Analysis Procedures Manual

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6 SEGMENT ANALYSIS

6.1 <u>Purpose</u>

For analysis purposes, roadway facilities are separated into categories that are specific to traffic flow type: Uninterrupted and Interrupted traffic flow.

This chapter presents commonly used segment (uninterrupted flow) analysis procedures and identifies specific methodologies and input parameters to be used on ODOT projects. Topics covered include:

- Freeways
- Multi-Lane Highways
- Two-Lane Highways

6.2 <u>Freeways</u>

The analysis of freeways is generally broken down into the major components of the freeway system including basic freeway segments, ramps and ramp junctions and weaving segments. The analysis procedures used for each of these components are described below.

6.2.1 Basic Freeway Segments

Basic freeway segments include the portions of freeway where flow is not influenced by the diverging, merging, or weaving associated with ramp/freeway connections. The common methodology used for analyzing basic freeway segment operations is from Chapter 23 of the *HCM 2000*. The primary factors that affect operations on basic freeway segments include: lane widths, lateral clearance, the number of lanes, interchange density, heavy vehicles, grades and driver familiarity. For a complete description of the analysis methodology, refer to Chapter 23 of the *HCM 2000*.

While the *HCM 2000* methodology uses level of service as a performance measure (based on vehicle density in passenger cars per mile per lane), volume/capacity ratios can be calculated from this analysis for comparison against ODOT's adopted mobility standards by following the steps listed below.

1. Assuming level of service E/F threshold represents capacity, determine the segment capacity by interpolating between the values for "maximum service flow rate" at level of service E displayed in Exhibit 23-2 of the *HCM 2000* for the appropriate free-flow speed. Free-flow speed will be either calculated by this methodology assumed to be 5 mph greater than posted, or observed in the field.
2. Divide the calculated flow rate (v_p) by the interpolated capacity to obtain a volume/capacity ratio. Note: The units are passenger cars per hour per lane (pcphpl), not vehicles per hour.

6.2.2 Ramps and Ramp Junctions

The analysis associated with operations at ramp junctions with the freeway mainline typically involves the effects of vehicles either merging onto or diverging from the mainline. The common methodologies used for analyzing these movements are those from Chapter 25 of the *HCM*.

These methodologies focus on an influence area of 1,500 feet (downstream from ramp if merging and upstream from ramp if diverging). It should be noted that while the *HCM* methodology defines the influence area of merging or diverging traffic to be within 1,500 feet, the effects can extend outside of this area. The analysis for merging and diverging areas is discussed further below.

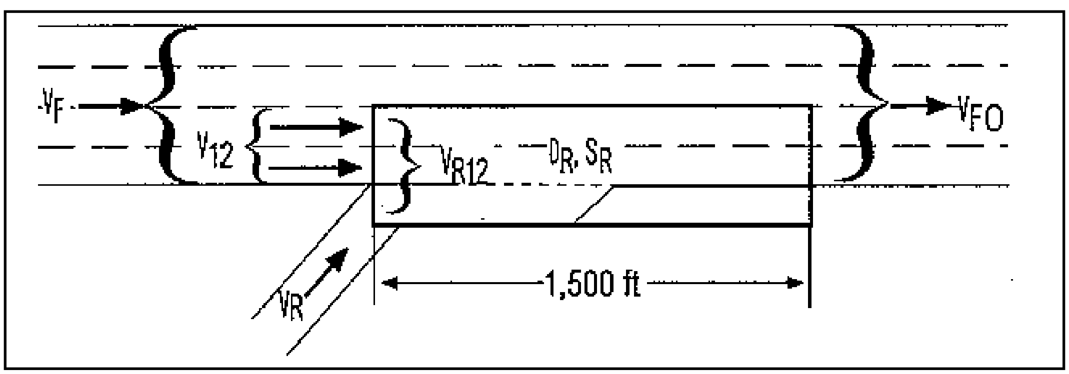
Merging Analysis

Merging analysis is often conducted at freeway on-ramps where vehicles from the ramp are entering a lane used by mainline traffic. In following the *HCM* methodology for merging analysis, there are three primary steps:

1. Predicting the flow rates entering lanes 1 and 2.
2. Determining capacity.
3. Determining level of service. Note that the performance measure of level of service is not used by ODOT and, therefore, this step will not be discussed.

The primary factors influencing the flow rates in lanes 1 and 2 (v_{12}) immediately upstream of the merge influence area are the total freeway flow rate approaching the merge area (v_F), the total ramp flow rate (v_R), the length of the acceleration lane and the ramp free-flow speed at the point of merging. The total flow rate entering the merge influence area (v_{R12}) is calculated by adding the flow rate remaining in lanes 1 and 2 (v_{12}) and the total ramp flow rate (v_R), as illustrated in Exhibit 6-1.

Exhibit 6-1 Freeway Merging Variables



Once the total flow rate entering the merge influence area (v_{R12}) has been calculated, it can be divided by the maximum desirable flow rate entering the merge influence area (4600 passenger cars per hour) to obtain a volume to capacity ratio for the merge influence area. When total flow rates for merge influence areas exceed capacity, locally high densities will occur, but freeway queuing will not always form as a result because mainline traffic will typically shift into the outermost lanes to avoid the merging traffic. Freeway queues are more likely to result in these situations where there are only two lanes for mainline traffic, forcing all vehicles to pass through the merge influence area. The *HCM* attempts to account for the amount of V_{12} traffic with the equations on *HCM* Exhibit 25-5. These equations are based on variables such as acceleration length, distance to next ramp, ramp volume, etc.

In addition to determining the volume to capacity ratio of the merge influence area, the volume to capacity of the downstream basic freeway segment should be checked to ensure the added traffic from the ramp does not create a downstream bottleneck. In cases where the total departing freeway flow rate (v_{FO}) is greater than the capacity of the downstream freeway segment (see Section 6.2.1), queues will form immediately downstream that will result in failure at the ramp connection, regardless of the whether flow rate entering the merge influence area has exceeded its capacity or not.

Exhibit 25-7 in the *HCM* displays capacities for merge areas including downstream freeway segment capacities (taken from Basic Freeway Segment chapter), as well as merge influence area capacities (where the maximum v_{R12} is always 4600 passenger cars per hour).

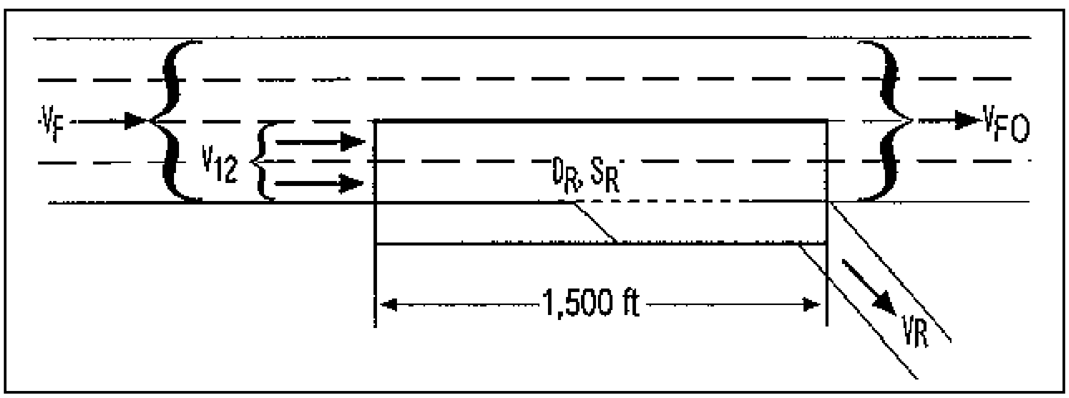
Diverging Analysis

Diverging analysis is often conducted at freeway off-ramps where vehicles from the mainline are departing to the ramp from a lane used by mainline traffic. The *HCM* methodology for diverging analysis is similar to that discussed above for merging, with three primary steps:

1. Predicting the approaching freeway flow in lanes 1 and 2.
2. Determining capacity.
3. Determining the density of flow within the ramp influence area. This step will not be discussed as the density is used to determine the performance measure of level of service, which is not used by ODOT.

For diverging analysis, the approaching flow rate in lanes 1 and 2 (v_{12}) is predicted for a point immediately upstream of the deceleration lane and includes the ramp flow rate (v_R) as illustrated in Exhibit 6-2. Models for predicting v_{12} can be found in Exhibit 25-12 of the *HCM*.

Exhibit 6-2 Freeway Diverging Variables



The primary cause of failure in diverge areas is inadequate capacity of an exit leg, whether on the freeway itself or the off-ramp. Capacities for downstream freeway legs can be obtained from Exhibit 25-14 (taken from Basic Freeway Segment chapter) from the *HCM*, and off-ramp capacities can be obtained from Exhibit 25-3. With these capacities known, volume to capacity ratios can be calculated by dividing the downstream freeway flow rate (v_{FO}) by the downstream freeway leg capacity and the ramp flow rate (v_R) by the ramp capacity.

Failure in diverge areas can also occur when the capacity of the freeway segment within the diverge area is exceeded. Capacities for upstream freeway segments can be obtained from Exhibit 25-14 (same as for downstream freeway segments) from the *HCM*. With this capacity known, a volume to capacity ratio can be calculated by dividing the freeway flow rate upstream of the diverge (v_F) by the capacity of the upstream freeway segment.

In addition to these conditions, the flow rate entering lanes 1 and 2 (v_{12}) immediately upstream of the deceleration lane should be checked to see if it exceeds the maximum desirable level. A volume to capacity ratio for this area can be calculated by dividing the approaching flow rate (v_{12}) by the maximum desirable flow rate of 4400 passenger cars per hour (Exhibit 25-14 of *HCM*). Unlike the other conditions described above, the condition where the flow rate entering lanes 1 and 2 exceeds the maximum desirable level may create locally high densities, but may not always result in freeway queuing because mainline traffic will typically shift into the outermost lanes to avoid the diverging traffic. Freeway queues are more likely to result in these situations where there are only two lanes for mainline traffic, forcing all vehicles to pass through the diverging area.

6.2.3 Weaving Segments

Weaving Configurations

Another necessary step before the analysis can be conducted is the determination of the weaving type, which is based on the number of lane changes required of each weaving movement. The *HCM* methodology identifies three types of geometric configurations for weaving areas. Each of these types of configurations is described below, with diagrams provided in Exhibit 6-4.

- **Type A**: Weaving vehicles in both directions must make one lane change to successfully complete a weaving maneuver.
- **Type B**: Weaving vehicles in one direction may complete a weaving maneuver without making a lane change, whereas other vehicles in the weaving segment must make one lane change to successfully complete a weaving maneuver.
- **Type C**: Weaving vehicles in one direction may complete a weaving maneuver without making a lane change, whereas other vehicles in the weaving segment must make two or more lane change to successfully complete a weaving maneuver.

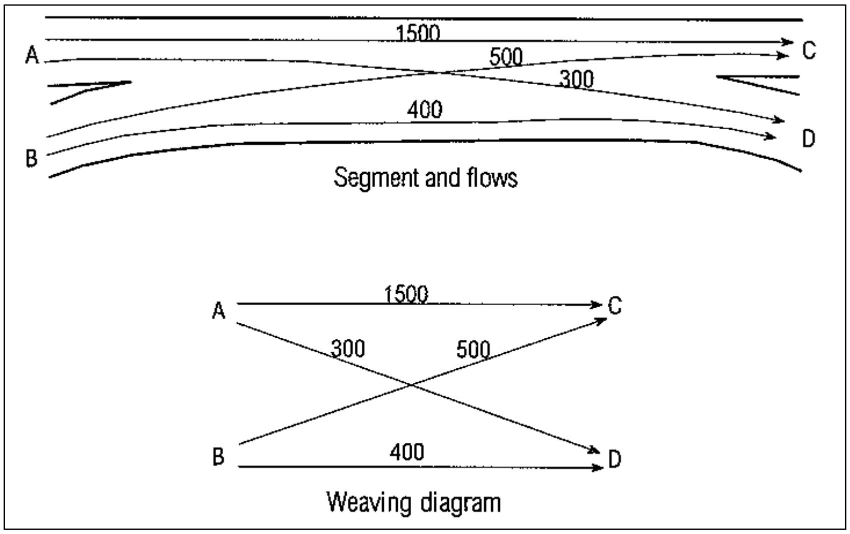
Typically weaving segments are formed when merge areas are followed closely by diverge areas (within 2,500 feet) and the two are joined by an auxiliary lane requiring the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway without the aid of traffic control devices. Note that when one-lane on-ramps are followed by one-lane off-ramps and the two are not connected by an auxiliary lane, weaving analysis is not conducted and the merge and diverge areas are analyzed independently using the procedures previously described. Recognition of configurations that could result in weaving is critical in highway operations analysis, as weaving areas require intense lane changing maneuvers that create a significant amount of turbulence. ODOT prefers the use of the *HCM* methodology for analyzing weaving maneuvers, but also supports the use of the Leisch Method in cases where engineering judgment suggests *HCM* results are not accurately reflecting conditions. For weaving areas greater than 2,500 feet use the more conservative of either the merge/diverge or Leisch methods.

The *HCM* discusses weaving concepts in Chapter 13 and the analysis methodology in Chapter 24. While most analysts will take advantage of the practicality of the Highway Capacity Software (HCS), which will perform all needed calculations to analyze weaving areas, it is important to have a basic understanding of weaving characteristics and key input parameters for use with HCS.

Weaving Diagrams

With a weaving area identified for analysis, a weaving segment diagram should be created to clearly identify the traffic flow rates associated with each movement, i.e., mainline to mainline, mainline to off-ramp, on-ramp to mainline, and on-ramp to off-ramp. An example of a weaving segment diagram is shown in Exhibit 6-3.

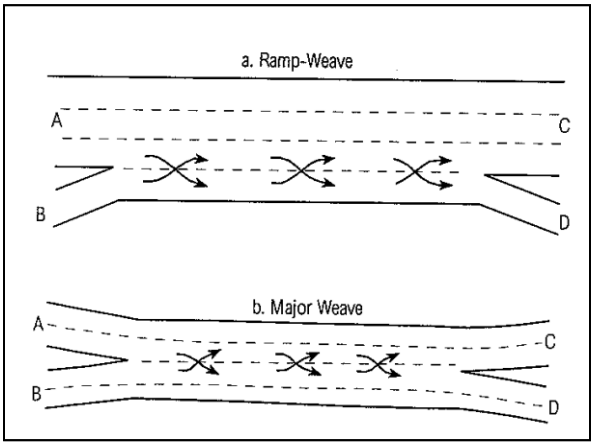
Exhibit 6-3 Weaving Diagram



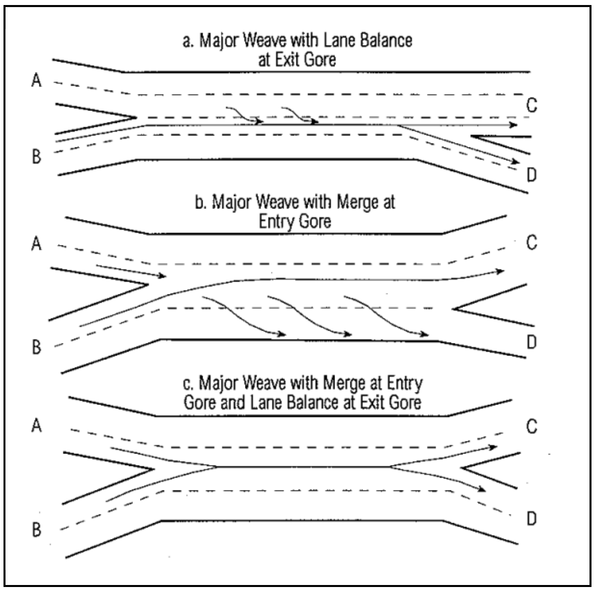
As can be seen, identifying the volume of traffic associated with each movement will require specialized data collection compared to typical counts that would only count traffic entering and leaving the area without noting its origin. Origin-destination surveys may provide the best data to use for a weaving analysis, but are not always practical to conduct and can be expensive. Some types of origin-destination studies, such as where traffic is stopped to conduct interviews, are more expensive than others such as license plate surveys. The less costly types of surveys are commonly through direct observation (usually recorded with video in the field and counted later) where all movements can be seen from one vantage point or through license plate identification. Another method sometimes available is the use of select-link output from a transportation demand model in combination with a common volume survey where a travel demand model has been created for the area. This method is especially useful for future scenarios where travel patterns may be different than current conditions. If either of these methods is not possible or practical for the particular area, the analyst may be required to apply engineering judgment in considering area characteristics such as land uses, topography and the area transportation network to create these movements from a common volume survey.

Weaving sections come in three configurations; Type A, B and C. Exhibit 6-4 shows the three types. Type A requires a lane change to get into or out of the auxiliary lane. Type A weaves are the most common type which occur mainly between interchanges that have a large portion of local trips that travel between them. High weaving volumes can cause Type A weaves to have poor operations. Type B weaves only require one lane change for either the mainline or ramp movement. These do not "trap" vehicles in the weaving section, so speeds are higher and operate much better than Type A weaves. Type C weaves require more than one lane change to perform the weaving maneuver and generally only operate well if the movement that must change lanes multiple times has a small volume. Type C weaves are relatively uncommon, are generally discouraged, but may exist in older highway alignments.

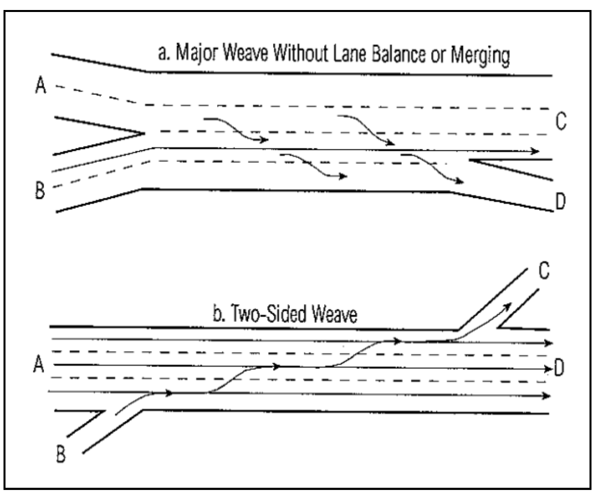
Exhibit 6-4 Weaving Configurations



Type A Configuration



Type B Configuration



Type C Configuration

Constrained vs. Unconstrained Conditions

Applying the weaving methodology, other geometric characteristics must be described including whether the weaving area is operating under constrained or unconstrained conditions and identifying the length of the weaving area. The determination of whether a weaving segment is operating under constrained or unconstrained conditions is based on the relationship between the number of lanes that must be used by weaving vehicles to achieve equilibrium with non-weaving vehicles (N_W) and the maximum number of lanes that can be used by weaving vehicles for a given configuration ($N_W(max)$). Where $N_W < N_W(max)$, conditions are described as unconstrained because there are no impediments to weaving vehicles' ability to achieve equilibrium with non-weaving traffic. Where $N_W > N_W$ (max), conditions are considered to be constrained because weaving vehicles are not provided enough roadway width as would be needed to reach equilibrium. Under constrained operation weaving vehicles often experience operating conditions much worse than those experienced by non-weaving vehicles, while under unconstrained conditions weaving and non-weaving vehicles usually experience similar operating conditions.

The calculation of N_W and $N_W(max)$ is determined by the configuration type, i.e., Type A, B, or C, and speeds of weaving and non-weaving vehicles. See Exhibit 24-7 in the *HCM*. When using the HCS to perform calculations, the analyst will only be required to determine the configuration type, free-flow speed and total number of lanes in the weaving section. However, an understanding of the characteristics of constrained and unconstrained conditions is important when analyzing weaving areas.

Weaving Length

Because weaving vehicles must execute all lane changes between the entry and exit gores, weaving lengths are measured from a point at the merge gore where the right edge of the freeway shoulder lane and the left edge of the merging lane are 2-feet apart to a point at the diverge gore where the two edges are 12-feet apart. Weaving lengths are limited to 2,500 feet in the *HCM* methodology. For weaving areas greater than 2,500 feet, use the more conservative of either the merge/diverge or Leisch methods.

Weaving Density

The key element of the *HCM* weaving analysis methodology is the calculation of the weaving area density, which is determined by incorporating weaving characteristics such as flow rate, configuration and free-flow speed. For a complete description of the density calculation refer to Chapter 24 of the *HCM*. The *HCM* uses the performance measure of level of service to rate weaving operations, which is directly related to the density calculated according to Exhibit 6-5.

Exhibit 6-5 Level of Service Criteria for Weaving Segments

Level of Service	Density (Passenger Cars/Mile/Lane)	
	Freeway Weaving Segment	Multi-Lane and Collector-Distributor* Weaving Segments
A	< 10.0	<12.0
B	10.0 – 20.0	12.0 – 24.0
C	20.0 – 28.0	24.0 – 32.0
D	28.0 – 35.0	32.0 – 36.0
E	35.0 – 43.0	36.0 – 40.0
F	>43.0	>40.0

* See page 24-19 of the HCM – research is unclear on applicability of LOS criteria to collector-distributor roads.

Weaving Capacity

While ODOT does not use level of service for evaluating facility performance, the density of the weaving section is still used to determine the volume to capacity ratio. If the capacity of the weaving section is equated to the level of service E/F threshold shown in Exhibit 6-5, then the capacity of a freeway weaving section would occur at a density of 43 passenger cars per mile per lane. The capacity in passenger cars per hour at this density can be found through the following iterative process.

1. Complete the analysis using the *HCM* methodology. While this methodology will produce a level of service, which is not needed, it will also produce a density.
2. The capacity of the weaving section will be equal to the total entering flow rate that results in a calculated density of 43 passenger cars per mile per lane (for freeways). Using the flow rates from the initial analysis, begin an iterative process by multiplying each movement flow rate by a common factor until the resulting density reaches, but does not exceed, 43 passenger cars per mile per lane.
3. Add the individual movement flow rates that produced the target density to obtain the total entering flow rate, which will be taken as the weaving section capacity.

The volume to capacity ratio for the section can now be calculated by dividing the original total entering flow rate by the capacity (total entering flow rate resulting in target density). This process of iteration will typically require fewer than ten attempts. The same procedure can be used for weaving analysis of non-freeway facilities, but a different target density for the capacity will be required, as shown in Exhibit 6-5 for multi-lane and collector-distributor roadways.

In addition to v/c ratio, the weaving section volume ration (VR) and speeds should be reported. The VR is the ratio of the weaving flow rate to the total flow rate. The *HCM* provides recommended upper limits on volume ratios. The difference between weaving and non-weaving speeds is a form of speed differential, which is preferred to be 10 mph or less for safety. Conditions exceeding these values should be examined using more detailed analysis methods such as simulation.

Example 6-1 Weave Capacity Example

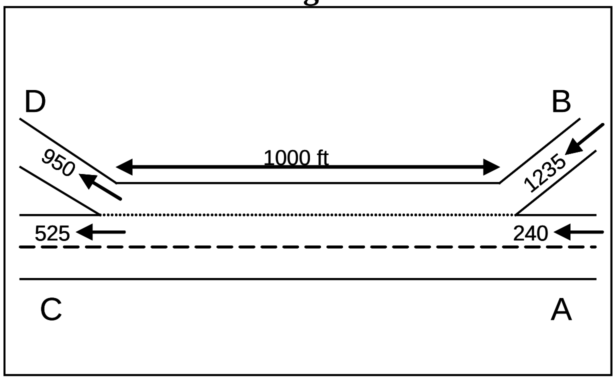
Given: Type A weave

- 12 ft lanes
- 6ft lateral clearance
- 1000 ft weaving distance
- 35 mph posted speed
- Multilane highway segment
- 5% Trucks
- PHF = 0.95
- Driver population factor = 0.95
- Volumes in vehicles per hour
- Weaving and non-weaving flow distributions

Find: Volume-to-Capacity ratio for weaving section

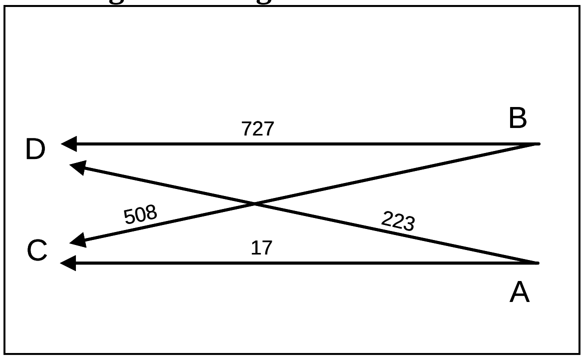
This example problem is based off of an actual project alternative. The lane and volume diagram shows the layout of the Type A weaving section and the volumes in vehicles per hour. The weaving section was created between a free-right turn at “B” and a loop off-ramp at “D” on a multilane roadway at an interchange.

Lane and Volume Diagram



The lane volumes were converted into weaving (A-D and B-C) and non-weaving (B-D and A-C) volumes as shown below. In this case, future distributions were available from a cumulative analysis procedure. Other sources of weaving volumes include field collected origin-destination data such as by tracking vehicle license plates. Where a travel demand model is present, select link runs can help estimate weaving movements.

Weaving Flow Diagram



The given information is then input into the *HCM* weaving procedure. The *HCM* result is a flow rate (in passenger cars per hour) of 1673 pc/h with a corresponding density of 17.41 pc/mi. The target density is 40 pc/mi, which is the density at capacity (v/c = 1.00) for a multilane or collector-distributor roadway. The table below was then iteratively created by multiplying the flow rates by a common factor until the density was as close as possible to 40 pc/mi.

Multiplied Flow Rates

Iteration	Factor	A-C (pc/h)	B-D (pc/h)	A-D (pc/h)	B-C (pc/h)	Flow rate (pc/h)	Density (pc/h)
1	1x	19	825	253	576	1673	17.41
2	2x	38	1650	506	1152	3346	47.83
3	1.5x	57	1238	380	864	2539	33.76
4	1.8x	34	1485	455	1037	3011	41.92
5	1.7x	32	1402	430	979	2843	39.04
6	1.74x	33	1436	440	1002	2911	40.20
7	1.73x	33	1427	438	996	2894	39.91
8	1.735x	33	1431	439	999	2902	40.05
9	**1.733x**	**33**	**1430**	**438**	**993**	**2899**	**39.99**

As can be seen from the last line in the table, the target density was reached at a flow rate of (rounded) 2900 pc/h. The 2900 pc/h flow rate is taken as the capacity in the v/c calculation. The sum of the original weaving and non-weaving flow rates is taken as the volume in the v/c calculation. The resulting v/c ratio would be:

Weaving v/c = 1673 / 2900 = 0.58

This v/c ratio is of an acceptable level. However, in doing the calculations it was found that the VR of 0.50 exceeds the maximum allowed by the methodology (0.45). Note: “c” at the end of *HCM* Exhibit 24-8 indicates that 3-lane type A segments do not operate well at volume ratios above .45, and may have poor operations and localized queuing. In addition, the difference between the weaving speeds (27 mph) and the non-weaving speeds (40 mph) is greater than 10 mph, which indicates a much greater potential crash risk. Simulation afterwards confirmed the poor operations as predicted even though the v/c ratio was acceptable.

It should also be noted, if using the HCS to perform calculations, that this program will provide warnings on the output sheet regarding limitations of this methodology that may not be reflected in the analysis results. It is the analyst’s responsibility to check these conditions to be sure the analysis results are valid. In addition, as with all types of analysis procedures, the analyst should verify that the results obtained appear to be reasonable for the given scenario. If they are not, the assumptions and input parameters should be reevaluated for errors. Should the results continue to appear inaccurate after making these types of adjustment, the analyst may consider applying a different methodology.

6.3 Multi-Lane Highways

Analysis procedures for uninterrupted-flow multi-lane highways are provided in Chapter 21 of the *HCM*. Highways analyzed with this procedure must maintain a minimum of two travel lanes in each direction, would typically have direct access allowed through driveways and at-grade intersections, and must maintain uninterrupted flow. Highways with access limited to on-ramps and off-ramps should be analyzed using the Basic Freeway Segment methodology. In addition, highways experiencing interrupted flow from influences such as traffic signals and on-street parking should be analyzed using a different methodology, such as the Urban Streets methodology from the *HCM*.

These procedures are very similar to those previously described for basic freeway segments, with slightly different input data needs. The most notable differences include the need to account for median type and access density. For a complete description of the analysis methodology, refer to Chapter 21 of the *HCM*.

While the *HCM* methodology uses level of service as a performance measure (based on vehicle density in passenger cars per mile per lane), volume/capacity ratios can be calculated from this analysis for comparison against ODOT's adopted mobility standards by following the steps listed below. Note that separate volume/capacity ratios must be calculated for each direction of travel.

1. Assuming level of service E/F threshold represents capacity, determine the segment capacity by interpolating between the values for "maximum service flow rate" at level of service E displayed in Exhibit 21-2 of the *HCM* for the appropriate free-flow speed. Free-flow speed will be either calculated by this methodology or assumed.
2. Divide the calculated flow rate (vp) by the interpolated capacity to obtain a volume/capacity ratio.

6.4 <u>Two-Lane Highways – SEE APM VERSION 2 ADDENDUM 11B, Two-Lane Highways</u>

6.4.1 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.

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.
1. Upstream of the passing lane.
2. The passing lane, including tapers.
3. Downstream of the passing lane, but within its effective length.
4. 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 capacity of a two-lane 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).

7 INTERSECTION ANALYSIS

7.1 Purpose

This chapter presents commonly used intersection (interrupted flow) analysis procedures and identifies specific methodologies and input parameters to be used on ODOT projects. Topics covered include:

- Turn Lane Criteria
- Intersection Capacity Analysis
- Traffic Signal Warrants
- Estimating Vehicle Queue Lengths

7.2 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.

7.2.1 Left Turn Lane Criteria – Unsignalized Intersections

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

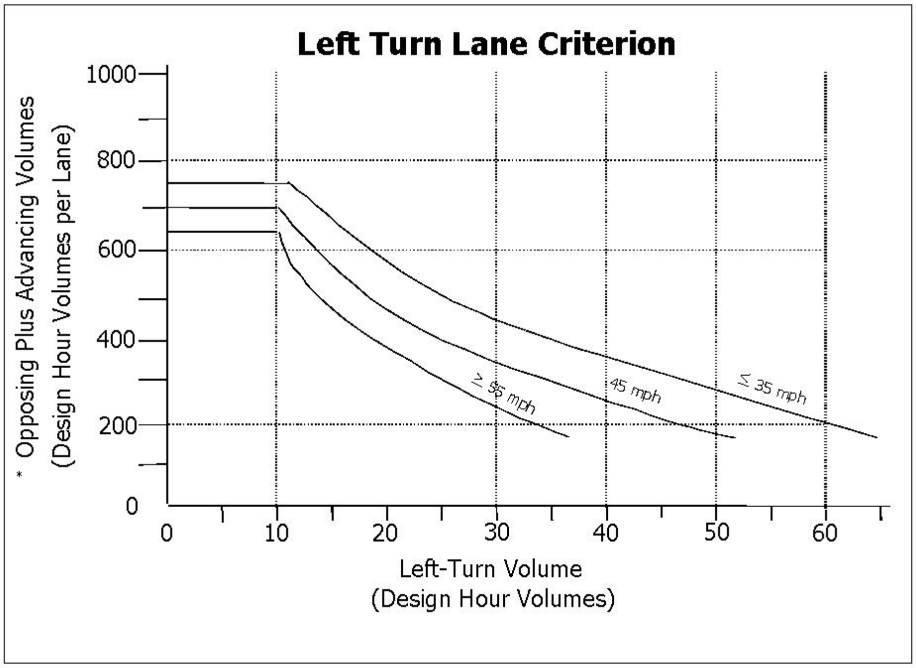
- A left turn lane should be installed, if criteria 1 (Volume) or 2 (Crash) or 3 (Special Cases) are met, unless a subsequent evaluation eliminate it as an option; and
- The Region Traffic Engineer must approve all proposed left turn lanes on state highways, regardless of funding source; and
- Complies with Access Management Spacing Standards; and
- 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 left turn lane. The volume criteria is determined by the Texas Transportation Institute (TTI) curves in Exhibit 7-1.

The criteria is not met from zero to ten left turn vehicle per hour, but indicates that careful consideration be given to installing a left turn lane due to the increased potential for accidents 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 7-1 Left Turn Lane Criterion (TTI)



*(Advancing Volume/Number of Advancing Through Lanes) + (Opposing Volume/Number of Opposing Through Lanes)

Criterion 2: Crash Experience

The crash experience criteria are 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 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.
5. **Signalized Intersection**: Consideration shall be given to installing left turn lanes at a signalized intersection. The State Traffic 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

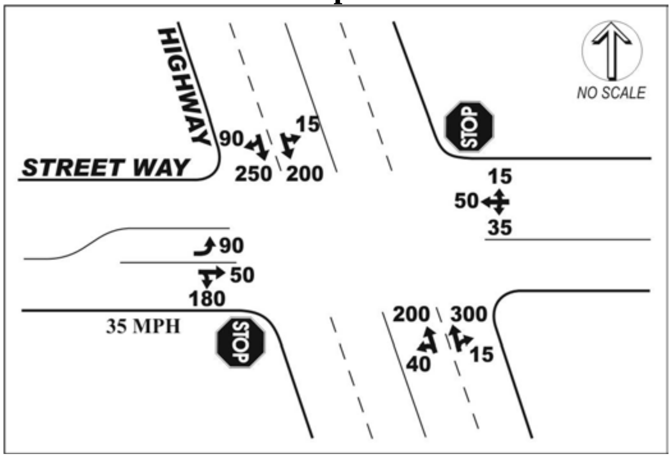
1. The evaluation should indicate the installation of a left turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the left turn lane should not be installed or, if already in place, not allowed to remain in operation.
2. **Alternatives 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 7-1 Left Turn Lane Criterion Example

Left Turn Volume Criterion Example

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**: The southbound advancing volume is 555 (90 + 250 + 200 + 15) 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 7-1 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 criteria is met, use the 45 mph curve in Exhibit 7-1, 535 for the y-axis and 15 left-turns for the x-axis. The volume criterion is not met in the southbound direction.

- **Northbound**: The northbound advancing volume is 555 (40 + 200 + 300 + 15) 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 7-1 is (555/2+ 540/2) = 548. To determine if the southbound left turn volume criteria is met, use the 45 mph curve in Exhibit 7-1, 548 for the y-axis and 40 left-turns for the x-axis. The volume criterion is met in the northbound direction.

7.2.2 Right Turn Lane Criteria – Unsignalized Intersections

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.

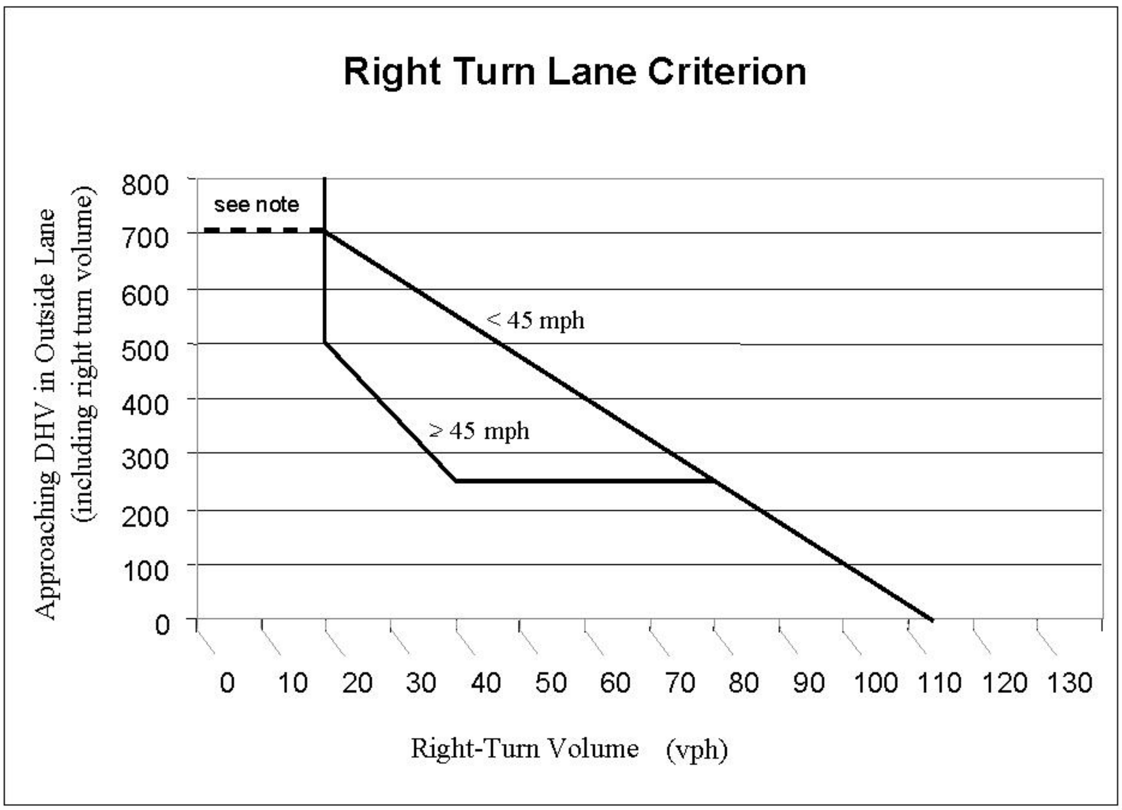
Right Turn Lane Evaluation Process

1. A right turn lane should be installed, if criteria 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. Complies with Access Management Spacing Standards; and
4. 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 right turn lane. The vehicular volume criteria are determined using the curve in Exhibit 7-2.

Exhibit 7-2 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 similar to that for railroad crossings.

Evaluation Guidelines

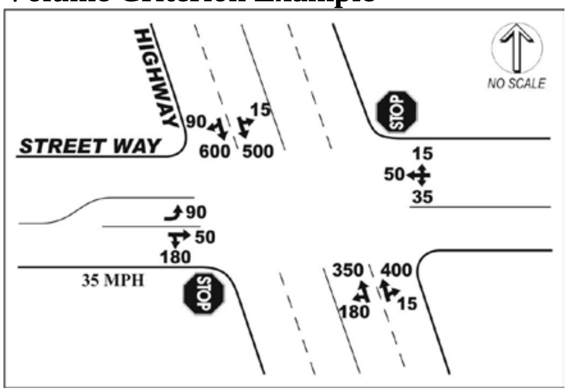
1. The evaluation should indicate the installation of a right turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the right turn lane should not be installed or, if already in place, should be reevaluated for continued use.
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 7-2 Right Turn Lane Criterion Example

Right Turn Vehicular Volume Criterion Example

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 criteria.

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 7-2, 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 7-2, along with 690 approaching vehicles and 90 right turning vehicles we find that the vehicular volume criterion is met.

7.2.3 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.

7.3 Intersection Capacity Analysis

7.3.1 Functional Area of Intersection - SEE APM V2 CHAPTER 4

7.3.2 Effects of Upstream or Downstream Bottlenecks

Intersection analysis can be affected by upstream and downstream bottlenecks on the roadway network. If there is an upstream bottleneck it could restrict the flow of vehicles, ultimately reducing the potential for vehicles to access a study intersection. This potential reduction in vehicle volume at the intersection could result in a lower v/c ratio, indicating that "additional" capacity is available at the intersection. Improving areas that bottleneck could create new areas that fail, but previously indicated available capacity due to the original bottleneck. The analyst should be aware of the potential for an improvement to push a problem elsewhere.

Downstream bottlenecks can have a similar effect by producing a queue spillback, which would prevent vehicles from passing through a study intersection upstream. In this situation, the unserved vehicles will queue beyond the study intersection, but will not be captured as part of the demand when a vehicle count is collected. This low vehicle count at the study intersection could result in a capacity analysis that shows a lower v/c ratio than would be calculated if the bottleneck were not occurring.

7.3.3 Peak Demand Exceeds Operational Capacity

In general, analysis of existing conditions should not render results for v/c ratio calculations of greater than 1.0. This would indicate that more vehicles actually proceeded through an intersection than there is available capacity for. If a v/c ratio of greater than 1.0 is calculated for existing conditions, the default parameters used in analysis should be checked for reasonableness. A common cause of this is the use of default saturation flow rates that do not reflect actual conditions. If the existing v/c ratio calculated is greater than 1.10, the local field data should be checked and possibly additional data collected to refine the analysis. Also check the parameters that are used to calculate the adjusted saturation flow rate (PHF, lane utilization, etc.).

During future year analysis a v/c ratio calculation may result in a value higher than 1.0. This condition may result from a latent demand of vehicles at an intersection. This should be considered as a demand-to-capacity ratio (d/c) rather than an actual v/c ratio and would indicate conditions where mitigation could be considered to improve intersection operations.

7.3.4 Actual Versus Theoretical Conditions

When analysis is conducted on an intersection, the analysis is typically representative of isolated intersection operations. In actuality multiple factors may play a part in the operations of the intersection. These could be factors such as upstream or downstream intersections, coordinated signal systems, closely spaced intersections, etc. These factors should be considered when conducting signalized intersection analysis to help replicate what is actually occurring in the field.

7.3.5 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.

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.

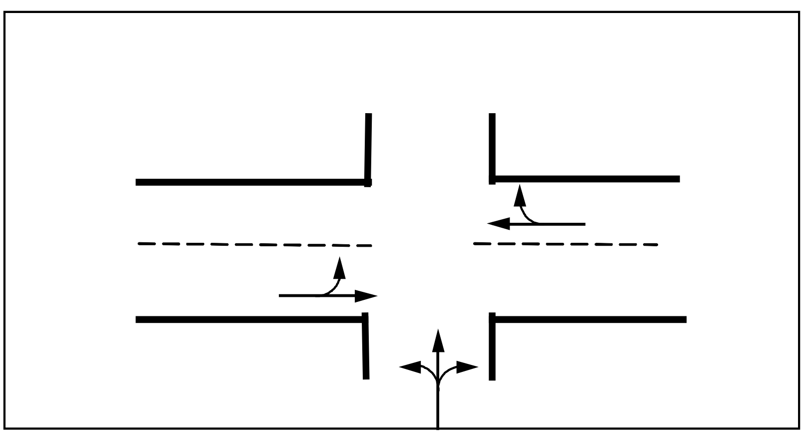
Substitution for the default values of critical gap and follow-up times used in the HCM shall only be permitted after conducting a thorough field investigation and obtaining ODOT approval.

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.

Special Note

For intersections where the minor street is one-way: Synchro 6 and 7 do not use the proper gap times for an intersection with a one-way minor street, such as at an interchange ramp terminal. Synchro 6/7 are using the gap times appropriate for a four-legged intersection with four approaches; however, one-way minor street intersections have four legs, but only three approaches. See Exhibit 7-3.

Exhibit 7-3 Two-Way Stop Control Intersection



The critical gap times (t_c) need to be changed for the minor street left turn only. The value is different depending on how many lanes are on the major street.

Critical Gap tc(s)

- Two Lane Major Street = 6.4
- Four/Six Lane Major Street = 6.8

All other critical gap times stay the same. After the value is changed it will be in red to indicate a user-overridden value. Deleting the value out and pressing “enter” will revert the value back to the default setting.

All-Way Stop Control
For all-way stop controlled intersections, 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 standard.

7.3.6 Roundabout Analysis

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. The ODOT Traffic Manual and HDM contain roundabout guidelines, standards and siting criteria. Roundabout automobile capacity analysis generally follows the 2010 HCM method. For further information, refer to Roundabouts: An Informational Guide, Second Edition, also known as NCHRP Report 672.

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 2010 Roundabout Automobile Methodology

2010 HCM Exhibit 21-9 shows 12 steps in the HCM 2010 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

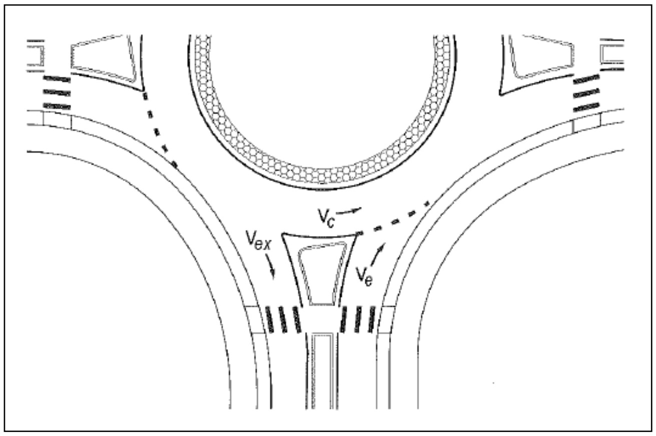
Step 11: Average Control Delay and LOS for entire roundabout

Step 12: Queues for each lane

Exhibit 7-4 (2010 HCM Exhibit 21-2) 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 7-4 2010 HCM Exhibit 21-2



Step 1: Flow rates from demand volumes, as per count

Use HCM 2010 Equation 21-8 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 the 2010 HCM (Exhibit 7-5).

Exhibit 7-5 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 the 2010 HCM. 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 2010, Equation 21-9.

$$v_{i,pce} = \frac{v_i}{f_{HV}}$$

Where:

$v_{i,pce}$ = demand flow rate for movement (passenger cars per hour; pc/hr)
v_i = demand flow rate for movement (veh/hr)
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 2010 Equation 21-11 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 2010 Equation 21-12, similar to Equation 21-11.

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 more than one entry lane, it is important to identify current lane utilization ratios and nearby attractions. Future developments should be considered as well. A travel demand model might show origin/destination routes or travel patterns through an intersection. See HCM 2010 Step 4 including Exhibits 21-13 and 21-14 for procedures.

Step 5: Capacity of entry lanes; uses value from step 3

For single lane roundabouts without a capacity and headway study (i.e. Bend, Oregon) one should use HCM 2010 Equation 21-1 to find the capacity for each entry lane using the circulatory flow rate calculated in Step 3.

$$C = 1130 \bullet \exp(-B \bullet V_c)$$

Where:

C = Entry capacity (pc/h) for single-lane roundabout
V_c = Circulating (conflicting) flow (pc/h)
B = Coefficient, 0.0010 for single lane roundabouts

The City of Bend, Oregon has more roundabouts than any city in Oregon. Therefore, Bend drivers have become accustomed to roundabouts which operate at a higher capacity. A study of single-lane roundabouts in Bend (City of Bend Roundabout Operational Analysis Guidelines, Kittelson & Associates, Inc., 2009) developed a locally calibrated capacity equation. Rather than the HCM equation, the Bend capacity equation is to be used for all single-lane roundabouts to be built in the Bend area. The local calibration of headways and capacities better match Bend driving habits.

Bend single-lane roundabout calibrated capacity equation:

$$C_{pce} = 1333 \bullet \exp(-B \bullet V_{c,pce})$$

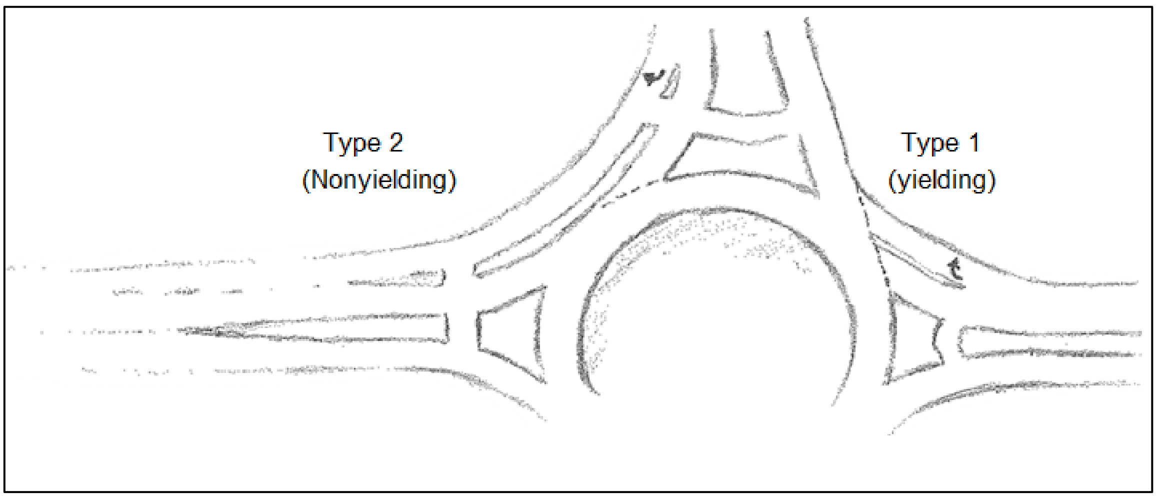
where

C_{pce} = Entry capacity (passenger cars per hour; pc/h), adjusted for heavy vehicles
$V_{c,pce}$ = Circulating (conflicting) flow (pc/h), adjusted for heavy vehicles
B = Bend coefficient, 0.0008 for single lane roundabouts

If considering a multi-lane roundabout with more than one entry lane, see the HCM 2010 Step 5 including Exhibit 21-15.

For a Type 1 Yielding Bypass lane as shown in Exhibit 7-6, 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 (2010 HCM Step 5, Equation 21-6 or 21-7) 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 7-6 Yielding and Non-Yielding



Step 6: Pedestrian impedance to vehicles

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 three equations, similar to HCM 2010 Exhibit 21-17, to find the entry capacity adjustment factor for pedestrians.

$$IF \qquad v_{c,pce} > 881 \text{ Or } n_{ped} < 40 \qquad f_{ped} = 1$$

$$Else \quad IF \qquad 40 \le n_{ped} \le 101 \qquad f_{ped} = 1 - 0.000137 n_{ped}$$

$$Else \qquad f_{ped} = \frac{1119.5 - 0.715 v_{c,ped} + 0.00073 v_{c,pce} n_{ped}}{1068.6 - 0.654 v_{c,pce}}$$

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 2010 Step 6 including Exhibits 21-19 and 21-20 for the entry capacity adjustment factor for pedestrians.

Step 7: Vehicles /hour /lane from capacities and factors

Step 7A: A weighted average of the heavy vehicle adjustment factor is created for each entry lane with HCM 2010 Equation 21-15.

$$f_{HVe} = \frac{f_{HV,U} v_{U,PCE} + f_{HV,L} v_{L,PCE} + f_{HV,T} v_{T,PCE} + f_{HV,R,e} v_{R,e,PCE}}{v_{U,PCE} + v_{L,PCE} + v_{T,PCE} + v_{R,e,PCE}}$$

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 2010, Equation 21-13, a rearrangement of Equation 21-9.

$$v_i = v_{i,PCE} \, f_{HV,e}$$

Where:

$v_{i,pce}$ = demand flow rate for lane i (pc/hr)
v_i = demand flow rate for lane i (veh/hr)
f_{HVe} = heavy vehicle adjustment factor

Step 7B: The capacity of a lane is converted back to vehicles per hour in Equation 21-14.

$$c_i = c_{i,PCE} \, f_{HVe} \, f_{ped}$$

Where:

$c_{i,pce}$ = demand flow rate for movement (Epc/hr)
c_i = demand flow rate for movement (veh/hr)
f_{HVe} = heavy vehicle adjustment factor
f_{ped} = pedestrian adjustment factor

Step 8: Volume/capacity ratio for each lane

The volume/capacity ratio of a lane is calculated in Equation 21-16.

$$x_i = \frac{v_i}{c_i}$$

Where:

x_i = volume-to-capacity ratio of the subject lane i
v_i = demand flow rate of the subject lane i (veh/hr)
c_i = capacity of the subject lane i (veh/hr)

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 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

The HCM 2010 states the delay to be similar to unsignalized intersections, per United Sates roundabout data. The 2010 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 the 2010 HCM describes as resembling STOP control.

The average control delay of a lane is calculated in 2010 HCM Equation 21-17.

$$d = \frac{3600}{c} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3600}{c}\right)x}{450T}}\right] + 5X\ \min[x,1]$$

Where:

d = average control delay (s/veh)
x = volume-to-capacity ratio of the subject lane
c = capacity of the subject lane (veh/hr)
T = time period (h) (T = 0.25 for a 15-min analysis)

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 7-7 (2010 HCM Exhibit 21-1) to determine the LOS of each lane.

Exhibit 7-7 HCM Unsignalized LOS table, HCM Exhibit 21-1

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a]	
	v/c ≤ 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.

Step 11: Average Control Delay and LOS for entire roundabout

The average control delay of a roundabout is calculated in 2010 HCM equations 21-18 and 21-19. For a single lane roundabout with single entry lanes, these equations will reduce to an average of approach (2010 HCM Equation 21-19):

$$d_{intersection} = \frac{\sum d_i v_i}{\sum v_i}$$

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 7-4 (2010 HCM Exhibit 21-1).

Step 12: Queues for Each Lane

The 95th percentile queue of a roundabout entry lane is calculated in HCM 2010 Equation 21-20.

$$Q_{95} = 900T\left[x - 1 + \sqrt{(1-x)^2 + \frac{\left(\frac{3600}{c}\right)x}{150T}}\right]\left(\frac{c}{3{,}600}\right)$$

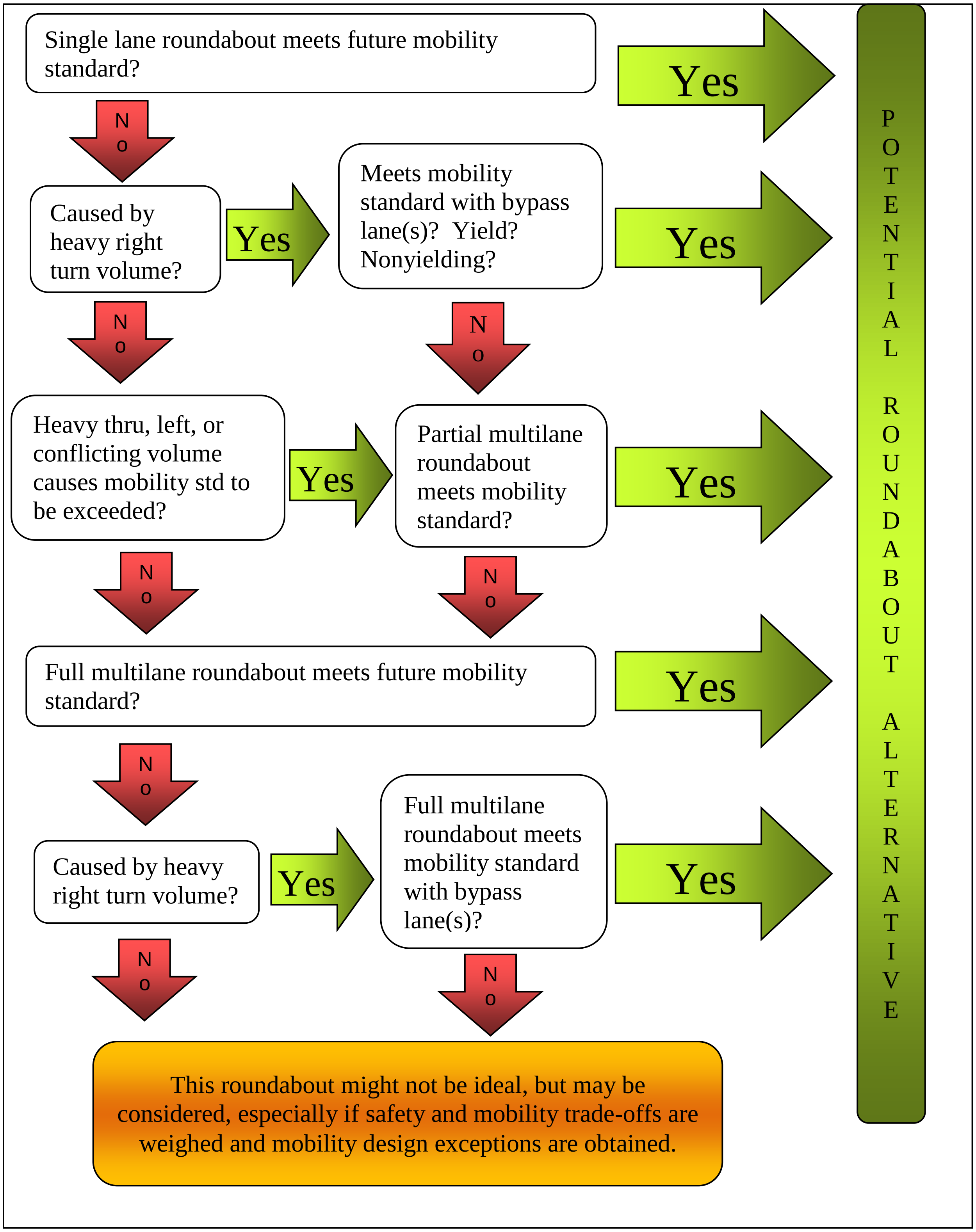
Where:

Q_{95} = 95th percentile queue (veh)
x = volume-to-capacity ratio of the subject lane
c = capacity of the subject lane (veh/hr)
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 (2010 HCM Example Problem 1, page 21-28). 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 7-8.

Exhibit 7-8 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

ODOT Single-Lane Roundabout Calculator

ODOT Single Lane Roundabout Calculator has been developed to expedite capacity and queuing calculations. The following example illustrates the analysis of a single-lane roundabout. This example uses the ODOT single-lane roundabout calculator to perform the analysis steps.

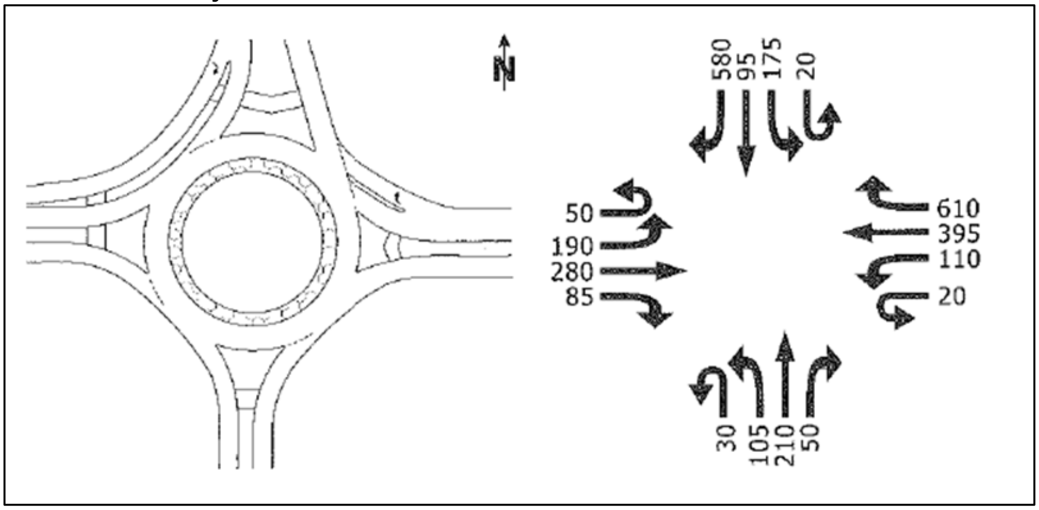
Example 7-3 Single Lane Roundabout Calculation

A single lane roundabout is proposed for the intersection of Mill Street and Elm Street. The intersection volumes could be 20 years beyond the build date for this example. This should be comparable to the HCM2010 Example Problem 1 on page 21-28.

The following traffic and geometric data is available:

- Four legs
- One-lane entries on each leg
- An east leg right-turn yielding bypass lane to be considered
- A north leg right-turn nonyielding bypass lane to be considered
- 2% heavy vehicles for all movements
- 0.94 Peak Hour Factor
- 50 pedestrians cross the south leg in the hour (negligible pedestrian crossings of other legs)
- Demand volumes and lane configurations as shown

Given Geometry and Volumes



Factors other than splitter island width to consider: proximity to a school (school crossing guards), inscribed diameter, and other peak hours (different traffic flows).

Use the tabs of the Single Lane Roundabout Calculator in order. The first step is to read the Notes Tab to gain understanding of the calculator. The second sheet describes some of the built-in checks to try to avoid input errors. Then input the data in the Input tab as shown.

Input Tab

Roundabout Input			Rec
3 or 4 legs		4	3 or 4
Portion of an hour:		0.25	0.25 for 1
Start of peak hour	5	45	PM

N
W E
S

Hourly volumes

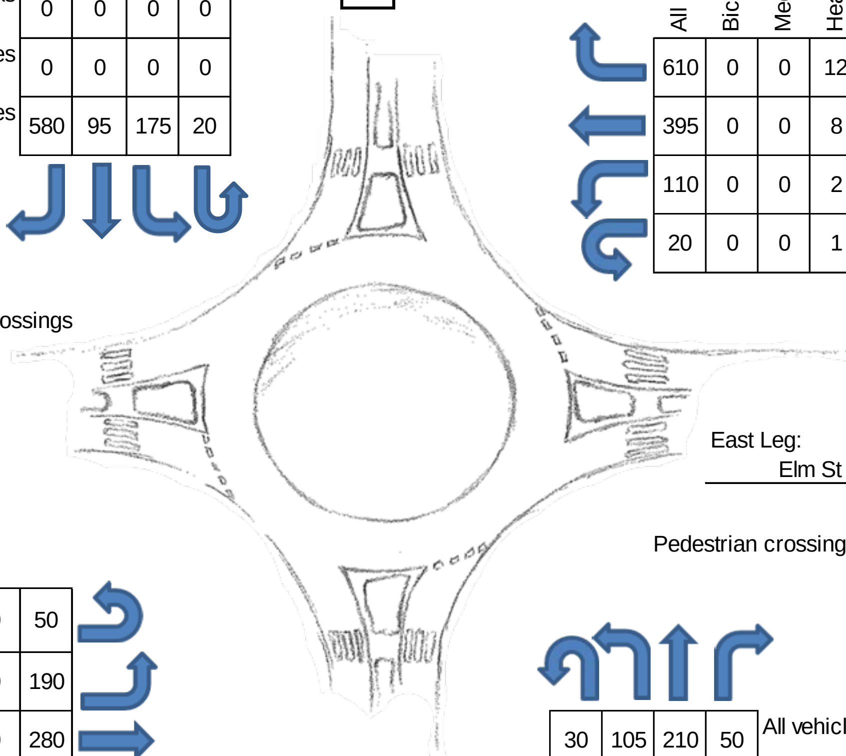
Peak Hour Factor (PHF)	0.94	0.94	0.94	0.94
Heavy trucks	12	2	4	1
Medium trucks	0	0	0	0
Bicycles	0	0	0	0
All vehicles	580	95	175	20

Pedestrian Crossings

0

All vehicles	Bicycles	Medium Trucks	Heavy Trucks	PHF
610	0	0	12	0.94
395	0	0	8	0.94
110	0	0	2	0.94
20	0	0	1	0.94

0 Pedestrian crossings



East Leg:
Elm St

Pedestrian crossings 0

PHF	Heavy Trucks	Medium Trucks	Bicycles	All Vehicles
0.94	1	0	0	50
0.94	4	0	0	190
0.94	6	0	0	280
0.94	2	0	0	85

South Leg:
Mill St

50

Pedestrian Crossings

30	105	210	50	All vehicles
0	0	0	0	Bicycles
0	0	0	0	Medium trucks
1	2	4	1	Heavy trucks
0.94	0.94	0.94	0.94	PHF

Entry Capacity	(+) Input	Rec	Bend
A intercept	1130	1130	1333
B coefficient	0.0010	0.001	0.0008
Bypass Capacity	**(+) Input**	**Rec**	**Bend**
A intercept	1130	1130	1333
B coefficient	0.0010	0.001	0.0008

Passenger Car Equivalents			Rec
bicycle	E_b	1	1
medium	E_m	0	1.5
heavy	E_h	2	2

Project Name

The next picture shows a closer view of the vehicle volume inputs. All peak hour vehicles (including bicycle and truck volumes) are located closest to the arrows. The next three inputs are the breakout of "all vehicles." The Peak Hour Factors are furthest from the arrows.

Volume Inputs

	All vehicles	Bicycles	Medium Trucks	Heavy Trucks	PHF
	610	0	0	12	0.94
	395	0	0	8	0.94
	110	0	0	2	0.94
	20	0	0	1	0.94

Bicycles that enter the roundabout as vehicles are to be part of the intersection volumes equaling a car for each movement. This does not involve the passenger car equivalent at this step. This is also the case for trucks; they are all input as one vehicle entering the roundabout.

Additional inputs (recommendations are in grey to the right):

- Number of legs (3 or 4)
- Portion of an hour studied (0.25 recommended)
- Peak hour (for documentation)
- Pedestrian crossings at each leg
- East and South leg street names (documentation)
- PCEs for bicycles, medium trucks, and heavy trucks
- Capacity A intercept and B coefficient (provide study/reasoning for non-recommended inputs)

Screen captures of these inputs are as follows.

Additional Inputs

Roundabout Input			Rec
3 or 4 legs		4	3 or 4
Portion of an hour:		0.25	0.25 for 1
Start of peak hour	5	45	PM

East Leg:	
Elm St	
Pedestrian crossings	0

Passenger Car Equivalents			Rec
bicycle	E_b	1	1
medium	E_m	0	1.5
heavy	E_h	2	2

Entry Capacity	(+) Input	Rec	Bend
A intercept	1130	1130	1333
B coefficient	0.0010	0.001	0.0008
Bypass Capacity	**(+) Input**	**Rec**	**Bend**
A intercept	1130	1130	1333
B coefficient	0.0010	0.001	0.0008

Single Lane Tab
Volumes are computed into passenger car equivalents on the Single Lane tab (Step 2). As the number of bicycles and trucks are entered, the proportions of these vehicles are calculated in the greyed out boxes to the right. Calculations are without bypasses; at this point the need is not known. Changes in this sheet do not affect the previous input sheet.

Additional inputs (recommendations are in grey to the right):

- Analyst
- Agency
- Date
- Project
- Year

Single Lane Tab

General Information	
Analyst:	Pat Stoplight PE
Agency	Safety City
Date:	9/22/2015
East leg:	Elm St
Project:	Project Name

Passenger Car Equivalents			Rec
bicycle	E_b	1	1
medium	E_m	0	1.5
heavy	E_h	2	2
South leg	Mill St		
Year:	20yrs > build		

Roundabout Input			
3 or 4 legs			4
Portion of an hour:			0.25
Peak hr	5	45	PM

N
W E
S

Pedestrian crossings per le	N	E	S	W
#	0	0	50	0

Hour Volumes vph		Approaches N	E	S	W
Exits	N	20	610	210	190
	E	175	20	50	280
	S	95	110	30	85
	W	580	395	105	50

Changes here do not go to Input tab.

Flow Rate v_i		Approaches N	E	S	W
Exits	N	21	649	223	202
	E	186	21	53	298
	S	101	117	32	90
	W	617	420	112	53

Peak Hour Factor PHF		Approaches N	E	S	W
Exits	N	0.94	0.94	0.94	0.94
	E	0.94	0.94	0.94	0.94
	S	0.94	0.94	0.94	0.94
	W	0.94	0.94	0.94	0.94

Vehicle Factor f_{hv}		Approaches N	E	S	W
Exits	N	0.952	0.980	0.981	0.979
	E	0.978	0.952	0.980	0.979
	S	0.979	0.982	0.968	0.977
	W	0.979	0.980	0.981	0.980

# of Bicycles vph		Approaches N	E	S	W
Exits	N	0	0	0	0
	E	0	0	0	0
	S	0	0	0	0
	W	0	0	0	0

Proportion of Bicycle P_b		Approaches N	E	S	W
Exits	N	0.000	0.000	0.000	0.000
	E	0.000	0.000	0.000	0.000
	S	0.000	0.000	0.000	0.000
	W	0.000	0.000	0.000	0.000

# of Medium Trucks vph		Approaches N	E	S	W
Exits	N	0	0	0	0
	E	0	0	0	0
	S	0	0	0	0
	W	0	0	0	0

Proportion of Medium P_m		Approaches N	E	S	W
Exits	N	0.000	0.000	0.000	0.000
	E	0.000	0.000	0.000	0.000
	S	0.000	0.000	0.000	0.000
	W	0.000	0.000	0.000	0.000

# of Heavy Trucks vph		Approaches N	E	S	W
Exits	N	1	12	4	4
	E	4	1	1	6
	S	2	2	1	2
	W	12	8	2	1

Proportion of Heavy P_h		Approaches N	E	S	W
Exits	N	0.050	0.020	0.019	0.021
	E	0.023	0.050	0.020	0.021
	S	0.021	0.018	0.033	0.024
	W	0.021	0.020	0.019	0.020

Adjusted Flow Rate v_i		Approaches N	E	S	W
Exits	N	22	662	227	206
	E	190	22	54	304
	S	103	119	33	92
	W	630	429	114	54
Entry Flow Rate (pc/h)		945	1232	428	656
Conflict Flow (pc/h)		771	656	798	489

Exits w/o right vol pcl		Weighted Entry Vehicle Factors			
N	455	0.979	0.980	0.980	0.979
E	516				
S	255	Weighted Conflict Vehicle Factors			
W	597	0.979	0.979	0.977	0.976

Output		Approaches N	E	S	W
Conflict flow (veh/h)	v_c	755	642	774	477
Entry flow (veh/h)	v_i	925	1207	419	642
Entry capacity (veh/h)	c_i	512	575	495	678
Pedestrian impedance	f_{ped}	1	1	0.993	1
Leg v/c ratio	x_i	1.81	2.10	0.85	0.95
Control delay (sec/veh)	d_i	391.6	517.9	40.4	47.8
LOS	n/a	F	F	E	E
HCM 95th% Queue (veh)	Q_m	58	84	9	14

Int cntrl delay (sec/veh)	d_{int}	324.06
Intersection LOS	n/a	F

Project Name

The single lane tab shows the initial results. V/C ratios that are over 1.0 are highlighted.

Single Lane Tab Close-up

Output		Approaches			
		N	E	S	W
Conflict flow (veh/h)	v_c	755	642	774	477
Entry flow (veh/h)	v_i	925	1207	419	642
Entry capacity (veh/h)	c_i	512	575	495	678
Pedestrian impedance	f_{ped}	1	1	0.993	1
Leg v/c ratio	x_i	1.81	2.10	0.85	0.95
Control delay (sec/veh)	d_i	391.6	517.9	40.4	47.8
LOS	n/a	F	F	E	E
HCM 95th% Queue (veh)	Q_m	58	84	9	14
Int cntrl delay (sec/veh)	d_{int}	324.06			
Intersection LOS	n/a	F			

BypassLane Tab

The next step is to improve a leg by considering a bypass lane using the bypass lane tab as shown in the next picture. As stated at the top of the sheet "Only two selections are necessary (cell E13 drop down and yield selection button)." The volumes are shown, with the right turns highlighted. The leg that the bypass lane originates from is chosen. With that selection, the exit leg is provided in print. The map at the right will then diagram the bypass that has been chosen. The only item left is to select the button to indicate if the bypass is yielding or nonyielding. If yielding is chosen, then the results for that bypass are shown below where the bypass was selected to be a Type 1 Yielding Bypass. The data is then recalculated and is available in the other print space.

The input coding and analysis results for this example for the first bypass lane are shown.

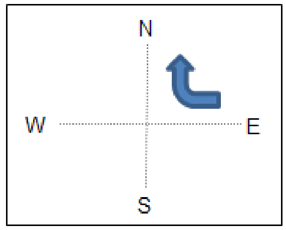
Bypass Lane Tab with Output Sheet (page 1)

Bypass Lane Merge Point Analysis

Only two selections are necessary (cell E13 and yield selection button).

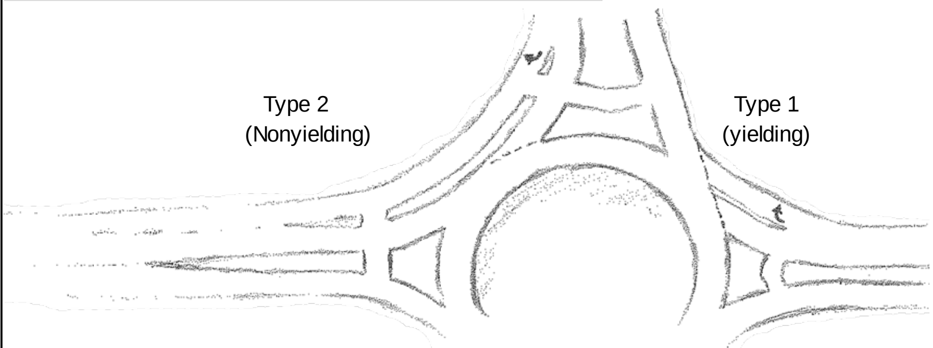
Entry on the Single Lane Roundabout Calculator:

Hour Volumes vph		Approaches N	E	S	W
Exits	N	20	610	210	190
	E	175	20	50	280
	S	95	110	30	85
	W	580	395	105	50



A heavy right turn volume approaches at the East leg.

The heavy right turn volume then exits on the North leg.



○ Type 2 Nonyielding Bypass lane

If there is room for a new lane, then bypass LOS is A and capacity is expected to be high (higher than yielding bypass values shown below) and the analysis is complete for this bypass lane.

Considerations for a Type 2 nonyielding bypass lane:

- A median refuge should ensure a pedestrian only crosses one lane at a time
- Bypass travel path geometrically slows traffic
- Is there a heavy left turn volume down this leg to create a demand to quickly merge?

◉ Type 1 Yielding Bypass lane

Items to keep in mind if constrained to a Type 1 nonyielding bypass lane:

- Angle that driver has to look over the shoulder to merge, then forward to yield to pedestrians
- All traffic volume is now in one lane, consider what gaps exist for pedestrian
- Safety of heavy right movement merging into all movements exiting roundabout

		717	pc/h
Capacity	c	703	veh/h
Entry Flow Rates	v	649	veh/h
Volume to Capacity ratio	v/c	0.92	
Delay		41.2	s/veh
LOS		E	
HCM Queue		12	veh

The roundabout analysis with the East approach to the North leg bypass volume removed is to the right. Please print and electronically save this information for your records.

Project Name

Bypass Lane Tab with Output Sheet (page 2)

General Information		Passenger Car Equivalent			Rec
Analyst:	Pat Stoplight PE	bicycle	E_b	1	1
Agency:	Safety City	medium	E_m	0	1.5
Date:	9/22/2015	heavy	E_h	2	2
East leg:	Elm St	South leg:	Mill St		
Project:	Project Name	Year:	20yrs > build		

Roundabout Input			
3 or 4 legs			4
Portion of an hour:			0.25
Peak hr	5	45	PM

N
W E
S

Pedestrian crossings per	N	E	S	W
#	0	0	50	0

Hour Volumes vph		Approaches N	E	S	W
Exits	N	20	0	210	190
	E	175	20	50	280
	S	95	110	30	85
	W	580	395	105	50

ONE BYPASS

Flow Rate v_i		Approaches N	E	S	W
Exits	N	21	0	223	202
	E	186	21	53	298
	S	101	117	32	90
	W	617	420	112	53

Peak Hour Factor PHF		Approaches N	E	S	W
Exits	N	0.94	0.00	0.94	0.94
	E	0.94	0.94	0.94	0.94
	S	0.94	0.94	0.94	0.94
	W	0.94	0.94	0.94	0.94

Vehicle Factor f_{hv}		Approaches N	E	S	W
Exits	N	0.952	1.000	0.981	0.979
	E	0.978	0.952	0.980	0.979
	S	0.979	0.982	0.968	0.977
	W	0.979	0.980	0.981	0.980

# of Bicycles vph		Approaches N	E	S	W
Exits	N	0	0	0	0
	E	0	0	0	0
	S	0	0	0	0
	W	0	0	0	0

Proportion of Bicycle P_b		Approaches N	E	S	W
Exits	N	0.000	0.000	0.000	0.000
	E	0.000	0.000	0.000	0.000
	S	0.000	0.000	0.000	0.000
	W	0.000	0.000	0.000	0.000

# of Medium Trucks vph		Approaches N	E	S	W
Exits	N	0	0	0	0
	E	0	0	0	0
	S	0	0	0	0
	W	0	0	0	0

Proportion of Medium P_m		Approaches N	E	S	W
Exits	N	0.000	0.000	0.000	0.000
	E	0.000	0.000	0.000	0.000
	S	0.000	0.000	0.000	0.000
	W	0.000	0.000	0.000	0.000

# of Heavy Trucks vph		Approaches N	E	S	W
Exits	N	1	0	4	4
	E	4	1	1	6
	S	2	2	1	2
	W	12	8	2	1

Proportion of Heavy P_h		Approaches N	E	S	W
Exits	N	0.050	0.000	0.019	0.021
	E	0.023	0.050	0.020	0.021
	S	0.021	0.018	0.033	0.024
	W	0.021	0.020	0.019	0.020

Adjusted Flow Rate v_i		Approaches N	E	S	W
Exits	N	22	0	227	206
	E	190	22	54	304
	S	103	119	33	92
	W	630	429	114	54
Entry Flow Rate (pc/h)		945	570	428	656
Conflict Flow (pc/h)		771	656	798	489
Bypass Delay		0.0	48.6	0.0	0.0
WeightedEntryVehFactor		0.979	0.979	0.980	0.979
1st Bypass Entry Flo		0	649	0	0
Weighted Conflict Factors		0.979	0.979	0.977	0.976

Output		Approaches N	E	S	W
Conflict flow (veh/h)	v_c	755	642	774	477
Entry flow (veh/h)	v_i	925	558	419	642
Entry capacity (veh/h)	c_i	512	574	495	678
Pedestrian impedance	f_{ped}	1	1	0.993	1
Leg v/c ratio	x_i	1.81	0.97	0.85	0.95
Control delay (sec/veh)	d_i	391.6	57.1	40.4	47.8
LOS	n/a	F	F	E	E
HCM 95th% Queue (veh)	Q_m	58	13	9	14

Int cntrl delay (sec/veh)	d_{int}	149.94
Intersection LOS	n/a	F

Project Name

The 2nd bypass lane tab clearly states that it is the 2nd Bypass Lane Merge Point analysis on the top line. It operates much the same way as the Bypass Lane Tab. It identifies the 1st bypass that was chosen and shows a zero where the 1st bypass removed the right turn volume from the entry lane. The 2nd leg that the bypass lane originates from and the yield or nonyielding choices are made again. If nonyielding is chosen, the results for a yielding bypass are not shown. In print, it states the bypass has LOS A and that capacity is expected to be very high. The data is then recalculated and is available in the other print space.

The input coding and analysis results for this example for the second bypass lane are shown.

2nd Bypass Lane Tab with Output Sheet (page 1)

2nd Bypass Lane Merge Point Analysis

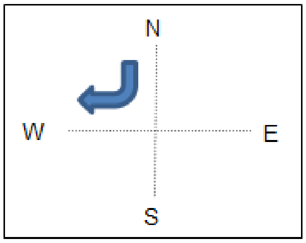
The first bypass you entered:

A heavy right turn volume approaches at the East leg.

The heavy right turn volume then exits on the North leg.

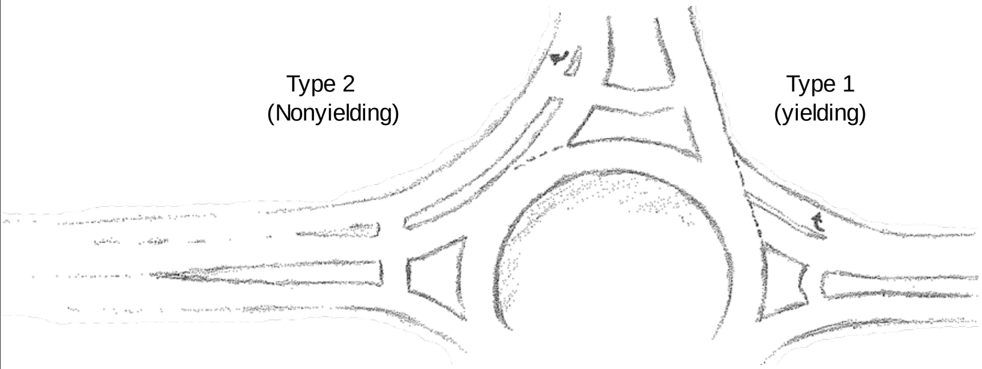
Entry on the Single Lane Roundabout Calculator with no volume from 1st Bypass:

Hour Volumes vph		Approaches N	E	S	W
Exits	N	20	0	210	190
	E	175	20	50	280
	S	95	110	30	85
	W	580	395	105	50



The heavy right turn volume enters from the North approach.

The heavy right turn volume then exits on the West leg.



◉ Type 2 Nonyielding Bypass lane

If there is room for a new lane, then bypass LOS is A and capacity is expected to be high (higher than yielding bypass values shown below) and the analysis is complete for this bypass lane.

Considerations for a Type 2 nonyielding bypass lane:

- A median refuge should ensure a pedestrian only crosses one lane at a time
- Bypass travel path geometrically slows traffic
- Is there a heavy left turn volume down this leg to create a demand to quickly merge?

○ Type 1 Yielding Bypass lane

Items to keep in mind if constrained to a Type 1 nonyielding bypass lane:

- Angle that driver has to look over the shoulder to merge, then forward to yield to pedestrians
- All traffic volume is now in one lane, consider what gaps exist for pedestrian
- Safety of heavy right movement merging into all movements exiting roundabout

Capacity	c
Entry Flow Rates	v
Volume to Capacity ratio	v/c
Delay	
LOS	
HCM Queue	

The roundabout analysis with the North approach to the West leg bypass and previous bypass volume removed is to the right. Please print and electronically save this information.

2nd Bypass Lane Tab with Output Sheet (page 2)

General Information		Passenger Car Equivalents			Rec
Analyst:	Pat Stoplight PE	bicycle	E_b	1	1
Agency:	Safety City	medium	E_m	0	1.5
Date:	42269	heavy	E_h	2	2
East leg:	Elm St	South leg:	Mill St		
Project:	Project Name	Year:	20yrs > build		

Roundabout Input			
3 or 4 legs			4
Portion of an hour:			0.25
Peak hr	5	45	PM

N
W E
S

Pedestrian Crossings per #	N	E	S	W
#	0	0	50	0

TWO BYPASSES

Hour Volumes vph		N	E	S	W
Exits	N	20	0	210	190
	E	175	20	50	280
	S	95	110	30	85
	W	0	395	105	50

Flow Rate v_i		N	E	S	W
Exits	N	21	0	223	202
	E	186	21	53	298
	S	101	117	32	90
	W	0	420	112	53

Peak Hour Factor PHF		N	E	S	W
Exits	N	0.94	0.00	0.94	0.94
	E	0.94	0.94	0.94	0.94
	S	0.94	0.94	0.94	0.94
	W	0.00	0.94	0.94	0.94

Vehicle Factor f_{hv}		N	E	S	W
Exits	N	0.952	1.000	0.981	0.979
	E	0.978	0.952	0.980	0.979
	S	0.979	0.982	0.968	0.977
	W	1.000	0.980	0.981	0.980

# of Bicycles vph		N	E	S	W
Exits	N	0	0	0	0
	E	0	0	0	0
	S	0	0	0	0
	W	0	0	0	0

Proportion of Bicycle P_b		N	E	S	W
Exits	N	0.000	0.000	0.000	0.000
	E	0.000	0.000	0.000	0.000
	S	0.000	0.000	0.000	0.000
	W	0.000	0.000	0.000	0.000

# of Medium Trucks vph		N	E	S	W
Exits	N	0	0	0	0
	E	0	0	0	0
	S	0	0	0	0
	W	0	0	0	0

Proportion of Medium P_m		N	E	S	W
Exits	N	0.000	0.000	0.000	0.000
	E	0.000	0.000	0.000	0.000
	S	0.000	0.000	0.000	0.000
	W	0.000	0.000	0.000	0.000

# of Heavy Trucks vph		N	E	S	W
Exits	N	1	0	4	4
	E	4	1	1	6
	S	2	2	1	2
	W	0	8	2	1

Proportion of Heavy P_h		N	E	S	W
Exits	N	0.050	0.000	0.019	0.021
	E	0.023	0.050	0.020	0.021
	S	0.021	0.018	0.033	0.024
	W	0.000	0.020	0.019	0.020

Adjusted Flow Rate v_i		N	E	S	W
Exits	N	22	0	227	206
	E	190	22	54	304
	S	103	119	33	92
	W	0	429	114	54
Entry Flow Rate (pc/h)		315	570	428	656
Conflict Flow (pc/h)		771	656	798	489
bypass delay		6.7	48.6	0.0	0.0
WeightedEntryVehFactor		0.976	0.979	0.980	0.979
1st		0.0	649.0	0.0	0.0
Weighted Conflict Factors		0.979	0.979	0.977	0.976
2nd Bypass Entry Flo		617	0	0	0

Output		N	E	S	W
Conflict flow (veh/h)	v_c	755	642	774	477
Entry flow (veh/h)	v_i	307	558	419	642
Entry capacity (veh/h)	c_i	510	574	495	678
Pedestrian impedance	f_{ped}	1	1	0.993	1
Leg v/c ratio	x_i	0.60	0.97	0.85	0.95
Control delay (sec/veh)	d_i	20.1	57.1	40.4	47.8
LOS	n/a	C	F	E	E
HCM 95th% Queue (veh)	Q_m	4	13	9	14

Int cntrl delay (sec/veh)	d_{int}	35.23
Intersection LOS	n/a	E

Project Name

7.3.7 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 of 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 Chapter 16 of the HCM.

Saturation Flow Rates

As previously discussed in Chapters 3-5, 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.

Signalized Intersection v/c Ratio

> *The HCM 2010 computational engine does not produce an overall signalized intersection v/c ratio. Some software such as Vistro does report out HCM 2010 overall intersection v/c ratio, while others such as Synchro do not. If the software being used does not report out this value, then this value shall be produced using HCM 2000 methods.*

For signalized intersections, the OHP v/c ratio is based on the overall intersection v/c ratio, not the movement v/c ratio as explained in Action 1F of the OHP. The intersection v/c ratio is also known as the critical v/c ratio or Xc in the HCM. The intersection v/c ratio is not generally affected by the approach green times (except in cases with shared left turns). See HCM 2010 equations 18-17 and 18-18 below.

$$X_C = \left(\frac{C}{C-L}\right)\sum_{i\in ci} y_{c,i}$$

$$L = \sum_{i \in ci} l_{t,i}$$

where:

X_c= critical intersection v/c ratio
C = cycle length (s)
$y_{c,i}$ = critical flow ratio for phase i = $\frac{v_i}{(Ns_i)}$
$l_{t,i}$ = phase i lost time = $l_{1,i}+l_{2,i}$(s)
ci = set of critical phases on the critical path, and
L = cycle lost time (s)

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 with regard to 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 maximum green split that should be used. 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. Otherwise, unless directed to do so by ODOT staff, the use of the existing timing is not required. 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. Note: If Synchro is to be used to optimize a series of coordinated intersections review Section 7.3.8 and ensure that all necessary data is entered. If SimTraffic will be eventually used, ensure that Section 7.3.8 and Chapter 8 is followed.

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-Roadway 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 of the included sheets are necessary, but it is important that all of the needed sheets are obtained. The following shows the important sheets (Exhibit 7-9 through Exhibit 7-15, 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.

Sheet 2 – Phase Rotation Diagram

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 other program. For complicated phasing the diagram is an invaluable source. Exhibit 7-9 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 7-9 Signal Timing Sheet 2

SHEET 2

Date sheet in effect:

Date sheet voided:

Location: Hwy 97 @ Pinebrook

Table Numbers refer to Trafficview & Translink

Phase Rotation Diagram

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	

	Key	Phase Number 1	2	3	4	5	6	7	8
	8								
Handicap Ped	E								

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	

	Key	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 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

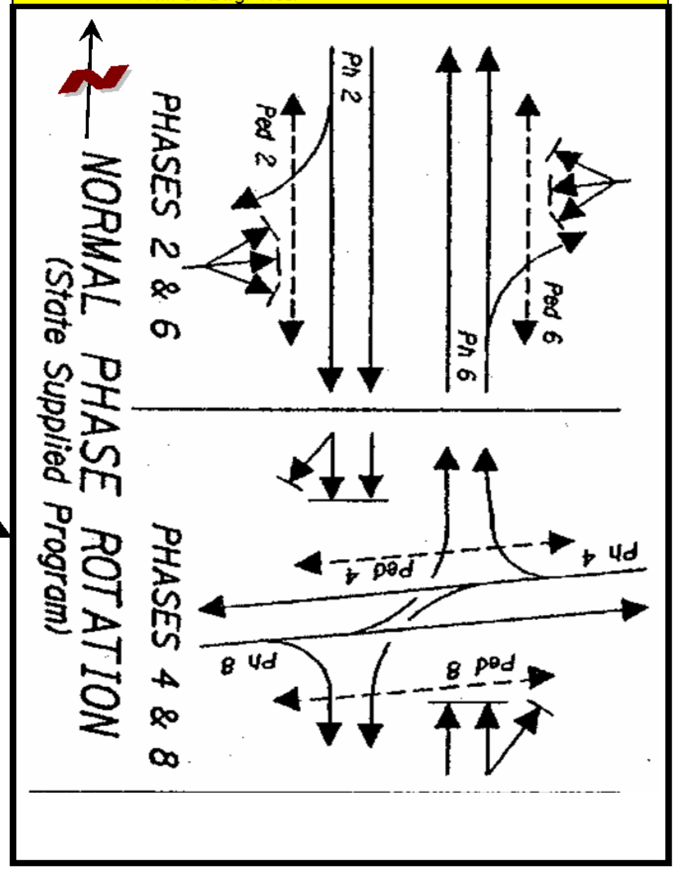
(3) C1 pins 54, 63, 64, 75, 76, and 77. See sheet 6, Location B+0+D.

(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 7-10) shows the basic phasing properties and Exhibit 7-11 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 7-10 Signal Timing Sheet 3 – Basic Phase Settings

Date sheet in effect ... Date sheet voided ... Hwy 97 @ Pinebrook

Vehicle Recall, Permitted Phases & Overlaps

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		

Function	Key	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

SHEET 3

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 Pinebrook
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		
Min Green	5		10		5		10		
TBR	6		10		5		10		
TTR	7		20		5		20		
Observe Gap	8								
Passage	9		5.2		3.5		5.2		
Min Gap	A		3.2		1.0		3.2		
Add per Act	B		1.5				1.5		
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								

Maximum Green and Max 2 Green Times

Sheet 8 indicates when each is in effect.

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		

Keyboard Entries when not in Free Display

A Advance D Column Advance
B Back E Enter and Advance
C Clear Display F Free Display

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)

Phase Conditions as shown on Free Display

00	Initial Entry	0C	Yellow
02	WALK	0D	Red C
03	Flashing DW	0E	Red R
05	Min Green	11	Gap Out
08	Rest	12	Force Off
09	Passage	14	Max Out
0B	Added Initial	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

Page I.D. 0

Exhibit 7-11 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

Function	Key	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

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

TABLE 2 Page 0

Function		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
- B Back
- C Clear Display
- D Column Advance
- E Enter and Advance
- F Free Display

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

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 7-4 Signal Phase Splits

Example values for Sheet 3 are (Exhibit 7-10):

- Vehicle Recall = Phases 2 and 6 (US 97)
- Permitted Phases = 2, 4, 6 and 8. From the phase rotation diagram in Exhibit 7-9 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 7-15), 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 a large number of 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.

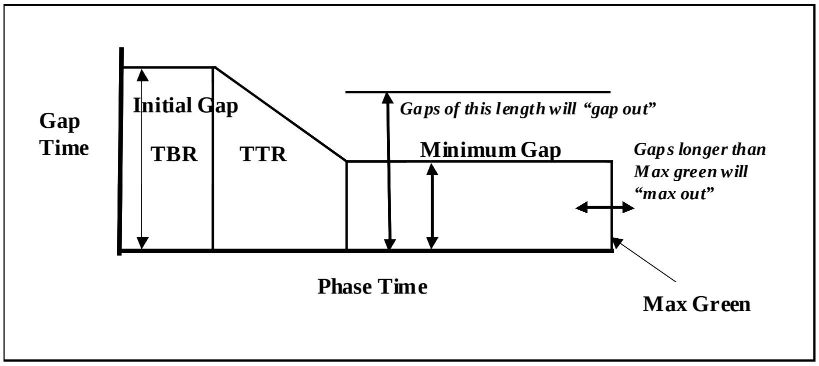
Table 1 also covers the actuated signal phasing parameters that are needed for creating timing plans and calibrated simulations. These five parameters are:

- **Minimum Green** (Key= ph + 5) - Minimum green time that a signal indication will occur for 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 7-12 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 down 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 7-12 Actuated Gap Time



<u>Sheet 6 – Table 6 Operation</u>

Table 6 indicates whether or not 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 7-13 shows that the example intersection is coordinated, but is not the master.

Exhibit 7-13 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

Key		Phase Number 1	2	3	4	5	6	7	8
C									
D	NEMA CNA								
E	Adv. Warn.								
F	MRI Phases		X		X		X		X

	Y	N
OSM ?		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)

(A + 3 + 9)

Sample Detectors (0 = off, 1 = on)

Sampling detectors are assigned using extended input codes on Sheet 11

For Protected / Permissive Left Turns

(A + 3 + A)

Left Turn Type (0, 1, or 2)

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.

Sheet 7 – Table 7 Coordination Timing

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 7-14 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 Phase 2 and 6 are 45 seconds and Phase 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 7-14 Signal Timing Sheet 7

Date sheet in effect: Date sheet voided:

Location: Hwy 97 @ Pinebrook

TABLE 7 (1 of 2)

Hardwire Conversion	1			2			3		
Dial 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
										Plan Number
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							

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) 10	11	12	13	14	15	16	17	18	Plan Number
		7	8	9	A	B	C	D	E	F	Key 1
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

<u>Sheet 8 – Table 5 Time Clock Control</u>

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 7-15 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 7-15 Signal Timing Sheet 8

SHEET 8

TABLE 5 (1 of 2)

Time Clock Control								(A+Code)		
vent N	S	M	T	W	T	F	S	Hour	Min.	Func
	1	2	3	4	5	6	7			
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)		
vent N	S	M	T	W	T	F	S	Hour	Min.	Func
	1	2	3	4	5	6	7			
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)		
vent N	S	M	T	W	T	F	S	Hour	Min.	Func
	1	2	3	4	5	6	7			
33	80							81	82	83
								85	86	87
35								89	8A	8B
36	8C							8D	8E	8F
37	90							91	92	93
39	98							99	9A	9B
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)		
vent N	S	M	T	W	T	F	S	Hour	Min.	Func
	1	2	3	4	5	6	7			
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
	D4							D5	D6	D7
55	D8							D9	DA	DB
	DC							DD	DE	DF
	E0							E1	E2	E3
	E4							E5	E6	E7
	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)

Date sheet in effect:

Date sheet voided:

Location: Highway 97 @ Pinebrook

Event numbers are for reference only.

Local TOD "Free" will override any plan received via an interconnect line.

7.3.8 Software and Tools Available for Analysis

There are many software programs and tools available for traffic analysis. The following is a brief discussion on a few of the most common tools. For more information on the selection of the appropriate tool, see the FHWA Traffic Analysis Toolbox, Volume II: Decision Support Methodology for Selecting Traffic Analysis Tools.

Critical Movement Analysis

The critical movement analysis method is a sketch planning-level tool used to get a quick ballpark estimate of whether the existing or forecasted volumes at a signalized intersection will be under, near or over the intersection's capacity. It is for estimation only, not used to report v/c ratios as a final product or to compare to mobility standards. The analysis requires the intersection approach volumes, number of lanes and lane geometry on each approach.

Each of the movement pairs in conflict at the intersection (e.g., the westbound left and the eastbound through movements) are the focus of the analysis. The total volume included in each conflict pair is calculated to find the highest (or critical movement pair) for each roadway. Where multiple lanes exist in a lane group, use available data on lane utilization; if there is no data on lane utilization, for this procedure assume an even distribution per lane.

The critical movement pairs for each roadway are then summed and compared with the following standards, as shown in Exhibit 7-16:

Exhibit 7-16 Intersection Performance Assessment by Critical Volume

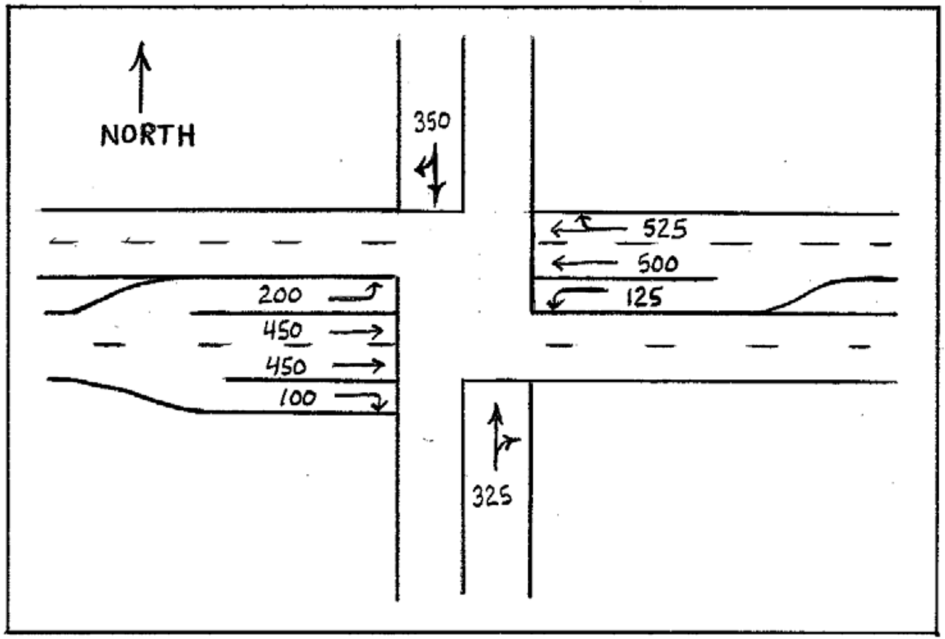
Sum of Critical Volumes (Vehicles/Hour/Lane)	Performance
0 to 1,200	Under Capacity
1,201 to 1,400	Near Capacity
1,401 and Above	Over Capacity

Critical movement analysis only estimates the intersection's ability to accommodate the projected volumes. It does not estimate vehicle delay, level of service or vehicle queue lengths.

Example 7-5 Critical Movement Analysis

The figure below illustrates 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 total volume in a conflict pair occurs for the 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, so the highest total volume on an approach is taken as the critical movement. Therefore, the critical movement volume for the north-south roadway is 350 vehicles.
The sum of the critical movement volumes for the intersection becomes:

725 (east-west) + 350 (north-south) = 1,075

Compared to the performance thresholds shown in Exhibit 7-16 this intersection is estimated to be operating under capacity.

Intersection Capacity Utilization
The Intersection Capacity Utilization (ICU) method is another signalized intersection sketch planning-level tool used to get a quick ballpark estimate of how much reserve capacity is

available or how much the intersection is over capacity. It compares the current traffic volume to the intersection's ultimate capacity. It is for estimation only not used to report v/c ratios as a final product or to compare to mobility standards. The method sums the amount of time required to serve all movements at saturation for a given cycle length and divides by that reference cycle length. This method is similar to taking a sum of critical volume to saturation flow ratios (v/s), yet allows minimum timings to be considered. While it does not predict delay, it can be used to predict how often an intersection will experience congestion. The ICU method can provide reasonable estimates for intersection capacity conditions, but should not be used for detailed operational analysis.

The ICU is timing plan independent, yet has rules to insure that minimum timing constraints are taken into account. This removes the choice of timing plan from the capacity results. The ICU can also be used on unsignalized intersections to determine the capacity utilization if the intersection were to be signalized.

The ICU Level of Service (LOS) should not be confused with delay-based levels of service such as the HCM. Both are providing information about the performance of an intersection, but are measuring a different objective function. The ICU LOS reports out the amount of reserve capacity or capacity deficit. The delay based LOS reports out on the average delay experienced by motorists.

SIGCAP2, UNSIG10

SIGCAP2 is an ODOT developed computer program similar to the ICU. It is also based on the 1985 HCM. It is a sketch planning-level tool, timing plan independent, used to get a quick ballpark estimate of a signalized intersection v/c ratio. It is for estimation only, not used to report v/c ratios as a final product nor to compare to mobility standards. It can be used to estimate LOS C volumes for Environmental traffic data.

UNSIG10 is an ODOT written computer program that analyzes unsignalized intersections. It is a sketch planning-level tool used to get a quick ballpark estimate of whether and by what magnitude the existing or forecasted volumes at a signalized intersection will be under, near or over the intersection's capacity. It is for estimation only, not used to report v/c ratios as a final product nor to compare to mobility standards. It can be used to estimate LOS C volumes for Environmental traffic data.

Traffix

Traffix is a computer program that calculates level of service at isolated signalized and unsignalized intersections based on the HCM methods. There is no interaction between the intersections, similar to the Highway Capacity Software (next software covered). This program is frequently used for evaluating the impacts of proposed developments. It facilitates the process of trip distribution and assignment over a street network making it easier to test multiple development scenarios and different mitigation measures. The Traffix program uses Zones, Gates, Paths, Routes and Attractions to simulate an existing network and the addition of a potential development. The program can be used to develop both existing and future traffic volumes for several alternatives, evaluate potential signal timing (but not progression) and generate Level of Service and HCM reports for intersections (signalized and unsignalized). A Traffix file can be converted over to a Synchro file (some details don't transfer), saving time creating new files and inputting different volume scenarios.

Local jurisdictions often use Traffix to track various development proposals and to keep an inventory of their network. Traffix is often used for TIAs and similar analysis work. This tool is also used when working with cumulative analysis of small communities and small regional projects. Traffix may be a better tool for analysis in an area with several new developments or experiencing unusually fast growth that out paces historical growth rates.

There are some limitations of Traffix. Traffix does not use ODOT's accepted analysis procedure for roundabouts. The electronic files (input and output) will need to be provided. Screen prints may also be required to show various inputs. Traffix queue lengths must not be used for unsignalized intersections and may only be used for isolated signalized intersections where no simulation is being performed. Gates may be needed between attractions to show trips occurring between attractions. In a model based forecast, volumes should be post processed and are not considered to be when using a factor or multiplier in this program.

Highway Capacity Software
Highway Capacity Software (HCS) implements the procedures defined in the HCM for analyzing capacity and determining LOS for signalized intersections, unsignalized intersections, urban streets (arterials), freeways, weaving areas, ramp junctions, multi-lane highways, two-lane highways and transit. Intersection analysis is based on the methodologies presented in Chapters 16 and 17 of the HCM. While the HCS is a widely used tool, it can only accurately analyze intersections in an isolated environment, free from the effects of other intersections.

Synchro/SimTraffic

If Synchro is being used for the analysis, Synchro 8 or 9 shall be used. Synchro 7 does not provide the correct lost time calculations.

Synchro is a complete software package for modeling and optimizing traffic signal timings. Synchro implements both the Intersection Capacity Utilization (ICU) 2003 method for determining intersection capacity, as well as the methods of the HCM, Chapters 15, 16 and 17; Urban Streets, Signalized Intersections and Unsignalized Intersections and reports both results. For analysis of ODOT facilities, the signalized intersection v/c ratio or the unsignalized highest movement v/c ratio obtained from the HCM Signalized and Unsignalized reports shall be used.

Synchro is the preferred analysis tool for areas where surrounding intersection operations can influence each other, as it will consider the effects of the coded transportation network on each intersection. This software is also suggested for projects where traffic simulation will be desired, because the street network and operational parameters used can be directly transferred to the SimTraffic program or other simulation programs. ODOT has conducted extensive research on the use of Synchro for analyzing state facilities and has documented several procedures for implementation and default values, which are provided in the next section. NOTE: Many of these procedures also apply to other programs, such as HCS-Signals and should be used where applicable.

7.3.9 Synchro Settings

This section shows the ODOT Synchro settings organized by window. The Simulation Settings Window is only used by SimTraffic and is covered in Section 8.3. The bullet points below only cover the important inputs.

Lane Window

- **Ideal Saturated Flow Rate –** default is 1900; however, ODOT's default is 1750 (see Section 3.5.2). The best way to determine the Saturated Flow Rate is to measure it in the field. (See HCM 2000 Chapter 16, Appendix H)
- **Lane Utilization Factor, F_{LU}, –** is calculated by Synchro, but may be overridden and can have a large impact on the movement saturation flow rate. This factor shall be calculated by the analyst if groups of two or more lanes exist including through and turn lanes that might be affected by uneven lane distribution. Uneven lane distribution can either occur with nearby downstream turn movements or where through lanes drop or add.

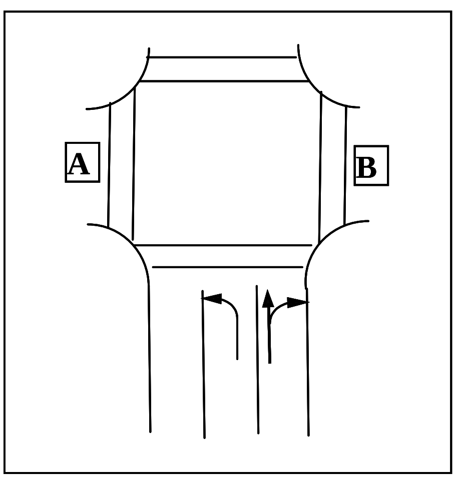
  F_{LU} = 1/(n * (Proportion in Heaviest Travel Lane))
  where:
  n= number of lanes in lane group

Volume Window

- **Conflicting Peds –** enter the number of pedestrians that conflict with the permissive right turn movements and the permissive left-turn movements. This value will generate pedestrians in SimTraffic for unsignalized intersections, so this value should only be coded for actual pedestrian paths.

  In Exhibit 7-17, pedestrians in Crosswalk A conflict with the northbound lefts and pedestrians in Crosswalk B conflict with the northbound rights. Pedestrians will not reduce the saturation flow rate for protected turn movements or through movements.

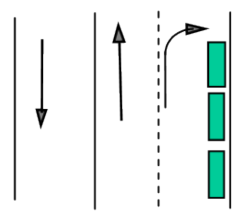
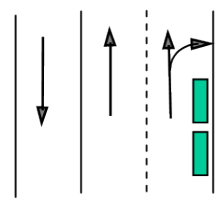
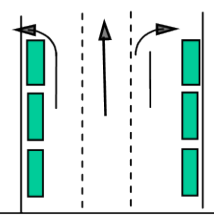
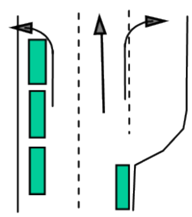
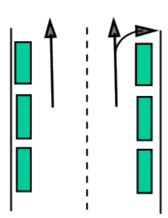
Exhibit 7-17 Conflicting Pedestrian Movements



- **Peak Hour Factor** (PHF) **–** enter the peak hour factors. For current year analysis, use the actual PHF determined from the manual counts. For future year analysis, use the ODOT PHF default values unless the current PHF's are larger as shown in Section 5.3.

- **Heavy Vehicles –** enter the percentage of trucks and buses for the hour being analyzed for each approach or movement. These values should match the classification count information.
- **Adjacent Parking Lane –** if there is on street parking for this approach, check the box for the adjacent parking lane and enter the number of parking maneuvers per hour. Enter parking maneuvers for each lane group that is affected. Note that parking with zero parking maneuvers per hour is different from no parking as the adjacent parking lane still has an impact. Exhibit 7-18 shows typical parking scenarios and shows the affected lane groups.

Exhibit 7-18 Parking Coding

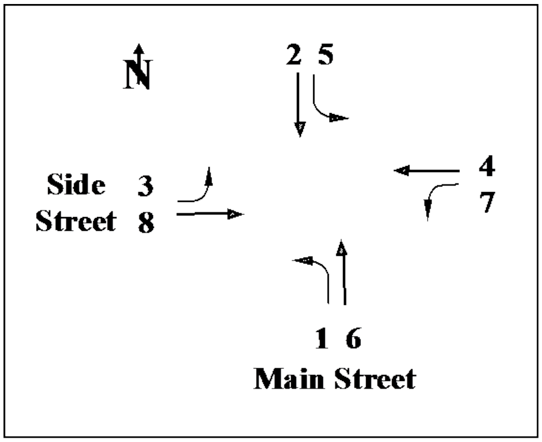
Lane Configurations		Affected Lane Groups		
		Left	Through	Right
	1 Through, 1 Right Long Storage			X
	2 Through		X	
	1 Left, 1 Through, 1 Right Long Storage	X		X
	1 Left Long Storage, 1 Through, 1 Short Right Storage	X	X	X
	2 Through		X	

- **Traffic From Mid-Block –** this is the proportion of traffic that comes from driveways and minor unsignalized intersections. This field should be used instead of trying to code multiple adjacent driveways which will result in excessive congestion.

Timing / Signing Window

- Exhibit 7-19 shows the ODOT default for signal phasing. This is different from the Synchro defaults. (Alternatively, if Main Street ran E-W, phase 6 would be WB.) This needs to be appropriately set for each intersection to have the signal operation work correctly.

Exhibit 7-19 Signal Phasing Diagram



- **Description** – This field can be used to record changes to settings when modifying alternatives, timing, calibrating simulations, or when reviewing other's files.
- **Controller Type:**
    - Actuated-Uncoordinated – This is the primary controller type used by ODOT in isolated situations. When analyzing for a new isolated signal, this is generally the correct controller type to assume;
    - Actuated-Coordinated – This is the primary controller type used by ODOT in progressed network situations;
    - Pretimed – This is used primarily in grid network situations (i.e. downtown networks) or older controllers on city streets;
    - Semi Actuated-Uncoordinated – No longer used by ODOT for permanent controller types, but it may be found on city or county facilities.
    - Unsignalized – Stopped controlled intersections;
    - Roundabouts – Synchro 8 and later versions will analyze single-lane roundabouts using the current HCM methodology.

  For existing networks, the type of controller can be determined by observation or through contacting your Region Traffic office. In new construction, the analysis will determine what controller type will be necessary.
- **Cycle Length (s)** – Good guidance for a maximum initial cycle length is determined by the number of phases: two phase = 60 s, three phase = 90 s, four phase =120 s.
- **Referenced to** – ODOT standard is set to "Beginning of yellow" For Type 170 signal controllers. Newer Type 2070 controllers use "Beginning of Green." This specifies the phase the offset is referenced to.
- **Reference Phase** – the coordinated phases for an actuated signal. ODOT uses the mainline phases 2 and 6.
- **Yellow and All-Red Time** – Use the ODOT defaults as shown in Exhibit 7-20 when

grades do not exceed 3 percent. For grades that do exceed 3 percent, the ITE formula below should be used for the yellow clearance intervals. Left turns may be treated as 25 mph approaches.

ITE Yellow Clearance Intervals

$$y = t + \frac{v}{2a + 2Gg}$$

Where:

y = length of the yellow interval, to the nearest 0.1 sec
t = driver perception-reaction time, recommended as 1.0 sec
v = velocity of approaching vehicle, in ft/sec (or m/sec)
a = deceleration rate, recommended as, 10 ft/sec^2 (3.05 m/sec^2)
g = acceleration due to gravity, 32 ft/sec^2 (9.8 m/sec^2)
G = grade of approach (3% downgrade would appear as -0.03)

- **Lost Time Adjust –** ODOT default for lost time is 4.0 seconds (unless unusual conditions exist that would warrant a longer time). Synchro 7 and later versions redefined the lost time calculation so it is necessary to adjust the lost time up or down to match the default. The lost time adjustment is equal to the difference between the sum of the yellow and all-red times and the lost time default. See Exhibit 7-20 for the default lost time adjustments.

Exhibit 7-20 Recommended Yellow, All-Red & Lost Time Adjustment Values*

85th Percentile Speed (mph)	Yellow (s)	All Red (s)	Lost Time Adjustment (s)
25	3.5	0.5	0.0
30	3.5	0.5	0.0
35	4.0	0.5	-0.5
40	4.3	0.5	-0.8
45	4.7	0.7	-1.4
50	5.0	1.0	-2.0
55	5.0	1.0	-2.0

* These yellow and all-red values are generally applicable where downgrades are less than or equal to 3 percent

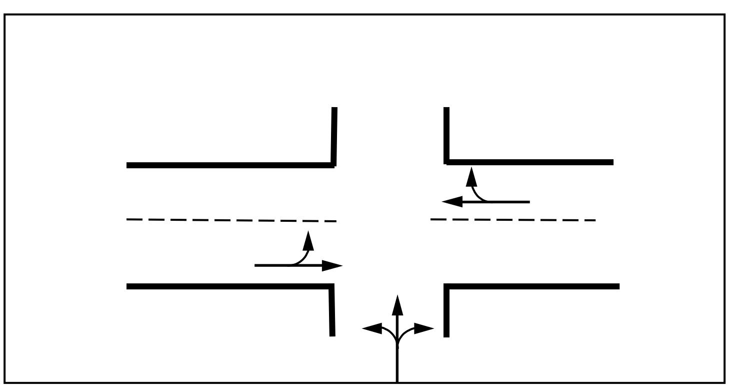
- **Lagging Phase? –** Checking this box will set this phase to lag the corresponding phases. This is generally for left turns, but can also be set for through and right turns provided that there are separate phases provided.
  Note: With "Dog-house" type permissive-protected left turn signals, lagging left turns are not allowed. Otherwise, a lagging left turn creates the "left-turn trap/yellow trap" where during the change from permissive movements in both directions to a lagging through phase in a single direction, the opposing movement does not stop as may be expected by a driver in the left-turn lane.

- **Recall Mode –** Defines what phases the signal can skip. There are four options: None, Min, Ped and Max. None will allow the phase to be skipped; Min requires the phase to occur for at least the minimum green and cannot be skipped. Ped requires a walk and flashing don't walk time to occur (i.e. in a downtown area) and Max (functionally equivalent to pre-timed) requires the phase to occur for the maximum green and cannot be skipped. Recall settings will only work correctly if consistent with mainline/side-street phase settings. Incorrect phase settings may result in a signal not giving green time to certain moves.
  For:
    - Major protected lefts & minor movements – Set to None;
    - Major through movements – Set to Min (Minimum).
    - Intersections that are at the junction of two progressed systems – Set to Min for all legs.

When the unsignalized controller type is selected, the Timing Window becomes the Signing Window. The below bullet points pertain to the Signing Window.

- **TWLTL Median** – Used to indicate whether a section is a two-way left-turn lane versus a regular median section. This will show the typical TWLTL striping on the screen, but Synchro will not analyze TWLTL operation. Using this setting will assume two-stage gap operation for the side-street even though this movement may not be compatible with a TWLTL. Two-stage gaps should only be coded if actual field observations show such behavior. New build alternatives should not be designed with two-stage gaps. However, the HCM considers wide medians where vehicles stop perpendicular to the mainline also to be a two-stage gap. Use this setting with caution.
- **Critical Gap(s)** – Leave the Synchro calculated default unless the unsignalized intersection is at an interchange ramp terminal or start of a one-way grid section where there are four legs but only three approaches. Synchro does not use the proper gap times for an unsignalized intersection with a one-way minor street such as at an interchange ramp terminal. Synchro is using the gap times appropriate for a four-legged intersection with four approaches, however, one-way minor street intersections have four legs but only three approaches (see Exhibit 7-21 below).

Exhibit 7-21 Four-Leg Three-Approach Intersection Illustration



The critical gap times (t_c) need to be changed for the minor street left turn only. The value is different depending on how many lanes are on the major street (see Exhibit

7-22). All other critical gap times stay the same. After the value is changed it will be in red to indicate a user-overridden value. Deleting the value out and pressing "Enter" will restore the value back to the default setting.

Exhibit 7-22 Critical Gaps for Four-Leg Three-Approach Intersections

	Two-Lane Major Street	Four/Six-Lane Major Street
Critical Gap t_c (s)	6.4	6.8

Phasing Window

Pedestrian Timing can have a significant impact on an intersection operation. Timing can be obtained from the signal timing sheets or the Region Traffic offices. Otherwise, the walk time is 7 seconds and the curb-to-curb "Flashing Don't Walk" time is generally calculated at 3.5 ft/sec for the length of the crosswalk. Areas with more pedestrians or older pedestrians may have different timings, so please check with Region Traffic or Traffic-Roadway Section.

Changing the defaults for the actuated signal phasing settings are only required if a calibrated simulation or actuated signal timing plans will be created. Exhibit 7-23 shows the required Phasing Window settings. These factors have a significant impact on the calibration. These settings can come either from signal timing (preferred) or from Exhibit 7-23.

These settings are defined as:

- Minimum Initial - Minimum green time
- Minimum Split - Minimum green + yellow + all-red + walk + flashing don't walk times. Leave higher values otherwise errors will result.
- Vehicle Extension (also known as "Passage" on a timing sheet) – Time that a detector extends the green time up to the maximum time available for that phase.
- Minimum Gap – Gap time after reduction until end of phase.
- Time Before Reduce (TBR) – Time elapsed before gap time is reduced
- Time To Reduce (TTR) - Time elapsed during gap time reduction to minimum.

Exhibit 7-23 ODOT Phasing Settings Defaults*

Parameter (s)	Left Turns (s)	Mainline Through's (s)	Side Street Through's (s)
Minimum Initial	4.0	10.0	6.0
Minimum Split	13.0 min.	14.0 min.	13.0 min.
Vehicle Extension	2.5	4.0	2.5
Minimum Gap	2.0	2.7	2.0
Time Before Reduce	8.0	10.0	8.0
Time To Reduce	4.0	13.0	4.0

*The values in this table are the general phasing settings from the Traffic-Roadway Section (TRS) except for the minimum gap for left turns and side streets. The TRS minimum gap values for these movements are 0.5 seconds. SimTraffic is too sensitive with the 0.5 second value because it does not allow enough time to adequately represent field conditions and a longer time is needed to prevent excessive queues and gap outs. The 0.5 second value should be retained for signal timing plan construction.

Note that these changes in the Phasing Window, especially minimum split times, might change the cycle length and maximum splits (especially for left turns and side streets), so the system optimization should be re-run.

Detector Window

If timing plans that involve actuated signals or a SimTraffic simulation needs to be created, the Detector Window data must be entered. Synchro uses this data to model actuated signal operation. Correct detector settings are critical to a successful simulation in Synchro. If actuated signal operation or simulation is not going to be utilized, the Synchro default detector settings can be used.

- **Number of Detectors –** Enter in number of detectors (1 to 3) for a given lane type.
- **Detector Phases –** Phase that is triggered by detection zone. This value is carried over from the Timing Window.
- **Leading and Trailing Detectors –** Not used in Synchro other than to maintain backwards compatibility with earlier versions. These values are automatically updated as more detailed detector position and size data is entered. The Leading Detector is the first detector that a vehicle encounters on an approach (furthest from the stop bar) while the Trailing Detector is the last detector on an approach and closest to the stop bar. **Warning: Do not modify these values if specific detector information is being entered as these will overwrite the detector data and create many errors.**
- **Detector Templates –** While all of the detector data can be added individually, it is very repetitive and the use of the Detector Template can quickly add the correct ODOT detector settings for new or existing signals in Synchro. The TPAU Analysis Tools web page has default Synchro template files that contain the full base ODOT detector settings.
- **Detector Position (ft) –** Enter distance to stop bar. Detector 1 is closest to the stop bar. Position varies by speed on approach and lane type.
- **Detector Type –** Synchro has three detector types – Call, Extend, or Call + Extend. ODOT uses the third type: Call + Extend (Cl+Ex). The detectors are capable of being both a “Call” or a “Extension” detector depending on the signal state. The call function occurs during the red time for phases that require a “call” for a green placed to the signal controller when a vehicle triggers the detector. The extend (or extension) function occurs during the green time and is used to extend the green time to allow vehicles to smoothly flow through the intersection approach.
- **Detector Size (ft) –** Enter size of detector. ODOT standard is a 6 foot diameter loop so enter “6” feet for all extension detectors. Detection on the side street is a pair of loops tied together, so these are coded as a single 16 foot detector.
- **Detector Delay –** Only used for side street exclusive right turn lanes to delay the detection call to the controller. This will limit the number of times that the side street phases will need to come up. This value is usually 10 seconds.

Detector Settings

There is a difference between ODOT detector placement standards which measure to the center versus Synchro which measures to the leading edge of the detector. Exhibit 7-24 shows the ODOT Synchro detector placement.

ODOT's detector settings are incompatible with the HCM 2010 as the maximum distance from the stop-bar is 20 feet. Signalized intersection analyses need to use HCM 2000 to obtain the intersection v/c ratio. When a HCM 2010-only analysis is desired for local non-state intersections where the intersection v/c ratio is not necessary, the detector placement is limited to the side-street placement style (one 16' detector at two feet from the stop-bar). All analyses that will use simulation ultimately, regardless of jurisdiction, will need to use the full detector settings and an HCM 2000 analysis.

If turn bays are shorter than 72' then eliminate Detector 2 as Synchro will give an error as the detection zone will exceed the storage length. Synchro may have trouble showing detectors in the Map Window in shorter turn bays, so it may be necessary to adjust the detector spacing or turn bay length. Short turn bays in the field likely only have a single detector (verify in the field).

If exclusive right-turn lanes on the mainline exist, there is only one detector at 137' from the stop bar. If the turn bay is shorter than 137' then adjust the distance to approximately 2/3 of the total bay distance, but do not put the detector at the extreme end of the turn bay as turning vehicles may miss the detector or through vehicles may trigger it.

Detectors for ramps are based on whether the ramp is a low speed ramp, such as a loop ramp (<45 mph), or a high speed ramp, such as a diagonal ramp (≥45 mph).

Exhibit 7-24 Synchro-Adjusted ODOT Detector Type and Position

Lane Type	Speed (mph)	Number of Detectors	Position (distance from stop bar to leading edge of detector, in feet)		
			Detector 1	Detector 2	Detector 3
Side or Left	All	2	2	72	
Right	All	1	137		
Ramp	<45	3	2	72	132
High-speed Ramp	45+	3	2	107	207
Mainline Through	25	1	137		
	30	1	177		
	35	2	107	217	
	40 & 45	2	157	317	
	50	2	187	377	
	55	2	222	447	

It is possible that detector type/placement differs in the field (i.e. call detectors at the stop bar on the mainline), so customization may be necessary. Local intersections can use the ODOT Side Street detector settings or default Synchro data, but should be field-checked for accuracy.

Detector spacing can also be modified to fit video detection zones for intersections that have cameras.

Optimizing Signal Operations

Existing conditions (base year) need to be optimized if the timing did not come exclusively from timing sheets. The only exception is when a calibrated existing condition analysis is being used for simulation. Short-term analyses may require optimization even if timing sheets were used. All future no-build conditions and future build alternatives must optimize the signal timing, either as an isolated case or a signal system. Mainline phase orientation, reference phase, offset style, and recall settings are set appropriately before optimizing.

> Note: If at any time a change is made to intersection geometry, volumes, signal timing, etc., the system shall be re-optimized.

Existing field timing may or may not be fully optimized; it is often set to minimize motorist delay or queue lengths. For planning purposes or traffic impact studies, ODOT's practice is to optimize the timing for the best intersection v/c ratio. Movement v/c's should be relatively even on the intersection approaches. The cycle length and phase splits should be optimized for each analysis (existing, no-build and build alternatives) except during simulation calibration work, see Chapter 8. Generally, optimizing with longer cycle lengths and fewer vehicle phases will result in a lower v/c, however longer cycle lenths will increase queuing.

- **Intersection Cycle Length Optimization–** This algorithm optimizes the cycle length for a single intersection, based on delay.
- **Intersection Splits Optimization–** This algorithm optimizes the phase splits for a single intersection. This is a good place to start when optimizing the v/c for an intersection. Subsequent adjustments to movement green time may improve the v/c.

  Make sure that Lead/Lag Optimize is set properly. This will allow Synchro to optimize the phase sequence (leading or lagging operation for left turns) when the signal functions as part of a coordinated system. Lagging operation may be inappropriate for high volumes, five-section "doghouse" heads or other concerns. Flashing yellow left turn heads can operate either in leading or lagging mode.

  When optimizing for part (zone) or a whole network the system optimization should use the manual cycle length option so the best system cycle length can be found. An increment of 5 seconds should be used. System cycle lengths:
  - Should not exceed ODOT's 60-90-120 second cycle limits for 2, 3 and 4 phase signals, respectively.
  - Show promise for minimizing delay and stops and maximizing bandwidth
  - Should have a low number of "dilemma vehicles", i.e. a low number of vehicles expected to be caught near the intersection when a signal turns yellow.
  - May change the phase splits at the intersections. Verify that the new split times are acceptable.

The optimized network should have good progression between signals in the system. The quality of the progression will generally be determined by noting the size of the bandwidth for the selected cycle length. The bandwidth should be maximized as much as possible. OAR Division 20 should be reviewed to ensure that the resulting bandwidth is acceptable. Link speeds can be dropped five mph in order to check minimum bandwidth requirements.

The Time-Space diagram will need to be reviewed as Synchro optimizes for delay, not progression bandwidth. The analyst will need to drag the intersection offsets on the time-space diagram with a mouse to improve the arterial bandwidths (the link bands are not used). Bandwidths should be visible for both directions in most cases. Experimentation is often necessary to determine which intersections are critical for increasing the bandwidth. Arterial bandwidths should be maximized for each direction unless it is desired to have a larger bandwidth in a given direction (i.e. outbound commuter flow). In addition, leading/lagging settings in the Timing window should be reviewed for opportunities to improve the bandwidth. The analyst should also consider alternative system cycle lengths to maximize the bandwidth.

Required Synchro Output Reports

The following reports should be retained for file documentation about the no-build conditions or an alternative.

- **Lanes, Volumes, Timings Report** - This report contains the information that is used as inputs into Synchro. When reviewing a Traffic Impact Study (TIS), it is essential to have this report in order to verify the analysis results. The Synchro default reports are adequate.
- **Queues Report** - This report contains information about estimated queue lengths and blocking. Synchro reports two queue lengths:
    - The 50th percentile queue is the largest queue length for a typical cycle;
    - The 95th percentile queue is the maximum back of queue with 95th percentile traffic volumes. This is the queue length used by ODOT to determine recommended storage lengths.
    - Note: Synchro calculates queue lengths as the maximum queue after only two cycles. In conditions where the v/c > 0.70, Synchro queues may not accurately reflect queuing projections, especially if the intersection/node spacing is less than the estimated queuing. Watch for presence of “m” or “#” codes next to the queuing values. The “m” means that an upstream signal is metering the queue, so queues in this movement are actually shorter than they would be if there was not a constraint elsewhere. The “#” means that the 95th percentile volume is over capacity so queues shown are likely much longer, even up to twice shown. In these cases, the Synchro-based queues are not adequate and SimTraffic simulations are required for intersection queuing. In constrained analyses where the v/c ≥ 0.90, arrival rates become unstable and the estimated queue lengths in Synchro are unreliable and SimTraffic simulation-based queues are required.
- **HCM 2010 Signalized or Unsignalized Report** - Synchro will report out analysis of both signalized and stop controlled intersections. These follow the methodologies in Chapter 18 (Signalized) and Chapter 19 (Unsignalized) of the HCM 2010.
    - HCM 2010 Signalized - This report generally follows the same outputs as in the HCM and the Highway Capacity Software (HCS). Some items may be different such as actuated green times which in turn may affect some of the calculations.
    - HCM 2010 TWSC/AWSC (Unsignalized) - Synchro will analyze two-way (TWSC) and all-way stop controlled intersections (AWSC) following HCM 2010 methodology. The effect of upstream traffic signals is now included in the Synchro analysis.

- HCM 2010 Roundabout - Synchro will analyze roundabouts using HCM 2010 methodology.
- HCM 2000 Signalized – This report is needed for obtaining the intersection v/c ratio (not an output with HCM 2010). Intersections with custom phasing or certain shared lane configurations may also require that this report is used instead of the HCM 2010 report when intersection v/c is not necessary.

7.4 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 MUTCD provides a set of 8 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 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.

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, the Region Traffic Engineer, input from TRS must be obtained prior to reaching any conclusions. Coordination with TRS 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, TRS 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 TRS and coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual.

7.4.1 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 8 traffic signal warrants found in the MUTCD, Page 4C-1. The signal warrants are:

- Warrant 1, Eight-Hour Vehicular Volume
    - Case A – Minimum Vehicular Volume
    - Case 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

OAR 734-020-0460 (1) stipulates that only MUTCD Warrant 1 Case A and Case B may be used to project future needs for traffic signals beyond three years from the present time (Corrected to reflect numbering used in the Millennium Edition of the MUTCD). Case A deals primarily with high volumes on the intersecting minor street. Case 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 Case A or Case B is met.

Information for Narrative
The following statement should be included in the Analysis Methodology section of the Narrative:

> TPAU uses Signal Warrants 1, Case A and Case 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 TRS. 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 Case 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 have already been completed for the analyst, as can be seen in Exhibit 7-25.[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 that 10,000, reduce the target volume for the warrants to 70 percent of the normal requirements. The warrant volumes, along with the number of lanes, are shown in the preliminary traffic signal warrant analysis sheet in Exhibit 7-25.

[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 7-25 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.

1. Major Street (Higher Volume Street)
   - Include only the through and through/turn lanes in the number of approach lanes.
   - 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.
   - 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.

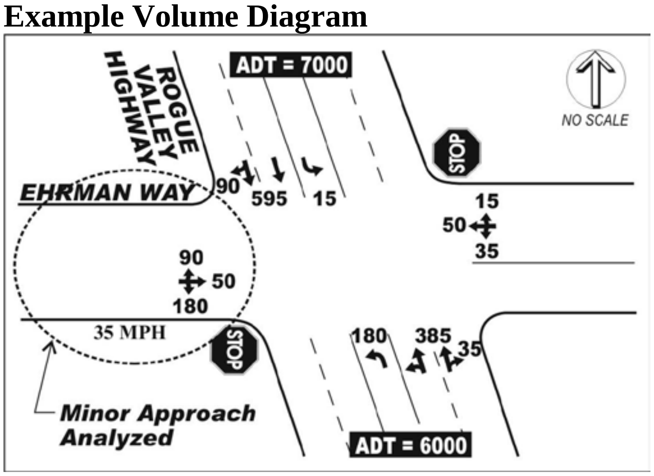
Example 7-6 Right Turn Discount for Shared Left/Through/Right Lane

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



Signal Warrant Analysis Example

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 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		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)

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+	10,600	13,000	N
	Minor	1	**2,650**	**2,180**	
Case B	Major	2+	**15,900**	**13,000**	N
	Minor	1	1,350	2,180	

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.

The figure above shows the Preliminary Signal Warrant Analysis for Example 7-9. The preliminary signal warrant is not met because the Minor Street ADT is less than the warrant volume in Case A and the Major Street ADT is less than the warrant volume in Case B, as shown in the darkened cells.

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.

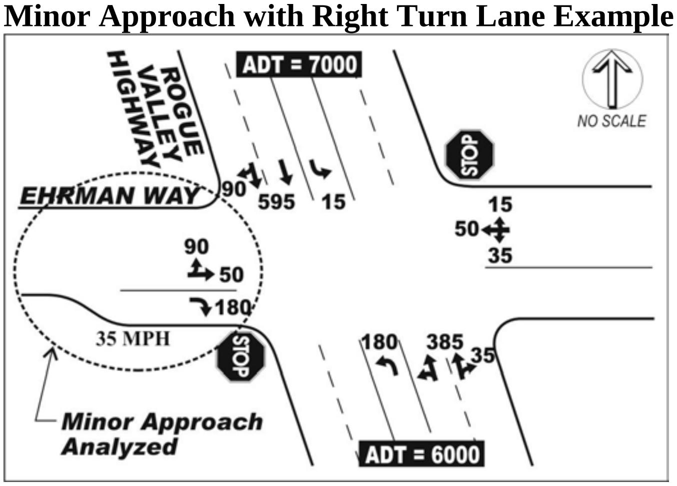
Example 7-7 Right Turn Discount for Exclusive Right Lane Lane

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 Example 7-10. The preliminary signal warrant is not met since the Minor Street ADT is less than the warrant volume in Case A and the Major Street ADT is less than the warrant volume in Case B, as shown in the darkened cells.

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, 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	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)					
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+	10,600	13,000	
	Minor	1	**2,650**	**1,400**	N
Case B	Major	2+	**15,900**	**13,000**	
	Minor	1	1,350	1,400	N
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.

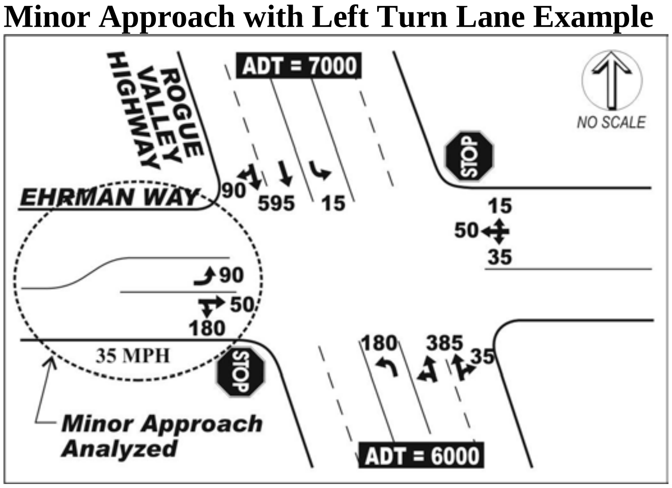
- Scenario # 3 – Shared Through-Right Lane: Some of the right turns are included in the approach ADT if the right turn demand is greater than 85% of the capacity of the shared through-right lane. Use unsignalized capacity analysis to calculate the capacity of the through-right shared lane. The right 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 7-8 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. 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 Example 7-6. The preliminary signal warrant is not met since the Minor Street ADT is less than the warrant volume in Case A and the Major/Minor Street ADT's are both less than the warrant volumes in Case B, as shown in the darkened cells.

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		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)					
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+	10,600	13,000	
	Minor	2	**3,550**	**1,400**	N
Case B	Major	2+	**15,900**	**13,000**	
	Minor	2	**1,750**	**1,400**	N
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.

- 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 of the 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.

7.4.2 Manual of Uniform Traffic Control Devices Signal Warrants

As previously noted, the MUTCD provides 8 warrants to be used in determining whether the installation of a traffic signal is justified for a given location. It should be noted that while the MUTCD states that a traffic signal should not be installed unless one or more of the warrants are met, it also emphasizes that meeting one or more warrant shall not in itself require the installation of a traffic signal and that the analysis of the warrants should be included as part of a comprehensive engineering study. The MUTCD warrants, if evaluated, should be evaluated along with all the other components of a full traffic signal engineering investigation as described in the ODOT Traffic Manual. MUTCD signal warrants should only be evaluated for existing and future short-term (up to 3 years in the future) conditions. Evaluating the need for a traffic signal over 3 years is not recommended as land uses and travel patterns can change within that time period, therefore, traffic conditions are not as predictable. A brief description of each warrant is included below. For a complete description of the warrants and their appropriate application, see the MUTCD.

- **Warrant 1, Eight-Hour Vehicular Volume**: Either can qualify. This warrant has two conditions.
    - Minimum Vehicular Volume, Condition A, is where a large volume of intersecting traffic is the principal reason to consider installing a traffic signal.
    - The Interruption of Continuous Traffic, Condition B, is for where the major street volume is so heavy that minor street traffic suffers excessive delay or conflicts with the major street.
- **Warrant 2, Four-Hour Vehicular Volume:** Applied where the volume of intersecting traffic is the principal reason to consider signal installation
- **Warrant 3, Peak Hour**: Used at locations where traffic conditions are such that for a minimum of one hour of an average day, the minor street traffic suffers undue delay when entering or crossing the major street.
- **Warrant 4, Pedestrian Volume**: Where the major street volume is so heavy that a large number of pedestrians experience excessive delay in crossing the major street.
- **Warrant 5, School Crossing**: For use where school children crossing the major street is the principal reason to consider installing a traffic signal.
- **Warrant 6, Coordinated Signal System**: For use where progressive movement in a coordinated signal system necessitates installing traffic signals at intersections where they would not otherwise be needed to maintain proper vehicle platoons.
- **Warrant 7, Crash Experience**: Intended where the severity and frequency of crashes are the reason to consider installing a traffic signal. This should include the three most recent calendar years for which data is available and only those crash types susceptible to correction by traffic signal control should be considered. Generally requires a minimum of 5 such crashes in a 12-month period.
- **Warrant 8, Roadway Network**: Is intended for use where installing traffic signals at some intersections might be justified to encourage concentration and organization of traffic flow on a roadway network.

The MUTCD and ODOT Traffic Signal Policy and Guidelines provide for the installation of traffic signals that meet criteria for special applications. These applications include providing access to fire and other emergency vehicles, regulating the flow of traffic at a freeway ramp, controlling traffic at a drawbridge or at a one-lane facility and temporary installations for construction projects.

7.5 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 through the use of simulation software. The 25-foot average does not apply to microsimulation, where vehicle lengths differ by vehicle type. Refer to Chapter 8.

7.5.1 Methodologies for Signalized Movements

For signalized movements queue length estimates are most often recommended to be calculated using traffic analysis software. 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.

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.

Left Turn Movement Queue Estimation Techniques

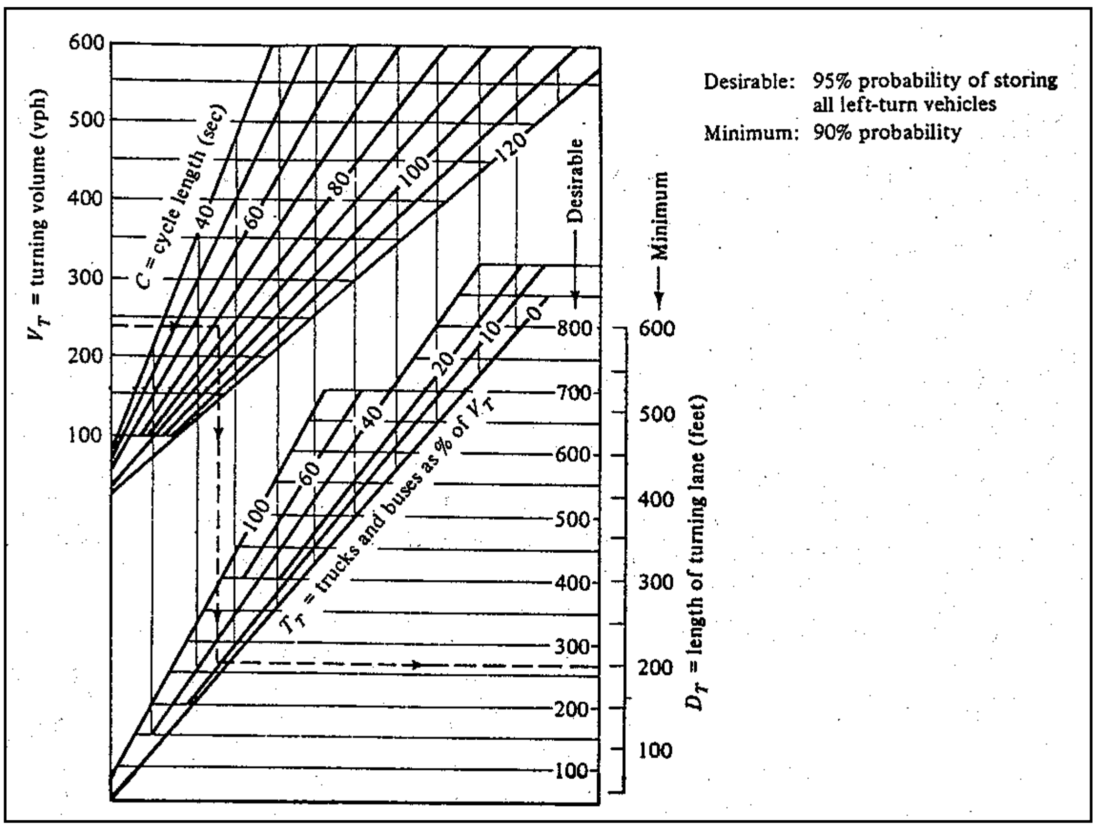
Three common methods of manually estimating vehicle queue lengths for single-lane left turn movements include the use of a nomograph[2] and two "rule of thumb" procedures. The nomograph (Exhibit 7-27) assumes a random rate of arrivals and uses the turning volumes, signal cycle length and a weighted average vehicle length based on the percentage of trucks in the turning volume to estimate vehicle queues at 90th and 95th percentile probabilities of storing all vehicles.

A "rule of thumb" equation[3] uses similar input while providing a simple procedure that can be applied without need to reference a manual. Using this method, single-lane left turn vehicle queue lengths are estimated as 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 7-28. Typically, transportation analysis uses the 95th percentile queue.

Exhibit 7-26 Nomograph for Estimating Single Lane Left Turn Vehicle Queue Lengths at Signalized Intersections



[2] J. E. Leish, *At-Grade Intersections*, A Design Reference Book and Text, Jack E. Leish & Associates, undated.
[3] *Discussion Paper No. 10: Left-Turn Bays*, Transportation Research Institute, Oregon State University, 1996, p. 17.

Exhibit 7-27 Selection of "t" Values

Minimum "t" Value	Percentile
2.0	98 %
1.85	95 %
1.75	90 %
1.0	50 %

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 7-29. This adjustment is only for the manual methods; software packages may require a different adjustment.

Exhibit 7-28 Storage Length Adjustments for Trucks

Percent Trucks in Turning Volume	Average Vehicle Storage Length
< 2%	25 ft
5%	27 ft
10%	29 ft

While both the nomograph and the rule of thumb equation are 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 these methods by 1.8. This value represents the assumption that queued vehicles will not be evenly distributed between the turn lanes.

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.

$$\text{Storage Length} = (1\text{-}G/C)\ (V)\ (K)\ (25\text{-feet})\ /\ (\text{Number of Cycles Per Hour})\ (N_L)$$

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.

[4]Koepke, F. J., Levinson, H. S., *Access Management Guidelines for Activity Centers*, NCHRP Report 348, TRB, Washington, D.C., 1992, p. 99.

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 Exhibit 7-29.

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.

Computer 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.

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 the SimTraffic simulation software be used to report the 95th percentile queues. Refer to Chapter 8 for further information on SimTraffic.

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.

7.5.2 Methodologies for Unsignalized Movements

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)[5], the Two-Minute Rule, the Harmelink Curves[6] and a method published by John T. Gard[7].

TPAU has conducted a study to evaluate the first three of these methodologies for estimating queue lengths. This study, Storage Estimates for Unsignalized Intersections, concluded that while the Two-Minute Rule provided conservative estimates for major street left turns and minor street right turns, it appeared to underestimate queue lengths for minor street left turns and shared left/right and left/through/right lanes. Despite this, the Two-Minute Rule was still found to produce more reliable results than the HCS or Harmelink methods. In particular, the HCS

[5] *Highway Capacity Software*, McTrans, University of Florida, Gainesville, Floriday.
[6] M.D.Harmelink, *Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections*, Highway Research Record 211, 1967.
[7] *Estimation of Maximum Queue Lengths at Unsignalized Intersection.* John T. Gard, ITE Journal/November 2001.

method was found to consistently underestimate queue lengths. This is confirmed in the previously mentioned study by John T. Gard. Therefore the HCS and Harmelink methods shall not be used. Either Simulation or the Two-Minute Rule may be used, until the John T. Gard method has been satisfactorily validated with local data.

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. If the effort to do a simulation analysis is not desired, the two-minute rule should be used. If the results of the two-minute rule analysis are deemed unacceptable, the option is to do a simulation analysis.

Two-Minute Rule

The Two-Minute Rule is a rule of thumb methodology that 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 95th 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 7-28.)
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.

8 TRAFFIC SIMULATION MODELS

8.1 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

8.2 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 (Section 4.6), 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 30^{th} 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 30^{th} 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 30^{th} 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 Section 4.4 to determine if the count is close enough to the 30^{th} highest hour. If the primary counts for the study area occurred during a time that is less than 90% of the 30^{th} 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.

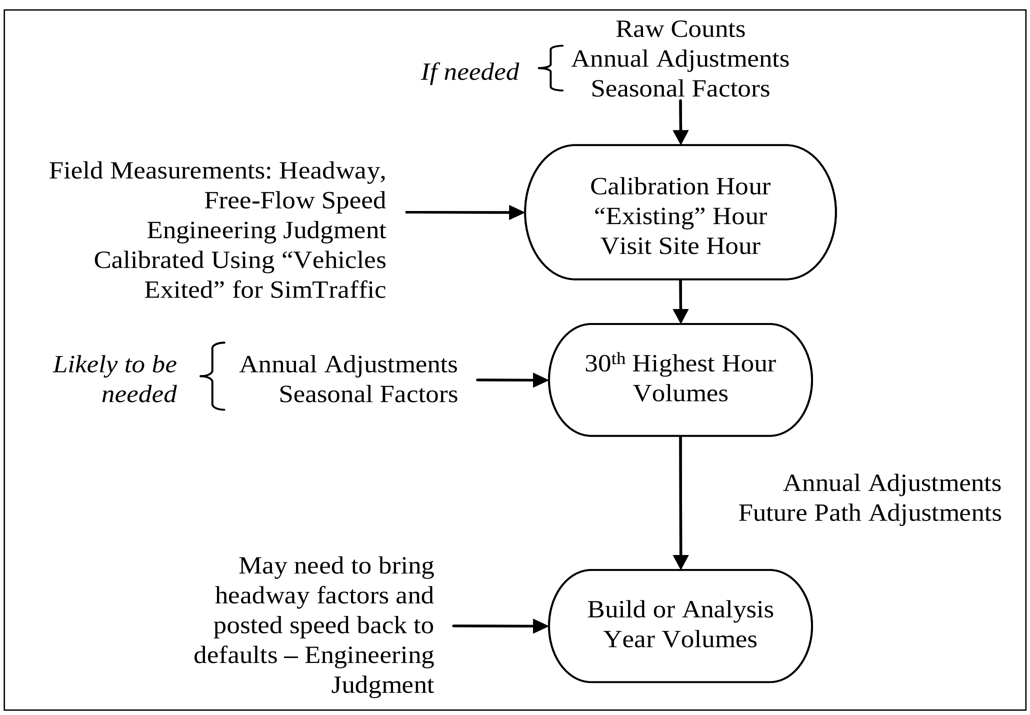
These rules are established to help ensure that calibration volumes 1) are near the 30^{th} highest hour and 2) represent conditions that have been witnessed in the field. The emphasis is placed on witnessed, as the analyst needs to visit the study area on or near the count day (30^{th} highest hour) 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. In Chapter 3, Transportation System Inventory, Exhibit 3-2 shows an example completed worksheet for a simulation project. 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 Exhibit 3-1).

The site visit should occur as close as possible to the 30^{th} highest hour. After the site a calibration scenario can be constructed. For the purpose of calibration, the peak hour volumes from the counts should be seasonally adjusted to the time period 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 Sections 7.3.9 and 8.3. 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 8-1 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 8.3 has the detailed procedures on calibrating a SimTraffic model using SimTraffic for ODOT projects.

Exhibit 8-1 Simulation Construction and Application Flow Chart



8.3 <u>SimTraffic</u>

8.3.1 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.

8.3.2 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 Section 3.2, 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 of time. 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 8-2 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 8-2 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.

8.3.3 Simulation Preparation

In addition to setting up the SimTraffic parameter file, there are a number of 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 Section 7.3.7. 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 Section 7.3.9 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 8.3.3 and 8.3.4, respectively. Earlier versions of SimTraffic only need to create the SimTraffic parameter file.

8.3.4 Simulation Settings Window

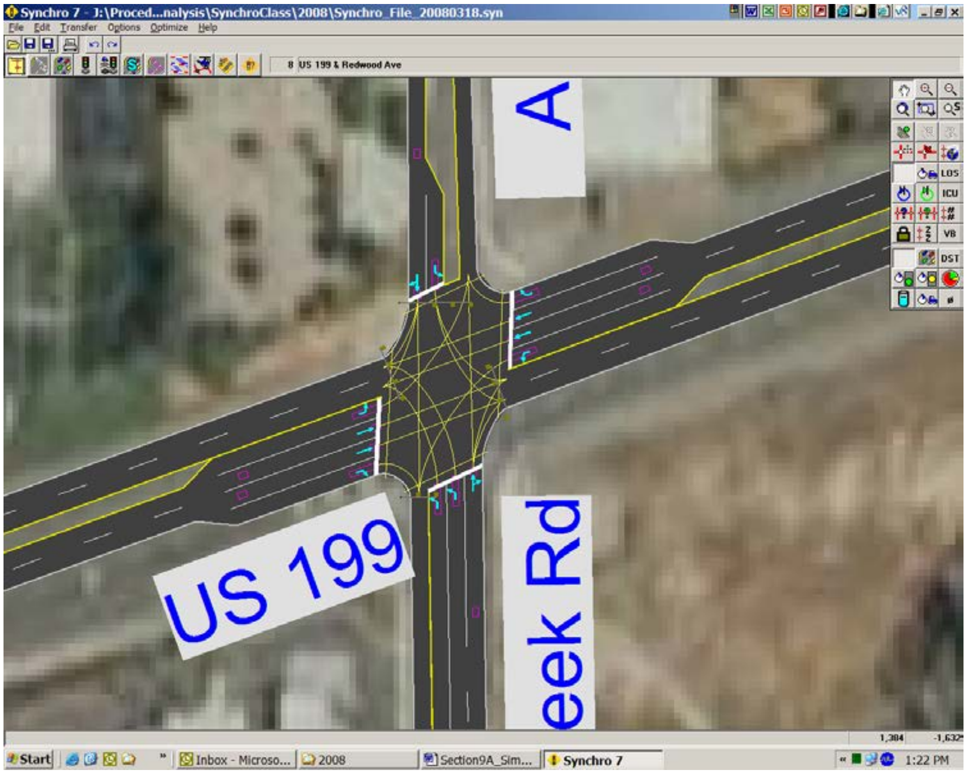
The following data is only used by SimTraffic and needs to be included for a proper simulation. This data allows for geometric refinement and operational behavior of the simulation. The data required by SimTraffic should be a part of the field collection/observation process and is included the Field 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 8-3 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 8-3 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 be always be coded legally.

Note that the northbound dual left turn lane shown in Exhibit 8-3 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 has a tendency 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 8-4 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 8-4 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 amount 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 of 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.

8.3.5 SimTraffic Parameter File

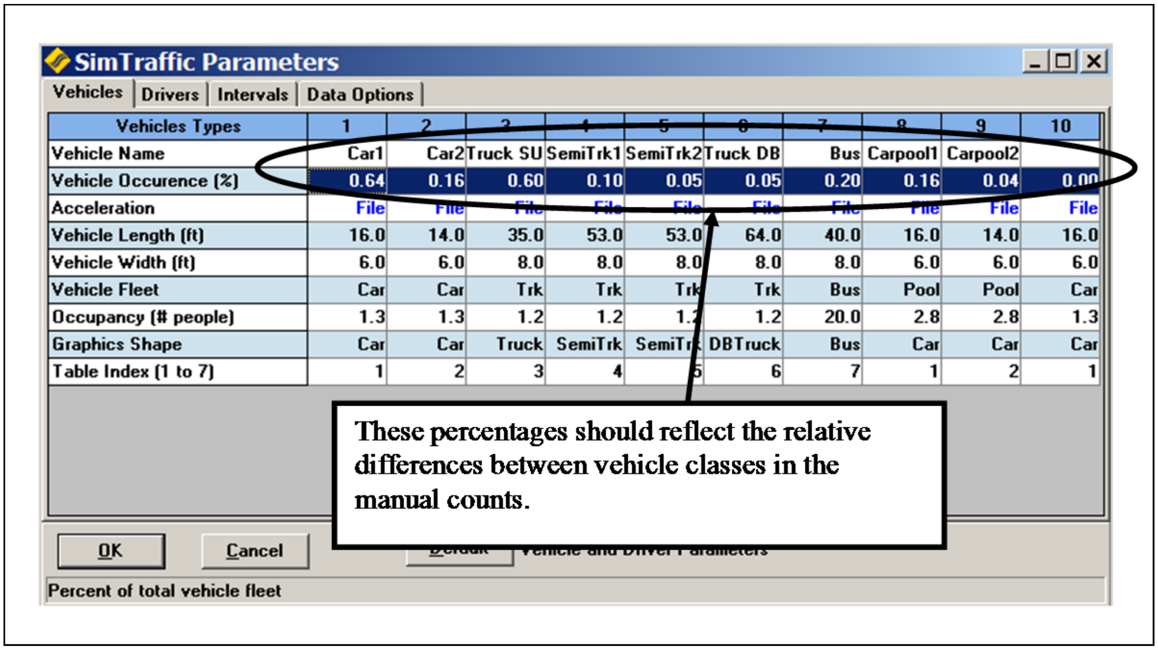
The SimTraffic parameter file controls the simulation operation and the defaults must be changed to reflect the proper impacts of queuing, travel time, etc. The parameter file has three major sections: Vehicles, Drivers and Intervals. The TPAU Analysis Tools webpage has a default SimTraffic template file with all of 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 8-5.
  - 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 http://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.

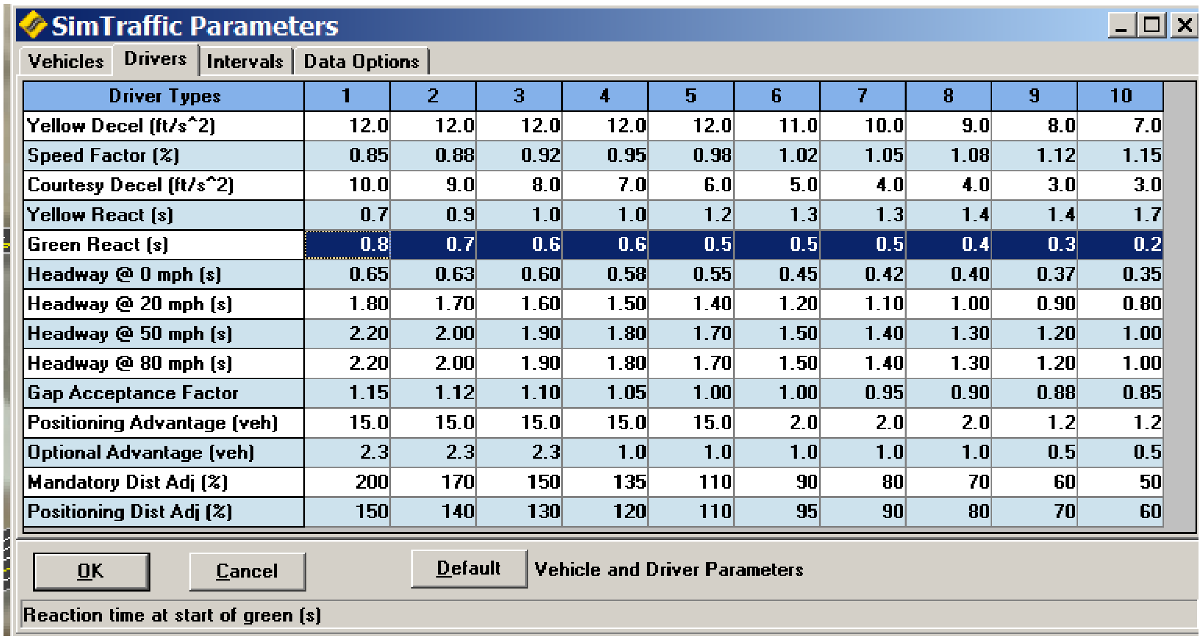
Exhibit 8-5 SimTraffic Default Vehicle Parameters



- **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 8-6) 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 8-7.

Exhibit 8-6 SimTraffic Default Driver Parameters



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

Exhibit 8-7 ODOT Green React Times

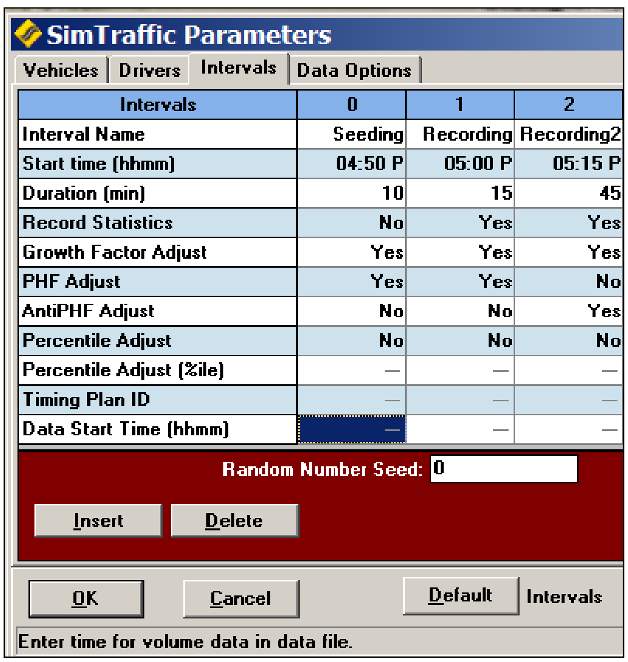
Driver Type	1	2	3	4	5	6	7	8	9	10
Green React (s)	2.0	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.5

The Intervals tab controls the actual operation and data recording of the simulation. Exhibit 8-8 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 8-8 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)	—	—	—

8.3.6 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.

8.3.7 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 8-9 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 8-9 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.

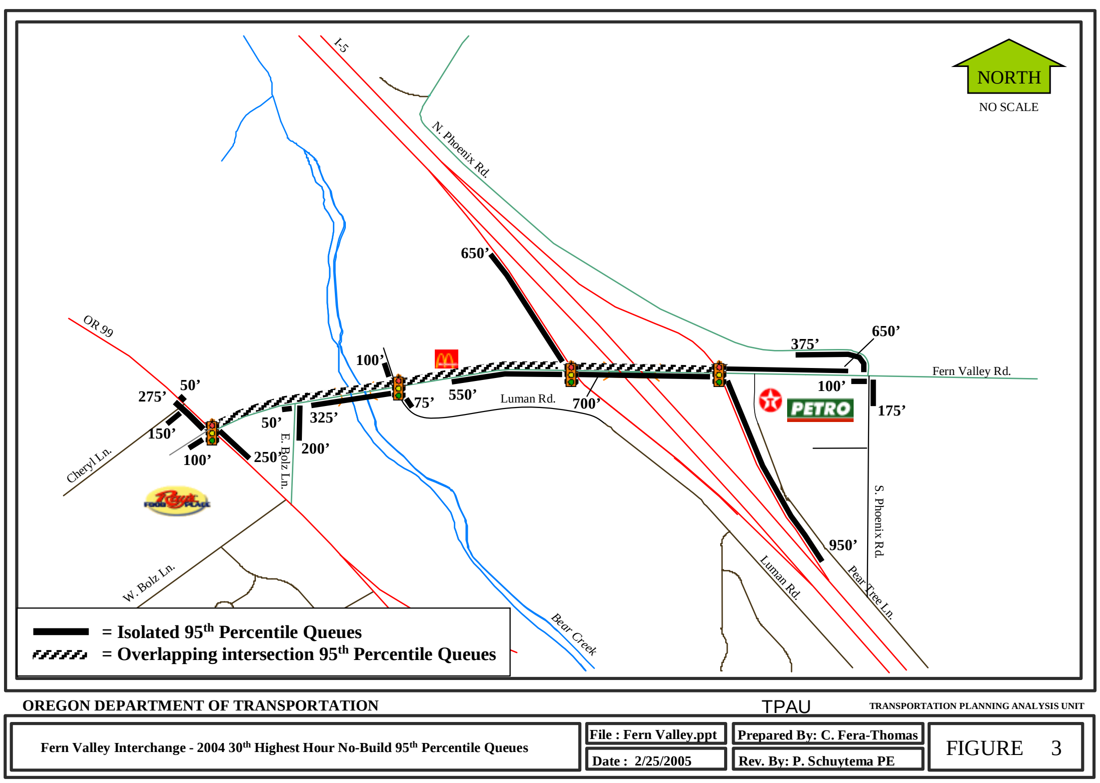
- **Upstream Blk (Block) Time (%)** - This is an estimated percentage of the peak hour in which the queue from the subject node blocks an upstream node. This is especially useful when analyzing a complex Synchro network, to determine the extent of queuing on a system when reporting out results. It can 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 completely 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 though 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 8-10 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 8-10 Sample Queuing Diagram



- **Performance Report -** The Performance report (Exhibit 8-11) 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 8.3.1 for more instruction.

Exhibit 8-11 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 8-12) 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 has to have the same road name without any special characters (i.e. dashes) along all of 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 8-12 Sample Arterial report

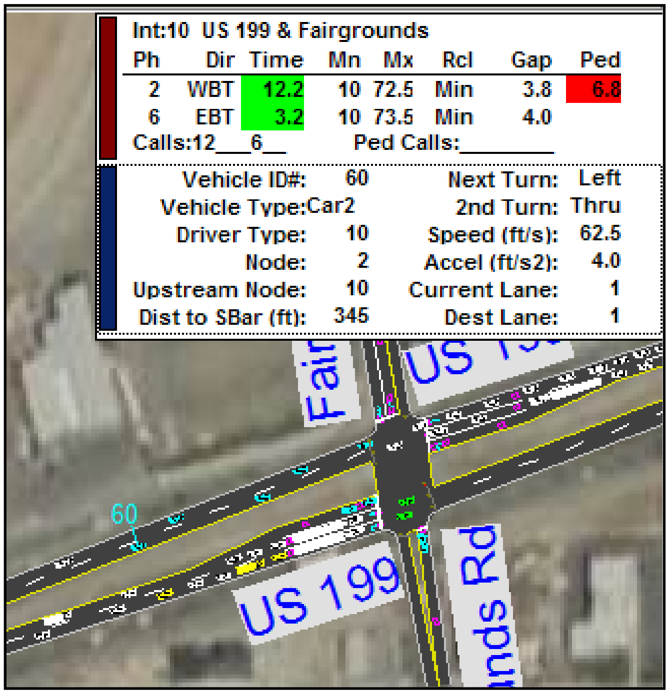
Arterial Level of Service: EB US 199, Entire Run

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 8-13) 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 of these can be useful in debugging a simulation. It is recommended that the simulation speed be set to real time or slower for best viewing.

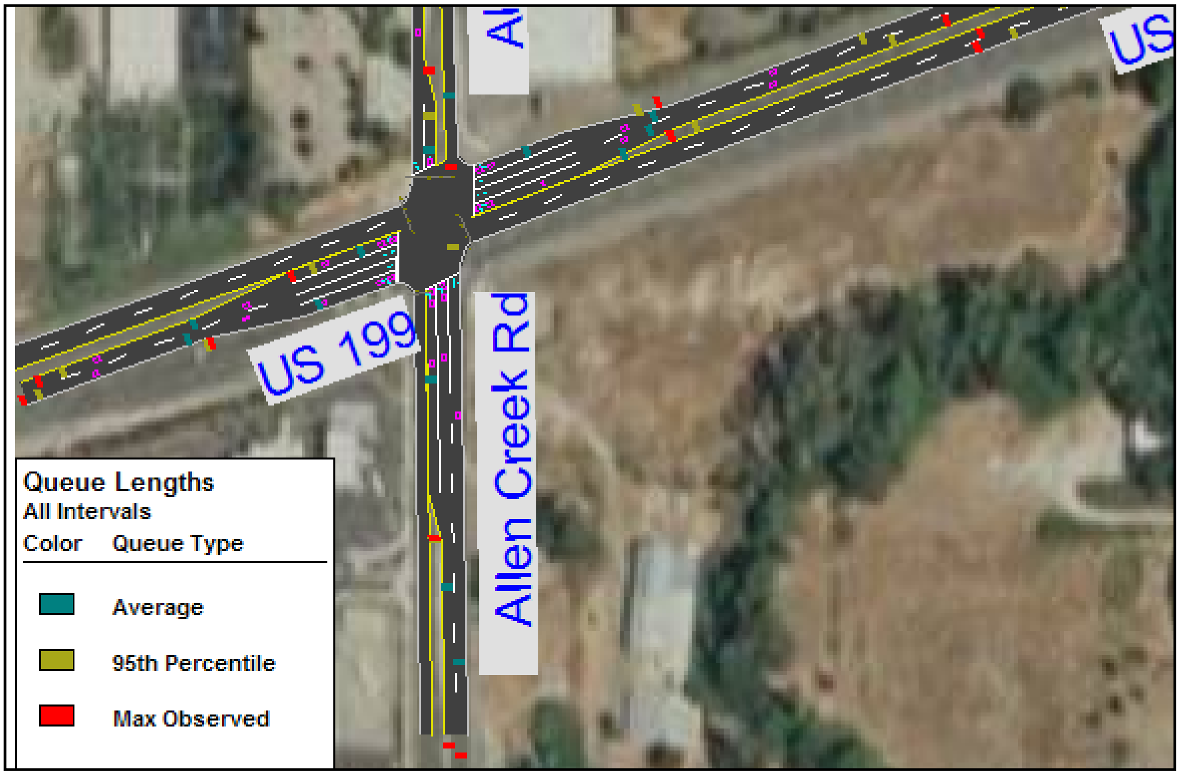
Exhibit 8-13 Animated Vehicle and Signal Tracking



Static Graphics

Other reports include the "Static Graphics" reports (Exhibit 8-14). 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 use to quickly visualize the issues for the analyst or for others.

Exhibit 8-14 Example Queue Length Static Report



8.4 <u>Vissim - Overview</u>

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 program) that can then import the files into Vissim for analysis. 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.

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.

Most ODOT region offices do not currently own the Vissim software (outside of Region 1). The ODOT Synchro defaults should be implemented in the Vissim model to the extent possible. Most region offices outside of Region 1 are unlikely to have the knowledge base to use Visum.

8.5 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 similar to 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.

8.6 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.

9 DETERMINING NEEDS

9.1 <u>Purpose</u>

The primary purpose for conducting the analysis presented in previous chapters is to determine how a given facility performs relative to the selected performance measures of the study. This chapter presents an overview of the process for comparing the results of the Existing and No-Build analysis with adopted OHP standards, in order to identify deficiencies in the performance of the facility. Solutions are addressed in Chapter 10. Topics covered include:

- Standards for Determining Needs
- Applicable Oregon Highway Standards
- Analysis of Transportation Systems

9.2 <u>Standards for Determining Needs</u>

The term ‘need’ as used by transportation professionals is defined as:

> “A ‘need’ has generally been defined by transportation analysts as any case where the current or planned facility conditions falls below an established standard.”

The above perspective assumes that the adopted standard is the minimum acceptable condition for a facility and any case where conditions fall below that level is a deficiency that should be corrected. The relevant standards presented in the latest version of the Oregon Highway Plan should be considered, as discussed in Chapter 2. Standards 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.

Selection of Performance Measures

Performance measures, sometimes referred to as measures of effectiveness, are quantitative criteria that indicate how well a function or activity is being performed. Some common performance measures used in traffic engineering include v/c ratio, LOS, vehicle delay, travel time, emissions, vehicle speed, mode shift and capacity.

Most road authorities (state, county or city) maintain adopted standards for operational efficiency that identify specific performance measures. It is important to identify all applicable standards and corresponding performance measures 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. The use of performance measures to identify needs and evaluate alternatives is discussed further in Chapters 9 and 10. ODOT measures highway mobility performance through volume to capacity (v/c) ratios and has adopted separate standards for identifying current and future needs and project design.

Operational standards for identification of current and future needs are documented in the 1999 OHP in Policy 1F. Tables 6 and 7 within Policy 1F list maximum allowable v/c ratios for various combinations of highway classifications and surrounding land uses, with Table 7 applying to the Metro Area and Table 6 applying to the remainder of the state. However, it should be noted that the text within Policy 1F contains exceptions to the standards listed in these tables and, therefore, must be consulted as well. Furthermore, the OHP Registry of Amendments webpage should be checked for amendments that may affect this policy.

As an example, Amendment 00-04, which was adopted on December 13, 2000, created alternate mobility standards for the South Medford Interchange and the Metro Area. These alternate standards can be found in the document “Amendment to 1999 Oregon Highway Plan Alternate Highway Mobility Standards Metro Area.” When using these standards, it should be noted that there is an error in the Table 7 footnote. The existing first bullet under OHP Table 7 was a leftover from the original Table 7 and is proposed to be stricken from the OHP with the next revision. Each of the hours needs to be analyzed separately, using an appropriate PHF, with the results compared to the respective v/c ratios provided in Table 7.

These standards are applicable to existing, future no-build and future build conditions for TISs (typically associated with comprehensive plan amendments, zone changes, development reviews and approach applications) and all no-build alternative work for existing and future conditions analyzed in other types of projects including transportation facility projects, transportation system plans, corridor plans, refinement plans, interchange area management plans and access management plans. In situations where an interchange or interstate freeway needs to be modified in association with proposed development impacts, it is necessary to coordinate with Federal Highway Administration (FHWA) and the developer to work out any issues relative to the OHP versus ODOT’s HDM guidelines.

Operational standards for project design are documented in Exhibit 10-1 of ODOT’s HDM. These standards (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 standards are applicable to future build alternatives 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 performance measures for different project types.

Exhibit 9-1 Sources of Performance Measures by Project Type

	TIS	Projects	Studies
Existing Conditions	OHP	OHP	OHP
Future No-Build	OHP	OHP	OHP
Future Build(s)	OHP	HDM	HDM

9.3 Applicable Oregon Highway Standards

9.3.1 Mobility

The OHP establishes the mobility standards for all state facilities. ODOT measures highway mobility performance through v/c ratios and has adopted separate standards for identifying current and future needs and project design.

Most of the analysis procedures summarized inExhibit 9-2 have direct (or equivalent) v/c ratio results for performance assessment. The compliance with the appropriate standard (maximum v/c ratio thresholds defined in the OHP) is the first tier of the evaluation. These procedures are noted in Exhibit 9-2.

Exhibit 9-2 Types of Performance Measures Applications

Type of Analysis	Volume to Capacity Ratio	Meets / Does Not Meet	Speed	Queue Length
Signalized Capacity	X			
Unsignalized Capacity	X			
Preliminary Signal Warrants		X		
Signal Warrants		X		
Turn Lane Criteria Analysis		X		
Queuing Analysis				X
Segment Analysis	X		X	
Progression Analysis			X	
Weaving Analysis	X		X	
Merge/Diverge Analysis	X		X	
Passing/Climbing Lanes	X		X	
Simulation Modeling	X		X	
Arterial Analysis			X	

However, several procedures do not yield v/c ratio outcomes. For example, traffic signal warrants are one guide to assess the readiness of an intersection or junction to be controlled by signals, but it is not, by itself, a performance indicator. However, these analyses are useful to flag potential modifications in traffic controls or facility designs that should be incorporated into Build scenario evaluations.

The other category of performance measures focuses on travel speeds, 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 should be used in conjunction with either intersection or segment analysis that do have v/c ratio related outcomes to determine the compliance with mobility standards.

9.3.2 Safety

The safety evaluations parameters are less discrete compared to mobility standards and generally rely on a comparative evaluation to other state facilities as a basis for acceptability. Section 5.2 of the Oregon State Highway Crash Rate Table states:

> "Table II presents a five-year comparison of crash rates for the state highway system, for urban and rural areas by functional classification."

For the crash analysis, use this table to compare the historical segment crash rate for a studied section to the statewide average rate in the table for a comparable type. The analyst must determine if the studied segment is within an urban or rural area, the roadway classification and whether it is a state primary or secondary highway. A listing of primary and secondary highways is included after Table IV in the Crash Rate Table. Note that the category "State Highway System" provided alongside the primary and secondary system categories is a combination that should NOT be used for most crash rate comparisons.

Exhibit 9-3 2008 Crash Rates by Jurisdiction and Functional Classification

JURISDICTION AND FUNCTIONAL CLASSIFICATION	MILES	ANNUAL VEHICLE MILES	CRASHES	FATALI-TIES	CRASH RATE*	FATALITY RATE*
TOTAL STATE HWY SYSTEM	**7,453.23**	**19,523,091,729**	**16,142**	**221**	**0.83**	**1.13**
Interstate Freeways	730.52	8,526,366,378	3,169	38	0.37	0.45
Other Fwys/Expressways	54.27	1,290,552,234	858	8	0.66	0.62
Non-Freeways (combined)	6,668.44	9,706,173,117	12,115	175	1.25	1.80
Other Principal Arterials	3,280.79	7,509,225,541	9,631	115	1.28	1.53
Minor Arterials	1,959.83	1,811,486,662	2,031	44	1.12	2.43
Urban Collectors	8.69	10,172,238	11	0	1.08	0.00
Rural Major Collectors	1,381.52	371,721,968	439	16	1.18	4.30
Rural Minor Collectors	34.72	3,432,935	3	0	0.87	0.00
Rural Local	2.89	133,773	0	0	0.00	0.00
URBAN HWY SYSTEM	**819.67**	**9,207,412,773**	**10,054**	**61**	**1.09**	**0.66**
Interstate Freeways	176.15	4,445,167,356	2,066	16	0.46	0.36
Other Fwys/Expressways	54.27	1,290,552,234	858	8	0.66	0.62
Non-Freeways (combined)	589.25	3,471,693,183	7,130	37	2.05	1.07
Other Principal Arterials	512.59	3,163,978,720	6,584	34	2.08	1.07
Minor Arterials	67.97	297,542,225	535	3	1.80	1.01
Urban Collectors	8.69	10,172,238	11	0	1.08	0.00
Urban Cities	**568.62**	**6,973,941,364**	**8,497**	**48**	**1.22**	**0.69**
Interstate Freeways	111.61	3,256,667,634	1,733	13	0.53	0.40
Other Fwys/Expressways	47.73	1,184,858,022	794	6	0.67	0.51
Non-Freeways (combined)	409.28	2,532,415,708	5,970	29	2.36	1.15
Other Principal Arterials	366.57	2,337,353,812	5,527	26	2.36	1.11
Minor Arterials	41.06	192,668,622	440	3	2.28	1.56
Urban Collectors	1.65	2,393,274	3	0	1.25	0.00
Suburban Areas	**251.05**	**2,233,471,409**	**1,557**	**13**	**0.70**	**0.58**
Interstate Freeways	64.54	1,188,499,722	333	3	0.28	0.25
Other Fwys/Expressways	6.54	105,694,212	64	2	0.61	1.89
Non-Freeways (combined)	179.97	939,277,475	1,160	8	1.23	0.85
Other Principal Arterials	146.02	826,624,908	1,057	8	1.28	0.97
Minor Arterials	26.91	104,873,603	95	0	0.91	0.00
Urban Collectors	7.04	7,778,964	8	0	1.03	0.00
RURAL HWY SYSTEM	**6,633.56**	**10,315,678,956**	**6,088**	**160**	**0.59**	**1.55**
Interstate Freeways	554.37	4,081,199,022	1,103	22	0.27	0.54
Non-Freeways (combined)	6,079.19	6,234,479,934	4,985	138	0.80	2.21
Other Principal Arterials	2,768.20	4,345,246,821	3,047	81	0.70	1.86
Minor Arterials	1,891.86	1,513,944,437	1,496	41	0.99	2.71
Rural Major Collectors	1,381.52	371,721,968	439	16	1.18	4.30
Rural Minor Collectors	34.72	3,432,935	3	0	0.87	0.00
Rural Local	2.89	133,773	0	0	0.00	0.00
Rural Cities	**218.91**	**491,825,707**	**536**	**0**	**1.09**	**0.00**
Interstate Freeways	14.05	89,782,362	26	0	0.29	0.00
Non-Freeways (combined)	204.86	402,043,345	510	0	1.27	0.00
Other Principal Arterials	109.77	271,934,779	321	0	1.18	0.00
Minor Arterials	53.24	92,629,842	148	0	1.60	0.00
Rural Major Collectors	41.60	37,222,524	41	0	1.10	0.00
Rural Minor Collectors	0.25	256,200	0	0	0.00	0.00
Rural Areas	**6,414.65**	**9,823,853,249**	**5,552**	**160**	**0.57**	**1.63**
Interstate Freeways	540.32	3,991,416,660	1,077	22	0.27	0.55
Non-Freeways (combined)	5,874.33	5,832,436,589	4,475	138	0.77	2.37
Other Principal Arterials	2,658.43	4,073,312,042	2,726	81	0.67	1.99
Minor Arterials	1,838.62	1,421,314,595	1,348	41	0.95	2.88
Rural Major Collectors	1,339.92	334,499,444	398	16	1.19	4.78
Rural Minor Collectors	34.47	3,176,735	3	0	0.94	0.00
Rural Local	2.89	133,773	0	0	0.00	0.00

* Crash Rate Formula: ((crashes*1,000,000)/VMT); Fatality Rate Formula: ((deaths*100,000,000)/VMT)

When comparing a statewide average rate to a segment crash rate for a study highway, simply exceeding the statewide average rate should not be interpreted as proof that a section is hazardous. Much like an intersection crash rate of 1.0 or greater, a segment crash rate that exceeds the statewide average crash rate should merely be considered as an indication that further investigation is necessary. The analyst should also examine the collision type and collision information such as time of day, milepost, roadway conditions and other factors to more accurately understand the crash history.

9.4 Analysis of Transportation System

9.4.1 Existing System

The analysis scoping, selecting performance measures and procedures for evaluating the existing transportation system are described in Chapters 2, 3, 5, 6, 7 and 8 of this manual. Refer to those sections for appropriate methods and techniques.

Elements of the existing transportation system that do not fall below current adopted performance standards should be flagged for consideration in developing facility alternatives. See Chapter 10.

Similarly the crash analysis procedures are described in Chapter 5. Locations that fall above the statewide average for a similar facility type and setting should be flagged for possible countermeasures or other improvements to be incorporated into the build plan alternatives. See Chapter 10.

9.4.2 Future No-Build System

The Future No-Build System typically includes the same street and intersection network, traffic controls and operational assumptions that were applied for the Existing System analysis without any improvement. In some cases, the Future No-Build System may include improvement projects that are assumed to be funded and constructed within the project planning horizon. The analyst should coordinate with Region Planning staff to identify these projects. Typically such projects would be listed in the STIP, city or county TSPs or MPO Regional Transportation Plans (RTPs). In these cases, it may be more useful to refer to this situation as the Future Base scenario, to reduce confusion with suggestion that no-build implies no improvement projects.

The same measures and analysis techniques applied for the Existing Transportation System will be applied on the Future No-Build System. However, the forecasted future volumes will be used in this analysis to assess how the future No-Build System operates. The future volumes should be developed according to the guidelines described in Chapter 4.

Elements of the transportation system that fall below current adopted performance standards should be flagged for consideration in developing facility alternatives. See Chapter 10.

There is no widely accepted method for assessing future traffic safety conditions. The detailed type of analysis used in Chapter 5 is not applied to future year traffic volumes. However, some project alternatives may help to resolve existing safety issues or deficiencies by upgrading substandard designs (modernization) or eliminating the primary conflicts (e.g., constructing a grade-separated crossing).

9.4.3 Travel Demand Management 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 demands within the project area. Common demand management techniques could include:

- Proposed changes to the current land use zoning.
- Restrictions to the intensity of development within an existing zone (e.g., trip caps).
- Increase or enhanced transit services.
- 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 give consideration to TDM components that can augment physical or operational improvements within the study area. Refer to Chapter 10 for more details about TDM options.

10 ANALYZING ALTERNATIVES

10.1 Purpose

The project alternatives should be developed and their effectiveness analyzed consistent with the goals and evaluation criteria selected for the project and to specifically address deficiencies identified through the Existing and No-Build System analysis. This chapter presents the process for conducting the transportation analysis of Build Alternatives. Topics covered include:

- Highway Design Manual Guidelines
- Screening Preliminary Alternatives
- Identifying Limitations to Design Concepts
- Documentation of Screening Process
- Evaluating Build Alternatives

10.2 Highway Design Manual Guidelines

The performance measures applied to flag deficiencies in the Existing or No-Build system, as described in Chapter 2, provide a basis for requiring improvements. However, when defining the scope and nature of improvements, these indicators are not sufficient. The project design guidelines identified in the Highway Design Manual should also be applied to measure acceptability of performance for the horizon forecast year. Refer to Chapter 2 for more detail.

The HDM has different design guidelines for different roadways and the expectation is that the guidelines will be followed. In some cases, however, the costs and impacts associated with a preferred improvement project are too great to fully comply with HDM guidelines and an exception to the design must be submitted and approved. Design exceptions are not intended as a commonplace occurrence, are not necessarily a quick process and should not be relied on prior to approval. Design exceptions may be needed for planning studies. Corridor studies are usually not developed at a level of detail that involves design exceptions. Transportation Growth Management (TGM) funded projects and refinement plans may have enough detail and information that would support design exception requests. As with normal project development projects, complete background information and sufficient justification as to why the guideline was unable to be met must be provided or be available to initiate the design exception process.

For a project that may be constructed within five years, the planner or project leader in charge of the planning project should contact the Region Technical Services Resource Manager (TSRM) to assist in putting together the design exception request. The design exception request should be processed in the same manner as a project development design exception, which is listed in Section 13.3 of the HDM. For projects that may be constructed within five to ten years, the design exceptions should be identified and the TSRM or the Roadway Engineering Manager should give an indication that a design exception is warranted and would probably be approved.

10.3 Screening Preliminary Alternatives

Alternatives for facilities should be developed, assessed and evaluated relative to the matrix of performance measures selected for this 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 screening process that allows for a large range of potential 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 sketch alternatives are then filtered to just a few alternatives through the first screening process. These alternatives would then be advanced to the next level in order to select the best candidates for the purposes of alternative performance evaluations. 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.

Projects that have an up-to-date travel demand model representation of the study area could use this tool to rapidly perform initial assessments of system performance without the need for detailed analytical calculations required for the full performance measures evaluation. These initial assessments typically focus on more general performance indicators, such as v/c ratios on arterials and highways, v/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.

10.3.1 Coordination with Stakeholders

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 2.

ODOT Engineers

Typically, the highway design and traffic operations engineers within ODOT have a key role in assisting the review and confirmation of the selected alternatives. The district or regional 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 refinements.

Local Agencies

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 or regional metropolitan planning organizations.

ODOT Rail Division

The Rail Division, which is 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 Rail Division should be contacted any time a project will have an impact directly to or within 500 feet of a railroad or rail crossing.

10.3.2 Potential Facility Solutions

Potential solutions to address existing or future deficiencies can range the following categories:

- Travel Demand Management TDM
- Potential Land Use or Regulatory Changes
- Access Control and Local Circulation Improvements
- Transportation System Management (TSM)
- Capacity Increases
- Intersection Control 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.

Travel Demand Management (TDM)
The initial assessment for 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. In general, they include:

- Carpooling/Ridesharing
- Shuttle Service/Transit Service Expansion
- Transit Fare Subsidies
- Flextime/Compressed Work Week
- Bike Parking/On-Site Lockers and Showers
- Telecommuting

The effectiveness of these types of programs can be estimated 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[8] that assessed the marginal reduction in traffic generation associated with various TDM options.

Potential Land Use or Regulatory Changes
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 allow less intense transportation uses.
- Restricting the intensity allowed within the current zoning by imposing trip caps that are regulated by local ordinance.
- Supporting mixed use development that minimizes trips onto the roadway system.

[8] Guidance for Estimating Trip Reductions From Commute Options, Oregon Department of Environmental Quality, August 1996.

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.

Access Control and Local Circulation Improvements

State facilities should be reviewed to compare background access provisions on state highways against the adopted standards as presented in OAR 734-051. Consolidating (or eliminating) existing vehicular access can substantially improve travel speeds and reduce vehicle conflicts along the highway. Typically, this would require coordination with affected property owners and implementation of necessary permits and easements to effect an alternative local circulation plan. This approach is most effective on a site that is making 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.

Transportation System Management (TSM)

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 are also discussed further in Chapter 7. The progression of potential solutions includes:

- **Reconfiguring Lanes:** 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.
- **Signal Phasing:** This involves signal phasing changes such as adding a right turn overlap or adding a u-turn.
- **Added Turn Lane Without Widening:** An example would be converting available shoulder or parking space for use as a turn lane.

Capacity Increases

<u>Added Turn Lane</u>

Review right and left-turn lane warrants to serve higher peak period demands. A good planning-level threshold is when turning volumes exceed roughly 150 to 200 vehicles per hour, a turn lane should be considered as an option. If the volumes satisfy warrants, review the intersection geometry to determine if improvements are required on the receiving side of the intersection to adequately serve the extra approach lane.

For example, a second left turn lane on one approach will require two lanes exiting the intersection for receiving the turning volumes. 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.

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.

- **Single Left or Right Turn 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.
- **Dual Left or Right Turning Lanes** (at intersections):Typically a dual left or right turning lanes 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.
- **Triple Left Turn Lanes**: When it starts to become apparent that dual left turn lanes are not sufficient to accommodate volumes, a grade separation should be considered as opposed to triple left turn lanes. Triple left turn lanes require a long run-out length of six-lane highway. ODOT presently has no triple left turn lanes.
- **Channelized 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 that requires no merging with other movements. This results in a free-flow movement with no conflict points.
- **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.

<u>Added Through Lane</u>

The addition of travel lanes on a highway facility may be appropriate to serve forecasted travel demands. As noted in the previous section, within urban areas the cross-section requirements of the highway may be influenced by the approach and departure lane requirements at the major intersections. Outside of urban areas, added through lanes may be needed to serve forecasted long-range growth in nearby communities or to reduce delays associated with trucks climbing extended grades. The limits of the recommended widening improvements should consider operational performance, study area intersections and the appropriate transition lengths back to the existing highway cross-section.

Intersection Control Improvements

<u>All-Way Stop Controls</u>

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. However, this solution should consider freight 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.

Roundabouts

This section will be updated when the HDM is updated with new roundabout siting considerations.

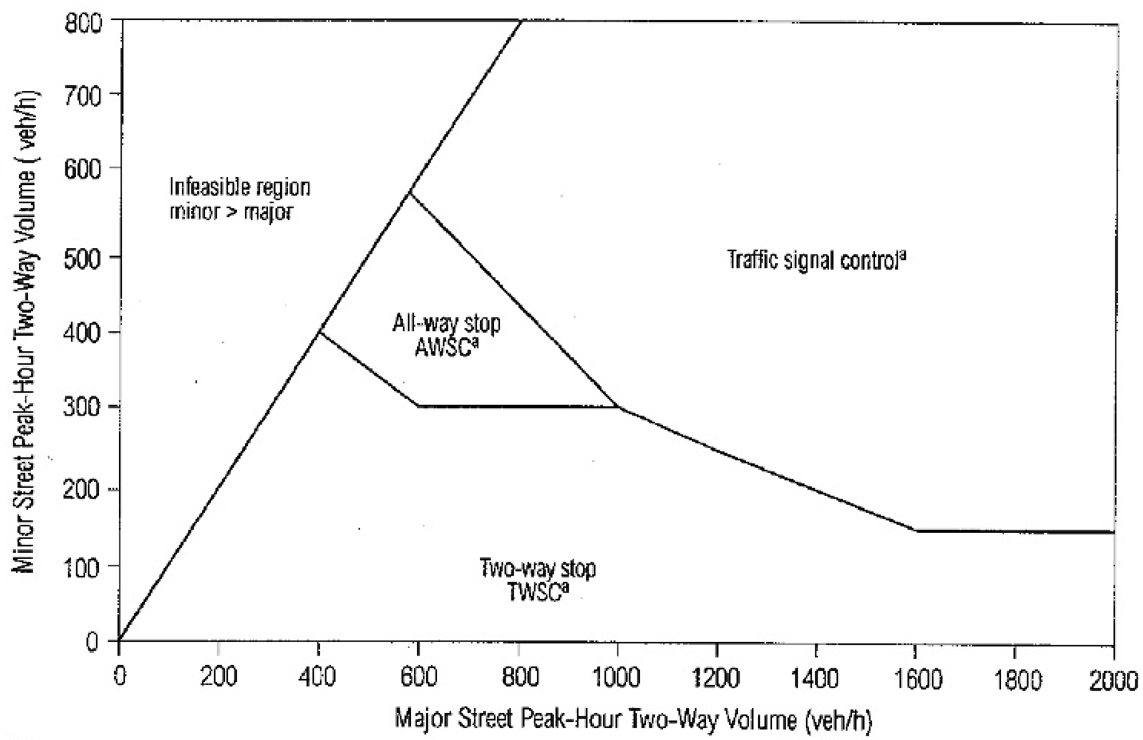
ODOT guidelines for consideration of siting roundabout facilities on state highways are contained in the Traffic Manual and the HDM. Both of these manuals are in the process of being updated. Currently, the Traffic Manual (2013 Edition, December, 2013) contains the following list of considerations.

- Freight mobility needs should be sufficiently defined and addressed prior to Conceptual Approval.
- Non-motorized user mobility needs such as the ability for bicyclists and pedestrians to safely move through the roundabout intersection should be balanced with the mobility needs of other motorized vehicles. Bicyclists should be given the option to use either the circulatory roadway with other vehicles or the pedestrian crossings outside the circulatory roadway. Special design consideration should be given for the pedestrian crossings at the entrances and exits on all legs of the roundabout where vehicles are either decelerating to enter the roundabout or accelerating to exit the roundabout.
- Roundabout design should consider the needs and desires of the local community including speed management and aesthetics.
- Intersection safety performance should be a primary consideration when pursuing a roundabout for intersection control. Predicted reductions in fatal and serious injury crashes should be compared with other types of intersection control such as traffic signals or other alternatives supported by crash modification factors (CMF) found in the AASHTO Highway Safety Manual.
- Roundabout entrance geometry, circulating geometry, and exit geometry should be designed to allow the design vehicle to traverse the roundabout in a reasonable and expected manner commensurate with best design practices as shown in NCHRP Report 672 and the Highway Design Manual. This design should utilize a representative template of the design vehicle and the vehicle path should be demonstrated through the use of computer generated path simulation software.
- Roundabouts should meet acceptable v/c ratios for the appropriate Design Life. (See the Design Life subsection for possible exceptions to this consideration.)
- Roundabouts proposed for state highways with posted speeds higher than 35 mph will require special design considerations (e.g. longer splitter islands, landscaping, reversing curves approaching the roundabout) to transition the roadside environment from higher to lower speeds approaching the roundabout intersection.
- For Roundabouts with more than 4 approach legs, special design considerations should be made for the layout of the approach legs.
- Roundabout proposals should address how roundabout operations would impact the corridor immediately upstream and downstream from the roundabout intersection. (If the proposed roundabout is in a location where exiting vehicles would be interrupted by queues from signals, railroads, drawbridges, ramp meters, or by operational problems created by left turns, accesses, these problems should be addressed by the Engineering Investigation.

Traffic Signal Controls

The ODOT standard signal warrant analysis is required to justify new signal installations. Issues to be considered include traffic volumes, freight volumes, pedestrian volumes, safety history and spacing relative to existing signal and the accepted standards for the highway facility. A general guideline for the appropriate type of intersection controls is presented in the HCM, Exhibit 10-15. A facsimile of that diagram is shown in Figure 10-1. As shown, the two-way vehicle volumes on the minor and major street facilities can be used to quickly determine possible traffic control schemes, ranging from two-way stop controls up to traffic signal controls. It is acknowledged that in some cases a roundabout installation may be an alternative solution to be considered.

Exhibit 10-1 Intersection Traffic Control Options



Notes

a. Roundabouts may be appropriate within portion of these ranges.

Source: Adapted from *Traffic Control Devices Handbook* (8, pp. 4–18) – peak-direction, 8-h warrants converted to two-way peak-hour volumes assuming ADT equals twice the 8-h volume and peak hour is 10 percent of daily. Two-way volumes assumed to be 150 percent of peak-direction volume.

Interchanges

Interchanges on highways are appropriate on 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. 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.

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 demands on the major arterials to a level that could be served by at-grade facilities.

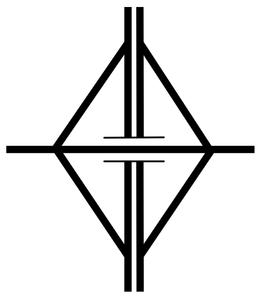
Grade-separated configurations should be developed to serve the forecasted travel demands consistent with the layout and spacing standards recommended in the HDM. Refer to that manual for more specific details that are useful in laying out interchange concepts. The following is a short review of the common elements of an interchange and a discussion of the conventional layout configurations that could be considered during alternative development:

Ramp Types

- **Jughandle Ramps**: These ramps are generally used at low-level interchanges, not for freeway connections and are characterized by low speeds. 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 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.
- **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. 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.
- **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. 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 stack) are semi-directional.

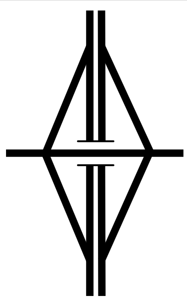
Interchange Types

Exhibit 10-2 Diamond Interchange



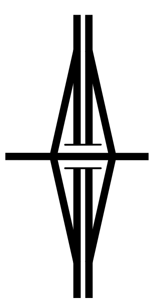
Diamond Interchange: An interchange that has straight ramps in all four quadrants is referred to as a diamond-shaped interchange. The capacity of this facility is typically determined by the operational analysis at the ramp terminals and merge/diverge areas on the mainline. The spacing of the intersections 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. 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. Some variations on the diamond interchange include:

Exhibit 10-3 Compressed Diamond Interchange



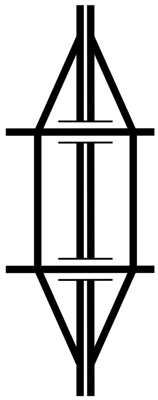
Compressed Diamond Interchange: A typically older interchange design where less than ideal ramp terminal spacing is present, between 400 and 800 feet. Sometimes the two ramp terminals can be operated with a single signal controller. Turn storage is done between the ramp terminals. Queue spillback between the ramp terminals is a common problem.

Exhibit 10-4 Tight Diamond Interchange



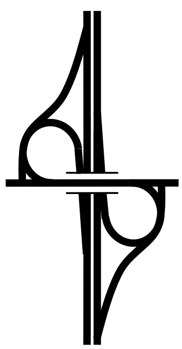
Tight Diamond Interchange: Typically found in urban areas, with ramp terminal spacing less than 400 feet. Usually the two ramp terminals can be operated with a single signal controller. Turn storage is done outside of the ramp terminals.

Exhibit 10-5 Split Diamond Interchange



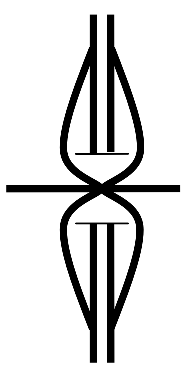
Split Diamond Interchange: Typically found on an urban grid system. Connections between each “half” of the interchange are one-way and are access-controlled.

Exhibit 10-6 Folded Diamond Interchange



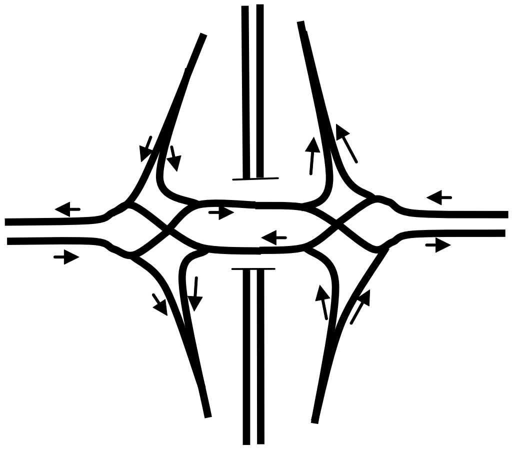
Folded Diamond Interchange: This interchange type “folds” one or two legs of the configuration to minimize impacts in one or two quadrants. Loop ramps can be located where topographical or environmental constraints adjacent to the interchange site do not favor the use of conventional straight ramps, e.g., where a railroad parallels the facility. Loop ramps that are located on the same side of a facility can create weaving sections on the mainline or crossroad that may not be desirable.

Exhibit 10-7 Single Point Urban Interchange



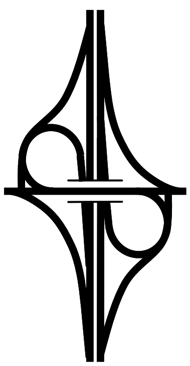
Single Point Urban Interchange (SPUI) also known as Single Point Urban Diamond (SPUD): The SPUI is a relatively recent development that evolved out of the need to limit ROW acquisition in built-up urban areas. SPUIs are a variation of the diamond interchange, which has two ramp terminals with the local 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. This feature 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.

Exhibit 10-8 Divergent Diamond



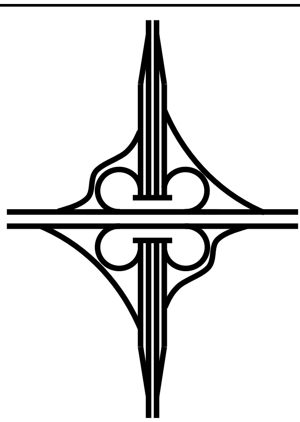
Divergent Diamond Interchange: This is a new type of interchange design that has very few installations in the U.S. This form of diamond interchange has the two directions of minor street traffic cross to the opposite side of the roadway under/over structure. This allows for two-phase signal operations since the left turns occur between the two signals in such a way that they do not cross the opposing through movements.

Exhibit 10-9 Partial Cloverleaf Interchange



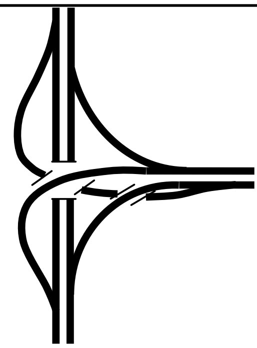
Partial Cloverleaf Interchange: A partial cloverleaf layout combines loop ramps and straight ramps to better serve areas with expected high turning volumes at the ramp terminals. In general, a partial cloverleaf configuration has a higher carrying capacity than a diamond interchange or folded diamond because it has more than four ramps, often providing two on-ramps in each direction as shown in the diagram. The preferred configuration is where loop ramps are located 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 10-10 Full Cloverleaf



Full Cloverleaf: This layout provides loop ramps in all four quadrants of the interchange, requiring a great deal of land area. It is a somewhat outdated design and should typically be used only where loop volumes are low. Loop ramps that are located on the same side of a facility can create weaving sections on the mainline or crossroad that may not be desirable.

Exhibit 10-11 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.

10.4 Identifying Limitations to Design Concepts

The facility design concepts are initially selected based on their ability to meet the needs of future travel demands, but each alternative must further balance those project features against the environmental constraints found at that location. A planning study should provide sufficient preliminary information about a range of environmental and physical 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 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. In general, 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 environmental parlance these are known as "4F" zones and may include historic sites, parks and other recreational properties, schools and cemeteries.
- **Built Environment**: Existing buildings and structures generally should not be disturbed, unless the owner is a willing seller and they can be purchased as a part of the improvement project. This requires consideration of historic buildings, schools, hospitals, parks, large developments, low income areas and environmental justice issues.
- **Right of Way**: 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.
- **Alternative Modes**: Depending on the functional designation of the highway facility and the adjoining land development there may be need to service pedestrians and bicycles in all solutions under consideration. Alternative concepts that create adverse conditions for non-auto travel, in these cases, will not be acceptable.

Based on the review of the above issues, the alternatives considered for evaluation may be modified or dismissed if any of these areas have substantial issues. An example case would be where the preferred operational solution for a freeway interchange indicated that a partial cloverleaf layout was superior, but because of proximity to EFU land the available configuration space was too constrained. The best solution to meet both the performance objectives and the environmental limitations was a tight diamond configuration.

10.5 Documentation of Screening Process

The alternatives analysis for potential improvement projects should be consistent with the established evaluation criteria.

10.5.1 Evaluation Criteria

The screening criteria should be readily assessable, without detailed evaluations. Examples include:

- Meets project purpose and needs.
- Meets project goals and objectives.
- Compliance with access spacing standards.
- Consistency with agency design guidelines.
- Avoid potential environmental impacts?
- Does the project impact adjacent private properties?

A screening matrix should be developed and applied to all the sketch level concepts and those alternatives that clearly do not meet these basic criteria should be dropped from further consideration. Other alternatives should be advanced to the broader assessment of operational performance analysis, project refinement and preliminary cost estimates, as appropriate.

10.5.2 Alternatives No Longer Considered

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. These discarded alternatives should be included in the Alternatives Considered but Dismissed appendix in the narrative report.

10.6 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. The evaluation of Build Alternatives is compared to the No-Build scenario to quantitatively compare relative performance benefits of the various alternatives.

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 is available.

10.6.1 Analysis of Future Conditions

The future conditions analysis should develop quantitative results sufficient to respond to all the selected performance measures for the study. Performance evaluation criteria typically include one or more of the following indicators. Refer to Chapter 5, 6 and 7 for details on how to make these assessments.

- **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 95th percentile queue and compare to storage length.
- **Queue Blocking Percentage**: Generally applied to through 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 indicators Include**: Travel time, total delay and total number of vehicle stops.

The evaluations for each alternative should assess all of the selected performance criteria. The results can be used to quantitatively compare and contrast the outcomes between alternative and No-Build and each of the respective alternatives to determine the best performing solution.

Analysis Assumptions Relative to No-Build Scenario

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.

- **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.

- **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 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 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 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 of the 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.
- **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. The analyst should coordinate with the agency responsible for operating the signals to identify upper and lower cycle limits for functional signal operations. 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.
- **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 demands 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 the example above, 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.

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 heavily congested in the base case, the analyst should verify through field studies, additional surveys or other measurements that the observed conditions are reasonably similar to the computer software results. These procedures were covered in Chapter 7, Intersection Analysis. 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. In this case, the demand is greater than the actual count 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.

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 maximum (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 a microsimulation, which accounts for interaction between locations, queue spillbacks, blocked intersections and serving excessive demand between signal cycles. See details in Chapter 8.

10.6.2 Progression Analysis

11 AIR AND NOISE TRAFFIC DATA

11.1 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. This chapter presents the general outline for the needs and processing of common data requested for the Air and Noise Analysis section of the Environmental Impact Statement (EIS) or Environmental Assessment (EA). Topics covered include:

- Input for Noise Analysis
- Input for Air Quality Analysis
- EISBase

11.2 Input for Noise Analysis

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 Geo-Environmental Section, or noise consultant, requires specific data from the project traffic analyst. This request is typically made through the Noise, Air and Energy Traffic Requirements Check List, which is filled out by the noise consultant or Geo-Environmental Section staff and delivered to the project traffic analyst. While this list identifies many different types of possible data needs, the collection and processing of the most common data requested is discussed below. This process should only be done on the No-Build and Selected Alternatives because of the time required to complete the work. Typically it will take a month for the no-build and a bit less for each build alternative.

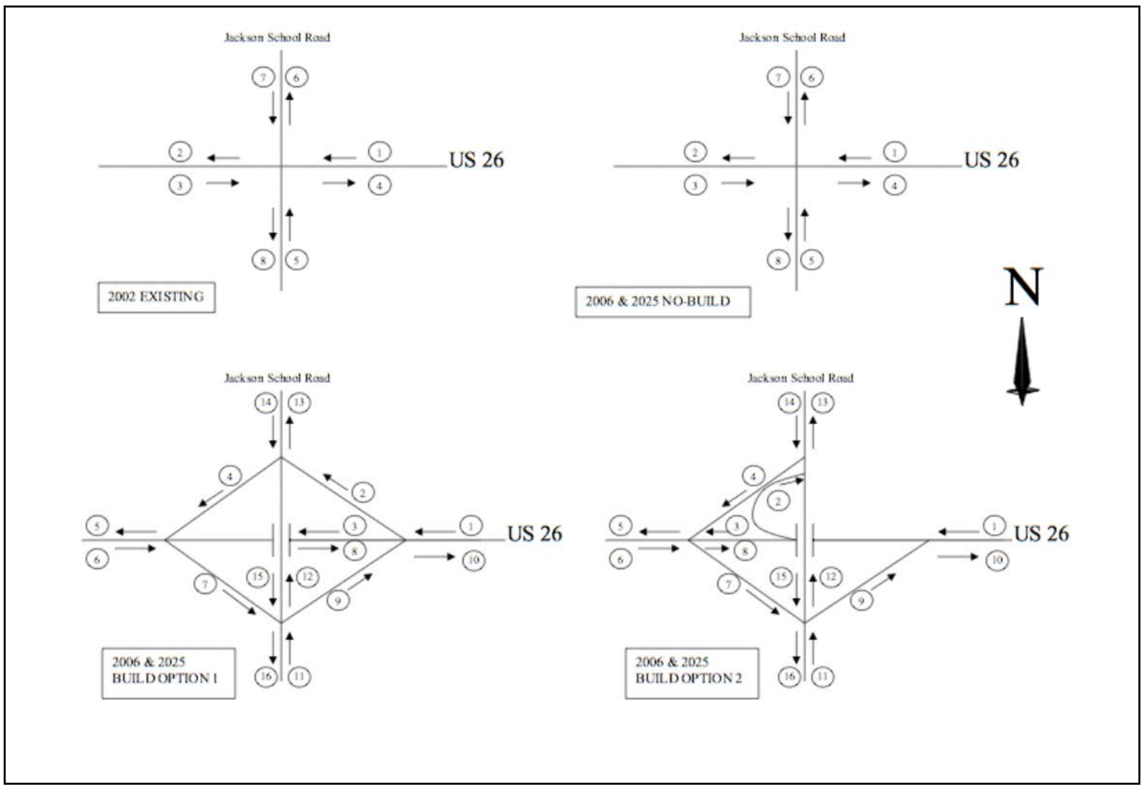
11.2.1 Common Data Needs

The traffic data requested will often be required for a no-build condition under the existing year, the year of opening and the future design year (typically 20 years from opening) as well as for build conditions for each alternative being considered under the year of opening and the future design year.

At the beginning of the project, 16-hour manual full federal (13 vehicle classes) classification turn movement counts need to be ordered at all signalized intersections in addition to all intersections with substantial traffic volumes or heavy vehicle movements. Shorter duration counts can be used at minor intersections between the classification count locations. These counts are also used to develop the traffic volumes for the project. See Chapter 4. The factors that are created from count data are based on the peak hour, the average hour, the average daily traffic, and the peak truck hour, which cannot be calculated from a peak period count. There will need to be enough classification counts so passenger car, medium and heavy vehicle movements can be calculated at the shorter duration count locations. Be sure to request an electronic version of all count data in spreadsheet format to aid in data processing.

The first step in the noise data process should be the creation of link diagrams depicting the study area roadway segments that will be included in the analysis for all no-build and build scenarios considered. 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. Each link should be given a unique number for identification purposes. It should be noted that the link number can be directional. Directional link numbers should be provided for freeways, expressways, interchanges, one-way streets, couplets, divided highways and facilities with separate roadbeds. Where possible, try to keep consistent numbering between the no-build and build link diagrams. This will mean adding extra links that have zero data into the no-build network that will accommodate the build Alternative. The more consistent the diagrams are, the easier it will be for the traffic analyst to troubleshoot and the noise analyst to follow. There is nothing wrong with having links with no data in the scenarios as long as they are labeled as not existing yet or not existing anymore. A set of sample link diagrams is provided in Exhibit 11-1.

Exhibit 11-1 Sample Link Diagram – Jackson School Road Interchange



The second step is recording the specific link characteristics, which include street name, length (in miles), posted speed and LOS C volume. Link length is the center-to-center intersection spacing. Links on the edge of the network should have a length of 0.25 mile. Speeds recorded for this analysis should be either the posted speed limit or the operating speed (highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions, without at any time exceeding the safe speed as determined by the design speed on a section-by-section basis) where it is determined to be consistently higher than the posted speed limit. Directional interim LOS C volumes for each link can be assumed to be as follows:

- For freeway segments, obtain from the latest version of the HCM. For example, in the HCM 2000 the LOS C volume for a basic freeway segment with a free-flow speed of 60 mph would be 1560 pcphpl. The LOS C volume is calculated using the HCM delay methodologies, via an iterative process based on the project volumes, to identify the volume where the LOS C threshold occurs (at the top end of LOS C, adjacent to LOS D). Use the following defaults only for links at the end of the network.
- For freeway ramps, assume 1000 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 700 pcphpl.
- For rural highways, assume 800 pcphpl.

In noise analysis, the LOS C volume is assumed to represent the maximum volume that can be sustained at free-flow speed. 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.

11.2.2 Calculations

When the count data ordered becomes available it will be necessary to regroup the 13 vehicle classes into the medium, heavy and all vehicles categories for the noise analysis. 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 8-feet above the ground. The following list provides information on the types of transportation noise sources that will be part of a typical roadway project, and describes the type of noise each produces[31].

- ***Passenger Vehicles***: Noise emitted from 0 to 2 feet above roadway, primarily from tire-roadway interface. This category includes normal passenger vehicles, small and regular pickup trucks, small to mid-size sport utility vehicles and mini- and full-size passenger vans.
- ***Medium Trucks***: Noise emitted from 2- to 5-feet above roadway, combined noise from tire-roadway interface and engine exhaust noise. This category includes delivery vans (e.g., UPS and Federal Express trucks) large sport utility vehicles

[31] *Traffic Noise Background Information*, Michael Minor & Associates.

with knobby tires, large diesel engine trucks, some tow-trucks, city transit and school buses with under vehicle exhaust, moving vans (U-Haul type trucks), small to medium recreational motor homes, and other larger trucks with the exhaust located under the vehicle. The federal vehicle classifications covered are: 2-axle other with trailer, 2-axle 6-tire single unit and buses. For practical application, include all trucks with 2 axles and 6 tires if insufficient information is available to provide for a more detailed analysis.

- ***Heavy Trucks***: Noise emitted from 6- to 8-feet above the roadway surface, combined noise sources includes tire-roadway interface, engine noise and exhaust stack noise. This category includes all long-haul tractor-trailers (semi-trucks), large tow-trucks, dump trucks, cement mixers, large transit buses, motor homes with exhaust located at top of vehicle, and other vehicles with the exhaust located above the vehicle (typical exhaust height of 12- to 15-feet). The federal vehicle classifications covered are: 3-axle and greater single units and all combinations. For practical application, include all trucks with 3 or more axles.

NOTE: 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 following are the federal classifications.

- ***Motorcycles (Optional)***: 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, motor scooters, mopeds, motor-powered bicycles and three-wheel motorcycles. This vehicle type may be reported at the option of the State.
- ***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.
- ***Other Two-Axle, Four-Tire Single Unit Vehicles***: All two-axle, four-tire vehicles, other than passenger cars. Included in this classification are pickups, vans and other vehicles such as campers, motor homes, ambulances, hearses, carryalls and minibuses. Other two-axle, four-tire single-unit vehicles pulling recreational or other light trailers are included in this classification. Because automatic vehicle classifiers have difficulty distinguishing class 3 from class 2, these two classes may be combined into class 2.
- ***Buses (Optional)***: 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 to be a truck and should be appropriately classified.

- ***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.
- ***Three-Axle Single-Unit Trucks***: All vehicles on a single frame including trucks camping and recreational vehicles, motor homes, etc., with three axles.
- ***Four or More Axle Single-Unit Trucks***: All trucks on a single frame with four or more axles.
- ***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.
- ***Five-Axle Single-Trailer Trucks***: All five-axle vehicles consisting of two units, one of which is a tractor or straight truck power unit.
- ***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.
- ***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.
- ***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.
- ***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.

There may be times when data related to buses and motorcycles is requested to be provided separately. 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.

The count data can be regrouped by hand or by spreadsheet, however the process can be long and cumbersome. An easy way to process ODOT-counted 12-hour or greater counts is to request the count in electronic form in "TruckSum" format from the Transportation Systems Monitoring Unit. A copy of the spreadsheet is available on the TPAU website. This format organizes the count into the three basic subgroups of medium, heavy and all vehicles for the macro-based TruckSum Excel spreadsheet. See Exhibit 11-2 and Exhibit 11-3, which calculates the initial truck factors. Note: The Exhibit margins were cut off due to a software issue.

The TruckSum spreadsheet calculates for each intersection leg the peak hour volume, the average daily traffic, the average daily truck volume, the average 8-hour volume and the peak truck hour volume. The spreadsheet also calculates the necessary truck factors explained later in this section. The analyst either types the summarized count values into the spreadsheet or copies and pastes the TruckSum-formatted count data into the spreadsheet. Use Paste – Special Values only, to avoid corrupting the output. The analyst also enters the length of the manual count (12 hours minimum), count date and location. Pressing the "Calculate" button on the spreadsheet will generate the factors and the rest of the data (page two of the output). It is best to print out the spreadsheet in landscape format double sided. Note: A side benefit of this spreadsheet, if used early in the project, is that it can be used to help determine the overall peak hour of the study area as will as determining directional factors, K-factors (percent of daily traffic in the peak hour) and truck percentages for each leg.

Exhibit 11-2 TruckSum Input

Summary of Manual Count 14 hour
Count Date: Oct 12 04
Location: US199 at Dowell

Analyst: Dorothy J Upton

time	type	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 LEG	EAST LEG	SOUTH LEG	WEST LEG
6 to 7	Medium Truck	1	0	1	0	1	14	1	2	0	0	6	0	26	3	24	4	21
	Heavy Truck	5	0	0	1	1	15	0	2	0	0	15	0	39	6	39	3	30
	All Vehicles	23	3	6	29	17	212	7	42	2	13	458	1	813	81	781	72	692
7 to 8	Medium Truck	9	4	4	2	1	20	0	4	0	4	13	0	61	23	49	9	41
	Heavy Truck	5	0	1	4	1	15	1	2	0	3	26	0	58	14	53	4	45
	All Vehicles	80	25	35	48	28	565	13	54	11	34	743	5	1641	235	1518	136	1393
8 to 9	Medium Truck	8	1	5	3	2	24	1	2	3	4	18	0	71	22	57	9	54
	Heavy Truck	8	0	1	2	7	25	1	5	0	3	27	0	79	15	74	13	56
	All Vehicles	102	12	45	59	33	512	16	46	11	55	706	6	1603	289	1458	124	1335
9 to 10	Medium Truck	2	0	6	6	0	26	1	2	0	5	15	0	63	20	51	3	52
	Heavy Truck	5	0	0	2	4	23	0	1	1	2	23	0	61	9	58	6	49
	All Vehicles	83	11	46	79	30	595	16	58	6	44	685	6	1659	279	1530	127	1382
10 to 11	Medium Truck	2	2	3	2	0	20	2	0	0	3	14	0	48	14	38	4	40
	Heavy Truck	2	1	5	10	1	27	0	6	0	2	21	0	75	20	67	8	55
	All Vehicles	86	15	47	83	43	562	10	61	4	45	657	3	1616	286	1492	136	1318
11 to 12	Medium Truck	8	2	3	5	2	23	1	1	5	4	12	1	67	23	51	12	48
	Heavy Truck	1	0	0	2	1	17	1	1	0	1	21	0	45	5	43	3	39
	All Vehicles	89	26	43	84	56	551	24	58	14	39	733	7	1724	305	1571	185	1387
12 to 1	Medium Truck	5	0	4	0	6	17	1	1	0	2	23	1	60	12	52	9	47
	Heavy Truck	9	0	1	5	1	25	0	3	0	1	26	0	71	16	69	4	53
	All Vehicles	103	16	46	75	68	737	17	68	9	57	762	6	1964	314	1813	184	1617
1 to 2	Medium Truck	2	1	5	11	4	20	1	4	2	3	23	0	76	23	64	12	53
	Heavy Truck	3	0	2	5	2	25	0	2	0	2	31	0	72	12	68	4	60
	All Vehicles	88	20	49	74	54	694	20	48	9	49	693	3	1801	300	1651	154	1497
2 to 3	Medium Truck	7	3	3	9	0	26	2	3	1	4	25	2	85	28	70	11	61
	Heavy Truck	4	0	3	3	4	27	0	1	0	1	28	0	71	11	67	5	59
	All Vehicles	120	18	53	80	58	793	13	61	8	50	821	9	2084	334	1933	167	1734
3 to 4	Medium Truck	6	2	7	5	2	23	2	2	1	3	15	0	68	25	53	9	49
	Heavy Truck	3	0	1	2	6	24	0	6	0	0	21	0	63	6	62	12	46
	All Vehicles	115	17	56	71	65	862	19	61	7	42	662	6	1983	320	1836	175	1635
4 to 5	Medium Truck	4	2	4	9	7	13	0	6	2	2	24	0	73	21	63	17	45
	Heavy Truck	1	0	1	2	2	19	0	1	0	2	32	0	60	6	57	3	54
	All Vehicles	103	24	69	90	87	932	12	73	12	36	717	8	2163	334	2002	216	1774
5 to 6	Medium Truck	0	1	1	0	2	14	0	0	0	1	10	0	29	3	26	3	26
	Heavy Truck	1	1	0	2	1	23	0	0	0	0	12	0	40	4	39	2	35
	All Vehicles	66	18	48	42	78	968	14	42	8	22	548	1	1855	210	1744	161	1595
6 to 7	Medium Truck	0	0	0	0	0	5	1	0	0	0	6	0	12	1	11	1	11
	Heavy Truck	0	0	0	1	1	9	0	1	0	0	9	0	21	1	21	2	18
	All Vehicles	35	19	22	24	51	713	8	37	7	19	504	3	1442	127	1364	125	1268
7 to 8	Medium Truck	0	0	0	0	0	1	0	0	0	0	8	0	9	0	9	0	9
	Heavy Truck	1	0	0	0	1	13	0	0	0	0	7	0	22	1	22	1	20
	All Vehicles	23	3	8	18	31	504	2	24	10	14	316	3	956	68	916	73	855
8 to 9	Medium Truck	0	0	0	0	0	3	0	0	0	0	3	1	7	0	6	1	7
	Heavy Truck	0	0	0	0	1	10	0	2	0	0	8	0	21	0	21	3	18
	All Vehicles	12	10	10	10	20	337	6	22	5	6	195	2	635	54	596	65	555
9 to 10	Medium Truck	0	0	0	0	0	3	0	0	0	0	6	0	9	0	9	0	9
	Heavy Truck	0	0	0	0	0	1	0	0	0	0	4	0	5	0	5	0	5
	All Vehicles	9	7	12	3	20	236	4	8	0	3	173	1	476	38	449	40	425
TOTAL	Medium Truck	54	18	46	52	27	252	13	27	14	35	221	5	764				
TOTAL	Heavy Truck	48	2	15	41	34	298	3	33	1	17	311	0	803				
TOTAL	All Vehicles	1137	244	595	869	739	9773	201	763	123	528	9373	70	24415				

Exhibit 11-3 TruckSum Output

Summary of Manual Count 14 hour
Count Date: Oct 12 04
Location: US199 at Dowell

Peak Hour is:
4 to 5 PM

HIGHEST 8 HOUR VOLUME FROM :		10 to 6	PM		
type	TOTAL	NORTH LEG	EAST LEG	SOUTH LEG	WEST LEG
Medium	506	149	417	77	369
Heavy	497	80	472	41	401
All Veh	15190	2403	14042	1378	12557

All vehicle ADT Factor 1.18

NORTH LEG		EAST LEG		SOUTH LEG		WEST LEG	
	volume		volume		volume		volume
NB	1886	EB	13302	SB	1243	WB	12379
SB	2332	WB	13430	NB	1283	EB	11766
TOTAL	4217	TOTAL	26732	TOTAL	2525	TOTAL	24145

All Vehicle PHV

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG		
	volume	factor		volume	factor		volume	factor		volume	factor
NB	138	0.073	EB	893	0.067	SB	119	0.096	WB	1013	0.082
SB	196	0.084	WB	1109	0.083	NB	97	0.076	EB	761	0.065
TOTAL	334	0.079	TOTAL	2002	0.075	TOTAL	216	0.086	TOTAL	1774	0.073

Truck ADT by leg * All factors apply to ADT volumes.

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG		
	volume	factor *		volume	factor *		volume	factor *		volume	factor *
Medium	218	0.052	Medium	633	0.024	Medium	104	0.041	Medium	573	0.024
Heavy	126	0.030	Heavy	765	0.029	Heavy	73	0.029	Heavy	642	0.027
Total	344	0.082	Total	1398	0.052	Total	177	0.070	Total	1215	0.050

TRUCK FACTOR

PHV Trucks ONLY *All factors apply to PHV for Trucks.

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG		
	volume	factor *		volume	factor *		volume	factor *		volume	factor *
Medium	21	0.063	Medium	63	0.031	Medium	17	0.079	Medium	45	0.026
Heavy	6	0.018	Heavy	57	0.028	Heavy	3	0.014	Heavy	54	0.030

Peak Hour Factor MEDIUM TRUCK
Peak Hour Factor HEAVY TRUCK

Peak 8 hour Avg.

If the All Vehicle factors are greater than 1.000, then use the corrected values shown. Apply the values as noted .

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG		
	volume	factor *		volume	factor *		volume	factor *		volume	factor *
All Veh	300	0.899	All Veh	1755	0.877	All Veh	172	0.797	All Veh	1570	0.886
All Trucks	29	0.096	All Trucks	111	0.063	All Trucks	15	0.086	All Trucks	96	0.061
All Veh	300	0.899	All Veh	1755	0.877	All Veh	172	0.797	All Veh	1570	0.886
All Trucks	29	0.095	All Trucks	111	0.063	All Trucks	15	0.086	All Trucks	96	0.061

Average Hour Factor ALL VEHICLES
Average Hour Factor TRUCKS

Peak Truck Hours By Leg
* Factors apply to Peak Hour Volumes
** Factors apply to Peak Truck Hour Volumes

If the All Vehicle factors are greater than 1.000, then use the corrected values shown. Apply the values as noted .

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG			
	volume	factor		volume	factor		volume	factor		volume	factor	
All Veh	334	1.000	All Veh	1933	0.966	All Veh	124	0.574	All Veh	1734	0.977	**
Medium	28	0.084	Medium	70	0.036	Medium	9	0.073	Medium	61	0.035	*
Heavy	11	0.033	Heavy	67	0.035	Heavy	13	0.105	Heavy	59	0.034	*
All Veh	334	[illegible]	All Veh	1933	0.966	All Veh	124	0.574	All Veh	1734	0.977	
Medium	28	[illegible]	Medium	70	0.036	Medium	9	0.073	Medium	61	0.036	
Heavy	11	[illegible]	Heavy	67	0.035	Heavy	13	0.105	Heavy	59	0.034	

Peak Truck Hour Factor ALL VEHICLES
Peak Truck Hour Factor MEDIUM
Peak Truck Hour Factor HEAVY

Analyst: Dorothy J Upton

Average daily traffic (ADT) volumes and peak hour volumes (PHV) maintain the same meaning in noise analysis as they do in other types of analysis described in this manual. They are simply the total number of all vehicle types experienced on a link over a 24-hour period and the highest hourly total of all vehicle types experienced during that 24-hour period, respectively. Both of these must be documented for each link studied during all years and analysis scenarios requested.

The "Truck Factor" represents the percent trucks in the average daily traffic volume, calculated as shown below.

$$\textit{Truck Factor} = \frac{(24\textit{-hour volume of all trucks})}{(24\textit{-hour volume of all vehicles})}$$

The "Peak Hour Factor" for noise analysis, refers to the percent trucks in the peak hour of traffic for all vehicle types. This factor is usually needed for both medium and heavy vehicles separately and is calculated as shown below.

$$\text{Peak Hour Factor, }_{\text{Medium Trucks}} = \frac{(\text{medium trucks in peak hour of all vehicles})}{(\text{all vehicles in peak hour of all vehicles})}$$

$$\text{Peak Hour Factor, }_{\text{Heavy Trucks}} = \frac{(\text{heavy trucks in peak hour of all vehicles})}{(\text{all vehicles in peak hour of all vehicles})}$$

Peak Truck Hour Factors refer to the percent of a specified vehicle type in the peak hour of all truck traffic. Most times the peak truck hour is different from the peak hour. These factors are typically calculated for all vehicle, medium truck and heavy truck classes, but the calculations for all vehicles is different than those for the truck classes. As an example, the peak truck hour factors for various classes are shown below.

$$\text{Peak Truck Hour Factor, }_{\text{All Vehicles}} = \frac{(\text{all vehicles in all truck peak hour})}{(\text{all vehicles in peak hour of all vehicles})}$$

$$\text{Peak Truck Hour Factor, }_{\text{Medium Trucks}} = \frac{(\text{medium trucks in all truck peak hour})}{(\text{all vehicles in all truck peak hour})}$$

$$\text{Peak Truck Hour Factor, }_{\text{Heavy Trucks}} = \frac{(\text{heavy trucks in all truck peak hour})}{(\text{all vehicles in all truck peak hour})}$$

Another type of factor that may be requested, though not often used, is the "Average Hour Factor." This represents the percent of a specified vehicle type in an average 8-hour period and is commonly requested for the all vehicle types and all truck classes.

When bus or motorcycle traffic is requested separately, provide percentages of these vehicle types in the all vehicle peak hour and truck peak hour. For example, if providing bus data, the following calculations would be used:

$$\text{\% Buses in All Vehicle Peak Hour} = \frac{(\text{buses in peak hour of all vehicles})}{(\text{all vehicles in peak hour of all vehicles})}$$

$$\text{\% Buses in Truck Peak Hour} = \frac{(\text{buses in all truck peak hour})}{(\text{all vehicles in all truck peak hour})}$$

Note that while updated ADT and PHV values for each analysis year must be provided, only one set of TruckSum factors must be calculated for each alternative. It is generally assumed that while traffic volumes will increase over time, the proportion of vehicle types in the total volume will remain approximately the same.

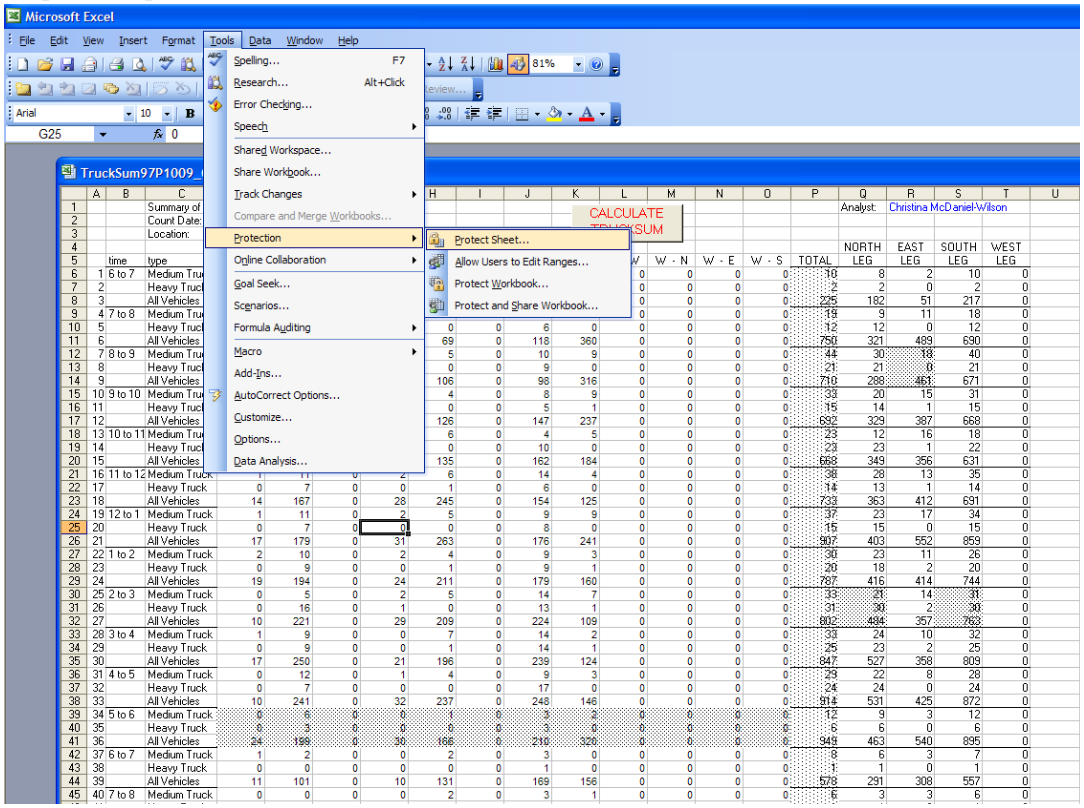
The factors in the TruckSum spreadsheet are for the indicated peak hour. Choose the peak hour that covers the most of the intersections. If the peak hour is different than the chosen hour, then the factors will need to be recalculated using the volumes for that hour. If buses are to be split out separately, then they will have to be done manually by splitting them from the medium trucks.
The factors shown in boxes on the right side of the spreadsheet are described below with the related equations.

Project wide peak hour adjustment using TruckSum

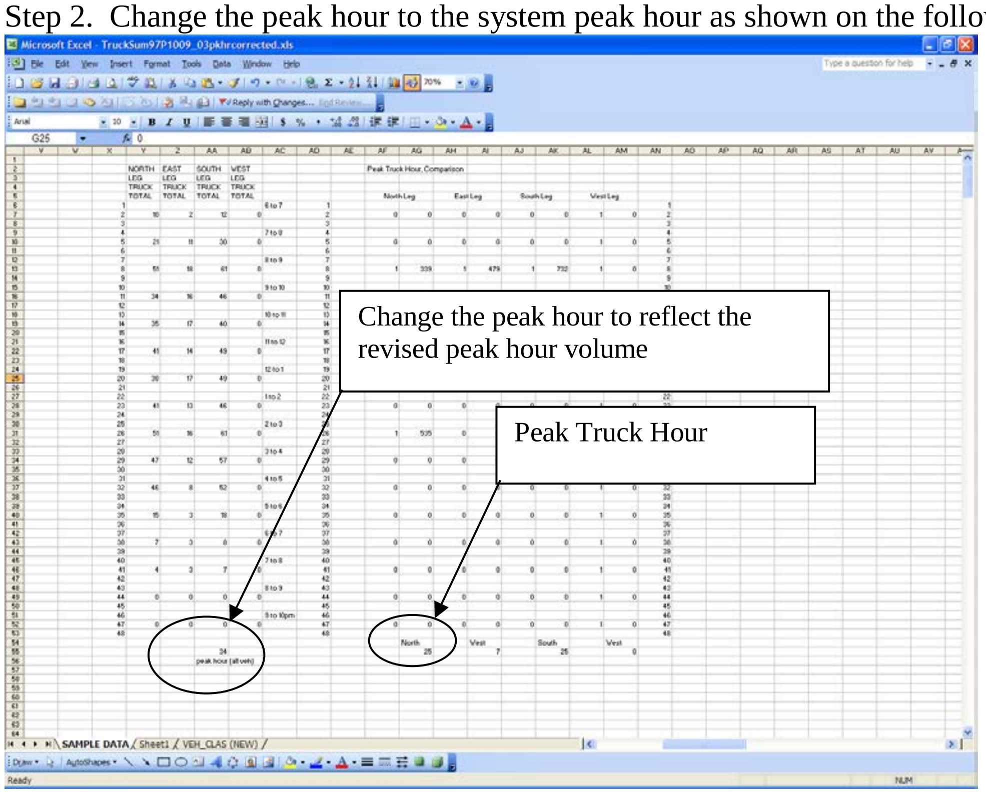
Often, different intersections on a project will have different peak hours. The peak hour calculated by TruckSum is the peak hour for each individual intersection. For those intersections where the peak hour is different from the system peak hour, the analyst can modify TruckSum to report the system peak hour, using the following procedure.

TruckSum factors are calculated using the VLOOKUP function in the EXCEL spreadsheet. To calculate factors for a system peak instead of the intersection peak, the peak hour needs to be changed in the spreadsheet. Over-ride the VLOOKUP function by manually typing in the system peak hour in the peak hour cell. The spreadsheet will no longer choose the peak hour from the table and instead will use the selected hour.

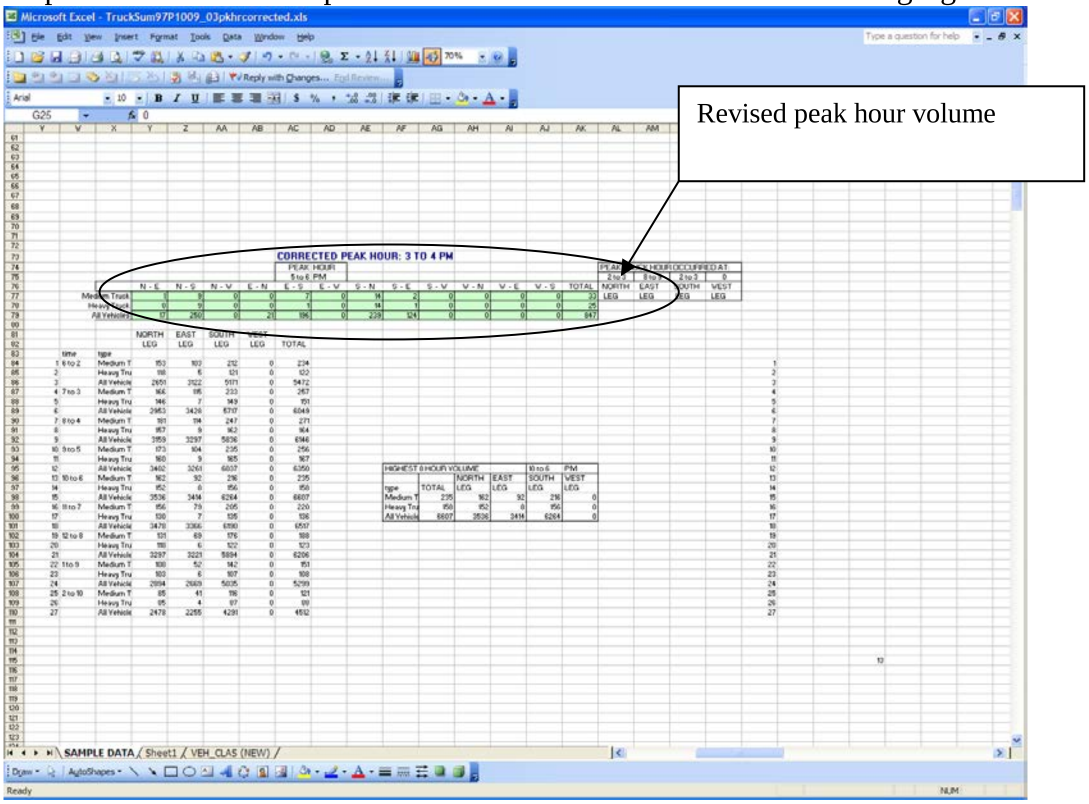
Step 1. Unprotect the worksheet as shown below.

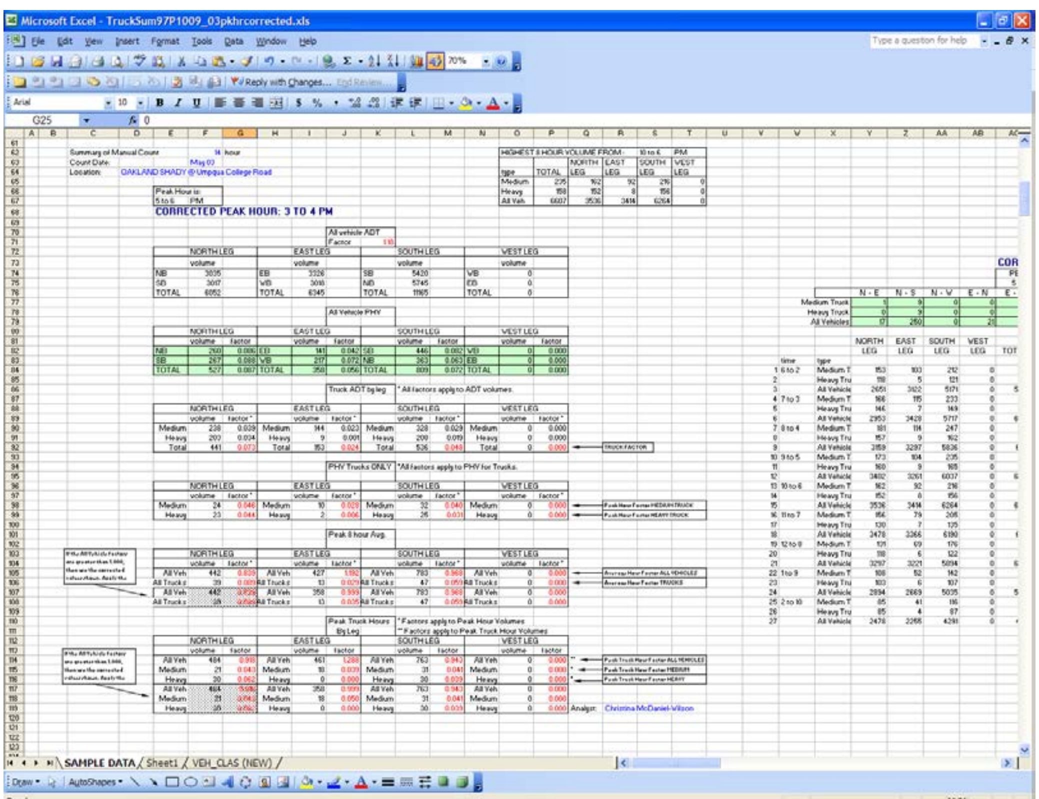


Step 2. Change the peak hour to the system peak hour as shown on the following figure.



Step 3. Observe revised peak hour volumes as shown on the following figures.





11.2.3 Process

The following procedure is suggested. Draw the no-build link diagram on a large blank piece of paper (11"x17") and put left, through and right arrows at each intersection approach. Take this master sheet and make about 10 copies. Starting with the "peak hour factor medium truck," write the factor value on each link at the midpoint. Average the factors between adjacent intersections. Multiply this factor times the peak hour volumes for each intersection approach (including the turning movements) for the entire study network. This will generate an initial set of medium truck peak hour volumes. Check and balance the medium trucks over the network. Once the network is balanced, re-compute the final medium truck factor for each link. These steps are necessary to help avoid errors (such as the average hour volume is greater than the peak hour volume) that may come up when the EISBase database program is used to enter all the data. These errors are very hard to track down unless there is this documentation on the factor development. Repeat for the heavy truck peak hour factor.

The average hour for all vehicles factor and the peak truck hour all vehicles factor modify the peak hour volumes. Create the average hour and peak truck hour volumes and the related factors using the above procedure. Use the average hour or peak truck hour volumes to distribute, balance and compute the factors for the average hour truck factor and the medium and heavy peak truck hour factors.

The link ADT should be computed by dividing the directional peak hour volumes by the directional K-factor (these are the factors listed in the "All Vehicle PHV" section of the TruckSum spreadsheet) for each intersection leg. Average the K-factors between adjacent intersections. Sum the directional average daily traffic to get the link average daily traffic. If the directional ADT is desired, multiply the total ADT by the directional split (calculated from the "All Vehicle ADT Factor" section). Average the directional factors between adjacent intersections as well. The average daily traffic should be consistent across the links.

11.3 Input for Air Quality Analysis

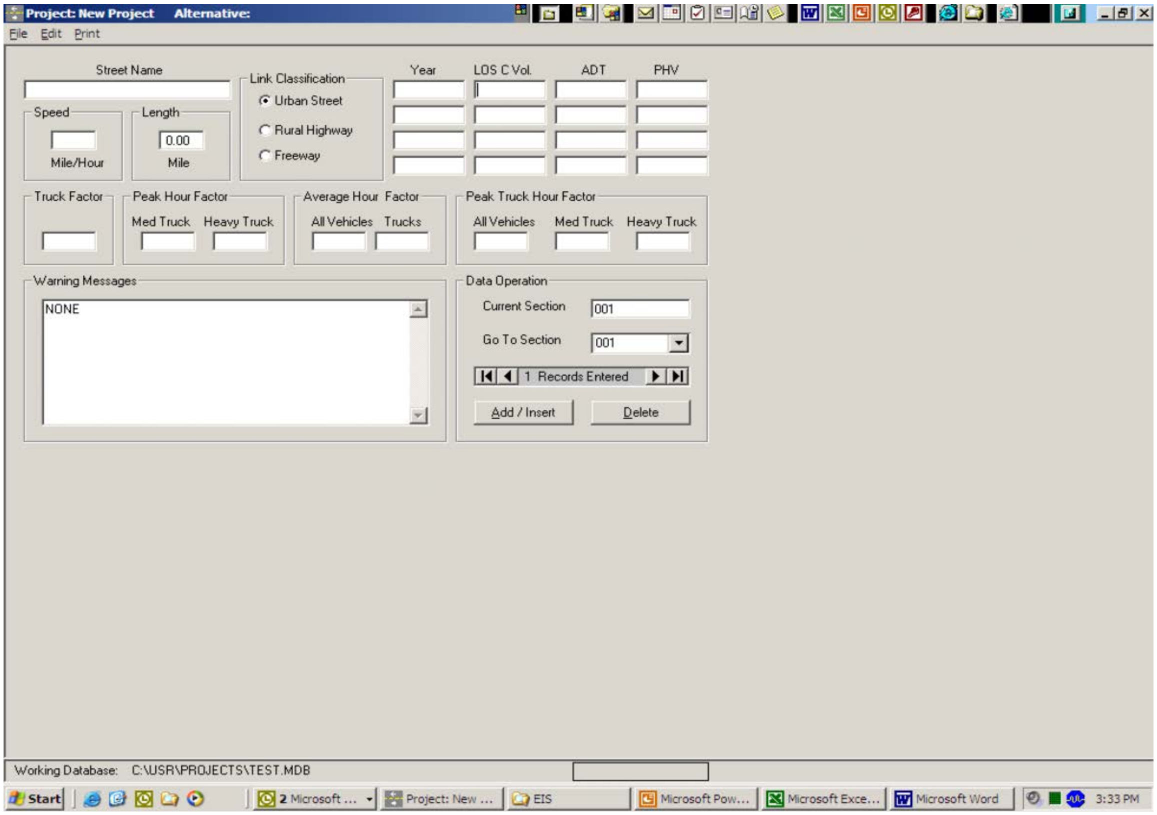
Note: This is an interim section, to be expanded.

Similar to 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 Geo-Environmental Section, or air quality consultant, requires specific data from the project traffic analyst. This request is typically made through the Noise, Air and Energy Traffic Requirements Check List, which is filled out by the air quality consultant or Geo-Environmental Section staff and delivered to the project traffic analyst.

11.4 EISBase

The EISBase software program is used by ODOT to produce final link volumes and speeds for all analysis scenarios, given the previously described data and factors as input. This represents the finished data needed by the Geo-Environmental staff or noise consultant. Exhibit 11-4 provides an image of an EISBase input screen prior to data entry. As shown in the figure, all of the data required for entry into the input windows has been described in this chapter. No decimal points are required allowing data entry to be sped up, however, data related to buses and motorcycles must be processed separately.

Exhibit 11-4 EISBase Input Screen (Replacement pending.)



While EISBase will calculate future link speeds, there may be times when manual adjustment is required. These instances will typically occur where a future plan shows the classification or use of a link to change in such a manner that the projected speed would no longer apply. Engineering judgment should be used to determine the appropriate link speed when altered manually.

It should be noted when using EISBase, any “All Vehicle Factors” under the “Peak Truck Hour Factor” category that are equal or greater to 1.000 (indicating that the peak truck hour volume and all vehicle peak hour volume are the same) must be input as 0.999 before saving the file. This adjustment is necessary due to a rounding error in the program.

In some cases special categories of vehicles are needed (additional refinement of vehicle types is desired). This is common for buses (such as where a transit mall is present) and passenger cars. For example, if buses are to be split out separately from the all vehicle category, compute all the factors as normal (i.e., heavy, medium and all vehicles) and generate a set of finalized and error-checked values before splitting the buses from the medium trucks. The analyst will have to return to the original manual count sheets to compute the number of buses. Split out the buses from the medium trucks on the factor sheets. Buses plus the remaining medium trucks should equal exactly (this is to avoid creating more errors) to the original medium trucks. The bus data should be inserted into the exported spreadsheet by adding buses peak hour and peak truck hour columns. Adjust the medium trucks downward to accommodate the buses. If passenger cars or motorcycles are desired, then the process is similar, but with the "All Vehicles" category.

After all data has been entered for the given scenarios, check the generated error report (click on "Print => Errors"). The error report checks for consistency between the different link factors and between the different scenarios. For example, the number of trucks in the peak truck hour, for a given link should be greater than the number of trucks in the peak or average hour or the future year ADT should be greater than the existing year ADT. One line is generated for each error. All errors, except for the "LOS C Volumes Exceeded by X%," need to be fixed. Many times the error is caused by rounding either within the program or by the analyst and may only be one or two vehicles different. In all the fixable errors, the analyst will have to go back to the individual volume sheets and do any adjustments to the volumes and the resulting factors.

The TruckSum error check only checks for errors on the link itself for a given year. The analyst should check for errors on the system and between years. It is the reviewer's responsibility to check for reasonability, e.g., the future year should be higher than intermediate years on all links. Also volumes should balance across links.

Example 11-1

Example pending.

11.4.1 Output and Final Product

When all of the data for the applicable analysis scenarios has been entered for each link, all errors have been fixed and the file has been saved, export the results from EISBase into a format that can be converted into a spreadsheet. Providing the analysis results in spreadsheet format facilitates the use of this information by the Geo-Environmental staff or noise consultant. This can be done by clicking the briefcase icon to export the data and selecting the "tab-separated text" as the desired format (.ttx extension). This file can then be opened in Microsoft Excel and adjustments to rows and columns, in addition to any manual data adjustments as described above, can be made, as needed. An example of the finished output that is ready for submittal is shown in Exhibit 11-5. It may be beneficial to ask the requesting noise analyst for preferences in data arrangement to facilitate their intended use (e.g., volumes in rows, vehicle class in columns).

The final submittal from the traffic analyst should include:

- Cover letter explaining contents of enclosures.
- Link diagrams.
- Spreadsheets containing traffic analysis output.
- The error report should be printed for the file only.

Exhibit 11-5 Traffic Analysis Output for Noise Analysis (Replacement pending.)

REGION 1 TRAFFIC ANALYSIS UNIT
EIS TRAFFIC DATA

PROJECT: Jackson School Road Interchange
LOCATION: Southeast of North Plains
ALTERNATIVE: Build Option 1 (with 80-acre UGB Expansion Fully Developed)

PAGE: 1 of 1
PRINTING DATE: November 27, 2002
UNIT: English

			PEAK HOUR					PEAK TRUCK HOUR				
SECT	DIST	YEAR	VOL	AUTO	MTR	HTR	SP	VOL	AUTO	MTR	HTR	SP
US 26 WB east of JSR 1	0.25	2006	1170	1069	39	62	55	1000	828	28	144	55
WB Off-Ramp 2	0.46	2006	370	338	12	20	45	320	265	9	46	45
US 26 WB between ramps 3	0.87	2006	800	717	30	53	55	730	589	22	119	55
WB On-Ramp 4	0.45	2006	40	36	1	3	45	40	32	1	7	45
US 26 WB west of JSR 5	0.25	2006	840	754	31	55	55	770	621	23	126	55
US 26 EB west of JSR 6	0.25	2006	1760	1670	39	51	55	850	748	25	77	55
EB Off-Ramp 7	0.45	2006	90	85	2	3	45	40	35	1	4	45
US 26 EB between ramps 8	0.87	2006	1670	1585	37	48	55	810	713	23	74	55
EB On-Ramp 9	0.46	2006	510	486	11	13	45	240	213	7	20	45
US 26 EB east of JSR 10	0.25	2006	2190	2087	46	57	55	1040	921	31	88	55
JSR NB south of US 26 11	0.5	2006	420	415	3	2	55	180	172	5	3	55
JSR NB overpass 12	0.21	2006	60	60	0	0	55	30	28	1	1	55
JSR NB north of US 26 13	0.38	2006	150	148	1	1	55	150	132	6	12	55
JSR SB north of US 26 14	0.38	2006	210	197	12	1	55	210	176	1	33	55
JSR SB overpass 15	0.21	2006	450	442	6	2	55	270	262	2	6	55
JSR SB south of US 26 16	0.5	2006	390	383	5	2	55	230	223	2	5	55

ABBREVIATION: SECT = SECTION NUMBER; VOL = TOTAL VOLUME; MTR = MEDIUM TRUCK VOLUME; SP = SPEED OF VEHICLE; AUTO = AUTOMOBILE VOLUME; HTR = HEAVY TRUCK VOLUME

ANALYST: ANN L. LIST
CHECKED BY: E. N. GINEER

12 TRAFFIC ANALYSIS REPORTS

12.1 <u>Purpose</u>

Traffic analysis reports are a comprehensive explanation of the final recommendations and the decision making process for a project. This chapter presents an overview of the elements that document the assumptions, methods, findings and recommendations of a traffic analyses. Topics covered include:

- Background
- Technical Memorandum
- Traffic Narrative Report

12.2 <u>Background</u>

This chapter presents an overview of the elements that document the assumptions, methods, findings and recommendations of a traffic analyses. In many cases the narrative 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 Narrative at the end stages of the study. The Final Traffic Narrative serves as the legacy document for the study, and should be comprehensive enough to explain and support the final recommendations and decision-making process that led up to it.

12.2.1 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 narrative 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 report include the following:

- **Target Audience**: The intended audience for the report 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, the majority of 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 should be clearly explained with a minimum of technical detail necessary to support and explain the narrative. This is very important because writing at the wrong level can generate unintended questions. More extensive technical calculations, findings and other reference materials should be attached to the document as appendices.

In most cases a document could be 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 narrative. Presentations to the CAC groups 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 narrative, in all cases, remain objective, impartial and impersonal so that the findings and recommendations are untainted by any biases. It should be recognized that any internal ODOT document might be released for public review outside of the designated committee groups. This could occur by informal sharing in the interest of coordination or, more formally, through a legal search warrant. All report narrative documents should be treated as if the general public and press will review them, even though many only circulate to the immediate committee members. No matter the purpose or scope of the report, it is vital to maintain a clear and objective style without introducing biases into a traffic report. To be clear that any recommendations are those of the author, not necessarily of ODOT, it is preferred to use the phrase, "It is recommended that . . ."
- **Readability and Document Structure**: The following sections of this chapter have suggestions about the narrative general layout of the document, but these should 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 narratives, summary tables and illustrations that are grouped according to key topics. In a report, 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 Glossary of this manual for reference purposes. However, care should be taken when developing the report narrative to limit the use of acronyms, except for the most common ones, that appear repeatedly throughout a particular document. The most common examples might include: ODOT, v/c ratio, OHP and HDM. Excessive use of acronyms can quickly degrade the readability of the narrative, 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 in the report as a quick reference guide.

12.2.2 Diagrams and Illustrations

Technical diagrams can be a powerful resource for quickly explaining report assumptions, findings and recommendations. One measure of a high quality report would allow a reader to scan through the study tables and figures, and then be able to glean the general conclusions without reading any of the narrative. For the purposes of traffic study 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.
- Trip patterns or trip distribution routes.
- Lane diagrams of existing or proposed intersection approaches.
- Existing or proposed circulation routes within the study area.
- Existing and proposed street or ramp centerline alignments.
- Alternative street improvement scenarios.
- Preferred street improvement scenario.
- Land use and zoning maps.

The best report graphics clearly label key reference streets, maintain a reasonable minimum 8-point font size, and avoid trying to illustrate many layers of new information at one time. 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 would be 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 should be illustrated on separate diagrams.

All documents need to be legible and usable in black and white.

12.2.3 Tables

Summary tables should be included for ease in making comparisons among scenarios and alternatives. Failing conditions should be denoted with white text on a black background. The preferred software to build tables is MS Word as opposed to MS Excel, due to formatting issues, although MS Excel may be acceptable for appendices.

12.3 Technical Memorandum

12.3.1 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.

12.3.2 Products

The focus of a technical memorandum can vary widely, but, in general, they include the following technical materials, in a typical 3-stage study development process.

TM #1 - Existing/No-Build System Analysis: This memo presents the key system inventory features and performance deficiencies that will shape development of study alternatives. The memo should include statements on the project purpose and need, study area background, and existing and future volume development. Discussed results should include the crash analysis and possible countermeasures, preliminary signal warrants, access or spacing issues, the volume-to-capacity ratios and LOS, if appropriate, and the 95th percentile queues.

TM #2 - Preliminary Alternatives Screening: This memo presents the screening criteria, the initial roster of project alternatives and the scoring of how well the preliminary alternative matched up with the screening criteria. Screening criteria are more general indicators of performance. This could include performance analyses, or these could be deferred until the next stage. Screening performance results typically include Level of Service results, volume-to-capacity ratio results and model-based results (travel times, speeds, v/c ratios, relative comparisons). Remember to keep track of the reasons why alternatives were dropped (will be needed for the narrative).

TM #3 - Future Alternatives Analysis: This memo presents the detailed evaluations of all alternatives that progressed through the screening process. These alternatives have full performance assessments and any other related evaluations (preliminary environmental, compliance with standards, etc.) as defined in the study criteria. Detailed performance results typically include Level of Service results, volume-to-capacity ratio results, 95th percentile queues, storage lengths required and simulation results.

For consultants doing ODOT analysis work, all input and output sheets shall be included with all technical memos and narratives.

12.3.3 Distribution

The technical memorandums should be distributed to the PT and the CAC for review and comment. The Region Traffic Manager should be added to the distribution list where he/she is not a member of the PT.

12.4 Traffic Narrative Report

12.4.1 Purpose

The majority of the traffic study analysis will be completed by the point that the Draft Traffic Narrative Report is developed. The purpose of this report is to present the final solution selected from the study alternatives.

12.4.2 Product

The Draft Traffic Narrative Report should present the full study process and outcomes, include the interim Technical Memorandums and any feedback from work team committees or other ODOT units that reviewed and commented on this effort. The major step to be completed with the Draft Traffic Narrative Report is to provide conclusions on the function of alternatives from a traffic analysis standpoint.

The project team selection process for a preferred alternative overall uses the analytical evaluation outcomes, relative scoring evaluations to isolate one alternative, or a hybrid of several alternatives that best meet the study objectives. This is necessarily a collaborative process with established Project Management Team members and affected ODOT technical units.

The report itself should be developed consistent with the following standard outline. Example Narratives have been provided in the APM Appendices.

Sample Outline

- **Cover Sheet**
  - Agency/Company Title, Division, Unit, City, State (in header, footer or along bound edge)
  - "Project Title Traffic Analysis" (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" (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, scope of alternatives, re-statement of conclusions and alternative recommendation.
- **Background Information**: Overview of study area including vicinity and study area maps, affected facilities and jurisdictions, past project or planning decisions that could influence outcomes, general problem statement and objectives for the study.
- **Existing Conditions**: Inventory and analysis of base year facility and operating conditions.
- **Future Year Forecasts and Needs (No-Build)**: Horizon year traffic forecasts and performance assessment on the existing street system with no project improvements. Agreed upon baseline projects should be included. See Chapter 9 for more details.
- **Preliminary Alternatives Screening**: Screening criteria, concept alternatives to address outstanding needs and preliminary screening of alternatives with highlight of those set aside from further evaluation.
- **Alternative Results**: Discussion of performance results for each analyzed alternative for the build, interim and design years.
- **Alternative Summary**: The alternatives are compared and contrasted against

each other, including a summary table, according to appropriate performance measures.

- **Conclusions**: The analyst should be careful to make conclusions based on the traffic analysis, rather than recommendations on a preferred alternative, as the best alternative from a pure traffic standpoint may not be the best overall. The analyst should coordinate with the PT Leader if it is desired to also report the recommendation by the project team as to the overall preferred alternative.
- **Further Areas of Study**: Optional
- **Appendices**
    - Crash History: Detailed crash analysis and listing of crashes in study area.
    - Record of Calibration: The calibration record will vary in detail level and length by project, but the record should address the following items;
        - A table or list citing all changes that were made to the inputs or model modules to achieve calibration, beyond the standard changes that would occur after collecting field inventory (standard list found in Section 3.2). This list or table should include
            - the issue that was occurring before the change was made,
            - the goal of the change, and
            - some record 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 should be with all standard inputs, but no changes beyond the standard adjustments. After results should recorded after all changes to achieve calibration were included in the model. Minimally, the APM requires that the MOE – "Vehicles Exited" be used to assess the calibration of microsimulations (for SimTraffic, for other software use a comparable measure that sums vehicles making individual movements).
        - The record should indicate that every movement met the calibration standards described in section 8.3 for "Vehicles Exited" (8.3 is specific to SimTraffic, but simulations in any software should meet this criteria).
    - Traffic Development: Count locations, explanation of base and future volume development, includes land use and zoning maps.
    - Existing Year Volumes: Volume diagrams for the existing (base) year.
    - Build Year Volumes: Volume diagrams for the build year.
    - Future No-Build Volumes: Volume diagrams for the future No-Build year.
    - Alternatives Considered but Dismissed: Short description of each dismissed alternative including why it was dropped.
    - Build Alternative Volumes: Volume diagrams for each alternative. Each build and design year for each alternative will be a separate appendix.
    - Analysis Methodologies: Boilerplate text on analysis methods used.
    - EIS Traffic Data: For No-Build and Build alternatives, including link diagrams.

The volume diagrams in the report should include the Preferred Alternative, and any other alternatives that were evaluated for the purposes of the environmental review process.

Technical appendices, including all data, and all software input and output files and reports should be burned to CD or DVD, and retained in the ODOT file. For consultants doing ODOT analysis work, all input and output sheets shall be included with all technical memos and narratives.

A draft of the narrative needs to be sent to Region Traffic, TEOS, the project leader, the Roadway designer, Environmental and any others who may be affected, for review and comment.

12.4.3 Distribution

Upon incorporation of comments received on the draft, the Traffic Narrative Report should be signed and stamped, and should be distributed to the following in addition to the draft reviewers:

- Project Teams
  - Project Development Team
  - Citizen Advisory Committee
- ODOT Region/District Groups
  - Traffic Operations
  - Region Traffic
  - Roadway
  - Environmental
  - Geo-Hydro
  - Bridge