicle to Everything (V2X) – is when connected vehicles "talk" to anything besides roadside infrastructure or other vehicles on the road, like mobile devices.

Vehicle to Infrastructure (V2I) – is when connected vehicles "talk" to roadside infrastructure, like traffic signals.

Vehicle to Vehicle (V2V) – is when connected vehicles "talk" to other vehicles on the road.

Vissim Protocol – The ODOT document used to guide the proper creation of Vissim-based microsimulations.

Vehicle Miles of Travel (VMT) – the sum of the distances traveled by specific motor vehicles in a specified system of highways for a given period of time.

Volume Balancing – Mathematically balancing the traffic volumes between two points.

Weather Management - Includes automated collection of weather condition data and the use of that data to detect hazards such as ice, high winds, snow, dense fog, etc. This information can be used to provide road condition information and more effectively deploy maintenance and construction resources.

Weave – A section of a highway where two or more vehicle flows must cross each other's path. Weaving areas are usually formed when merge areas are closely followed by diverging areas.

Weaving Lane – A type of auxiliary lane typically occurring within an interchange between successive on and off ramps.

Weigh-In-Motion (WIM) – a permanently installed device for weighing vehicles in the traveled lanes.

Windowing Model – A cut out portion or "window" of an existing model that contains a subarea of the transportation network and creates cordon areas at the edge of the subarea. The window allows additional refinement, and greater detail while still maintaining consistency with the original model.

Work Zones - Uses traffic control devices (signs, channeling devices, barriers, etc.) and traveler information to maximize the availability of roadways during construction or maintenance while minimizing the impact on the traveling public and highway workers.

Zero-Volume Delay – Delay associated with traffic control devices. This is the expected delay that a single vehicle would encounter even if it were the only vehicle on the road.

Zonal Cumulative Analysis – A manual three-step analysis that involves the creation of zones and utilizes ITE Trip Generation methodologies, external trip O-D pairs, and gravity-based distribution.

Zoning Maps – Map of the local geographic area that defines current zoning designation and land use.

ABBREVIATIONS AND ACRONYMS

%HV – Heavy Vehicle Percentage

AADT – Annual Average Daily Traffic

AASHTO – American Association of State Highway and Transportation Officials

ABM – Activity-Based (Demand) Model

ACS – American Community Survey

ACT – Area Commission on Transportation

ADT – Average Daily Traffic

AES – Average Effective Speed

ALD – Aggregate Land Development

AMP – Access Management Plan

AMStrat – Access Management Strategy

AMU – Access Management Unit

AOC – Association of Oregon Counties

APLVM – Aggregate Probabilistic Limiting Velocity Model

API – Application Programming Interface

APM – Analysis Procedures Manual

APTS – Advanced Public Transportation System

AQMA – Air Quality Maintenance Area

ArcGIS – Geographic Information Software by ESRI.

ARTS – All Roads Transportation Safety Program

ATIS – Advanced Traveler Information System

ATL – Auxiliary Through Lane

ATR – Automatic Traffic Recorder

ATS – Average Travel Speed

ATSS – Advanced Transit Scheduling Systems

AVC – Automatic Vehicle Counter

AVCSS – Advanced Vehicle Control and Safety System

Ave – Avenue

AVL – Automatic Vehicle Location

AWDT – Average Weekday Traffic

AWSC – All Way Stop-Controlled

BCR – Benefit Cost Ratio

BFFS - Base Free-Flow Speed

BLI – Buildable Lands Inventory

BLOS – Bicycle Level of Service

BLTS – Bicycle Level of Traffic Stress

Blvd – Boulevard

BMP – Beginning Mile Post

BRT – Bus Rapid Transit

CAC – Citizen's Advisory Committee

CAF – Capacity Adjustment Factors

CAR Unit – Crash Analysis and Reporting Unit

CAV – Connected and Automated Vehicles

CCTV – Closed Circuit Television

CDS – Crash Data System

CHAMPS – Central Highway Approach Maintenance Permit System

CIP – Capital Improvement Program

CMF – Crash Modification Factors

CMS – Congestion Management System

COG – Council of Governments

CONVOL – Conflicting Flow Rates

CORSIM – Corridor Simulation Software by FHWA

CRF – Crash Reduction Factor

CT – Commercial Transport

CVO – Commercial Vehicle Operations

d/c – demand-to-capacity ratio

DDI – Diverging Diamond Interchange

DHV – Design Hour Volumes

DLCD – Department of Land Conservation and Development

DMS – Dynamic Message Signs

DMV – Department of Motor Vehicles

Dr – Drive

DTA – Dynamic Traffic Assignment

DUE – Dynamic User Equilibrium

DXFS – Data Exchange Format Specification

EA – Environmental Assessment

EB – Empirical Bayes

EBL – Eastbound Left

EBR – Eastbound Right

EBT – Eastbound Through

ED – Economics and Demographics

E-E – External – External (for model matrices)

EFU – Exclusive Farm Use

EIS – Environmental Impact Statement

EJ – Environmental Justice

EMME/2 – Travel demand modeling software by INRO

EP – Excess Proportion

EPM – Environmental Program Managers

ET – External Transport

EZCA – Enhanced Zonal Cumulative Analysis

FADT -– Future Average Daily Traffic

FC – Functional Class

FEIS – Final Environmental Impact Statement

FEMA – Federal Emergency Management Agency

FFS – Free Flow Speed

FHWA – Federal Highway Administration

FONSI – Finding of No Significant Impact

FR – Freight Route

FREEVAL – FREeway EVALuation (Software)

FTA – Federal Transit Authority

FYA – Flashing Yellow Arrow

GHG – Greenhouse Gas

GIS – Geographic Information System

GISU – GIS Unit

GreenSTEP – Greenhouse Gas Strategic Transportation Energy Planning Model

HAR – Highway Advisory Radio

HCM – Highway Capacity Manual

HCS – Highway Capacity Software

HDM – Highway Design Manual

HERS-ST – Highway Economic Reporting System **–** State Version

HGV – Vissim abbreviation for heavy vehicles such as trucks

HOC – Hours of Congestion

HOT – High Occupancy Toll

HOV – High Occupancy Vehicle

HPMS – Highway Performance Monitoring System

HSEC – Highway Safety Engineering Committee

HSIP – Highway Safety Improvement Program

HSM – Highway Safety Manual

Hwy – Highway

IAMP – Interchange Area Management Plan

ICU – Intersection Capacity Utilization

IDAS – ITS Deployment Analysis System

IHSDM – Interactive Highway Safety Design Model

I-E – Internal – External (for model matrices)

I-I – Internal – Internal (for model matrices)

ISATe – Enhanced Interchange Safety Analysis Tool

ISD – Intersection Sight Distance

ITE – Institute of Transportation Engineers

ITIS – Integrated Transport Information System

ITS – Intelligent Transportation Systems

JEMnR – Joint Estimated Model in R

LCDC – Land Conservation and Development Commission

LCOG – Lane Council of Governments

LCP – Least Cost Planning

LOS - Level of Service

LRT – Light Rail Transit

LSN – Local Street Network

LTS – Level of Traffic Stress

LUSDR – Land Use Scenario DevelopeR

MAC - Mobility Advisory Committee

MAZ – Micro-Analysis Zone

MCE – Multi-Criteria Evaluation

MEV – Million Entering Vehicles

MIS – Major Investment Study

MJL – Major street left turn approach

MMA – Multimodal Mixed-Use Areas

MMLOS – Multi-Modal Level of Service

MNL – Minor street exclusive left turn lane

MNLR – Minor street shared left-right approach

MNLTR – Minor street shared left-through-right approach

MNR – Minor street exclusive right turn lane

MOE – Measure of effectiveness

MOVES – MOtor Vehicle Emission Simulator

MP – Milepoint

MPMLUC – Modeling Procedures Manual for Land-use Changes

MPO – Metropolitan Planning Organization

MR3ST – Multilane Rural 3-leg, minor stop

MR4ST - Multilane Rural 4-leg, minor stop

MR4SG - Multilane Rural 3-leg, signalized

MRD – Multilane Rural Divided

MRU – Multilane Rural Undivided

MS2 – Modern Traffic Analytics

MUT – Multi Unit Truck

MUTCD – Manual of Uniform Traffic Control Devices

MVM – Million Vehicle Miles

MVMT – Million Vehicle Miles Traveled

MWVCOG – Mid-Willamette Valley Council of Governments

NAICS – North American Industry Classification System

NBL – Northbound Left

NBR – Northbound Right

NBT – Northbound Through

NCHRP – National Cooperative Highway Research Program

NEMA - National Electrical Manufacturers Association

NEPA – National Environmental Policy Act

NHI – National Highway Institute

NHS – National Highway System

NHTSA – National Highway Traffic Safety Administration

NIMS – National Incident Management System (FEMA)

NMDS – Non-Motorized Database System

OAR – Oregon Administrative Rules

OASIS – Oregon Adjustable Safety Index System

O-D – Origin-Destination

ODME – Origin-Demand Matrix Estimation

ODOT – Oregon Department of Transportation

OFP – Oregon Freight Plan

OHAS – Oregon Household Activity Survey

OHP – Oregon Highway Plan

OPL – Office of Project Letting

ORS – Oregon Revised Statutes

OSOW – Oversize/Overweight Vehicle

OSUM – Oregon Small Urban Model

OTC – Oregon Transportation Commission

OTMS – Oregon Traffic Monitoring System

OTP – Oregon Transportation Plan

P&N – Purpose and Need

PCE – Passenger Car Equivalent

pcph – Passenger Cars Per Hour

pcphgl – Passenger Cars Per Hour of Green Per Lane

pcphpl – Passenger Cars Per Hour Per Lane

PDLT – Project Delivery Leadership Team

PDO – Property Damage Only

PDT – Project Development Team

PE – Professional Engineer

PEL – Planning and Environmental Linkage

PFFS – Percent Free-Flow Speed

PHEV – Plug-in Hybrid Electric Vehicles

PHF – Peak Hour Factors

PHV – Peak Hour Volume

PI – Productions and Interactions

PIEV – Perception, intellection, emotion and volition

PLOS – Pedestrian Level of Service

PLTS – Pedestrian Level of Traffic Stress

PM – Particulate Matter

PMS – Pavement Management System

PPEAG – Planning and Preliminary Engineering Application Guide

PRC – Comprehensive (Crash) Report

PSA – Public Service Announcement

PSU – Pavement Services Unit

PSW – Preliminary Signal Warrants

PT – Project Team

PT – Person Travel

PTSF – Percent-Time Spent Following

PUD – Planned Unit Development

QMA – Qualitative Multimodal Assessment

R2U – Rural 2-lane undivided

R3ST – Rural 3-leg signalized

R3ST – Rural 3-leg minor stop

R4SG – Rural 4-leg signalized

R4ST – Rural 4-leg minor stop

RAME – Regional Access Management Engineer

RBC – Ring Barrier Controller

Rd – Road

REA – Revised Environmental Assessment

RICS Unit – Roadway Inventory and Classification Services Unit

RIRO – Right-In/Right-Out

RITIS - Regional Integrated Transportation Information System

RM – Residential Urban Medium Density

ROD – Record of Decision

ROW – Right of Way

RPAT – Rapid Policy Assessment Tool

RS – Residential Urban Standard Density

RSPM – Regional Strategic Planning Model

RTM – Regression to the mean

RTOR – Right Turn on Red

RTP – Regional Transportation Plan

RV – Residual Value

RRR - Reduction of Review Routes

RVC – Reduction of Vehicle Carrying Capacity

RVMPO – Rogue Valley Metropolitan Planning Organization

SAF - Speed Adjustment Factor

SBL – Southbound Left

SBR – Southbound Right

SBT - Southbound Through

SCFT – Small Community Forecast Tool

SCIP – Statewide Communication Interoperability Plan

SHRP2 -– Strategic Highway Research Program 2

SIDRA – Signalized and Unsignalized Intersection Design and Research Aid

SIP – State Implementation Plan (Air Quality)

SOV – Single Occupant Vehicle

SOW – Scope of Work

SPF – Safety Performance Functions

SPG – Synthetic Population Generator

SPIS – Safety Priority Index System

SPR – State Planning and Research Program

SPUI – Single Point Urban Interchange

St – Street

ST – Steering Team

STA -– Special Transportation Area

STIP – Statewide Transportation Improvement Program

STS – Statewide Transportation Strategy

SUT - Single Unit Trucks

SWIM – Statewide Integrated Model

TAC – Technical Advisory Committee

TANR – Traffic Analysis Narrative Report

TAZ – Transportation Analysis Zone

TCDS – Traffic Count Database System

TCM – Traffic Count Management

TCQSM – Transit Capacity Quality of Service Manual

TDD – Transportation Development Division

TDM – Transportation Demand Management

TDMS – Transportation Data Management System

TDS – Transport Data Section

TEV – Total Entering Volume

TGM – Transportation and Growth Management Program

TIA – Traffic Impact Analysis

TIGER – Transportation Investment Generating Economic Recovery

TIP – Transportation Improvement Program

TIS – Transportation Impact Study

TMC – Turning Movement Count

TPR – Transportation Planning Rule

TRD – Total Ramp Density

TRI – Transportation Research Institute

TRS – Traffic-Roadway Section

TEV – Total Entering Volume

TGM – Transportation Growth Management

TIA – Traffic Impact Analysis

TIP – Transportation Improvement Program

TIS – Transportation Impact Study

TM – Technical Memorandum

TMC – Traffic Management Center

TOC – Traffic Operations Center

TMIP – Travel Model Improvement Program

TPAU – Transportation Planning Analysis Unit

TPOD – Transportation Planning Online Database

TPR – Transportation Planning Rule

TPU – Transportation Planning Unit

TR – Traffic Responsive

TRB – Transportation Research Board

TRI – Transportation Research Institute

TRS – Traffic-Roadway Section

TS – Transport Supply

TSM – Transportation System Monitoring

TSMO – Transportation Systems Management and Operations

TSP – Transportation System Plan

TSRM – Technical Services Resource Manager

TT – Tractor Trailer

TTI – Texas A& M Transportation Institute

TTI – Travel Time Index

TUDI – Tight Urban Diamond Interchange

TVT – Transportation Volume Tables

TWLTL – Two-Way Left-Turn Lane

TWSC – Two-Way Stop Controlled

U2U – Urban 2-lane undivided

U3SG – Urban 3-lane signalized

U3ST – Urban 3-leg minor stop

U3T – Urban 3-lane with two-way left turn lane

U4D – Urban 4-lane divided

U4SG – Urban 4-leg signalized

U4ST – Urband4-leg minor stop

U4U – Urban 4-lane undivided

U5T – Urban 5-lane with two-way left turn lane

UBA – Urban Business Area

UGB – Urban Growth Boundary

UTDF – Universal Traffic Data Format

v/c – Volume to capacity ratio

VCR – Volume to capacity ratio

vdf – volume delay function

VHD – Vehicle Hours of Delay

VHT – Vehicle Hours Traveled

VMS – Variable Message Sign

VMT – Vehicle Miles Traveled

Vph – Vehicles Per Hour

vphpl – Vehicle Per Hour Per Lane

VR – Volume Ratio

WBL – Westbound Left

WBR – Westbound Right

WBT – Westbound Through