12 UNSIGNALIZED INTERSECTION ANALYSIS

12.1 Purpose

This chapter presents commonly used unsignalized intersection deterministic analysis procedures and identifies specific methodologies and input parameters to be used on ODOT projects. Simulation procedures are covered in APM Chapter 15. Software settings are covered in Appendix 12A/13A. Topics covered include:

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

> *The scope of this chapter is limited to auto mode analysis at unsignalized intersections. A complete evaluation of unsignalized intersections requires a broader evaluation including of non-auto modes. Refer to APM Chapter 10 for modal considerations such as for left and right turn lanes, and to Chapter 14 for multimodal analysis procedures such as MMLOS. The need for other evaluations such as per the Traffic Manual and HDM should be coordinated with Region Roadway/Traffic or Traffic-Roadway Section.*

For software-specific settings and parameters for unsignalized intersection analysis, refer to Appendix 12A/13A.

12.2 Turn Lane Criteria

Proposed left or right turn lanes at unsignalized intersections and private approach roads must meet the installation criteria contained in the Highway Design Manual (HDM). Meeting the criteria does not require a turn lane to be installed. Engineering judgment must be used to determine if an installation would be safe and practical. The ODOT Traffic Manual provides further guidance on the use of right and left turn lanes.

12.2.1 Left Turn Lane Criteria

Purpose: A left turn lane improves safety and increases the capacity of the roadway by reducing the speed differential between the through and the left turn vehicles. Furthermore, the left turn lane provides the turning vehicle with a potential waiting area until acceptable gaps in the opposing traffic allow them to complete the turn. Installation of a left turn lane must be consistent with the access management strategy for the roadway.

Left Turn Lane Evaluation Process

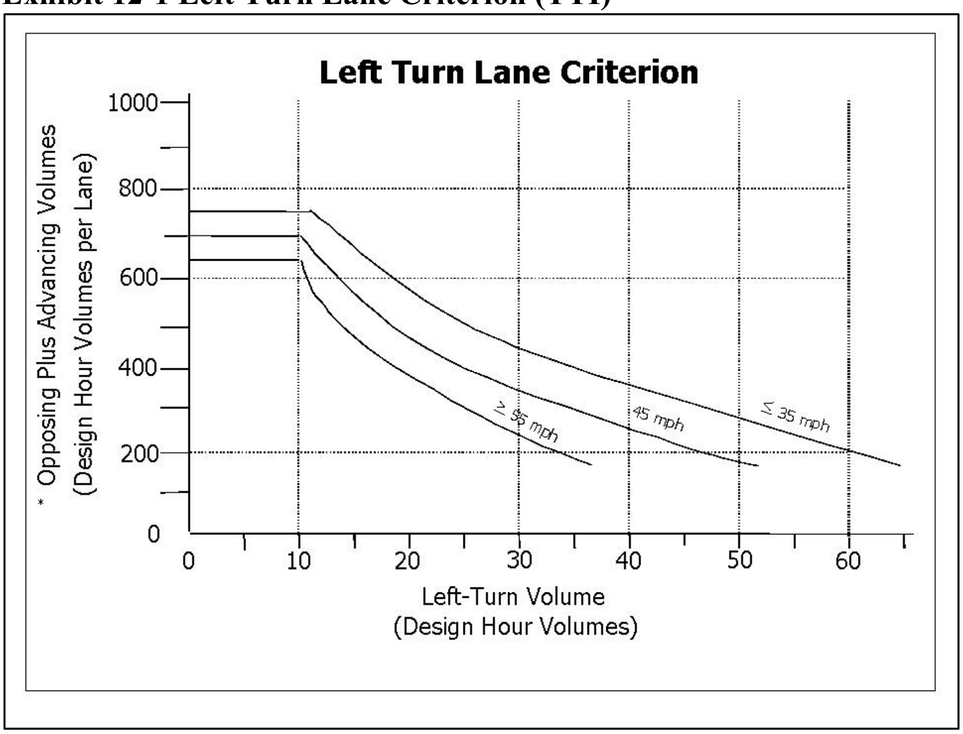
- A left turn lane should be installed, if criterion 1 (Volume) or 2 (Crash) or 3 (Special Cases) are met, unless a subsequent evaluation eliminate it as an option; and
- The Region Traffic Engineer must approve all proposed left turn lanes on state highways, regardless of funding source; and
- Left turn lane complies with Access Management Spacing Standards; and
- Left turn lane conforms to applicable local, regional, and state plans.

Criterion 1: Vehicular Volume

The vehicular volume criterion is intended for application where the volume of intersecting traffic is the principal reason for considering installation of a left turn lane. The volume criterion is determined by the Texas Transportation Institute (TTI) curves in Exhibit 12-1.

The criterion is not met from zero to ten left turn vehicles per hour but indicates that careful consideration be given to installing a left turn lane due to the increased potential for rear-end collisions in the through lanes. While the turn volumes are low, the adverse safety and operations impacts may require installation of a left turn. The final determination will be based on a field study.

Exhibit 12-1 Left Turn Lane Criterion (TTI)



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

Opposing left turns are not counted as opposing volumes

Criterion 2: Crash Experience

The crash experience criterion is satisfied when:

1. Adequate trial of other remedies with satisfactory observance and enforcement has failed to reduce the accident frequency; and
2. There is a history of crashes of the type susceptible to correction by a left turn lane (such as where a vehicle waiting to make a left turn from a through lane was struck from the rear); and
3. The safety benefits outweigh the associated improvement costs; and
4. The installation of the left turn lane does not adversely impact the operations of the roadway.

Criterion 3: Special Cases

1. **Railroad Crossings**: If a railroad is parallel to the roadway and adversely affects left turns, a worst-case scenario should be used in determining the storage requirements for the left turn lane design. The left turn lane storage length depends on the amount of time the roadway is closed, the expected number of vehicle arrivals and the location of the crossing or other obstruction. The analysis should consider all of the variables influencing the design of the left turn lane and may allow a design for conditions other than the worst-case storage requirements, providing safety is not compromised.
2. **Passing Lane**: Special consideration must be given to installing a left turn lane for those locations where left turns may occur and other mitigation options are not acceptable.
3. **Geometric/Safety Concerns**: Consider sight distance, alignment, operating speeds, nearby access movements and other safety related concerns.
4. **Non-Traversable Median**: As required in the Median Policy, a left turn lane must be installed for any break in a non-traversable median (OHP Action 3B.4).
5. **Signalized Intersection**: Consideration shall be given to installing left turn lanes at a signalized intersection. The State Traffic-Roadway Engineer shall review and approve all proposed left turn lanes at signalized intersection locations on the state highway system.
6. **Other Conditions**: Other surrounding conditions, such as a drawbridge, could adversely affect left turns and must be treated in a manner similar to that for railroad crossings.

Evaluation Guidelines

1. The **evaluation** should indicate the installation of a left turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the left turn lane should not be installed or, if already in place, not allowed to remain in operation.
2. **Alternatives Considered**: List all alternatives that were considered, including alternative locations. Briefly discuss alternatives to the left turn lane considered to diminish congestion/delays resulting in criteria being met.
3. **Access Management**: Address access management issues such as the long-term

access management strategy for the state roadway, spacing standards, other accesses that may be located nearby, breaks in barrier/curb, etc.

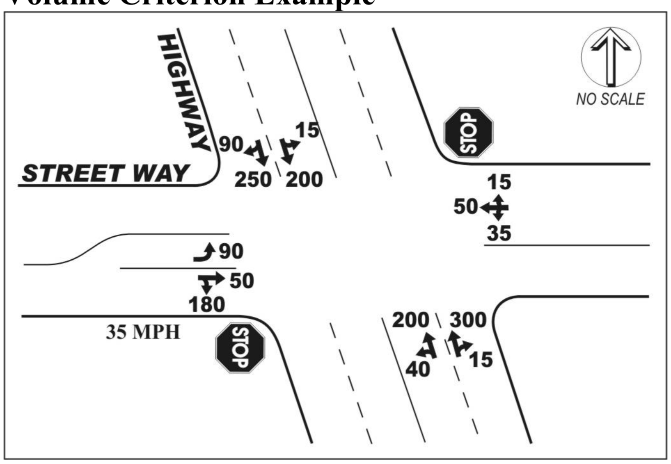
4. **Land Use Concerns**: Include how the proposed left turn lane addresses land use concerns and transportation plans.
5. **Plan**: Include a plan or diagram of proposed location of left turn lane.
6. **Operational Requirements**: Consider storage length requirements, deceleration distance, desired alignment distance, etc. For signalized intersections, installing a left turn lane must be consistent with the requirements in the Traffic Signal Guidelines.

Example 12-1 Left Turn Lane Criterion

Left Turn Volume Criterion

Volume Criterion Example shown below shows an unsignalized intersection with a shared through-right lane and a shared through-left lane on the Highway. The peak hour volumes and lane configurations are included in the figure. The 85th percentile speed is 45 mph and the intersection is located in a city with a population of 60,000. Do the NB and SB left turn movements meet the volume criterion?

Volume Criterion Example



- **Southbound Left**: The southbound advancing volume is 90 + 200 + 250 + 15 = 555, and the northbound opposing volume is 515 vehicles (the opposing left turns are not counted as opposing volumes). The volume for the y-axis on Exhibit 12-1 is determined using the equation:

  y-axis volume = ((Advancing Volume/Number of Advancing Lanes) + (Opposing Volume/Number of Opposing Lanes)) y-axis
  = (555/2 + 515/2) = 535

  To determine if the southbound left turn volume criterion is met, use the 45-mph curve in Exhibit 12-1, 535 for the y-axis and 15 left-turns for the x-axis. The volume criterion is not met in the southbound direction.

- **Northbound Left**: The northbound advancing volume is 40 + 300 + 200 + 15 = 555, and the southbound opposing volume is 540 vehicles (the opposing left turns are not counted as opposing volumes). The volume for the y-axis on Exhibit 12-1 is (555/2+ 540/2) = 548. To determine if the northbound left turn volume criterion is met, use the 45-mph curve in Exhibit 12-1, 548 for the y-axis and 40 left-turns for the x-axis. The volume criterion is met in the northbound direction.

12.2.2 Right Turn Lane Criteria – Unsignalized Intersections

> *Not all intersections that meet the siting criteria below should have a right turn lane installed. Refer to APM Chapter 10 for modal considerations for right turn lanes, and to Chapter 14 for multimodal analysis procedures such as MMLOS. The need for other evaluations such as per the Traffic Manual and HDM should be coordinated with Region Roadway/Traffic or Traffic-Roadway Section.*

Purpose

The purpose of a right turn lane at an unsignalized intersection is to improve safety and to maximize the capacity of a roadway by reducing the speed differential between the right turning vehicles and the other vehicles on the roadway.

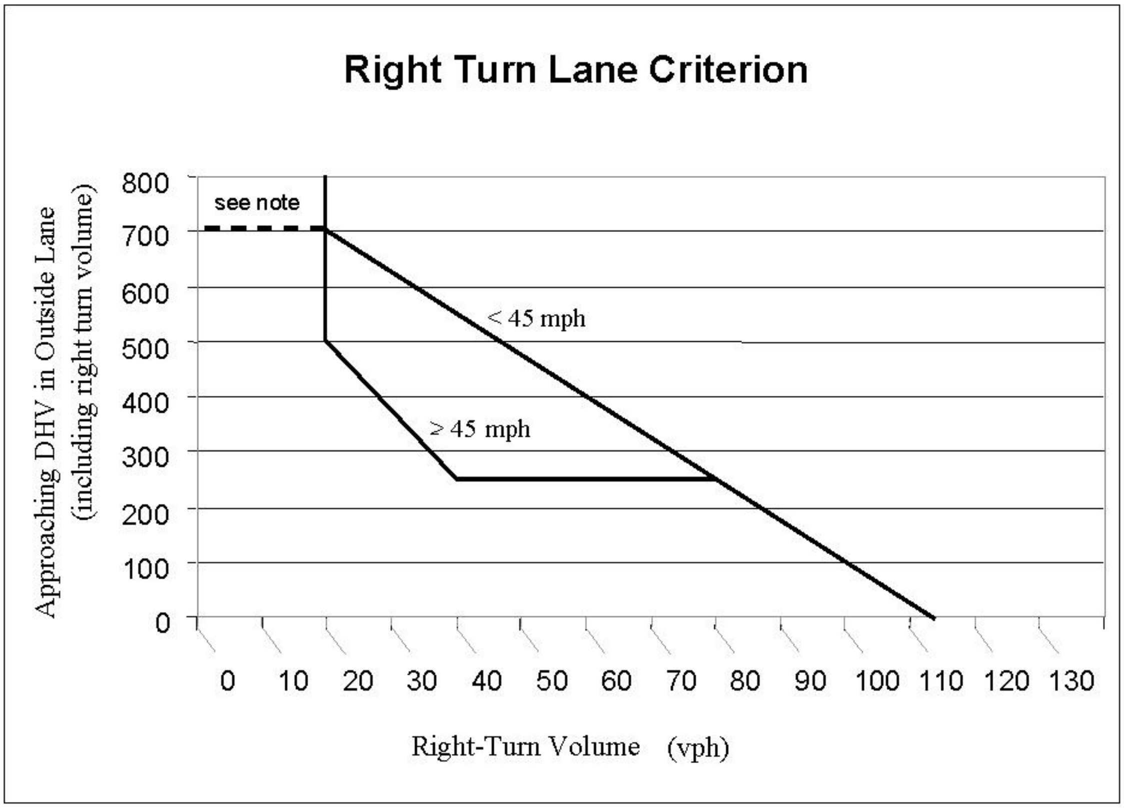
Right Turn Lane Evaluation Process

1. A right turn lane should be installed, if criterion 1 (Volume) or 2 (Crash) or 3 (Special Cases) are met, unless a subsequent evaluation eliminates it as an option; **and**
2. The Region Traffic Engineer must approve all proposed right turn lanes on state highways, regardless of funding source; **and**
3. The right turn lane complies with Access Management Spacing Standards; **and**
4. The right turn lane conforms to applicable local, regional and state plans.

<u>Criterion 1: Vehicular Volume</u>

The vehicular volume criterion is intended for application where the volume of intersecting traffic is the principal reason for considering installation of a right turn lane. The vehicular volume criterion is determined using the curve in Exhibit 12-2.

Exhibit 12-2 Right Turn Lane Criterion



Note: If there is no right turn lane, a shoulder needs to be provided. If this intersection is in a rural area and is a connection to a public street, a right turn lane is needed.

<u>Criterion 2: Crash Experience</u>

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.

<u>Criterion 3: Special Cases</u>

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

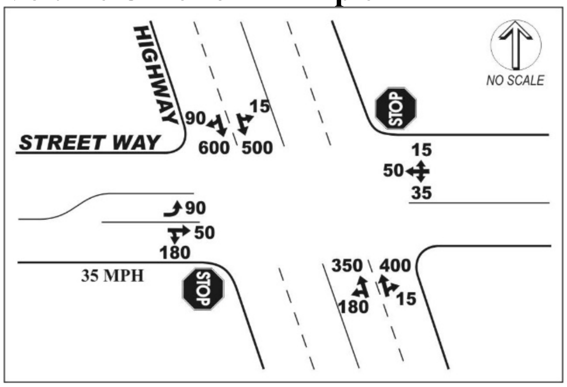
1. The **evaluation** should indicate the installation of a right turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the right turn lane should not be installed or, if already in place, should be reevaluated for continued 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 12-2 Right Turn Lane Criterion

<u>Right Turn Vehicular Volume Criterion</u>

Volume Criterion Example shown below shows an unsignalized intersection with a shared through-right lane and a shared through-left lane on the Highway. The peak hour volumes and lane configurations are included in the figure. The 85th percentile speed is 45 mph and the intersection is located in a city with a population of 60,000. Determine if a NB or SB right turn lane meets the criterion.

Volume Criterion Example



The northbound outside lane has 400 through vehicles and 15 right turning vehicles for a total of 415 vehicles. Using the 45-mph curve in Exhibit 12-2, along with 415 approaching vehicles and 15 right turning vehicles we find that the vehicular volume criterion is not met.

The southbound outside lane has 600 through vehicles and 90 right turning vehicles for a total of 690 vehicles. Using the 45-mph curve in Exhibit 12-2, along with 690 approaching vehicles and 90 right turning vehicles we find that the vehicular volume criterion is met.

12.3 Unsignalized Intersection Capacity

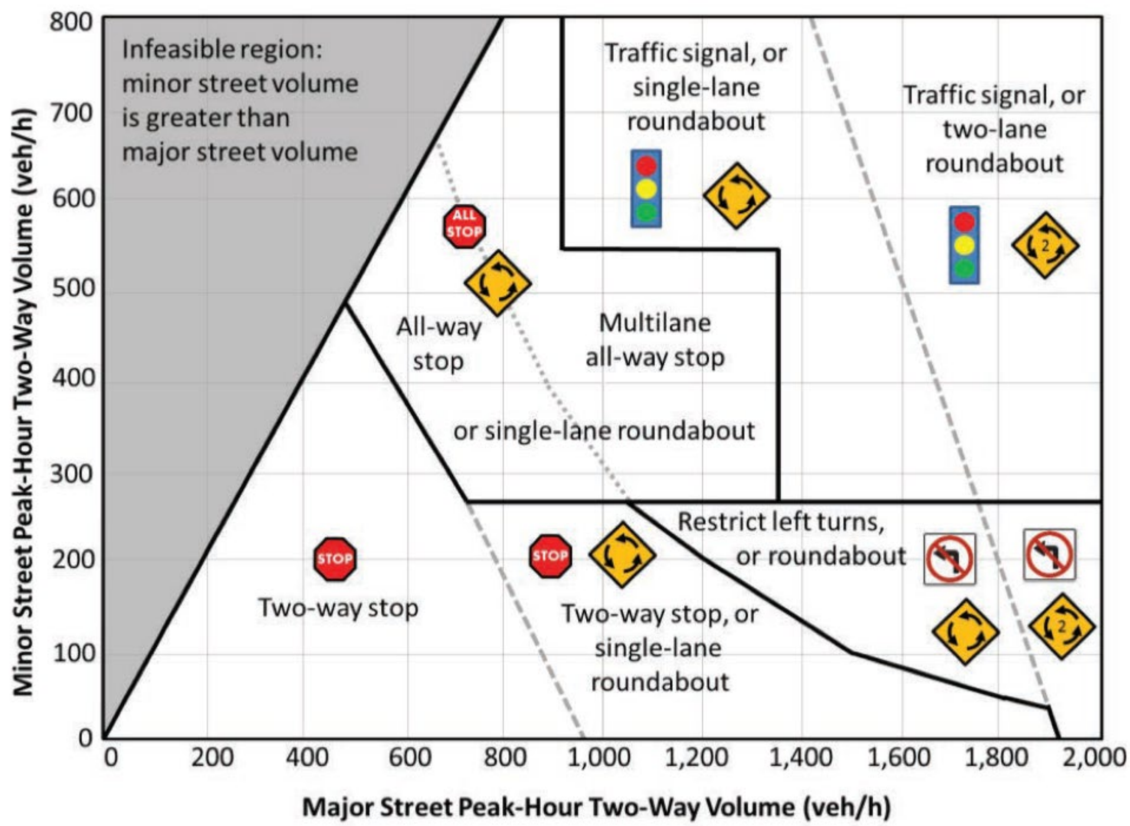
Capacity analysis for unsignalized intersections should generally follow the established methodology of the current HCM for both two-way and all-way stop control. For ODOT facilities, the highest movement v/c ratio of the major and minor approaches at an unsignalized intersection should be reported. Many jurisdictions require delay and level of service as the actual threshold performance measures.

Refer to OHP Action 1F.1 for clarification on how the OHP mobility targets are applied at different segment and intersection facilities. Different OHP v/c ratio targets apply to mainline versus minor approaches. HDM v/c ratios apply to all approaches as they do not specify minor or mainline.

If operational performance measure targets or criteria indicate a need, such as where the minor approach exceeds the v/c ratio target, the analyst needs to investigate multiple traffic control type solutions from lowest impact to highest. Potential solutions that could be considered range from, but are not limited to additional channelization, changes to lane alignments/designations, conversion to all-way stop, realignment, roundabout, and j turns as well as more intensive solutions such as signals and grade separations. Supplemental operational measures and considerations also come into play. Volume to capacity ratio is just one factor. Other operational factors to be investigated include multimodal considerations, safety and crash history, Level of Service and delay, sight distance, conflict points, functional area adequacy, and availability of alternate routes. The decision-making process may involve an intersection control evaluation and/or a design exception process. For further guidance on solution development, refer to Chapter 10.

For a sketch planning level estimation of future traffic control needs, the Planning & Preliminary Engineering Applications Guide (PPEAG) provides a graphical method as shown in Exhibits 12-3 and 12-4. Refer to the PPEAG for guidance on appropriate use of this method.

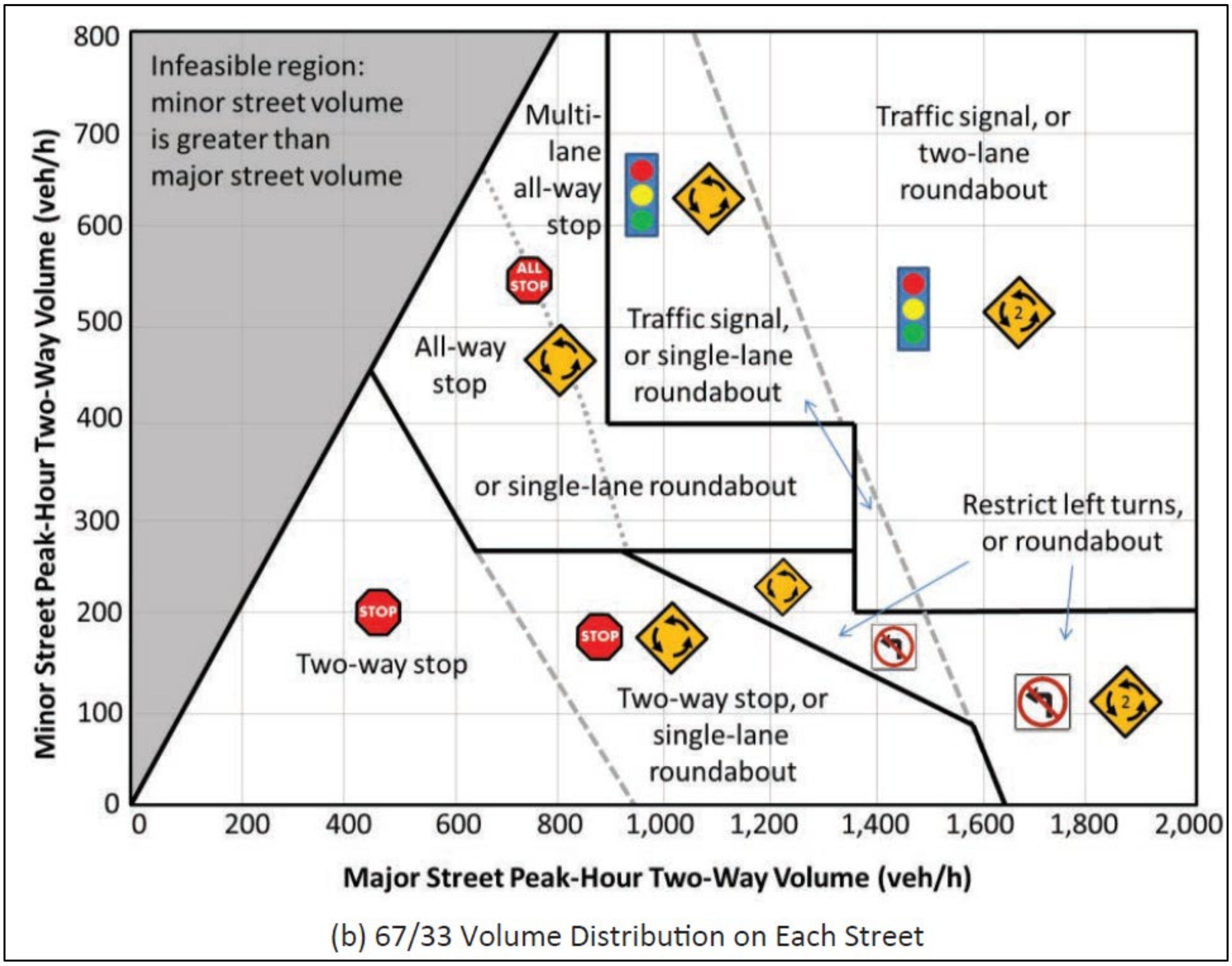
Exhibit 12-3 Planning Level Estimate of Traffic Control Needs - 50/50 Directional Volume Distribution



(a) 50/50 Volume Distribution on Each Street

Source: PPEAG Exhibit 17

Exhibit 12-4 Planning Level Estimate of Traffic Control Needs - 67/33 Volume Distribution



Source: Calculated from MUTCD 8-hour signal warrant, MUTCD all-way STOP warrant, and HCM methods for roundabout capacity and STOP-controlled intersection delay.
Notes: Assumes eighth-highest-hour volumes = 55% of peak hour volumes, peak hour factor = 0.92, 10% left turns and 10% right turns on each approach, and a single lane on each approach as the base case.
See text for an explanation of how boundaries between regions in the graphs were determined.

Source: PPEAG Exhibit 17

12.3.1 Two-Way Stop Control

For two-way stop control, the HCM employs a procedure for analyzing unsignalized intersections that is primarily based on an established hierarchy of intersection movements (based on assigned ROW) and a gap acceptance model. The major components of the gap acceptance model include the critical gap and follow-up time; where the critical gap is the minimum time interval in the major street traffic stream that allows intersection entry for one minor street vehicle and the follow-up time is the time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major street gap under a condition of continuous queuing on the minor street. A simplified planning level analysis method is available in the PPEAG, including a simplified spreadsheet tool.

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

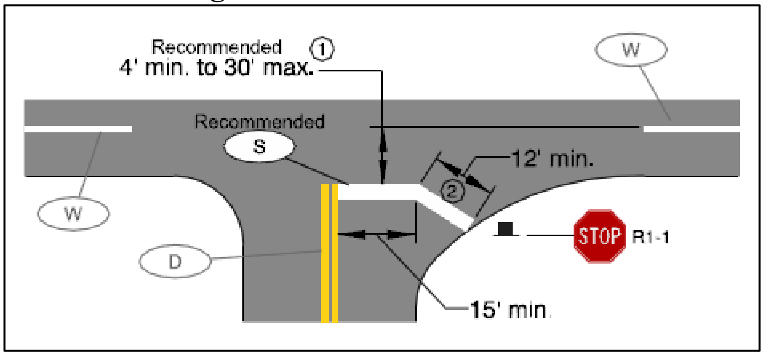
At two-way stop intersections, the controlling movement (usually a minor street left turn) often controls the overall intersection performance. Therefore, the v/c ratio for that movement will typically be the one reported and evaluated against the adopted mobility standard. This is especially important to recognize when analyzing two-way stop-controlled intersections where the very low v/c ratios for the unimpeded, high-volume major street movements will overshadow the higher v/c ratios for the lower-volume minor street movements. In these situations the unimpeded v/c ratio is often very low, even though the minor street movements are near or over capacity. However, as there may be times when the mainline v/c ratio is near the mobility standard, it should always be acknowledged before deferring to minor street movements. For ODOT facilities, the mainline through movement v/c ratio should be reported, as programs generally only report out minor v/c and mainline left.

The analyst should also check for heavy traffic flows that may occur in the opposite direction of peak hour volumes. For example, a high volume right turn movement in the pm peak period can be an indicator of a paired high volume left turn movement in the am peak period.

Right Turn Flares

A right turn flare is where, on the stop-controlled approach at a two-way stop controlled intersection, a shared lane allows right-turning vehicles to complete their movement while other vehicles are occupying the lane (see Exhibit 12-5). Current analysis procedures/processes/software differ as to how a right turn flare on the minor street is analyzed at unsignalized intersections.

Exhibit 12-5 Right Turn Flare



Source: ODOT Traffic Line Manual 2018 Figure 150-B

The HCM 6, HCS and Vistro provide a method for directly coding and evaluating the capacity of a flared right turn lane.
Synchro/SimTraffic and SIDRA do not allow directly for a flare, so in some cases it may be appropriate to code in a separate (short) turn lane. However, Synchro and SIDRA both see the added lane as having full capacity which is not the case as a flare is limited in its capacity. Therefore the capacity of an intersection is over-estimated when a flare is coded as a separate short turn lane regardless of the "storage" length of the flare.

If SimTraffic is being used, it may still be appropriate to include a scenario with a short turn lane with an appropriate length taper measured from field conditions or from design guidance (i.e., HDM). This will reflect the impact of a flare in SimTraffic when modeling driver behavior and vehicle characteristics for determining measures such as queuing and stop delay.

Engineering judgment is needed to determine when a right turn flare should be coded. There is not a single way to analyze/report the v/c ratios since the factors above vary widely across analysis areas. The analyst should observe operations in the field to understand existing usage. Considerations include:

- Purpose: What is the purpose of the analysis – broad versus specific? Plan versus project? What measures are needed? What is the correct effort for the work?
- Physical Conditions: Width, length, curbed section or not, available sight distance. A flare may be created by a large radius to accommodate trucks. Are there other constraints such as parking?
- Volumes: Total, turn moves
- Characteristics: Drivers, vehicles, traffic flow volumes, bicycles, pedestrians
- Operations – Are vehicles observed using/creating a flare? Is access to the flare blocked by queues?

There are three ways to handle right turn flares for reporting v/c ratio:

1. The most conservative is to not code any flare (the outside lane is a full shared lane).
2. The most correct mathematical way is to input the data (directly or by importing) into the HCM/HCS/Vistro unsignalized processes and coding the flare to account for the partial increase in capacity.

3. A third approach is to code the intersection both ways in HCM/HCS/Vistro and see if the values are different enough to warrant reporting the difference (perhaps as a range).

When the decision is made to include the effects of a flare on an unsignalized intersection, the analyst must use the HCM/HCS/Vistro process to report v/c ratios.

Shared Major Street Left Turn Approaches

There is a limitation of the HCM unsignalized intersection methodology for shared left turn approaches. Major street left turns are always treated as exclusive turn lanes regardless of how they are coded. This can result in very low shared left turn v/c ratios (like 0.01) on an approach that should be over capacity.

Shared major left turn vehicles are approximated in the HCM methodology by adjusting the potential for a "queue-free state" in the case of delaying through movement vehicles. This calculation ratchets down the through lane capacity (1700 for an unstopped lane) to reflect the capacity for the left turning vehicles. Note that this value is for the left turns not for the through movement (which is ignored).

The resulting reported HCM v/c is the left turn volume divided by the capacity of the shared lane for the lefts only. The v/c of the major street (non-stopped) left turn only reflects the left turn volume regardless if it is in a shared or an exclusive lane (v/c = volume of left turns / shared lane capacity). Other through movements and the stopped movements use the total lane volume divided by the shared lane capacity to obtain v/c.

In most cases this won't make a difference as the minor approaches will tend to control. However, in cases of small minor leg movements and a high volume on the mainline, the major through or the major left will control.

To calculate the correct shared approach v/c requires that you add the through v/c (volume of through vehicles divided by 1700) to the left turn v/c.

Example 12-3 Shared Through-Left V/C Calculation

Software programs that follow HCM 2010/6th Edition report out a value for the v/c ratio for the major left turn movement. However, this v/c is only of the left turn and does not include the through movement.

From a HCM software report: NBL v/c = 0.02

Divide the major street flow rate (pcph) by 1700 pcph to obtain the v/c of the through movement.
In this case the northbound major street flow rate is 191 pcph.

NBT v/c = 191 pcph / 1700 pcph = 0.11

Add the major left to the through movement to obtain the total reportable v/c:

NBLT v/c = NBL v/c + NBT v/c = 0.02 + 0.11 = **0.13**

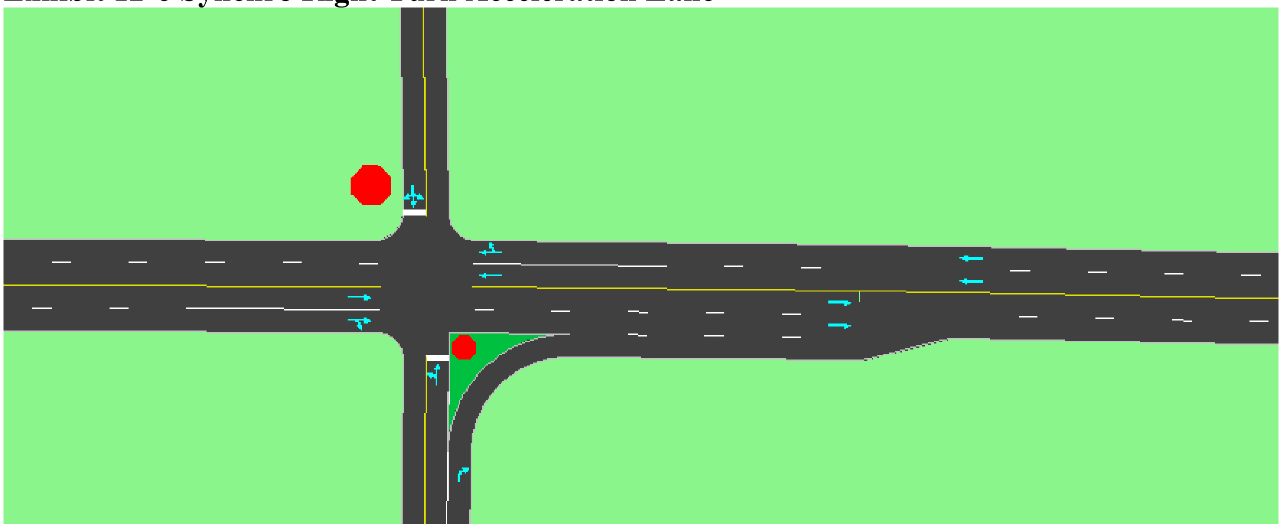
Unsignalized Intersection Acceleration Lanes

An unsignalized intersection acceleration lane is an added lane for vehicles turning from a side street at an at-grade intersection that allows the turning vehicle to accelerate from the turning speed to highway speed, typically on rural limited access highways. The v/c ratio of intersection acceleration lanes is performed using segment analysis. The worst v/c ratio is reported out of either the upstream segment before the merge point, or of the downstream segment after the merge point. Refer to Chapter 11 for segment v/c ratio calculation procedures. Additional analysis of intersection acceleration lane operations may be performed using microsimulation. Refer to Chapter 10 for general considerations on intersection acceleration lanes. An engineering study, Roadway Design Exception, and State Traffic-Roadway Engineer approval is required for acceleration lanes from at-grade intersections on state highways. Refer to Chapter 8 of the HDM and Section 6 of the ODOT Traffic Manual for more information.

Right-Turn Acceleration Lanes

A right turn acceleration lane is created in Synchro by coding a minor stop-controlled approach right turn movement with one Add Lane, entering the curb radius, and designating the sign control as Free, Stop or Yield as appropriate. If the acceleration lane is a drop lane, a bend node is coded at the end of the lane drop. This will draw an add lane on the departure side of the intersection that will merge with the through travel lanes downstream. In the simulation window, the lane alignment for through traffic is coded as L-NA so through vehicles do not enter the right turn acceleration lane. Likewise, right turning traffic is coded as R-NA so right turn vehicles turn into the acceleration lane and not the through lane. See Exhibit 12-6.

Exhibit 12-6 Synchro Right Turn Acceleration Lane

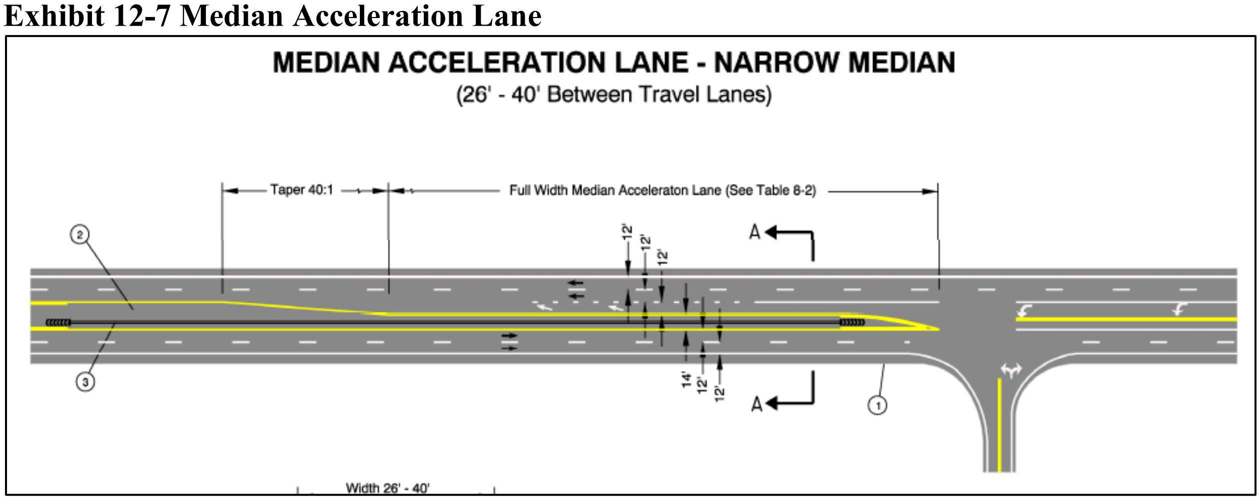


This coding does not provide a v/c ratio of the right turn acceleration lane. The v/c ratio analysis is performed using segment analysis for a two-lane highway. The worst v/c ratio is reported out of either the upstream segment before the merge point, or of the downstream segment after the merge point. For a multilane highway a merge analysis would be performed.

<u>Median Acceleration Lanes and Left Turn Add Lanes</u>

A median acceleration lane is shown in Exhibit 12-7. The acceleration lane drops downstream of the intersection.

Exhibit 12-7 Median Acceleration Lane



Source: 2012 ODOT Highway Design Manual

A left turn add lane is shown in Exhibit 12-8. This differs from the median acceleration lane in that the added lane does not drop downstream of the intersection. This design requires a barrier separating the through lane from the add lane.

Exhibit 12-8 Left Turn Add Lane

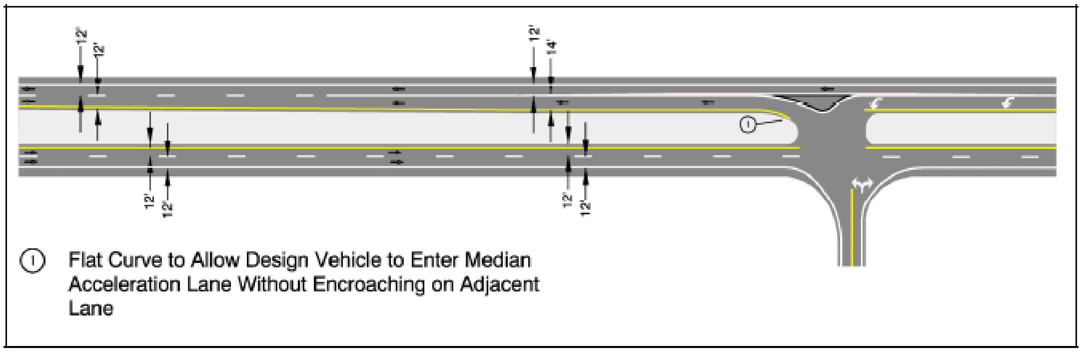


Figure 8-14: Left Turn Add Lane

Source: 2012 ODOT Highway Design Manual

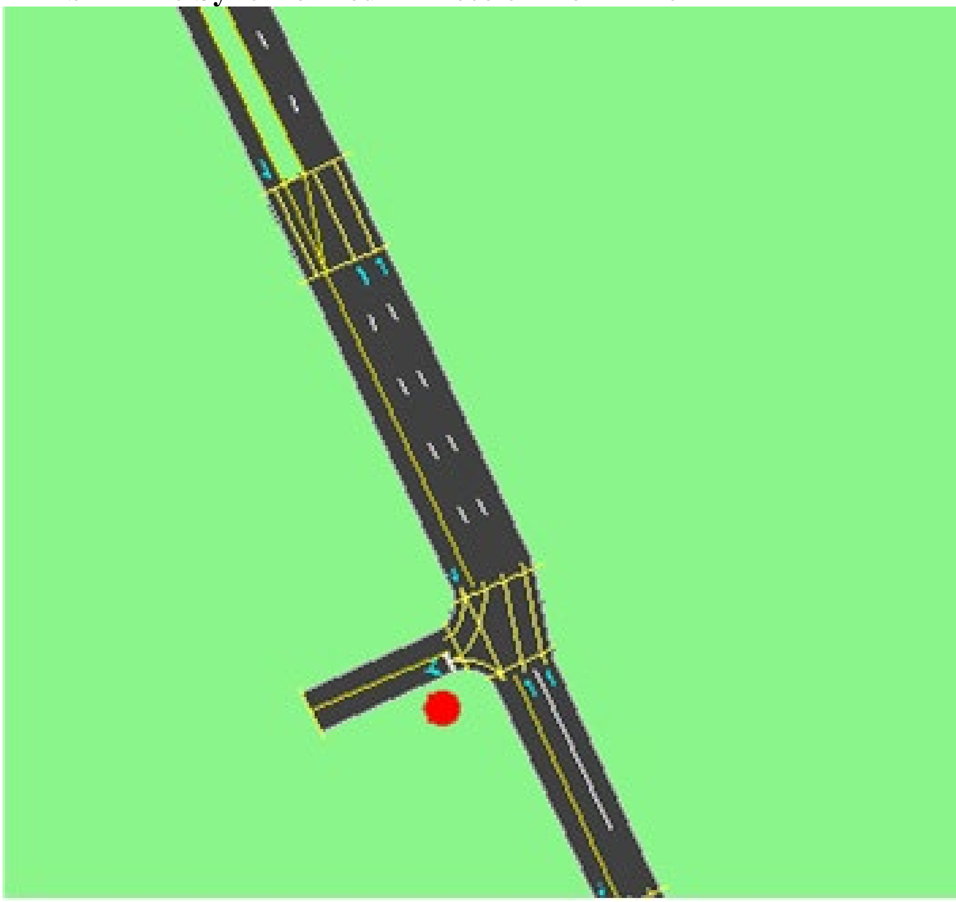
A median acceleration lane or left turn add lane can be created in Synchro by coding the movement as an Add Lane. Synchro provides a v/c ratio for the left turn into the median acceleration lane using a non-HCM methodology. The segment downstream of the merge point still needs to be evaluated using segment analysis, unless it is a left turn add lane where there is no merge point.

For simulation of an add lane, Synchro includes a Lane Alignment setting to establish whether vehicles are allowed to enter the added lane as they pass through an intersection or where through movements need to stay in their own lane. For a median acceleration lane, to prevent through vehicles from entering the median acceleration lane the movement is coded as R-NA. The left-out movement is coded as L-NA to force those vehicles to turn into the median acceleration lane only. See Exhibit 12-9 and Exhibit 12-10.

Exhibit 12-9 Synchro Lane Alignment Settings

SIMULATION SETTINGS	EBL	EBR	NBL	NBT	SBT	SBR
Lanes and Sharing (#RL)						
Traffic Volume (vph)	70	35	0	1125	905	55
Future Volume (vph)	70	35	0	1125	905	55
Storage Length (ft)	0	0	0	—	—	0
Storage Lanes (#)	—	—	—	—	—	—
Taper Length (ft)	—	—	—	—	—	—
Lane Alignment	L-NA	Right	Left	R-NA	Left	Right
Lane Width (ft)	12	12	12	12	12	12
Enter Blocked Intersection	1 veh	1 veh	No	Yes	Yes	Yes
Median Width (ft)	12	—	—	0	0	—

Exhibit 12-10 Synchro Median Acceleration Lane

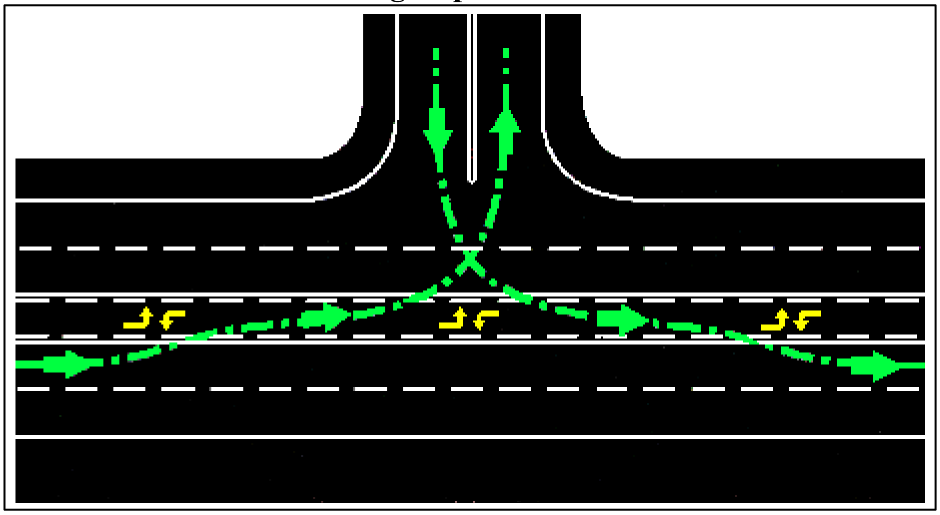


Two-Way Left Turn Lanes (TWLTLs)

Synchro provides a TWLTL feature. With this feature Synchro assumes two-stage left turn out from the minor approach at an intersection, as shown in Exhibit 12-11. This is coded by inputting a Median Width and checking the TWLTL option. Two vehicles can be stored in the median. This does not model driveway operations along a TWLTL. Synchro allows coding of TWLTL operation at four-leg intersections – this configuration is only allowed at minor crossroads. Consult with Region Traffic on whether TWLTL striping is appropriate. SimTraffic does not model two-stage gap acceptance.

Refer to Chapter 10 for general considerations on TWLTLs.

Exhibit 12-11 TWLTL 2-Stage Operation



Connected and Automated Vehicles

No conclusive research has been conducted yet on the potential future effects of connected and automated vehicles (CAVs) at two-way stop controlled (TWSC) intersections. However, operational issues at TWSC intersections typically arise on the stop-controlled approaches and CAVs are unlikely to improve the capacity of those approaches. This is because capacity improvements due to CAVs primarily arise because of CAVs being able to form platoons of closely spaced vehicles. In contrast, stop control disrupts side-street platoons, particularly if CAVs obey the legal requirement to come to a full and complete stop before proceeding. Similar to signalized intersections (see Chapter 13), at high percentages (>60–80%) of CAVs in the traffic stream, CAV platoons on the main street may create larger gaps that can be utilized by major-street left-turning traffic and by side-street traffic. However, assuming 100% human-driven vehicles—even for planning analyses—is a conservative approach that is recommended to be applied until further research occurs.

No guidance is presented for the effects of CAVs on TWSC intersections, and it is recommended that no adjustments to capacity are made until further research becomes available.

12.3.2 All-Way Stop Control (AWSC)

Under low volume conditions, two-way stop control (TWSC) is sufficient at most intersections. However, in some circumstances AWSC may be justified, for example as a safety treatment or as an interim improvement such as prior to installation of a roundabout or traffic signal. An Intersection Traffic Control Study is required for multiway stop installation. The ODOT Traffic Manual contains guidance on the engineering study required for AWSC as well as the approval process. AWSC requires approval of the State Traffic-Roadway Engineer.

The MUTCD contains threshold criteria for AWSC based on crashes or volumes. These are guidelines rather than mandatory requirements. They should not be regarded as an absolute minimum that must be met to consider AWSC.

For AWSC intersection operational analysis, the HCM procedure is based on an analysis of each approach independently. The procedure determines the capacity of each approach, which is used to calculate v/c ratios. The highest v/c ratio approach will be the one reported and evaluated against the adopted mobility standard. Some programs report out only degree of saturation, which should be assumed equivalent to v/c ratio. A simplified planning level analysis method is available in the Planning & Preliminary Engineering Applications Guide. Refer to Chapter 10 for guidance on the consideration of AWSC as a solution.

No research has been conducted yet on the potential future effects of connected and automated vehicles (CAVs) at AWSC intersections. However, CAVs are unlikely to improve the capacity of an AWSC intersection because the all-way stop will disrupt any CAV platoons that might exist on any intersection approach.

No guidance is presented for the effects of CAVs on AWSC intersections, and it is recommended that no adjustments to capacity are made until further research becomes available.

12.3.3 Non-HCM Compatible Stop Configurations

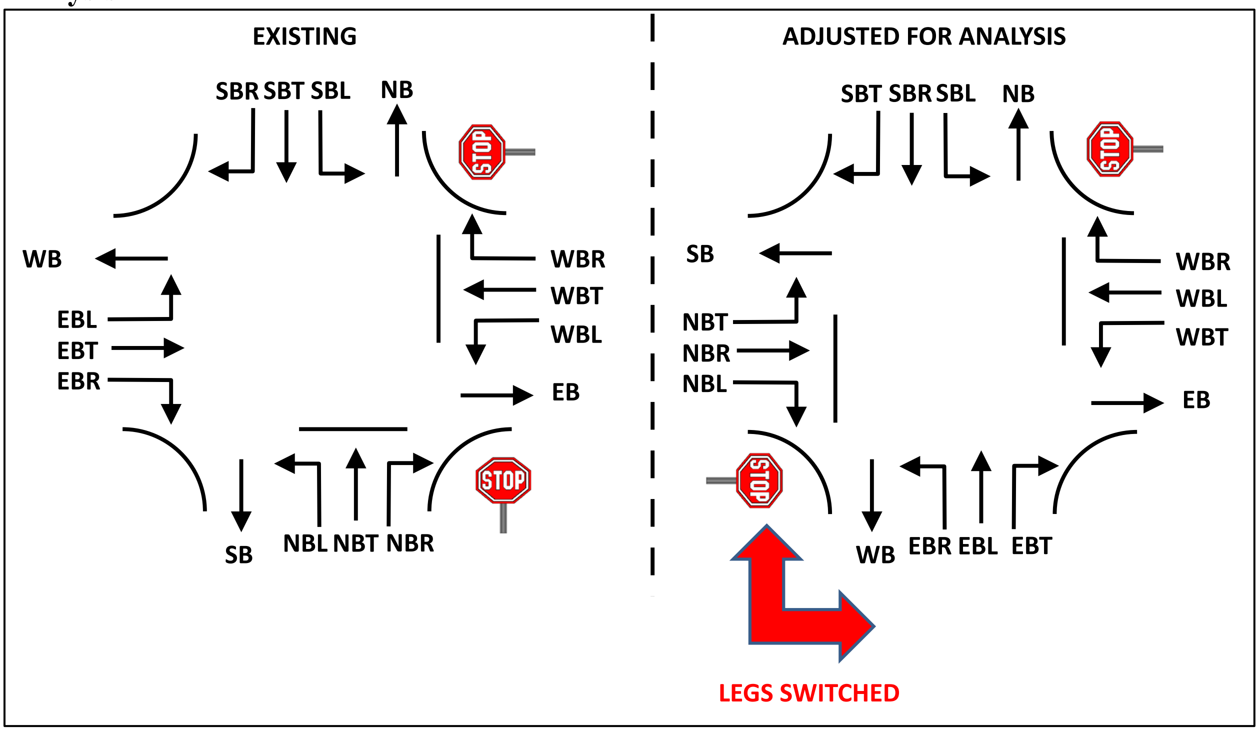
The HCM does not include methods for analysis of certain unsignalized intersections with unusual configurations. These configurations can be analyzed using microsimulation, but simulation does not produce a v/c ratio. SIDRA is the only program able to compute v/c ratios for configurations such as all-way stops with more than 4 legs, or non-standard stop sign placement such as where the mainline turns at an intersection.

An HCM based workaround procedure to obtain an approximate v/c ratio can be done for a 4-leg two-way stop intersection where the stop signs have non-standard placement by moving the volumes to mimic an HCM analyzable configuration. An example is shown in Exhibit 12-12. In this example, the highest volumes occur between the SB and EB approaches which are not stopped, as shown in the Existing configuration. The WB and NB approaches are low volume and are stopped. The workaround to analyze this configuration is to model the approaches having the major flow as if they were opposite each other. This was done in the Adjusted for Analysis configuration shown in Exhibit

12-12, where the EB and NB approaches were switched with each other. Note that Exhibit 12-12 shows turn movements rather than actual lane configurations. All movements still go to the same departure leg as in the Existing configuration. In other words, the directional approach and departure volumes on each leg of the intersection remain unchanged. The Adjusted for Analysis configuration can then be analyzed using HCM TWSC methodology.

The v/c ratios resulting from this method should be considered as approximate only. This method can also be used to estimate preliminary signal warrants. It should be noted that the resulting volumes are only for approximating the analysis and should not be shown on flow diagram figures.

Exhibit 12-12 Non-HCM Compatible Intersection with Directions Adjusted for Analysis



The workaround described above does not work for T intersections where the stem leg is not stopped while the other two legs are stopped. The v/c ratio for such a configuration can be obtained using SIDRA or can be approximated by analyzing the intersection under all-way stop control, or by taking the average v/c ratio between AWSC and TWSC.

12.3.4 Roundabouts

Roundabouts are a safe and efficient intersection option with more free flow than a stop sign or signal provides. Roundabouts can be a gateway or transition feature, roadway connection point, or key element of an access management project. Research has shown roundabouts generally reduce crashes and vehicle delay as compared to signals.

Roundabouts have fewer conflict points and severe injury crashes in comparison to other intersection designs.

For roundabouts on state highways, refer to the ODOT Traffic Manual and HDM for roundabout guidelines, standards, siting criteria and the approval process. The State Traffic-Roadway Engineer has been delegated the authority to approve the installation of roundabouts on State Highways, which is divided into two phases: Conceptual Approval and Design Approval.

Unlike traffic signals, there are no roundabout warrants because roundabouts are intersection designs and not traffic control devices. As such the decision to convert an intersection to a roundabout is an engineering design decision and not a traffic control device decision. Roundabout automobile capacity analysis generally follows the current HCM method. For further information, refer to Roundabouts: An Informational Guide (i), Second Edition, also known as NCHRP Report 672. A simplified planning level analysis method is available in the Planning & Preliminary Engineering Applications Guide, including a simplified spreadsheet tool.

Studies have shown that U.S. drivers use roundabouts more conservatively than international drivers. Therefore, U.S. roundabout capacities are generally lower than international values.

ODOT HCM Roundabout Automobile Methodology

HCM 7 Exhibit 22-10 shows 12 steps in the HCM analysis

Step 1: Flow rates from demand volumes

Step 2: Passenger car equivalents (bicycle, medium trucks, and heavy trucks)

Step 3: Circulating and exiting flow rates, addition of movements

Step 4: Entry flow rates by lane

Step 5: Capacity of entry lanes

Step 6: Pedestrian impedance to vehicles

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

Step 8: Volume/capacity ratio for each lane

Step 9: Average control delay, similar to unsignalized intersections

Step 10: LOS for each lane on each approach

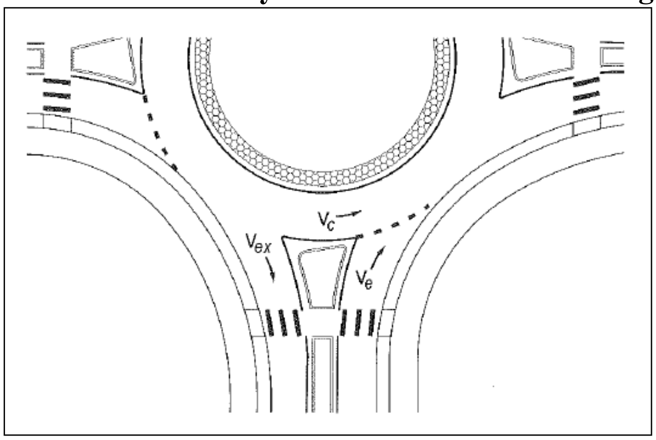
Step 11: Average Control Delay and LOS for entire roundabout

Step 12: Queues for each lane

Exhibit 12-13 (HCM 7 Exhibit 22-12) shows a single lane roundabout with an entry flow conflicting with a circulatory flow. Please note the subscripts: “c” is for circulatory, “e” is for entry, and “ex” is for exiting flow. Entry vehicles yield to circulatory vehicles.

Bicycles that enter the roundabout as a vehicle should be included in the intersection volumes for each movement (including U-turns).

Exhibit 12-13 Analysis on One Roundabout Leg



Source: HCM 7 Exhibit 22-1

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

Use HCM 7 Equation 22-8 to find the demand flow rate for each movement.

$$v_i = \frac{V_i}{PHF}$$

Where:

v_i = demand flow rate for movement (veh/h)
V_i = demand volume recorded for movement, include bicycles as a vehicle (veh/h)
PHF = peak hour factor

Step 2: Passenger car equivalents (bicycle, medium trucks, and heavy trucks)

Flow rates in vehicles per hour (veh/h) are converted to equivalent passenger cars per hour (pc/h) using vehicle factors. The bicycle equivalent factor should be 1.0, rather than 0.5 as suggested in HCM 7 (Exhibit 12-14).

Exhibit 12-14 Recommended Passenger Car Equivalents

Vehicle Type	Passenger Car Equivalents (E)
Passenger Car	1.0
Bicycle	1.0
Medium truck (two axles, UPS truck)	1.5
Heavy vehicle	2.0

Demand volumes (vph) are converted to equivalent passenger cars per hour (pc/h) using a heavy vehicle factor equation similar to that found in HCM 6. E_m and E_h are the equivalent factors for medium and heavy vehicles, 1.5 and 2, respectively. Heavy vehicles should be WB-67 or long trucks, such as fire engines. This designation is the engineer's judgment and also dependent on the counting methodology. The proportion that these vehicle types occur in a count is designated as P_m and P_h.

An adjusted heavy vehicle adjustment factor equation:

$$f_{HV} = \frac{1}{1 + P_m(E_m - 1) + P_h(E_h - 1)}$$

Where:

f_{HV} = heavy vehicle adjustment factor
P_m = proportion of demand volume that consists of medium trucks (decimal)
P_h = proportion of demand volume that consists of heavy vehicles (decimal)
E_m = passenger car equivalent for medium trucks (Passenger Car Equivalents given)
E_h = passenger car equivalent for heavy vehicles (Passenger Car Equivalents given)

This f_{HV} is then used in HCM 7, Equation 22-9.

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

Where:

$v_{i,pce}$ = demand flow rate for movement (passenger cars per hour; pc/h)
v_i = demand flow rate for movement (veh/h)
f_{HV} = heavy vehicle adjustment factor

Step 3: Circulating and exiting flow rates; addition of movements

The circulating flow rates in front of each entry are summed in terms of passenger car equivalents. See HCM 7 Equation 22-11 below.

$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

Where:

v_c = circulating flow rates in front of specified entry; in passenger car equivalents

$v_{WBU,pce}$ = flow rates of a specified movement

Step 3B: If considering a bypass lane, calculate the conflicting flow rates. The conflicting flow rates for where the bypass lane merges into the exiting lane can be calculated with HCM 7 Equation 22-12, similar to Equation 22-11.

Step 4: Entry flow rates by lane, if more than one lane

This step is for a multi-lane roundabout approach with more than one entry lane.

For approaches with multiple lanes, lane utilization must be estimated. If field data are not available, HCM 7 Exhibit 22-9 provides guidance on potential default values for different lane configurations.

For approaches with movements that may use more than one lane, follow HCM 7 Exhibit 22-14 to determine the assumed lane assignment.

Using the assumed lane assignment, assign flow rates to each lane using the formulas provided in HCM 7 Exhibit 22-15.

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

For roundabouts without a capacity and headway study, one should use HCM 7 Equation 33-1 with Exhibit 12-15 below to find the capacity for each entry lane using the circulatory flow rate calculated in Step 3.

$$C = Ae^{(-B \times V_c)}$$

Where:

C = roundabout entry lane capacity (pc/h)

A = intercept parameter, from Exhibit 12-15

V_c = circulating (conflicting) flow (pc/h)

B = slope parameter, from Exhibit 12-15

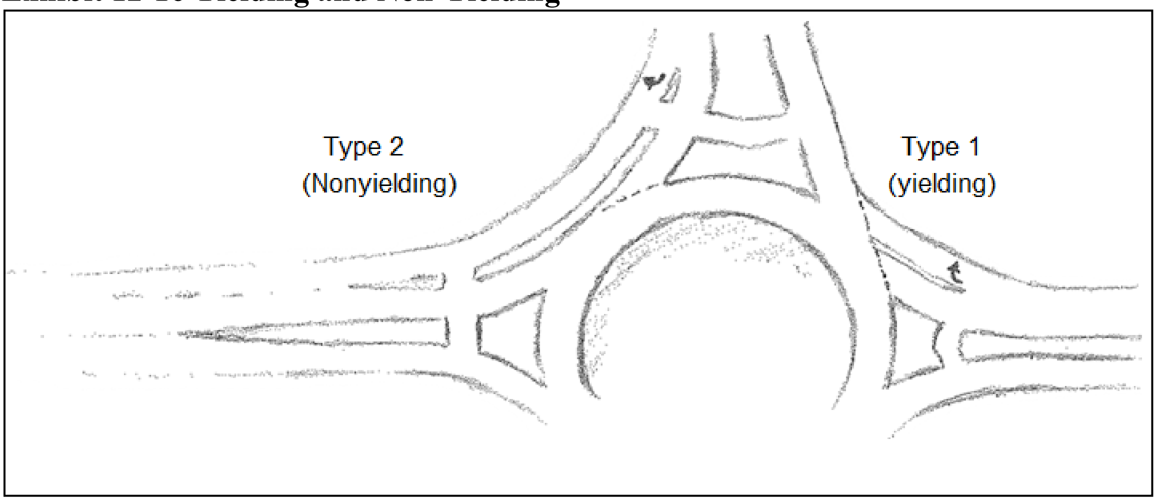
Exhibit 12-15 Roundabout Entry Lane Capacity Model Parameters

Entry Lane Type	*A*	*B*
One-lane entry conflicted by one circulating lane	1,380	0.00102
Two-lane entry conflicted by one circulating lane (both entry lanes)	1,420	0.00091
One-lane entry conflicted by two circulating lanes	1,420	0.00085
Two-lane entry conflicting by two circulating lanes (right entry lane)	1,420	0.00085
Two-lane entry conflicting by two circulating lanes (left entry lane)	1,350	0.00092

Source: HCM 7, Exhibit 33-12

For a Type 1 Yielding Bypass lane as shown in Exhibit 12-16, the capacity of the bypass lane should also be calculated. The exiting flow is used as the circulating or conflicting flow and the bypass lane volume must yield as the entry flow. Use of the single or multilane capacity equation (HCM 7 Step 5, Equation 22-6 or 22-7) depends on the number of opposing exit lanes. No calculation is necessary if the bypass lane is a Type 2, non-yielding bypass entering an add-lane. The capacity of an add-lane is expected to be high.

Exhibit 12-16 Yielding and Non-Yielding



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 the following three equations, similar to HCM 7 Exhibit 22-18, to find the entry capacity adjustment factor for pedestrians.

1. If the conflicting flow rate exceeds 881 pc/h, or if the number of conflicting pedestrians per hour is less than 40, the entry capacity adjustment factor for pedestrians is 1.0

$$\text{If } v_{c,pce} > 881 \text{ or } n_{ped} < 40,\ f_{ped} = 1$$

2. If the number of conflicting pedestrians per hour is equal to or greater than 40, but less than 101, use the following formula to calculate the entry capacity adjustment factor for pedestrians.

$$\text{Else, if } 40 \leq n_{ped} \leq 101, \; f_{ped} = 1 - 0.000137 n_{ped}$$

3. If either of the above two conditions are not met, use the following formula to calculate the entry capacity adjustment factor for pedestrians.

Else,

$$f_{ped} = \frac{1{,}119.5 - 0.715 v_{c,ped} + 0.00073 v_{c,pce} n_{ped}}{1{,}068.6 - 0.654 v_{c,pce}}$$

Where:

f_{ped} = entry capacity pedestrian adjustment factor
v_c = conflicting flow (pc/h)
n_{ped} = conflicting pedestrians (p/h)

An adjustment factor for pedestrians of 1.0 is recommended if there are fewer than 40 pedestrians crossing a leg in an hour. Less than 40 pedestrians crossing a leg in an hour do not have a significant effect on single lane roundabout operation.

If the hourly number of passenger car equivalent vehicles circulating in front of an entrance is over 881, then the adjustment factor for pedestrians is a factor of 1.0. If that is not the case and the number of pedestrians crossing at a crosswalk is greater than 40 and less than or equal to 101, then the second equation determines the adjustment factor for pedestrians.

Step 6B: If considering more than one entry lane, see HCM 7 Step 6 including Exhibits 22-20 and 22-21 for the entry capacity adjustment factor for pedestrians.

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 6 Equation 22-15.

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

Where:

f_{HVe} = averaged heavy vehicle adjustment factor for entry lane
f_{HVi} = heavy vehicle adjustment factor for movement *i*
$v_{i,PCE}$ = demand flow for movement i (pc/h)

The entry lane flow rate is converted back to vehicles per hour with HCM 7 Equation 22-13, a rearrangement of Equation 22-9.

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

Where:

$v_{i,PCE}$ = demand flow rate for lane *i* (pc/h)
v_i = demand flow rate for lane *i* (veh/h)
$f_{HV,e}$ = heavy vehicle adjustment factor

Step 7B: The capacity of a lane is converted back to vehicles per hour in Equation 22-14.

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

Where:

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

Step 8: Volume/capacity ratio for each lane

The volume/capacity ratio of a lane is calculated in Equation 22-16.

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

Where:

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

The v/c ratio is calculated for each lane on each approach. The highest lane v/c ratio calculated should be reported. An approach with a v/c ratio exceeding a standard, such as the applicable OHP/HDM v/c ratio, calls for further analysis and potential improvement, such as a bypass lane.

The decision to build a roundabout is determined by the State Traffic-Roadway Engineer (with consultation from Region Traffic). Considerations for further study may include highway classification, traffic characteristics, and system continuity.

Step 9: Average control delay, similar to unsignalized intersections

HCM 7 states the delay to be similar to unsignalized intersections, per United Sates roundabout data. The HCM makes a good point about delay at peak hour or design hour:

> "At higher volume-to-capacity ratios, the likelihood of coming to a complete stop increases, thus causing behavior to resemble STOP control more closely."

At higher volumes, it is likely that motorists may make stops before the crosswalk as well as the yield/stop that HCM 7 describes as resembling STOP control.

The average control delay of a lane is calculated in HCM 7 Equation 22-17.

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

Where:

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

The delay is calculated for each lane on each approach.

Step 10: LOS for each lane on each approach

The delay from Step 9 and the v/c ratio from Step 8 are used with Exhibit 12-17 (HCM 7 Exhibit 22-8) to determine the LOS of each lane on each approach.

Exhibit 12-17 HCM Unsignalized LOS Table

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a] $v/c \leq 1.0$	$v/c > 1.0$
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: [a] For approaches and intersectionwide assessment, LOS is defined solely by control delay.

Source: HCM Exhibit 22-8

Step 11: Average Control Delay and LOS for entire roundabout

The average control delay of a roundabout is calculated in HCM 7 equations 22-18 and 22-19. For a single lane roundabout with single entry lanes, these equations will reduce to an average of approach (HCM 7 Equation 22-19):

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

Where:

$d_{intersection}$ = average control delay for entire intersection (s/veh)
d_i = control delay for approach *i* (s/veh)
v_i = flow rate for approach *i* (veh/h)

With the average intersection delay, the intersection LOS is found from Exhibit 12-16 (HCM 7 Exhibit 22-8).

For multilane approaches and approaches with bypass lanes, the full Equation 22-18 is used, which calculates a weighted average delay for the approach. An overall intersection delay and LOS can also be determined using Equation 22-19.

Step 12: Queues for Each Lane

The 95th percentile queue of a roundabout entry lane is calculated in HCM 7 Equation 22-20.

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

Where:

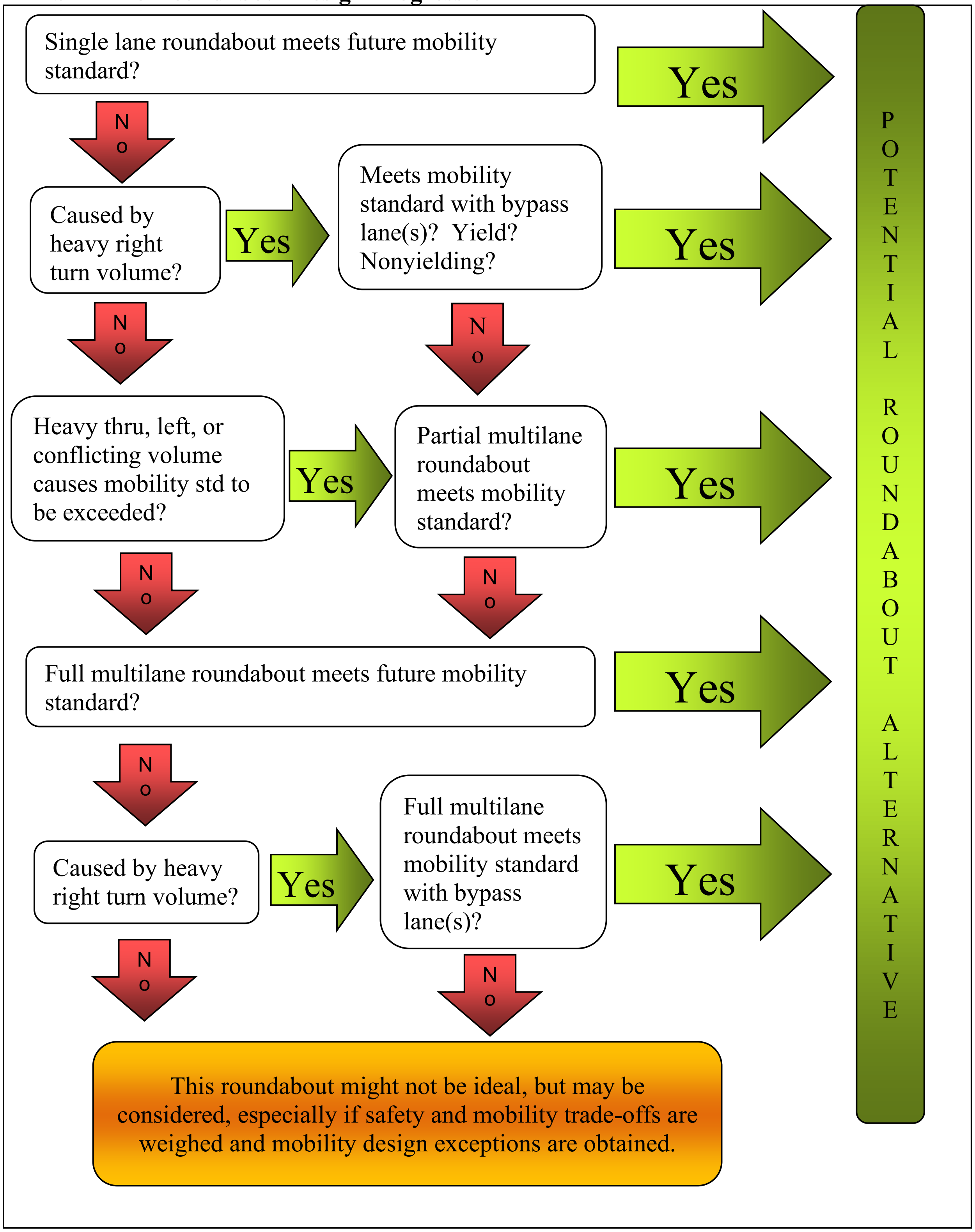
Q_{95} = 95th percentile queue (veh)
x = volume-to-capacity ratio of the subject lane
c = capacity of the subject lane (veh/h)
T = time period (h) (T = 0.25 for a 15-min analysis)

Logical Design Progression

Start analysis of a single lane roundabout with existing and future volumes. If an entry lane exceeds the mobility standard, then analyze a bypass lane for that approach. The bypass lane volume is subtracted out of the roundabout entry lane volume. This affects flow rate calculations of Steps 1 through 5. This may also affect capacity, v/c, delay, LOS, or 95th percentile queue. If a bypass lane merges into an existing lane (Yielding

Type 1), then calculate the capacity of the bypass lane (HCM 7 Chapter 33, Example Problem 1). If not due to a heavy right turn movement, then a multilane roundabout should be considered (not all of the circulating lanes must have more than one lane). If a multilane roundabout entry lane exceeds the mobility standard, then again consider a bypass lane. A flow chart showing this process is shown in Exhibit 12-18.

Exhibit 12-18 Roundabout Design Progression



Reporting

ODOT required outputs:

- Highest entry lane V/C
- Each bypass lane V/C
- Predicted queue lengths

Other jurisdictions may require:

- Intersection LOS and delay
- Bypass LOS
- Lane capacities
- Delay and LOS on each leg
- Entry and conflicting flows

Connected and Automated Vehicles

Chapter 33 of HCM 7 provides a process for adjusting the capacity of a roundabout entry lane to account for the effects of connected and automated vehicles (CAVs) in the traffic stream. This process, described in Appendix 6B, adjusts the parameters *A* and *B* used in the entry lane capacity equation in Step 5, resulting in somewhat higher entry lane capacities, depending on the percentage of CAVs in the traffic stream. No CAV capacity adjustment is yet available for roundabout bypass lanes, due to a lack of research.

As of 2023, no vehicles were available commercially that met the definition of a CAV for the purposes of the capacity adjustments provided for roundabout analyses in the HCM (i.e., a vehicle with an operating cooperative adaptive cruise control system that can communicate with other vehicles and driving without human intervention in any situation). The capacity adjustment process for CAVs presented in Appendix 6B is intended for use only in longer-range planning analyses. That appendix also provides guidance on estimating the percentage of CAVs in the traffic stream in a future year and example problems.

Because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in near-term analyses such as traffic impact studies.

12.4 Traffic Signal Warrants

Because the presence of traffic signals can degrade some aspects of overall traffic operations on a highway in addition to the improvements they provide, traffic signal warrants are used to determine when installation may be justified by identifying conditions where the benefits may outweigh the costs. The Manual on Uniform Traffic Control Devices (MUTCD) provides a set of 9 warrants to be used in determining if the installation of a traffic signal should be considered. In addition to these, the ODOT Transportation Planning Analysis Unit has also developed a set of "preliminary" traffic signal warrants, which are based on a portion of the MUTCD warrants but require less data for analysis. The preliminary warrants are generally not accepted as a basis for approving the installation of a traffic signal but are useful for projecting signalization needs for future years. Full warrants are evaluated later as part of the engineering study required by the MUTCD. Many other considerations go into determining whether a signal should be installed. For example, a signal installation is generally not appropriate in a rural area. The MUTCD and Preliminary Signal Warrant (PSW) methodologies are described below.

> *The MUTCD warrants are part of the Traffic-Roadway Section (TRS) signal approval process. For more information contact TRS. For all other applications/projections, only the ADT-based Preliminary Signal Warrant process can be used.*

When evaluating signal warrants (preliminary or MUTCD), it is important to include only the appropriate lane configurations and traffic volumes. Incorrect modeling of intersections is a very common mistake and can make a significant difference to the outcome of the analysis. There may be times when minor streets need to be modeled as major streets because of high side-street volumes (e.g., rural interchange) or left turns behave as right turns when dealing with one-way streets. In such cases, sound engineering judgment is critical to obtaining accurate analysis. Direction for proper modeling of intersections when analyzing signal warrants is included in the next section.

Traffic signal warrants must be met and the State Traffic-Roadway 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 Traffic-Roadway Section (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 Region Traffic and coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. If Region Traffic supports the concept of a signal installation, they will forward the analysis to TRS.

12.4.1 Preliminary Signal Warrants

Introduction

The single most important criterion for preliminary signal warrant analysis is engineering judgment. In the following procedures only the fundamental parameters of volumes and approach lanes are provided.

Background

There are 9 traffic signal warrants found in the MUTCD, listed in Part 4. The signal warrants are:

- Warrant 1, Eight-Hour Vehicular Volume
    - Condition A – Minimum Vehicular Volume
    - Condition B – Interruption of Continuous Traffic
- Warrant 2, Four-Hour Vehicular Volume
- Warrant 3, Peak Hour
- Warrant 4, Pedestrian Volume
- Warrant 5, School Crossing
- Warrant 6, Coordinated Signal System
- Warrant 7, Crash Experience
- Warrant 8, Roadway Network
- Warrant 9, Intersection Near a Grade Crossing

OAR 734-020-0460 (1) stipulates that only MUTCD Warrant 1 Condition A and Condition B may be used to project future needs for traffic signals beyond three years from the present time. Condition A deals primarily with high volumes on the intersecting minor street. Condition B addresses high volumes on the major street and the delays and hazards to vehicles on the minor street trying to either access or cross the major street. The preliminary warrant is considered satisfied if either Condition A or Condition B is met for either 100% of the thresholds or 70% of the thresholds when the major-street speed exceeds 40 mph or in an isolated community with a population of less than 10,000. The 80% and 56% thresholds that may apply after adequate trial of other remedial measures are not used for preliminary signal warrants. MUTCD Warrant 3, Peak Hour cannot be used to project needs for future traffic signals.

Information for Narrative

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

> TPAU uses Signal Warrant 1, Condition A and Condition B (MUTCD), which deal primarily with high volumes on the intersecting minor street and high volumes on the major-street. Meeting preliminary signal warrants does not guarantee that a signal shall be installed. Before a signal can be installed a field warrant analysis is conducted by the Region. If warrants are met, the State Traffic-Roadway 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 Condition A to be met the MUTCD requires a minimum total of 500 vehicles per hour on both approaches of the major street, where the major and minor streets both have only one lane for moving traffic (at 100%, assuming no reductions). To convert this to ADT volumes, the following calculations are made:

$$ADT = \frac{500}{0.0565} = 8{,}850$$

These calculations of ADT thresholds have already been completed for the analyst on the Preliminary Traffic Signal Warrant Analysis Form, as can be seen in Exhibit 12-18[1]

If the 85th percentile speed of major street traffic exceeds 40 mph in either an urban or rural area or when the intersection lies within the built-up area of an isolated community (typically non-MPO) having a population of less than 10,000, reduce the target volume for the warrants to 70 percent of the normal requirements, as shown in the preliminary traffic signal warrant analysis sheet in Exhibit 12-19.

[1] Note that the value of 8,850 calculated in the analysis example is the same as the value on the worksheet for this scenario.

Exhibit 12-19 Preliminary Traffic Signal Warrant Analysis Form

Oregon Department of Transportation Transportation Development Branch Transportation Planning Analysis Unit					
Preliminary Traffic Signal Warrant Analysis[1]					
Major Street:			**Minor Street:**		
Project:			**City/County:**		
Year:			**Alternative:**		
Preliminary Signal Warrant Volumes					
Number of Approach Lanes		ADT on Major Street Approaching From Both Directions		ADT on Minor Street, Highest Approaching Volume	
Major Street	Minor Street	Percent of Standard Warrants		Percent of Standard Warrants	
		100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8,850	6,200	2,650	1,850
2 or more	1	10,600	7,400	2,650	1,850
2 or more	2 or more	10,600	7,400	3,550	2,500
1	2 or more	8,850	6,200	3,550	2,500
Case B: Interruption of Continuous Traffic					
1	1	13,300	9,300	1,350	950
2 or more	1	15,900	11,100	1,350	950
2 or more	2 or more	15,900	11,100	1,750	1,250
1	2 or more	13,300	9,300	1,750	1,250
	5.65% of the above ADT volumes is equal to the MUTCD vehicles per hour (vph)				
	100 percent of standard warrants				
	70 percent of standard warrants[2]				
Preliminary Signal Warrant Calculation					
	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major				
	Minor				
Case B	Major				
	Minor				
Analyst and Date:			Reviewer and Date:		

[1] Meeting preliminary signal warrants does not guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

[2] Used due to 85th percentile speed in excess of 40 mph or isolated community with population of less than 10,000.

Determining the number of approach lanes and determining the approach volumes to use in the warrant analysis requires knowledge of the involved intersection. A spreadsheet calculator is available on the Technical Tools page that streamlines these calculations including the right turn discounts.

1. Major Street (Higher Volume Street)

   - Include only the through and through/left turn lanes in the number of approach lanes.
   - The major street number of approach lanes is directional, so a left-through in one direction and a through right (like at a ramp terminal) is considered as only a 1 lane major street, not a 2 lane major street.
   - An exclusive left turn lane and a through lane in one direction would be considered as a 2 lane major street, even if the other direction had only one lane.
   - An exclusive right turn lane is not counted in the number of directional approach lanes. An exclusive right turn lane and a through lane in one direction, and one lane in the other direction, would be considered as a 1 lane major street.
   - For the ADT, count total volume approaching from both directions, including all turn movements.

2. Minor Street (Lower Volume Street)

   - Include only the through, through/turn and left turn lanes in the number of approach lanes. However, in cases of where a minor street approach is just a right turn lane, code this as a lane in the worksheet. The right turn discount is applied normally as described below.
   - For the ADT, count the highest approaching volume (one direction only, do not include the ADT approaching from both directions) including some or none of the right turn volume as discussed in the following scenarios and examples:
     - **Scenario # 1 – Shared Left-Through-Right Lane**: Some of the right turns are included in the minor street approach ADT if the right turn demand is greater than 85% of the capacity of the shared lane. Use unsignalized capacity analysis to calculate the capacity of the shared lane. The right turn discount is 85% of the shared lane capacity (85% of the capacity is used because once the v/c exceeds 0.85, drivers suffer longer delay and begin to take unsafe gaps). Subtract the right-turn discount from the total right turn volume to determine the number of right turns in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

Example 12-4 Right Turn Discount for Shared Left/Through/Right Lane

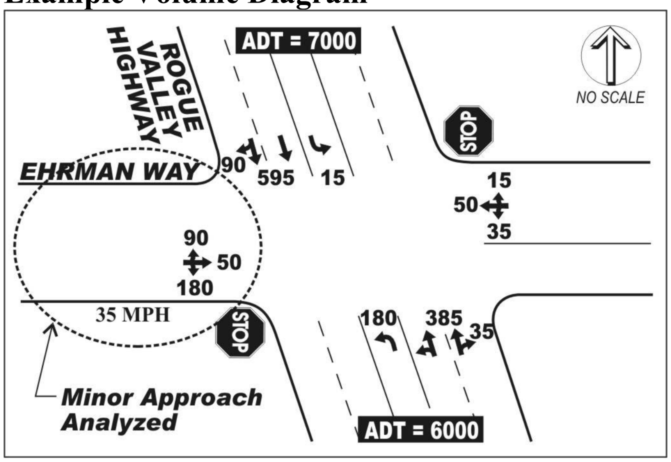
Example Application: Right Turn Discounts (Only for the minor road.)

The diagram below shows a typical unsignalized intersection, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The

85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound shared lane capacity is 120 vph. The right-turn discount is 85% of the shared lane capacity, 120 x 0.85 = 102 right turns. The number of right turns included in the warrant would be 180 – 102 = 78.
- Determine the minor approach ADT. The minor street approach peak hour volume used in the warrant is 90+50+78 = 218. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (218 / 0.10) = 2,180.

Example Volume Diagram



The figure below shows the Preliminary Signal Warrant Analysis for this example. The preliminary signal warrant is not met because the Minor Street ADT is less than the warrant volume in Condition A and the Major Street ADT is less than the warrant volume in Condition B.

Warrant Analysis of Minor Approach #1 Example Conditions

Oregon Department of Transportation

Transportation Development Branch

Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	Rogue Valley Highway	**Minor Street:**	Ehrman Way
Project:	Ehrman Way	**City/County:**	Medford
Year:	1995	**Alternative:**	Single Ln Minor Appr L/T/R

Preliminary Signal Warrant Volumes

Number of Approach lanes		ADT on major street approaching from both directions		ADT on minor street, highest approaching volume	
Major Street	Minor Street	Percent of standard warrants 100	70	Percent of standard warrants 100	70
Case A: Minimum Vehicular Traffic					
1	1	8850	6200	2650	1850
2 or more	1	10600	7400	2650	1850
2 or more	2 or more	10600	7400	3550	2500
1	2 or more	8850	6200	3550	2500
Case B: Interruption of Continuous Traffic					
1	1	13300	9300	1350	950
2 or more	1	15900	11100	1350	950
2 or more	2 or more	15900	11100	1750	1250
1	2 or more	13300	9300	1750	1250
X	100 percent of standard warrants				
	70 percent of standard warrants[2]				

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2	10600	13000	N
	Minor	1	2650	2180	
Case B	Major	2	15900	13000	N
	Minor	1	1350	2180	

Analyst and Date:	Reviewer and Date:

[1] Meeting preliminary signal warrants does **not** guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

- **Scenario # 2 – Exclusive Right-Turn Lane**: Some of the right turns are included in the approach ADT if the right turn lane demand is greater than 85% of the capacity of the right turn lane. Use unsignalized capacity analysis to calculate the capacity of the right turn lane. The right turn discount is 85% of the right turn lane capacity. Subtract the right turn discount from the total right turning volume to determine the number of right turns that will be included in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

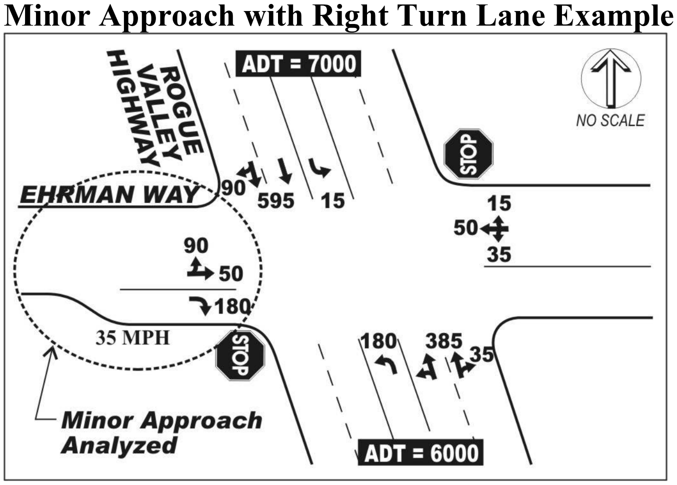
Example 12-5 Right Turn Discount for Exclusive Right Lane

The diagram below shows a typical unsignalized intersection with a separate right turn lane on the eastbound approach, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The 85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound right turn lane capacity is 639 vph. The right turn discount is 85% of the shared lane capacity, 0.85 x 639 = 543 right turns. The number of right turns included in the warrant is 180-543 = -363 = 0. If the number is less than or equal to zero, do not include any right turns in the warrant. The EB right turn lane is not included in the number of approach lanes.
- Determine the minor approach ADT. The minor approach peak hour volume used in the warrant is 90+50+0 = 140. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (140/0.10) = 1,400.

The form below shows the Preliminary Signal Warrant Analysis for this example. The preliminary signal warrant is not met since the Minor Street ADT is less than the warrant volume in Condition A and the Major Street ADT is less than the warrant volume in Condition B.

Minor Approach with Right Turn Lane Example



Warrant Analysis of Minor Approach #1 Example Conditions

Oregon Department of Transportation

Transportation Development Branch

Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	Rogue Valley Highway	**Minor Street:**	Ehrman Way
Project:	Ehrman Way	**City/County:**	Medford
Year:	1995	**Alternative:**	2 Lane Minor Approach L/T, R

Preliminary Signal Warrant Volumes

Number of Approach lanes		ADT on major street approaching from both directions		ADT on minor street, highest approaching volume	
Major Street	Minor Street	Percent of standard warrants 100	70	Percent of standard warrants 100	70
Case A: Minimum Vehicular Traffic					
1	1	8850	6200	2650	1850
2 or more	1	10600	7400	2650	1850
2 or more	2 or more	10600	7400	3550	2500
1	2 or more	8850	6200	3550	2500
Case B: Interruption of Continuous Traffic					
1	1	13300	9300	1350	950
2 or more	1	15900	11100	1350	950
2 or more	2 or more	15900	11100	1750	1250
1	2 or more	13300	9300	1750	1250
X	100 percent of standard warrants				
	70 percent of standard warrants[2]				

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2	10600	13000	N
	Minor	1	2650	1400	
Case B	Major	2	15900	13000	N
	Minor	1	1350	1400	
Analyst and Date:			Reviewer and Date:		

[1] Meeting preliminary signal warrants does **not** guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

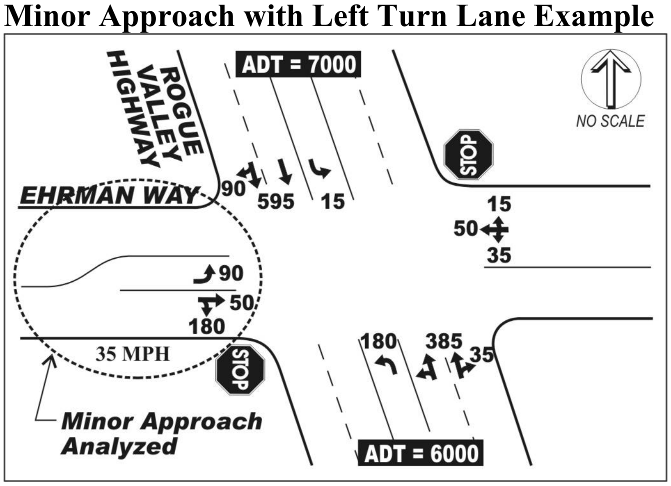
- Scenario # 3 – Shared Through-Right Lane: Some of the right turns are included in the approach ADT if the right turn demand is greater than 85% of the capacity of the shared through-right lane. Use unsignalized capacity analysis to calculate the capacity of the through-right shared lane. The right turn discount is 85 % of the shared lane capacity. Subtract the right turn discount from the total right turn volume to determine the number of right turns in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

Example 12-6 Right Turn Discount for Shared Through/Right Lane

The diagram below shows a typical unsignalized intersection with a shared through-right lane on the eastbound approach, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The 85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound shared lane capacity is 277 vph. The right turn discount is 85% of the shared lane capacity, 0.85 x 277 = 235 right turns. The number of right turns included in the warrant is 180 – 235= -55 = 0. If the number is less than or equal to zero, do not include any right turns in the warrant.
- Determine the minor approach ADT. The minor approach peak hour volume used in the warrant is 90+50+0= 140. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (140 / 0.10) = 1,400.
- The form below shows the Preliminary Signal Warrant Analysis for this example. The preliminary signal warrant is not met since the Minor Street ADT is less than the warrant volume in Condition A and the Major/Minor Street ADT's are both less than the warrant volumes in Condition B.

Minor Approach with Left Turn Lane Example



Warrant Analysis of Minor Approach #1 Example Conditions

Oregon Department of Transportation

Transportation Development Branch

Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	Rogue Valley Highway	**Minor Street:**	Ehrman Way
Project:	Ehrman Way	**City/County:**	Medford
Year:	1995	**Alternative:**	2 Lane Minor Approach L, T/R

Preliminary Signal Warrant Volumes

Number of Approach lanes		ADT on major street approaching from both directions		ADT on minor street, highest approaching volume	
Major Street	Minor Street	Percent of standard warrants		Percent of standard warrants	
		100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8850	6200	2650	1850
2 or more	1	10600	7400	2650	1850
2 or more	2 or more	10600	7400	3550	2500
1	2 or more	8850	6200	3550	2500
Case B: Interruption of Continuous Traffic					
1	1	13300	9300	1350	950
2 or more	1	15900	11100	1350	950
2 or more	2 or more	15900	11100	1750	1250
1	2 or more	13300	9300	1750	1250
X	100 percent of standard warrants				
	70 percent of standard warrants[2]				

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2	10600	13000	N
	Minor	2	3550	1400	
Case B	Major	2	15900	13000	N
	Minor	2	1750	1400	

Analyst and Date:	Reviewer and Date:

[1] Meeting preliminary signal warrants does **not** guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

- **Scenario # 4 – Double Right-Turn Lane**: Include all of the right turning volume in the approach ADT if a double right turn lane is required. If such is the case, the number of approach lanes for warrant analysis is 2 or more.

The above information is meant to serve as general guidelines only. Engineering judgment may be required when one or both 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.

12.4.2 MUTCD Signal Warrants

PSWs are used to project signalization needs. MUTCD warrants are limited to 3 years or less and are for actual approval of installation of a traffic signal. This requires an engineering study including an evaluation of the full set of 9 MUTCD signal warrants. Provisions for this evaluation are found in the ODOT Traffic Signal Policy and Guidelines, OAR 734-020-0400 thru 734-020-0500 and ODOT Traffic Manual.

12.5 Estimating Vehicle Queue Lengths

Vehicle queues can have a significant effect on highway safety and operation. Queues that exceed the provided storage at turn lanes can block the adjacent through lanes creating a temporary reduction in capacity as well as an unexpected obstruction in the travel lane that could result in a crash. In through lanes long queues can block access to turn lanes, driveways and minor street approaches, in addition to spilling back into upstream intersections. Under these conditions there are significant losses in capacity that can quickly spread to other upstream intersections and adjacent streets. There can also be a higher potential for crashes as drivers turning onto or off of the highway are required to pass through gaps in the queue that provide limited visibility and other drivers incurring long delays become more aggressive. Therefore, the estimation of vehicle queue lengths is an important traffic analysis procedure that should be included in most operational and safety projects.

Estimates of queue lengths should be based on the anticipated arrival patterns, duration of interruptions and the ability of the intersection to recover from momentary heavy arrival rates. The average queue length and the 95th percentile queue length should be shown in the report. The 95th percentile queue length shall be used for design purposes. A queue blockage or spillback condition is considered a problem when the duration exceeds 5 percent of the peak hour. The average vehicle length, including buffer space between vehicles, to be used in analysis shall be 25-feet, unless a local study indicates otherwise, with all queue length calculations rounded up to the next 25-foot increment. Queue lengths subject to over-capacity conditions can only be adequately assessed with simulation software. The 25-foot average does not apply to microsimulation, where vehicle lengths differ by vehicle type. Refer to Chapter 15 for microsimulation guidance.

The minimum storage length for urban or rural left turn lanes at unsignalized intersections on state highways is 100 feet. Left Turn Lane layouts/dimensions are available in HDM Chapter 8 Figure 8-9 and Traffic Line Manual (TLM) Section 310.

12.5.1 Two-Way Stop Control Intersection Queuing

TPAU Models

At unsignalized intersections, the movements of interest are often the major street left turns and all minor street movements. The most common methodologies used for estimating queue lengths for these movements include the Highway Capacity Software (HCS)[2] and the Two-Minute Rule.

[2] *Highway Capacity Software*, McTrans, University of Florida, Gainesville, Florida.

TPAU has conducted studies on modeling queue lengths at two-way stop controlled (TWSC) intersections[3]. The studies checked the relative performance of the two-minute rule and the HCM 2000 method. One of the conclusions was that the two-minute rule was overestimating and the HCM methodology was underestimating the queue lengths. In addition, existing methods were not found to be accurately predicting queue lengths for more than 50 percent of the cases.

Poisson regression models were developed to improve the queue length estimations. Model validation shows that the refined models are predicting queue lengths better than other methods. The HCM methodology was found to consistently underestimate the queue length. A Two-Way Stop Queue Length Calculator is available on the Planning Section website under Tools. Exhibit 12-19 summarizes the developed models, and applicable ranges of input data for each model type. When the range of independent variables exceeds the limiting value, use queue length models with caution.

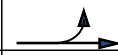
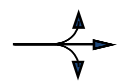
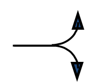
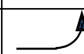
Exhibit 12-20 TPAU Two-way Stop Controlled Intersection Queue Length Models

Lane Group	Queue Length Model Equation[1]
MJL[2]	Queue Length = $e^{(0.3925+0.0059*VOL+0.00104*CONVOL+0.49*Signal-0.81*LT)}$
MNLTR[3]	Queue Length = $e^{(-0.7844+0.01636*VOL+0.0006*CONVOL-0.0000043*VOL*CONVOL)}$
MNLR[4]	Queue Length = $e^{(-0.6319+0.0173*VOL+0.00066*CONVOL-0.000007913*VOL*CONVOL)}$
MNL[5]	Queue Length = 0.95+ 0.014*VOL +0.00074*CONVOL+3.01*(VOL/ CONVOL)
MNR[6]	Queue Length = 0.865+ 0.0000534*VOL*CONVOL +0.2372*(VOL/CONVOL)

[1] Use this method with caution if volumes fall outside the variable ranges shown below:
[2] MJL VOL = 0 to 300 vph; CONVOL = 0 to 2,000 vph; SIGNAL = 0 or 1; LT = 0 or 1
[3] MNLTR VOL = 0 to 300 vph; CONVOL = 0 to 3,000 vph
[4] MNLR VOL = 0 to 300 vph; CONVOL = 0 to 3,000 vph
[5] MNL VOL = 0 to 300 vph; CONVOL = 0 to 2,000 vph
[6] MNR VOL = 0 to 250 vph; CONVOL = 0 to 1,500 vph

[3] Development of Queue Length Models at Two-way STOP Controlled Intersection: A Surrogate Method (accessed on January 23, 2014)

Where:

VOL	Traffic volume on the subject approach in vehicles per hour	
CONVOL	Conflicting traffic volume in vehicles per hour	
SIGNAL	Presence of an upstream signal within ¼ mile of an intersection, applicable for major left turn only, 1 if there is a signal, otherwise 0	
LT	Presence of a separate left turn lane, applicable for major left turn only (1 if there is an exclusive left turn lane/median left turn lane/ two-way left turn lane, otherwise 0)	
MJL	Major street left turn approach	 or 
MNLTR	Minor street shared left-through-right approach	
MNLR	Minor street shared left-right approach	
MNL	Minor street exclusive left turn lane	
MNR	Minor street exclusive right turn lane	

As the HCM method was found to consistently underestimate queue lengths, and two-minute rule consistently overestimates queue lengths, neither method should be used for two-way stop control queue length estimation. Either simulation or the models in Exhibit 12-19 may be used. Example 12-7 and Example 12-8 outline the step-by-step process of queue length estimation using developed models. The queues in this methodology represent the maximum queues for the peak 15-minute period which are an acceptable approximation of the 95th percentile queue length.

This procedure estimates the number of vehicles in queue. This number is multiplied by the appropriate average vehicle storage length obtained from Exhibit 12-20 to determine the queue length.

Exhibit 12-21 Storage Length Adjustments for Trucks

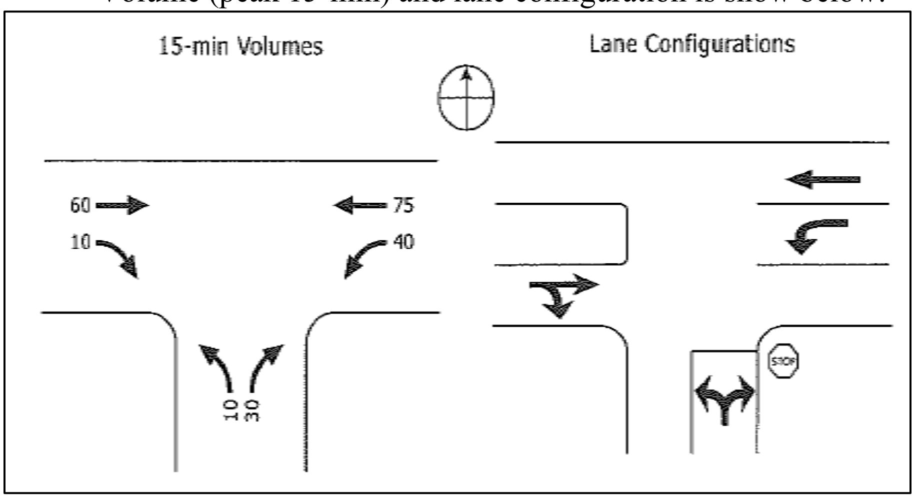
Percent Trucks in Turning Volume	Average Vehicle Storage Length
< 2%	25 ft
5%	27 ft
10%	29 ft

Example 12-7 Queue Length Estimation at a Three-legged Stop-controlled Intersection

This example demonstrates the application of the TPAU queue length estimation models at a three- legged Stop- controlled intersection.
(Source: Example Problem 1 of HCM 6 Chapter 32)

Data

- Volume (peak 15-min) and lane configuration is show below:



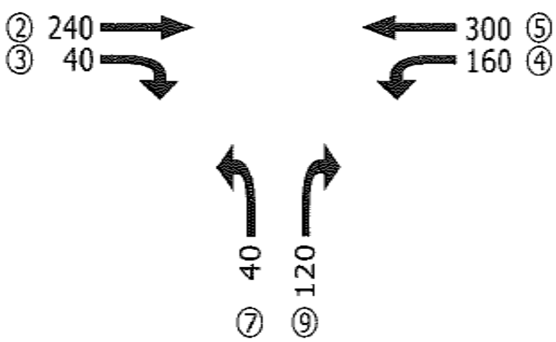
- Level grade on all approaches;
- Percent of heavy vehicles on all approaches is 10 percent;
- No flared approaches;
- No upstream signal;
- No pedestrians;
- Length of analysis is 1 hour (This example in HCM 6 uses 0.25 h);

Step1: Choose Lane Groups to Apply the Queue Length Models

- MNLR: North bound (minor approach)
- MJL: West bound TWLT lane

Steps 2 and 3: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities According HCM 6, Chapter 20 Methodology

Peak 15-min volume is multiplied by 4 to get the flow rate. Movement numbers are circled.



Step 4: Compute Conflicting Flow Rates (CONVOL) as per HCM 6 Equations 20-4 through 20-29

<u>WB MJL</u>

$V_{c,MJL} = V_{c,4}$

$V_{c,4} = V_2 + V_3$

$V_{c,4}$ = 240+40 = 280 veh/h

<u>NB MNLR</u>

$\mathrm{V}_{c,MNLR} = \mathrm{v}_{c,7} + \mathrm{v}_{c,9}$

$V_{c,7} = v_2 + 0.5v_3 + 2v_4 + v_5$

$V_{c,7}$ = 240+ 0.5(40)+ 2(160) + 300= 880 veh/h

$V_{c,9} = v_2 + 0.5v_3$

$V_{c,9}$ = 240 + 0.5(40) = 260 veh/h

$V_{c,MNLR}$ = 880+260 = 1140 veh/h

Step 5: Compute Queue Lengths using Models

WB MJL

VOL = 160 veh/h is within the range (0, 300]

CONVOL = 280 veh/h is within the range (0, 2000]

SIGNAL = 0 ; LT = 1

$QL = e^{(0.3925 + 0.0059 * VOL + 0.00104 * CONVOL + 0.49 * Signal - 0.81 * LT)}$

$QL = e^{(0.3925 + 0.0059 * 160 + 0.00104 * 280 + 0.49 * 0 - 0.81 * 1)}$

QL = 2.3 ≈ 3 vehicles

NB MNLR

VOL = 160 veh/h is within the range (0, 300]

CONVOL = 1140 veh/h is within the range (0, 3000]

$QL = e^{(-0.6319+0.0173*VOL+0.00066*CONVOL-0.000007913* VOL* CONVOL)}$

$QL = e^{(-0.6319 + 0.0173*160 + 0.00066*1140 -0.000007913* 160* 1140)}$

QL = 4.2 ≈ 5 vehicles

Summary

Maximum QL for WB LT = 3 Vehicles

Maximum QL for NB approach = 5 Vehicles

Estimates from other queue length models are presented below:

Method	Queue Length for WB LT (veh)	Queue Length for NB (veh)
HCM	2	1
Two-minute Rule	10	10
QL Model	3	5

Based on the percentage of trucks in the traffic stream, queue lengths (number of queued vehicles) from models are converted to feet using Exhibit 12-20.

From the above heavy vehicle percentage conversion table, the queue lengths for:

WB LT = 3 x 29 ft. = 87 ft. ≈ 100 ft.

NB approach = 5 x 29 ft. =145 ft. ≈ 150 ft.

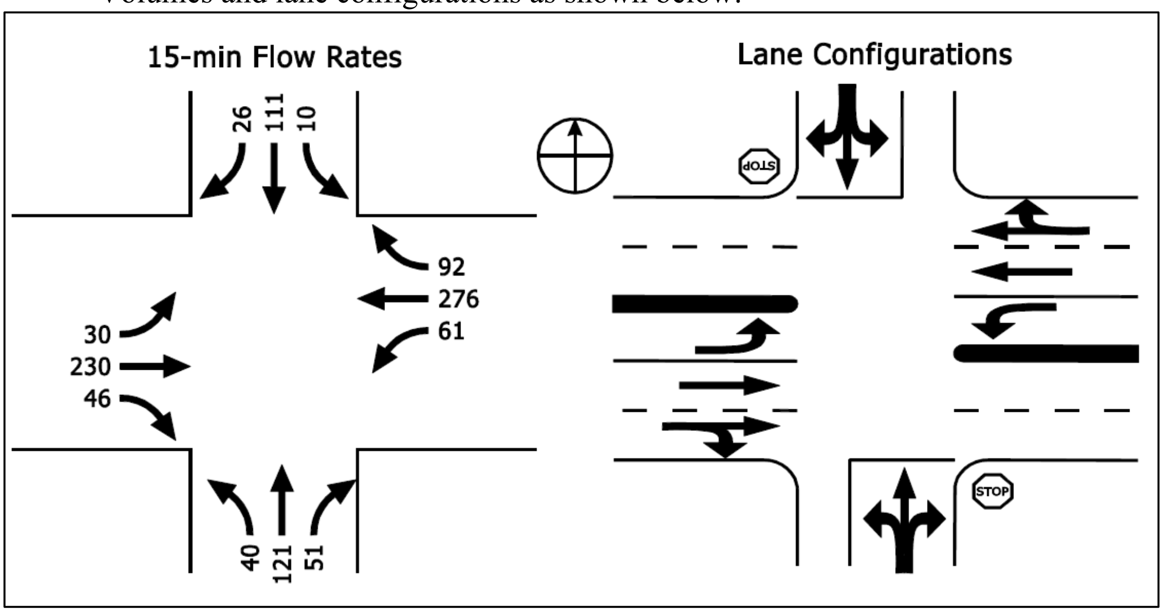
Example 12-8 Queue Length Estimation at a Four-legged Two-Way Stop-controlled Intersection

This example demonstrates the application of the TPAU queue length estimation models at a four-legged two-way STOP controlled intersection.

(Source: Example Problem 3 of 2010 HCM Chapter 32, Page 32-7)

Data

- Volumes and lane configurations as shown below:



- Major street with two lanes in each direction, minor street with one lane on each approach that flares with storage for one vehicle in the flare area, and median storage for two vehicles at one time available for minor-street through and left-turn movements;
- Level grade on all approaches;

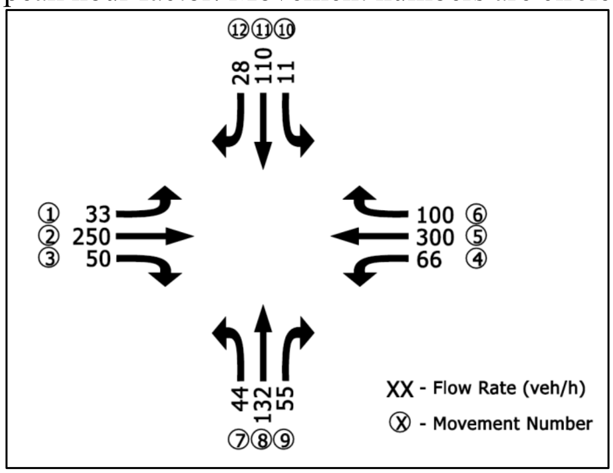
- Percent heavy vehicles on all approaches = 10%;
- Peak hour factor on all approaches = 0.92;
- Length of analysis period = 1.0 h.

Step1: Choose Lane Groups to Apply the Queue Length Models

- MNLTR – NB and SB
- MJL – EB and WB

Steps 2 and 3: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities According HCM 6, Chapter 20 Methodology

For each movement, peak hour flow rate is obtained by dividing the hourly volume by the peak hour factor. Movement numbers are circled.



Step 4: Compute Conflicting Flow Rates (CONVOL) as per HCM 6 Equations 19-4 through 19-29

<u>EB MJL</u>

$v_{c,1} = v_5 + v_6 + v_{16}$ = 300 + 100 + 0 = 400 veh/h

<u>WB MJL</u>

$v_{c,4} = v_2 + v_3 + v_{15}$ = 250 + 50 + 0 = 300 veh/h

<u>NB MNLTR</u>

$v_{c,NB} = v_{c,7} + v_{c,8} + v_{c,9}$

$v_{c,7} = v_{c,I,7} + v_{c,II,7}$ (two-stage gap acceptance)

$v_{c,I,7} = 2(v_1 + v_{1u}) + v_2 + 0.5v_3 + v_{15}$

= 2(33+0) + 250 + 0.5(50) + 0 = 341 veh/h

$v_{c,II,7} = 2(v_4 + v_{4u}) + 0.5v_5 + 0.5v_{11} + v_{13}$

= 2(66+0) + 0.5(300) + 0.5(110) + 0 = 337 veh/h

$v_{c,7} = v_{c,I,7} + v_{c,II,7} =$ 341 + 337 = 678 veh/h

$v_{c,8} = v_{c,I,8} + v_{c,II,8}$ (two-stage gap acceptance)

$v_{c,I,8} = 2(v_1 + v_{1u}) + v_2 + 0.5v_3 + v_{15}$

= 2(33+0) + 250 + 0.5(50) + 0 = 341 veh/h

$v_{c,II,8} = 2(v_4 + v_{4u}) + v_5 + v_6 + v_{16}$

= 2(66+0) + 300 + 100 + 0 = 532 veh/h

$v_{c,8} = v_{c,I,8} + v_{c,II,8} =$ 341 + 532 = 873 veh/h

$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{4U} + v_{14} + v_{15}$

= 0.5(250) + 0.5(50) + 0 + 0 + 0 = 150 veh/h
$v_{c,NB} = v_{c,7} + v_{c,8} + v_{c,9}$ = 678 + 873 + 150 = 1701 veh/h
SB MNLTR
$v_{c,SB} = v_{c,10} + v_{c,11} + v_{c,12}$
$v_{c,10} = v_{c,I,10} + v_{c,II,10}$ (two-stage gap acceptance)
$v_{c,I,10} = 2(v_4 + v_{4u}) + v_5 + 0.5v_6 + v_{16}$
= 2(66+0) + 300 + 0.5(100) + 0 = 482 veh/h
$v_{c,II,10} = 2(v_1 + v_{1u}) + 0.5v_2 + 0.5v_8 + v_{14}$
= 2(33+0) + 0.5(250) + 0.5(132) + 0 = 257 veh/h
$v_{c,10} = v_{c,I,10} + v_{c,II,10} =$ 482 + 257 = 739 veh/h
$v_{c,11} = v_{c,I,11} + v_{c,II,11}$ (two-stage gap acceptance)
$v_{c,I,11} = 2(v_4 + v_{4u}) + v_5 + 0.5v_6 + v_{16}$
= 2(66+0) + 300 + 0.5(100) + 0 = 482 veh/h
$v_{c,II,11} = 2(v_1 + v_{1u}) + v_2 + v_3 + v_{15}$
= 2(33+0) + 250 + 50 + 0 = 366 veh/h
$v_{c,11} = v_{c,I,11} + v_{c,II,11} =$ 482 + 366 = 848 veh/h
$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{1U} + v_{13} + v_{16}$
= 0.5(300) + 0.5(100) + 0 + 0 + 0 = 200 veh/h
$v_{c,SB} = v_{c,10} + v_{c,11} + v_{c,12}$ = 739 + 848 + 200 = 1787 veh/h

Step 5: Compute Queue Lengths using Models
EB MJL
VOL = 33 veh/h is within the range (0, 300]
CONVOL = 400 veh/h is within the range (0, 2000]
SIGNAL = 0 ; LT = 1
QL= $e^{(0.3925 + 0.0059 * VOL + 0.00104 * CONVOL + 0.49 * Signal - 0.81 * LT)}$
QL= $e^{(0.3925 + 0.0059 * 33 + 0.00104 * 400 + 0.49 * 0 - 0.81 * 1)}$
QL = 1.2 ≈ 2 veh

WB MJL
VOL = 66 veh/h is within the range (0, 300]
CONVOL = 300 veh/h is within the range (0, 2000]
SIGNAL = 0 ; LT = 1
QL= $e^{(0.3925 + 0.0059 * VOL + 0.00104 * CONVOL + 0.49 * Signal - 0.81 * LT)}$
QL= $e^{(0.3925 + 0.0059 * 66 + 0.00104 * 300 + 0.49 * 0 - 0.81 * 1)}$
QL = 1.3 ≈ 2 veh

NB MNLTR
VOL = 231 veh/h is within the range (0, 300]
CONVOL = 1701 veh/h is within the range (0, 3000]
QL= $e^{(-0.7844 + 0.01636 * VOL + 0.0006 * CONVOL - 0.0000043 * VOL * CONVOL)}$
QL= $e^{(-0.7844 + 0.01636 * 231 + 0.0006 * 1701 - 0.0000043 * 231 * 1701)}$
QL = 10.2 ≈ 11 veh

SB MNLTR
VOL = 149 veh/h is within the range (0, 300]
CONVOL = 1787 veh/h is within the range (0, 3000]
$QL = e^{(-0.7844 + 0.01636 * VOL + 0.0006 * CONVOL - 0.0000043 * VOL * CONVOL)}$
$QL = e^{(-0.7844 + 0.01636 * 149 + 0.0006 * 1787 - 0.0000043 * 149 * 1787)}$
QL = 4.8 ≈ 5 veh

Summary
Maximum QL for
EB LT = 2 veh
WB LT = 2 veh
NB approach = 11 Veh
SB approach = 5 Veh
Estimates from other queue length models are presented below:

Method	EB LT (Veh)	WB LT (Veh)	NB (Veh)	SB (Veh)
HCM	0	0	6	3
Two-minute Rule	2	4	15	10
QL Model	2	2	11	5

Based on the percentage of trucks in the traffic stream, queue lengths (number of queued vehicles) from models are converted to feet using Exhibit 12-20.
From the above heavy vehicle percentage conversion table, the queue lengths for:
EB LT = 2 x 29ft. = 58 ft. ≈ 75 ft.
WB LT ≈ 75 ft.
NB approach = 11 x 29 ft. = 319 ft. ≈ 325 ft.
SB approach = 5 x 29ft. = 145 ft. ≈ 150 ft.

Simulation

If simulation is being performed as part of the analysis, queue lengths should be taken from the simulation results. If simulation is not being done, it should be considered especially if the v/c ratios are approaching 0.90. If the effort to do a simulation analysis is not desired, the TPAU queue length estimation models should be used. Refer to APM version 1 Chapter 8 for simulation procedures.

Two-Minute Rule

The Two-Minute Rule is a rule of thumb methodology that shall only be used for sketch planning level analysis or for lane groups not addressed in the TPAU method. This method estimates queue lengths for major street left turns and minor street movements by using the queue that would result from a two-minute stoppage of the turning demand volume. This method does not consider the magnitudes and impacts of the conflicting flows on the size of the queue. The calculation of the 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 12-21)
L = average length of the vehicles being stored and the gap between vehicles; 25 ft. for cars. This value can be increased where a significant number of trucks are present in the turning volume using the same relationship between average vehicle storage length and percent trucks in turning volumes shown for the signalized movement rule of thumb method discussed earlier in this chapter.

Exhibit 12-22 Selection of "t" Values

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

Appendix 12A/13A – Software and Settings for Intersection Analysis

References

(i) Robinson, Bruce W., et al. *Roundabouts: An Informational Guide*. No. FHWA-RD-00-067. 2000.