Capacity and Level of Service at Signalized Intersections

1. Introduction

The level of service at any intersection on a highway has a significant effect on the overall operating performance of that highway. Thus, improvement of the level of service at each intersection usually results in an improvement of the overall operating performance of the highway. An analysis procedure that provides for the determination of capacity or level of service at intersections is therefore an important tool for designers, operation personnel, and policy makers. Factors that affect the level of service at intersections include the flow and distribution of traffic, the geometric characteristics, and the signalization system.

A major difference between consideration of level of service on highway segments and level of service at intersections is that only through flows are used in computing the levels of service at highway segments as discussed in previous lectures for freeway segment, whereas turning flows are significant when computing the levels of service at signalized intersections. The signalization system (which includes the allocation of time among the conflicting movements of traffic and pedestrians at the intersection) is also an important factor.

For example, the distribution of green times among these conflicting flows significantly affects both capacity and operation of the intersection. Other factors such as lane widths, traffic composition, grade, and speed also affect the level of service at intersections in a similar manner as for highway segments.

2. Definitions

1. **Permitted turning movements** are those made within gaps of an opposing traffic stream or through a conflicting pedestrian flow. For example, when a right turn is made while pedestrians are crossing a conflicting crosswalk, the right turn is a permitted turning movement. Similarly, when a left turn is made between two consecutive vehicles of the opposing traffic stream, the left turn is a permitted turn. The suitability of permitted turns at a given intersection depends on the geometric characteristics of the intersection, the turning volume, and the opposing volume.

2. **Protected turns** are those turns protected from any conflicts with vehicles in an opposing stream or pedestrians on a conflicting crosswalk. A permitted turn takes more time than a similar protected turn and will use more of the available green time.

3. **Change and clearance interval** is the sum of the "yellow" and "all-red" intervals (given in seconds) that are provided between phases to allow vehicular and pedestrian traffic to clear the intersection before conflicting movements are released.

4. **Geometric conditions** is a term used to describe the roadway characteristics of the approach. They include the number and width of lanes, grades, and the allocation of the lanes for different uses, including the designation of a parking lane.
5. **Signalization conditions** is a term used to describe the details of the signal operation. They include the type of signal control, phasing sequence, timing, and an evaluation of signal progression on each approach.
6. **Flow ratio (v/s)** is the ratio of the actual flow rate or projected demand v on an approach or lane group to the saturation flow rate s.
7. **Lane group** consists of one or more lanes that have a common stop line, carry a set of traffic streams, and whose capacity is shared by all vehicles in the group.

Capacity at Signalized Intersections

The capacity at a signalized intersection is given for each lane group and is defined as the maximum rate of flow for the subject lane group that can go through the intersection under prevailing traffic, roadway, and signalized conditions. Capacity is given in vehicles per hour (veh/h) but is based on the flow during a peak 15-minute period.
The capacity of the intersection as a whole is not considered; rather, emphasis is placed on providing suitable facilities for the major movements of the intersections.
Capacity therefore is applied meaningfully only to major movements or approaches of the intersection. Note also that in comparison with other locations such as freeway segments, the capacity of an intersection approach is not as strongly correlated with the level of service. It is therefore necessary that both the level of service and capacity be analyzed separately when signalized intersections are being evaluated.

Saturation Flow or Saturation Flow Rate

The concept of a saturation flow or saturation flow rate (s) is used to determine the capacity of a lane group. The saturation flow rate is the maximum flow rate on the approach or lane group that can go through the intersection under prevailing traffic and roadway conditions when 100 percent effective green time is available. The saturation flow rate is given in units of veh/h of effective green time.

The capacity of an approach or lane group is given as:

$$c_i = s_i(g_i/C)$$

where

c_i = capacity of lane group i (veh/h)
s_i = saturation flow rate for lane group or approach i
(g_i/C) = green ratio for lane group or approach i
g_i = effective green for lane group i or approach i
C = cycle length

The ratio of flow to capacity (v/c) is usually referred to as the degree of saturation and can be expressed as:

$$(v/c)_i = X_i = \frac{v_i}{s_i(g_i/c)}$$

where

v_i = Flow rate for lane i

X_i = (v/c) ratio for lane group or approach i
v_i = actual flow rate or projected demand for lane group or approach i (veh/h)
s_i = saturation flow for lane group or approach i (veh/h/g)
g_i = effective green time for lane group i or approach i (sec)

It can be seen that when the flow rate equals capacity, X_i equals 1.00; when flow rate equals zero, X_i equals zero.
When the overall intersection is to be evaluated with respect to its geometry and total cycle time, the concept of critical volume-to-capacity ratio (X_c) is used. The critical (v/c) ratio is usually obtained for the overall intersection but considers only the critical lane groups or approaches which are those lane groups or approaches that have the maximum flow ratio (v/s) for each signal phase. For example, in a two-phase signalized intersection, if the north approach has a higher (v/s) ratio than the south approach, more time will be required for vehicles on the north approach to go through the intersection during the north−south green phase, and the phase length will be based on the green time requirements for the north approach. The north approach will therefore be the critical approach for the north−south phase. The critical v/c ratio for the whole intersection is given as:

$$X_c = \sum_i (v/s)_{ci} \frac{C}{C - L}$$

where

X_c = critical v/c ratio for the intersection
$\sum_i (v/s)_{ci}$ = summation of the ratios of actual flows to saturation flow (flow ratios) for all critical lanes, groups, or approaches
C = cycle length (sec)
L = total lost time per cycle computed as the sum of the lost time, (t_ℓ), for each critical signal phase, $L = \sum_i t_\ell$

Equation above can be used to estimate the signal timing for the intersection if this is unknown and a critical (v/c) ratio is specified for the intersection. Alternatively, this equation can be used to obtain a broader indicator of the overall sufficiency of the intersection by substituting the maximum permitted cycle length for the jurisdiction and determining the resultant critical (v/c) ratio for the intersection. When the critical (v/c) ratio is less than 1.00, the cycle length provided is adequate for all critical movements to go through the intersection if the green time is proportionately distributed to the different phases, that is, for the assumed phase sequence, all movements in the intersection will be provided with adequate green times if the total green time is proportionately divided among all phases. If the total green time is not properly allocated to the different phases, it is possible to have a critical (v/c) ratio of less than 1.00 but with one or more individual oversaturated movements within a cycle.

Level of Service at Signalized Intersections

The procedures for determining the Level of Service (LOS) at an intersection can be used for either a detailed or operational evaluation of a given intersection or a general planning estimate of the overall performance of an existing or planned signalized intersection. At the design level of analysis, more input data are required for a direct estimate of the level of service to be made. It is also possible at this level of analysis to determine the effect of changing signal timing.

The procedures presented here for the operational evaluation are those given in the 2000 edition of the Highway Capacity Manual. These procedures deal with the computation of the level of service at the intersection approaches and the level of service at the intersection as a whole. Control delay is used to define the level of service at signalized intersections since delay not only indicates the amount of lost travel time and fuel consumption but it is also a measure of the frustration and discomfort of motorists. Control or signal delay, which is that portion of total delay that is attributed to the control facility, is computed to define the level of service at the signalized intersection. This includes the delay due to the initial deceleration, queue move up time, stopped time, and final acceleration. Delay, however, depends on the red time, which in turn depends on the length of the cycle. Reasonable levels of service can therefore be obtained for short cycle lengths, even though the (v/c) ratio is as high as 0.9. To the extent that signal coordination reduces delay, different levels of service may also be obtained for the same (v/c) ratio when the effect of signal coordination changes.

Operational Analysis Procedure

The procedure at the operation level of analysis can be used to determine the capacity or level of service at the approaches of an existing signalized intersection or the overall level of service at an existing intersection. The procedure also can be used in the detailed design of a given intersection. In using the procedure to analyze an existing signal, operational data such as phasing sequence, signal timing, and geometric details (lane widths, number of lanes) are known. The procedure is used to determine the level of service at which the intersection is performing in terms of control or signal delay. In using the procedure for detailed design, the operational data usually are not known and therefore have to be computed or assumed. The delay and level of service are then determined.

The LOS criteria are given in terms of the average control delay per vehicle during an analysis period of 15 minutes. Six levels of service are prescribed. The criteria for each are described below and are shown in Table below:

Table: Level-of-Service Criteria for Signalized Intersections.

Level of Service	*Control Delay Per Vehicle (sec)*
A	≤ 10.0
B	> 10.0 and ≤ 20.0
C	> 20.0 and ≤ 35.0
D	> 35.0 and ≤ 55.0
E	> 55.0 and ≤ 80.0
F	> 80.0

Level of Service A describes that level of operation at which the average delay per vehicle is 10.0 seconds or less. At LOS A, vehicles arrive mainly during the green phase, resulting in only a few vehicles stopping at the intersection. Short cycle lengths may help in obtaining low delays.

Level of Service B describes that level of operation at which delay per vehicle is greater than 10 seconds but not greater than 20 seconds. At LOS B, the number of vehicles stopped at the intersection is greater than that for LOS A, but progression is still good, and cycle length also may be short.

Level of Service C describes that level of operation at which delay per vehicle is greater than 20 seconds but not greater than 35 seconds. At LOS C, many vehicles go through the intersection without stopping, but a significant number of vehicles are stopped. In addition, some vehicles at an approach will not clear the intersection during the first cycle (cycle failure). The higher delay may be due to the significant number of vehicles arriving during the red phase (fair progression) and/or relatively long cycle lengths.

Level of Service D describes that level of operation at which the delay per vehicle is greater than 35 seconds but not greater than 55 seconds. At LOS D, more vehicles are stopped at the intersection, resulting in a longer delay. The number of individual cycles failing is now noticeable. The longer delay at this level of service is due to a combination of two or more of several factors that include long cycle lengths, high (v/c) ratios, and unfavorable progression.

Level of Service E describes that level of operation at which the delay per vehicle is greater than 55 seconds but not greater than 80 seconds. At LOS E, individual cycles frequently fail. This long delay, which is usually taken as the limit of acceptable delay by many agencies, generally includes high (v/c) ratios, long cycle lengths, and poor progression.

Level of Service F describes that level of operation at which the delay per vehicle is greater than 80 seconds. This long delay is usually unacceptable to most motorists. At LOS F, oversaturation usually occurs–that is, arrival flow rates are greater than the capacity at the intersection. Long delay can also occur as a result of poor progression and long cycle lengths. Note that this level of service can occur when approaches have high (v/c) ratios which are less than 1.00 but also have many individual cycles failing.

It should be emphasized once more that, in contrast to other locations, the level of service at a signalized intersection does not have a simple one-to-one relationship with capacity. For example, at freeway segments, the (v/c) ratio is 1.00 at the upper limits of LOS E. At the signalized intersection, however, it is possible for the delay to be unacceptable at LOS F although the (v/c) ratio is less than 1.00 and even as low as 0.75. When long delays occur at such (v/c) ratios, it may be due to a combination of two or more of the following conditions.

- Long cycle lengths
- Green time is not properly distributed, resulting in a longer red time for one or more lane groups−that is, there is one or more disadvantaged lane group
- A poor signal progression, which results in a large percentage of the vehicles on the approach arriving during the red phase It is also possible to have short delays at an approach when the (v/c) ratio equals
- 1.00−that is, saturated approach−which can occur if the following conditions exist.
- Short cycle length
- Favorable signal progression, resulting in a high percentage of vehicles arriving during the green phase

Clearly, LOS F does not necessarily indicate that the intersection, approach, or lane group is oversaturated, nor can it automatically be assumed that the demand flow is below capacity for a LOS range of A to E. It is therefore imperative that both the capacity and LOS analyses be carried out when a signalized intersection is to be fully evaluated.

<u>Methodology of Operation Analysis Procedure</u>

The tasks involved in an operational analysis are presented in the flow chart shown in Figure below. The tasks have been divided into five modules:
(1) input parameters,
(2) lane grouping and demand flow rate,
(3) saturation flow rate,
(4) capacity analysis v/c, and
(5) performance measures.
Each of these modules will be discussed in turn, including a detailed description of each task involved.
Input Parameters
This module involves the collection and presentation of the data that will be required for the analysis. The tasks involved are

- Identifying and recording the geometric characteristics.
- Identifying and specifying the traffic conditions.
- Specifying the signalized conditions.

Table below gives the input data required for each analysis lane group.

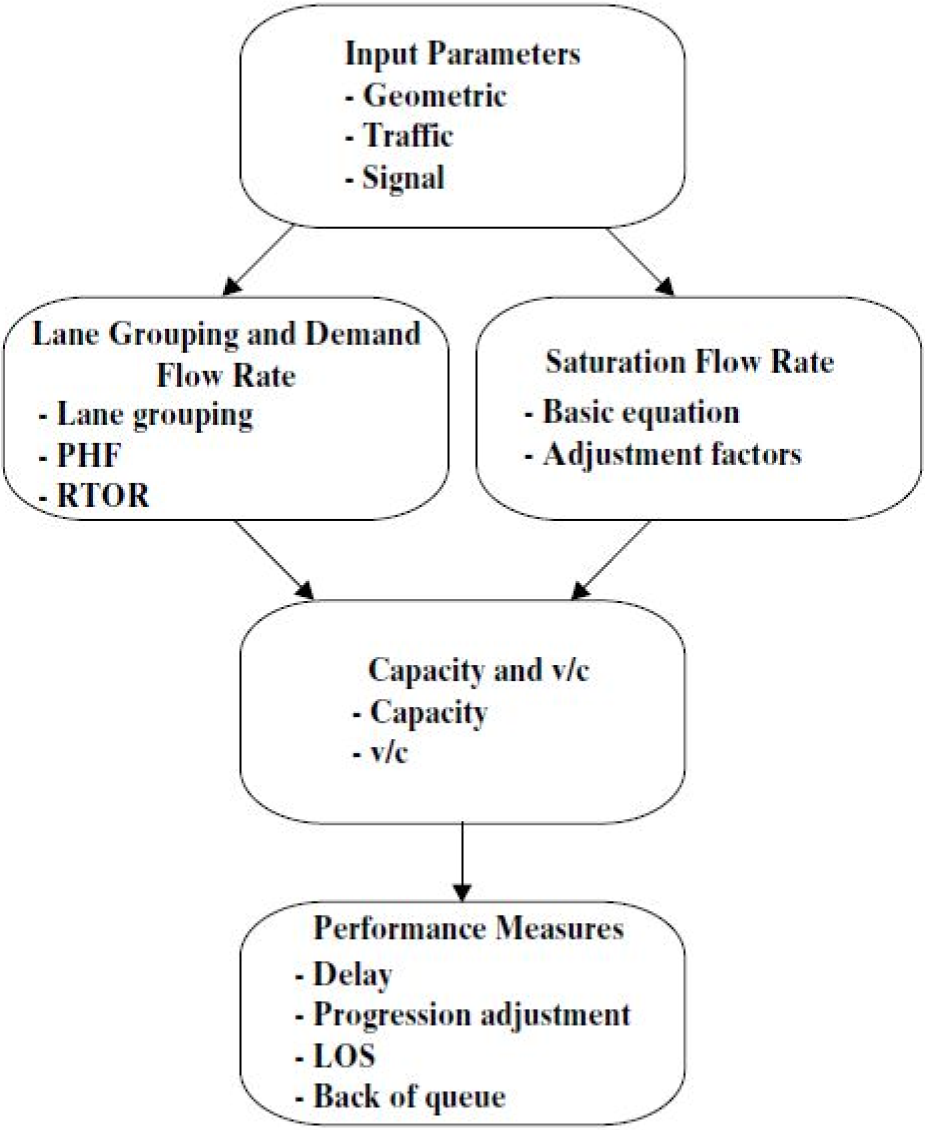


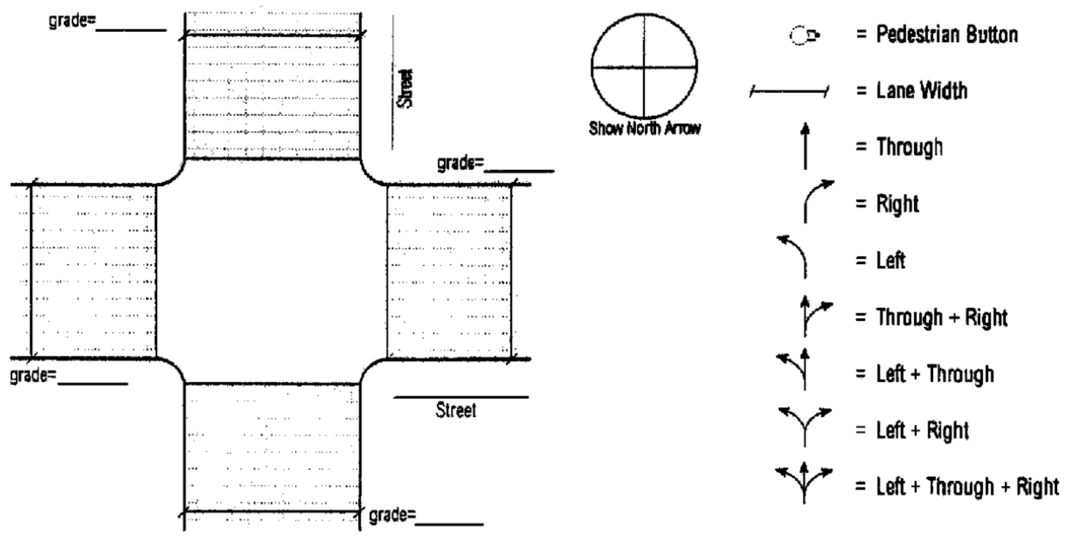
Figure: Flow Chart for Operation Analysis Procedure.

Recording Geometric Characteristics. The physical configuration of the intersection is obtained in terms of the number of lanes, lane width, grade, movement by lane, parking locations, lengths of storage bays, and so forth and is recorded on the appropriate form shown in Figure below. In cases where the physical configuration of the intersection is unknown, the planning level of analysis may be used to determine a suitable configuration or state and local policies and/or guidelines can be used. If no guidelines are available.

INPUT WORKSHEET

General Information		Site Information	
Analyst	________	Intersection	________
Agency or Company	________	Area Type	☐ CBD ☐ Other
Date Performed	________	Jurisdiction	________
Analysis Time Period	________	Analysis Year	________

Intersection Geometry



Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]
Volume, V (veh/h)												
% heavy vehicles, % HV												
Peak-hour factor, PHF												
Pretimed (P) or actuated (A)												
Start-up lost time, l_1 (s)												
Extension of effective green time, e (s)												
Arrival type, AT												
Approach pedestrian volume,[2] v_{ped} (p/h)												
Approach bicycle volume,[2] v_{bic} (bicycles/h)												
Parking (Y or N)												
Parking maneuvers, N_m (maneuvers/h)												
Bus stopping, N_B (buses/h)												
Min. timing for pedestrians,[3] G_p (s)												

Signal Phasing Plan

	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
DIAGRAM								
Timing	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =

Protected turns	Permitted turns Pedestrian	Cycle length, C = _____ s

Notes

1. RT volumes, as shown, exclude RTOR.
2. Approach pedestrian and bicycle volumes are those that conflict with right turns from the subject approach.
3. Refer to Equation 8-11 & 8-12

Figure: Input Worksheet for Operation Level of Analysis.

Recording Traffic Conditions. This phase involves the recording of bicycle, pedestrian, and vehicular hourly volumes on the appropriate cell of the form shown in previous Figure. Pedestrian and bicycle volumes are recorded such that those that conflict with a given stream of right-turning vehicles are in the same direction as the conflicting right-turning vehicles. For example, pedestrians on the north crosswalk will conflict with the westbound (WB) right-turning vehicles and should be recorded in the WB row of the form. Similarly, pedestrians on the east crosswalk will conflict with the northbound (NB) right-turning vehicles and should be recorded in the NB row of the form. The traffic volumes are the flow rates (equivalent hourly volumes) for the analysis period, which is usually taken as 15 minutes (T=0.25). This flow rate also may be computed from the hourly volumes and the peak-hour factors. Control delay is significantly influenced by the length of the analysis period where v/c is greater than 0.9. Therefore, when v/c is greater than 0.9 and the 15-minute flow rate is relatively constant for periods longer than 15 minutes, the analysis period (T) in hours should be the length of time the flow remains relatively constant. In cases of oversaturation (v/c > 1) in which the flow rate remains relatively constant, the analysis period should be extended to cover the period of oversaturation. However, when the resulting analysis period is longer than 15 minutes and different flow rates are observed during sub-periods of equal length within the longer analysis period, a special multiple period analysis should be conducted. Details of traffic volume should include the percentage of heavy vehicles(%HV)in each movement, where heavy vehicles are defined as all vehicles having more than four tires on the pavement. In recording the number of buses, only buses that stop to pick up or discharge passengers on either side of the intersection are included. Buses that go through the intersection without stopping to pick up or discharge passengers are considered heavy vehicles.
The level of coordination between the lights at the intersection being studied and those at adjacent intersections is a critical characteristic and is determined in terms of the type of vehicle arrival at the intersection. Six arrival types (AT) have been identified:

- **Arrival Type 1**, which represents the worst condition of arrival, is a dense platoon containing over 80 percent of the lane group volume arriving at the beginning of the red phase.
- **Arrival Type 2**, which while better than Type 1, is still considered unfavorable, is either a dense platoon arriving in the middle of the red phase or a dispersed platoon containing 40 to 80 percent of the lane group volume arriving throughout the red phase.
- **Arrival Type 3**, which usually occurs at isolated and noninter connected intersections, is characterized by highly dispersed platoons and entails the random arrival of vehicles in which the main platoon contains less than 40 percent of the lane group volume. Arrivals at coordinated intersections with minimum benefits of progression also may be described by this arrival type.
- **Arrival Type 4**, which is usually considered a favorable platoon condition, is either a moderately dense platoon arriving in the middle of a green phase or a dispersed platoon containing 40 to 80 percent of the lane group volume arriving throughout the green phase.

- **Arrival Type 5**, which represents the best condition of arrival that usually occurs, is a dense platoon containing over 80 percent of the lane group volume arriving at the start of the green phase.
- **Arrival Type 6**, which represents exceptional progression quality, is a very dense platoon progressing through several closely spaced intersections with very low traffic from the side streets.

It is necessary to determine, as accurately as possible, the type of arrival for the intersection being considered, since both the estimate of delay and the determination of the level of service will be significantly affected by the arrival type used in the analysis. Field observation is the best way to determine the arrival type, although time−space diagrams for the street being considered could be used for an approximate estimation. In using field observations, the percentage of vehicles arriving during the green phase is determined and the arrival type is then obtained for the platoon ratio for the approach. The HCM gives the platoon ratio as:

$$R_p = P(C/g)$$

where:
R_p = platoon ratio.
P= proportion of all vehicles in the movement arriving during the green phase.
C = cycle length (sec).
g= effective green time for the movement (sec).

The arrival type is obtained from Table below which gives a range of platoon ratios and progression quality for each arrival type.
The number of parking maneuvers at the approach is another factor that influences capacity and level of service of the approach. A parking maneuver is when a vehicle enters or leaves a parking space. The number of parking maneuvers adjacent to an analysis lane group is given as the number of parking maneuvers per hour (Nm) that occur within 250 ft upstream of the intersection.

Table: Relationship Between Arrival Type and Platoon Ratio (R_p).

Arrival Type	*Range of Platoon Ratio (R_p)*	*Default Value (R_p)*	*Progression Quality*
1	≤ 0.50	0.333	Very poor
2	> 0.50 and ≤ 0.85	0.667	Unfavorable
3	> 0.85 and ≤ 1.15	1.000	Random arrivals
4	> 1.15 and ≤ 1.50	1.333	Favorable
5	> 1.50 and ≤ 2.00	1.667	Highly favorable
6	> 2.00	2.000	Exceptional

Specifying Signalized Conditions. Details of the signal system should be specified, including a phase diagram and the green, yellow, and red cycle lengths. The phasing scheme at an intersection determines which traffic stream or streams are given the right of way at the intersection, and therefore has a significant effect on the level of service. A poorly designed phasing scheme may result in unnecessary delay. Different phasing plans for pretimed and actuated signals.

Lane Grouping and Demand Flow Rate

Three main tasks are involved in this module: identifying the different lane groups, adjusting hourly volumes to peak 15-minute flow rates using the PHF, and adjusting for a right turn on red (RTOR). Figure below shows a worksheet that can be used for this module.

Identifying the Different Lane Groups

The lane groups at each approach must be identified as the HCM methodology considers each approach at the intersection and individual lane groups on each approach. However, when a lane group includes a lane that is shared by left-turning and straight through vehicles (shared lane), it is necessary to determine whether the shared lane is operating as a de facto left-turn lane. The shared lane is considered a de facto left lane if the computed proportion of left turns in the shared lane is 1.0 (i.e., 100%).

Adjustment of Hourly Volumes

Earlier, we saw that the analysis for level of service requires that flow rates be based on the peak 15-minute flow rate. It is therefore necessary to convert hourly volumes to 15-minute flow rates by dividing the hourly volumes by the peak hour factor (PHF). Note that although not all movements of an approach may peak at the same time, dividing all hourly volumes by a single PHF assumes that the peaking occurs for all movements at the same time, which is a conservative assumption.

Adjustment for Right-Turn-on-Red (RTOR)

The right-turn volume in a lane group may be reduced by the volume of vehicles turning right during the red phase. rates. It is also recommended to consider field data on the number of vehicles turning right during the red phase when existing intersections are being considered. When a future intersection is being considered, it is suggested that the total right-turn volumes be used in the analysis since it is very difficult to estimate the number of right-turning vehicles that move on the red phase. An exception to this is when an exclusive left turn phase for the cross street "shadows" the right-turn-lane movement. For example, an eastbound exclusive left-turn phase will "shadow" the southbound right-turning vehicles. In such a case, the shadowing left-turn volume per lane may be used as the volume of right-turning vehicles that move on the red phase and can be deducted from the right-turn volumes. Also, right turns that are free-flowing and not controlled by the signal are not included in the analysis.

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET

General Information

Project Description____

Volume Adjustment

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)												
Peak-hour factor, PHF												
Adjusted flow rate, v_p = V/PHF (veh/h)												
Lane group												
Adjusted flow rate in lane group, v (veh/h)												
Proportion[1] of LT or RT (P_{LT} or P_{RT})		-			-			-			-	

Saturation Flow Rate

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Base saturation flow, s_o (pc/h/ln)												
Number of lanes, N												
Lane width adjustment factor, f_w												
Heavy-vehicle adjustment factor, f_{HV}												
Grade adjustment factor, f_g												
Parking adjustment factor, f_p												
Bus blockage adjustment factor, f_{bb}												
Area type adjustment factor, f_a												
Lane utilization adjustment factor, f_{LU}												
Left-turn adjustment factor, f_{LT}												
Right-turn adjustment factor, f_{RT}												
Left-turn ped/bike adjustment factor, f_{Lpb}												
Right-turn ped/bike adjustment factor, f_{Rpb}												
Adjusted saturation flow, s (veh/h) $s = s_o N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$												

Notes

1. P_{LT} = 1.000 for exclusive left-turn lanes, and P_{RT} = 1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group.

Figure: Volume Adjustment and Saturation Flow Rate Worksheet.

Saturation Flow Rate

This module provides for the computation of a saturation flow rate for each lane group. The saturation flow rate is defined as the flow rate in veh/h that the lane group can carry if it has the green indication continuously, that is, if $g/C = 1$.

Base Equation for Saturation Flow Rate

The saturation flow rate (s) depends on an ideal saturation flow (s_0), which is usually taken as 1900 passenger cars/h of green time per lane. This ideal saturation flow is then adjusted for the prevailing conditions to obtain the saturation flow for the lane group being considered. The adjustment is made by introducing factors that correct for the number of lanes, lane width, the percent of heavy vehicles in the traffic, approach grade, parking activity, local buses stopping within the intersection, area type, lane utilization factor, and right and left turns.

The HCM gives the saturation flow as:

$$s = (s_o)(N)(f_w)(f_{HV})(f_g)(f_p)(f_a)(f_{bb})(f_{Lu})(f_{RT})(f_{LT})(f_{Lpb})(f_{Rpb})$$

where

s = saturation flow rate for the subject lane group, expressed as a total for all lanes in lane group under prevailing conditions (veh/h/g)

s_o = ideal saturation flow rate per lane, usually taken as 1900 (veh/h/ln)

N = number of lanes in lane group

f_w = adjustment factor for lane width

f_{HV} = adjustment factor for heavy vehicles in the traffic stream

f_g = adjustment factor for approach grade

f_p = adjustment factor for the existence of parking lane adjacent to the lane group and the parking activity on that lane

f_a = adjustment factor for area type (for CBD, 0.90; for all other areas, 1.00)

f_{bb} = adjustment factor for the blocking effect of local buses stopping within the intersection area

f_{Lu} = adjustment factor for lane utilization

f_{RT} = adjustment factor for right turns in the lane group

f_{LT} = adjustment factor for left turns in the lane group

f_{Lpb} = pedestrian adjustment factor for left-turn movements

f_{Rpb} = pedestrian adjustment factor for right-turn movements

Adjustment Factors

Although the necessity for using some of these adjustment factors, the basis for using each of them is given again here to facilitate comprehension of the material. The equations which are used to determine the factors are given in Table below:

Table: Adjustment Factors for Saturation Flow $Rates^a$

Factor	*Formula*	*Definition of Variables*	*Notes*
Lane width	$f_w = 1 + \frac{(W-12)}{30}$	W = lane width (ft)	$W \geq 8.0$ If $W > 16$, two-lane analysis may be considered
Heavy vehicles	$f_{HV} = \frac{100}{100 + \%HV(E_T - 1)}$	$\%HV$ = percent heavy vehicles for lane group volume	$E_T = 2.0$ pc/HV
Grade	$f_g = 1 - \frac{\%G}{200}$	$\%G$ = percent grade on a lane group approach	$-6 \leq \%G \leq +10$ Negative is downhill
Parking	$f_p = \frac{N - 0.1 - \frac{18N_m}{3600}}{N}$	N = number of lanes in lane group N_m = number of parking maneuvers/h	$0 \leq N_m \leq 180$ $f_p = \geq 0.050$ $f_p = 1.000$ for no parking
Bus blockage	$f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N}$	N = number of lanes in lane group N_B = number of buses stopping/h	$0 \leq N_B \leq 250$ $f_{bb} = \geq 0.050$
Type of area	$f_a = 0.900$ in CBD $f_a = 1000$ in all other areas		
Lane utilization	$f_{LU} = v_g/(v_{g1}N)$	v_g = unadjusted demand flow rate for the lane group, veh/h v_{g1} = unadjusted demand flow rate on the single lane in the lane group with the highest volume N = number of lanes in the lane group	
Left turns	Protected phasing: Exclusive lane: $F_{LT} = 0.95$ Shared lane: $f_{LT} = \frac{1}{1.0 + 0.05P_{LT}}$	P_{LT} = proportion of LTs in lane group	See pages 474 through 483 for non-protected phasing alternatives
Right turns	Exclusive lane: $f_{RT} = 0.85$ Shared lane: $f_{RT} = 1.0 - (0.15)P_{RT}$ Single lane: $f_{RT} = 1.0 - (0.135)P_{RT}$	P_{RT} = proportion of RTs in lane group	$f_{RT} = \geq 0.050$
Pedestrian-bicycle blockage	LT adjustment: $f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$ RT adjustment: $f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$	P_{LT} = proportion of LTs in lane group A_{pbT} = permitted phase adjustment P_{LTA} = proportion of LT protected green over total RT green P_{RT} = proportion of RTs in lane group P_{RTA} = proportion of RT protected green over total RT green	See pages 485 to 490 for step-by-step procedure

[a] The table contains formulas for all adjustment factors. However, for situations in which permitted phasing is involved, either by itself or in combination with protected phasing, separate tables are provided, as indicated in this exhibit.

· **Lane Width Adjustment Factor,** f_w. This factor depends on the average width of the lanes in a lane group. It is used to account for both the reduction in saturation flow rates when lane widths are less than 12 ft and the increase in saturation flow rates when lane widths are greater than 12 ft. When lane widths are 16 ft or greater, such lanes may be divided into two narrow lanes of 8 ft each. Lane width factors should not be computed for lanes less than 8 ft wide. See previous Table for the equation used to compute these factors.

· **Heavy Vehicle Adjustment Factor**, f_{HV}. The heavy vehicle adjustment factor is related to the percentage of heavy vehicles in the lane group. This factor corrects for the additional delay and reduction in saturation flow due to the presence of heavy vehicles in the traffic stream. The additional delay and reduction in saturation flow are due mainly to the difference between the operational capabilities of heavy vehicles and passenger cars and the additional space taken up by heavy vehicles. In this procedure, heavy vehicles are defined as any vehicle that has more than four tires touching the pavement. A passenger-car equivalent (Et) of two is used for each heavy vehicle. This factor is computed by using the appropriate equation given in previous Table .

· **Grade Adjustment Factor**, f_{g}. This factor is related to the slope of the approach being considered. It is used to correct for the effect of slopes on the speed of vehicles, including both passenger cars and heavy vehicles, since passenger cars are also affected by grade. This effect is different for up-slope and down slope conditions; therefore, the direction of the slope should be taken into consideration. This factor is computed by using the appropriate equation given in previous Table.

· **Parking Adjustment Factor**, f_{p}. On-street parking within 250 ft upstream of the stop line of an intersection causes friction between parking and nonparking vehicles which results in a reduction of the maximum flow rate that the adjacent lane group can handle. This effect is corrected for by using a parking adjustment factor on the base saturation flow. This factor depends on the number of lanes in the lane group and the number of parking maneuvers/h. The equation given in previous Table for the parking adjustment factor indicates that the higher the number of lanes in a given lane group, the less effect parking has on the saturation flow; the higher the number of parking maneuvers, the greater the effect. In determining these factors, it is assumed that each parking maneuver (either in or out) blocks traffic on the adjacent lane group for an average duration of 18 seconds. It should be noted that when the number of parking maneuvers/h is greater than 180 (equivalent to more than 54 minutes), a practical limit of 180 should be used. This adjustment factor should be applied only to the lane group immediately adjacent to the parking lane. When parking occurs on both sides of a single lane group, the sum of the number of parking maneuvers on both sides should be used.

· **Area Type Adjustment Factor**, f_{a}. The general types of activities in the area at which the intersection is located have a significant effect on speed and therefore on saturation volume at an approach. For example, because of the complexity of intersections located in areas with typical central business district characteristics, such as narrow sidewalks, frequent parking maneuvers, vehicle blockades, narrow streets, and high-pedestrian activities, these intersections operate less efficiently than intersections at other areas. This is corrected for by using the area type adjustment factor f_a , which is 0.90 for a central business district (CBD) and 1.0 for all other areas. It should be noted, however, that 0.90 is not automatically used for all areas designated as CBDs, nor should it be limited only to CBDs. It should be used for locations that exhibit the characteristics referred to earlier that result in a significant impact on the intersection capacity.

· **Bus Blockage Adjustment Factor**, f_{bb}. When buses have to stop on a travel lane to discharge or pick up passengers, some of the vehicles immediately behind the bus will also have to stop. This results in a decrease in the maximum volume that can be handled by that lane. This effect is corrected for by using the bus blockage adjustment factor which is related to the number of buses in an hour that stop on the travel lane within 250 ft upstream or downstream from the stop line, to pick up or discharge passengers, as well as the number of lanes in the lane group. This factor is also computed using the appropriate equation given in previous Table.

· **Lane Utilization Adjustment Factor**, f_{Lu}. The lane utilization factor is used to adjust the ideal saturation flow rate to account for the unequal utilization of the lanes in a lane group. This factor is also computed using the appropriate equation given in previous Table. It is given as

$$f_{Lu} = \frac{v_g}{v_{gi} N}$$

where

v_g = unadjusted demand flow rate for lane group (veh/h)
v_{gi} = unadjusted demand flow rate on the single lane in the lane group with the highest volume
N = number of lanes in the lane group

It is recommended that actual field data be used for computing f_{Lui}. Values shown in Table below can, however, be used as default values when field information is not available.

Table: Default Lane Utilization Factors.

Lane Group Movements	*No. of Lanes in Lane Group*	*Percent of Traffic in Most Heavily Traveled Lane*	*Lane Utilization Factor (f_{Lu})*
	1	100.0	1.000
Through or shared	2	52.5	0.952
	3[a]	36.7	0.908
Exclusive left turn	1	100.0	1.000
	2[a]	51.5	0.971
Exclusive right turn	1	100.0	1.000
	2[a]	56.5	0.885

[a]If lane group has more lanes than number shown in this table, it is recommended that surveys be made or the largest f_{Lu}-factor shown for that type of lane group be used.

· **Right-Turn Adjustment Factor**, f_{RT}. This factor accounts for the effect of geometry as other factors are used to account for pedestrians and bicycles using the conflicting crosswalk. It depends on the lane from which the right turn is made, (i.e., exclusive or shared lane) and the proportion of right-turning vehicles on the shared lane. This factor is also computed using the appropriate equation given in Table below.

· **Left-Turn Adjustment Factor**, f_{LT}. This adjustment factor is used to account for the fact that left-turn movements take more time than through movements. The values of this factor also depend on the type of phasing (protected, permitted, or protected-plus-permitted), the type of lane used for left turns (exclusive or shared lane), the proportion of left-turn vehicles using a shared lane, and the opposing flow rate when there is a permitted left-turn phase. The left turns can be made under any one of the following conditions:
Case 1: Exclusive lanes with protected phasing.
Case 2: Exclusive lanes with permitted phasing.
Case 3: Exclusive lanes with protected-plus-permitted phasing.
Case 4: Shared lane with protected phasing.
Case 5: Shared lane with permitted phasing.
Case 6: Shared lane with protected-plus-permitted phasing.
Case 7: Single-lane approaches with permitted left turns.

Cases 1 through 6 are for multilane approaches. Case 7 is for single-lane approaches in which either the subject approach and/or the opposing approach consists of a single lane. The methodology for computing the left-turn factors for the multilane approaches is first presented. These computations take into account :

- The portion of the effective green time in seconds during which left turns cannot be made because they are blocked by the clearance of an opposing saturated queue of vehicles, or g_{q}
- The portion of the effective green time in seconds that expires before a left turning vehicle arrives, or g_{f}
- The portion of the effective green time in seconds during which left turns filter through the opposing unsaturated flow (after the opposing queue clears), or g_{u}

The appropriate left-turn adjustment factor is determined through the following computations for the different cases.

Case 1 Exclusive Left-Turn Lane with Protected Phasing. As shown in previous Table, a left-turn factor of $f_{\mathrm{LT1}} = 0.95$ is used.
Case 2A Exclusive Left-Turn Lane with Permitted Phasing (Multilane permitted left turns opposed by a multilane approach). The left-turn factor f_{LT2A} is computed from the expression:

$$f_{LT2A} = \left(\frac{g_u}{g}\right)\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right] \qquad (f_{\min} \leq f_{LTA} \leq 1.00)$$

$$f_{\min} = 2(1 + P_L)/g$$

$$P_L = \left[1 + \frac{(N-1)g}{g_u/(E_{L1} + 4.24)}\right]$$

$$g_u = g - g_q \qquad g_q \geq 0$$

or

$$g_u = g \qquad g_q < 0$$

$$g_q = \frac{v_{olc}\, qr_o}{0.5 - [v_{olc}(1 - qr_o)/g_o]} - t_l,\ v_{olc}(1 - qr_o)/g_o \leq 0.49$$

<u>where:</u>

qr_o = opposing queue ratio

$= \max[1 - R_{po}(g_o/C), 0]$

R_{po} = opposing platoon ratio.

v_{olc} = adjusted opposing flow per lane, per cycle.

$$= \frac{v_o C}{3600 N_o f_{LU_o}} \text{(veh/}C\text{/ln)}$$

$$= \frac{v_o C}{3600 N_o f_{LU_o}} \text{(veh/}C\text{/ln)}$$

<u>Table</u>: Through-Car Equivalents, E_{L1}, for Permitted Left Turns.

	Effective Opposing Flow, $v_{oe} = v_o/f_{LUo}$						
Type of Left-Turn Lane	*1*	*200*	*400*	*600*	*800*	*1000*	*1200*[a]
Shared	1.4	1.7	2.1	2.5	3.1	3.7	4.5
Exclusive	1.3	1.6	1.9	2.3	2.8	3.3	4.0

Notes:

[a]Use formula for effective opposing flow more than 1200; v_{oe} must be >0.

$$E_{L1} = S_{HT}/S_{LT} - 1 \text{ (shared)}$$
$$E_{L1} = S_{HT}/S_{LT} \text{ (exclusive)}$$

$$S_{LT} = \frac{v_{oe} e^{\left(\frac{-v_{oe} t_c}{3600}\right)}}{1 - e^{\left(\frac{-v_{oe} t_f}{3600}\right)}}$$

where

E_{L1} = through-car equivalent for permitted left turns

S_{HT} = saturation flow of through traffic (veh/h/ln) - 1900 veh/h/ln

S_{LT} = filter saturation flow of permitted left turns (veh/h/ln)

t_c = critical gap = 4.5 s

t_f = follow-up headway = 4.5 s (shared), 2.5 s (exclusive)

g_u = portion of the effective green time during which left turns filter through the opposing saturated flow (sec).
g_q = portion of the effective green time during which left turns cannot be made because they are blocked by the clearance of an opposing saturated queue of vehicles.
C = cycle length (sec).
g = effective permitted green time for left-turn lane group (sec).
g_o = opposing effective green time (sec).
N = number of lanes in exclusive left-turn group.
N_o= number of lanes in opposing approach.
t_l = lost time for left-turn lane group.
v_o= adjusted flow rate for opposing approach (veh/h).
E_{L1} = through car equivalent for permitted left turns.

Case 2B Exclusive Left-Turn Lane with Permitted Phasing (Multilane permitted left turns opposed by a single lane approach). The left-turn
factor f_{LTB} is computed from the equation

$$f_{LT2B} = \left(\frac{g_u}{g}\right)\left[\frac{1}{1 + (E_{L2} - 1)}\right] + \left[\frac{g_{diff}/g}{1 + (E_{L2} - 1)}\right],$$
$$\text{where } (f_{min} \le f_{LTB} \le 1)$$

$$f_{min} = \frac{4}{g}$$

$$E_{L2} = \max[(1 - P_{THo}^{n})/P_{LTo}, 1.0]$$
$$g_{diff} = \max[g_q, 0] \qquad \text{(when opposing left-turn volume is 0, } g_{diff} \text{ is zero)}$$
$$n = \max[(g_q/2), 0]$$

$$P_{THo} = 1 - P_{LTo}$$

$$g_u = g - g_q \qquad \text{if } g_q \ge 0 \text{ or}$$
$$g_u = g \qquad \text{if } g_q < 0$$

$$v_{olc} = v_o C/3600\, f_{Luo} \quad \text{(veh/hr)}$$
$$g_q = 4.943 v_{olc}^{0.762}\, q_{ro}^{1.061} - t_L$$
$$q_{ro} = \max[1 - R_{po}(g_o/C), 0]$$

<u>where:</u>
g = effective permitted green time for left-turn lane group (sec)
P_{LTo} =proportion of opposing left-turn volume in opposing flow
g_{u} = proportion of the effective green time during which left turns filter through the opposing saturated flow (sec)
g_{q} = proportion of the effective green time during which left turns cannot be made because they are blocked by the clearance of an opposing saturated queue of vehicles.
q_{ro} = opposing queue ratio
g_{o} = opposing effective green time (sec)
v_{o} = adjusted flow rate for opposing flow (veh/h)
t_{L} = lost time for left-turn lane group
E_{L1} = through car equivalent for permitted left turns, previous table.
C = cycle length (sec)
R_{po} = opposing platoon ratio.

Since the proportion of left turns on an exclusive left-turn lane is 1, then:

$$f_{LT2}(\min) = \frac{2(1+1)}{g} = \frac{4}{g}$$

<u>where:</u>
g= effective green time for the lane group (sec).
f_{LT2} (min) =practical minimum value for left-turn adjustment factor for exclusive lanes-permitted left turns and assuming an approximate average headway of 2 seconds per vehicle in an exclusive lane.

<u>Case 3 Exclusive Lane with Protected-Plus-Permitted Phasing</u>
In determining the left-turn factor for this case, the protected portion of the phase is separated from the permitted portion, and each portion is analyzed separately. That is, the protected portion of the phase is considered a protected phase with a separate lane group, and the permitted portion is considered a permitted phase with its own separate lane group. The left-turn factor for the protected portion is then obtained as 0.95, and the left-turn factor for the permitted phase is computed from the appropriate equation. However, care should be taken to compute the appropriate values of G, g, g_{f} , and g_{q}, as discussed later. Doing so may result in different saturation flow rates for the two portions. A method for estimating delay in such cases is defined later.

<u>Case 4 Shared Lane with Protected Phasing</u>
In this case, the left-turn factor f_{LT4} is computed from the expression:

$$f_{LT4} = \frac{1.0}{1.0 + 0.05P_{LT}}$$

where P_{LT} is the proportion of left turns in the lane group.

Case 5A Shared Lane with Permitted Phasing (Permitted left turns opposed by multilane approach).

In this case, the resultant effect on the entire lane group should be considered. The left-turn factor f_{LT5A} is computed from the expression:

$$f_{LT5A} = \frac{f_{m5A} + 0.91(N-1)}{N}$$

where f_{m5} is the left-turn adjustment factor for the lane from which permitted left turns are made. This value is computed from the expression:

$$f_{m5} = \frac{g_{f5A}}{g} + \frac{g_u}{g}\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right] f_{\min} \leq f_{m5A} \leq 1.00$$

where:

g = effective permitted green time for the left-turn lane group (sec).

g_{f5A} = portion of the effective green time that expires before a left-turning vehicle arrives (sec).

g_u = portion of the effective green time during which left-turning vehicles filter through the opposing flow (sec).

$$= g - g_q \text{ when } g_q \geq g_{f5}$$
$$= g - g_{f5} \text{ when } g_q \leq g_{f5}$$

E_{L1} = through-car equivalent for each turning vehicle, as obtained from previous Table.

P_L = proportion of left turns from shared lane(s).

The value of g_{f5A} is calculated from:

$$g_{f5A} = G \exp(-0.882\ LTC^{0.717}) - t_L$$

and the proportion of left turns from shared lanes, P_L, is calculated from:

$$P_L = P_{LT}\left[1 + \frac{(N-1)g}{g_{f5A} + \dfrac{g_u}{E_{L1}} + 4.24}\right]$$

where
G= actual green time (sec)
LTC = left turns per cycle = v_{LT} C/3600
C = cycle length (sec)
t_{L} = lost time per phase (sec)
PLT= proportion of left turns in the lane group
N = number of lanes in the lane group
E_{L1} = through-car equivalent for each turning vehicle, as obtained from

Also, note that in order to account for sneakers, the practical minimum value of f_{m5A} is estimated as $2(1 + PL)/g$.

Case 5B Shared Lane with Permitted Phasing (Permitted left turns opposed by a single-lane approach)

In this case, the left-turn factor is computed from the expression:

$$f_{LT5B} = [g_f/g] + \left[\frac{g_u/g}{1 + P_{LT}(E_{L1} - 1)}\right] + \left[\frac{g_{\text{diff}}/g}{1 + P_{LT}(E_{L2} - 1)}\right]$$

$$(f_{\min} \leq f_{LT5B} \leq 1)$$
$$f_{\min} = 2(1 + P_{LT})/g$$

$$g_{\text{diff}} = \max[(g_q - g_f), 0] \text{ (when left turn volume is 0, } g_{\text{diff}} \text{ is 0)}$$
$$E_{L2} = \max[(1 - P_{THo}^n)/P_{LTo}, 1.0]$$
$$P_{THo} = 1 - P_{LTo}$$

$$g_u = g - g_q \quad (\text{if } g_q \geq 0)$$
$$g_u = g - g_f \quad (\text{if } g_q < 0)$$
$$g_q = 4.943 v_{olc}^{0.762} q_{ro}^{1.061} - t_L \qquad (\text{if } g_q \leq g)$$

$$v_{olc} = v_o C/3600 \quad (\text{veh/h})$$
$$q_{ro} = \max[1 - R_{po}(g_o/C), 0]$$
$$g_f = G[e^{0.860(LTC^{0.629})}] - t_L \qquad (g_f \leq g)$$
$$LTC = v_{LT} C/3600$$

where:
g = effective permitted green time for left-turn lane group (sec).
G = total actual green for left-turn lane group (sec).
P_{LTo} = proportion of opposing left-turn volume in opposing flow.
g_{u} = proportion of the effective green time during which left turns filter through the opposing saturated flow (sec).

g_q = proportion of the effective green time during which left turns cannot be made because they are blocked by the clearance of an opposing saturated queue of vehicles.
q_{ro} = opposing queue ratio.
g_o = opposing effective green time (sec).
v_o = adjusted flow rate for opposing flow (veh/h).
v_{LT} = adjusted left-turn flow rate.
P_{LT} = proportion of left turn volume in left-turn lane group.
t_L = lost time for left-turn lane group.
E_{L1} = through car equivalent for permitted left turns.
C = cycle length (sec).
R_{po} = opposing platoon ratio.

Case 6 Shared Lane with Protected-Plus-Permitted Phasing

In determining the left-turn factor for this case, the protected portion of the phase is separated from the permitted portion and each portion is analyzed separately. The protected portion is considered as a protected phase and Eq. (case 4) is used to determine the left-turn factor for this portion.

The left-turn factor for the permitted portion is determined by using the procedure for Case 5A or 5B depending on whether the left turns are opposed by a multilane approach or a single-lane approach.

Case 7 Single-Lane Approaches with Permitted Left Turns

Three different conditions exist under this case:Case7A,a single-lane approach opposed by a single-lane approach; Case 7B, a single-lane approach opposed by a multilane approach; and Case 7C, a multilane approach opposed by a single-lane approach.

In Case 7A (single-lane approach opposed by a single-lane approach), the left-turn adjustment f_{LT7A} factor is computed from:

$$f_{LT7A} = \frac{g_{f7A}}{g} + \frac{g_{\text{diff}}}{g}\left[\frac{1}{1 + P_{LT}(E_{L2} - 1)}\right] + \frac{g_{u7A}}{g}\left[\frac{1}{1 + P_{LT}(E_{L1} - 1)}\right]$$

where:

$g_{\text{diff}} = \max[(g_q - g_{f\,7A}), 0]$ (when no opposing left turns are present g_{diff} is zero)

$$g_{f7A} = G\exp(-0.860LTC^{0.629}) - t_L \text{ (sec)} \qquad 0 \le g_{f7A} \le g$$
$$g_{q7A} = 4.943v_{\text{olc}}^{0.762}\, qr_o^{1.061} - t_L(\text{sec}) \qquad 0 \le g_{q7A} \le g$$
$$g_{u7A} = g - g_{q7A} \text{ when } g_{q7A} \ge g_{f7A}$$
$$g_{u7A} = g - g_{f7A} \text{ when } g_{q7A} < g_{f7A}$$

g = effective green time (sec)
G = actual green time for the permitted phase (sec)
P_{LT} = proportion of left turns in the lane group
L_{TC} = left turns per cycle
= $v_{LT}C/3600$
v_{LT} = adjusted left-turn flow rate (veh/h)
C = cycle length (sec)
t_L = lost time for subject left-turn lane group (sec)
v_{olc} = adjusted opposing flow rate per lane per cycle (veh/ln/c)
= $v_oC/(3600\, f_{Luo})$ (veh/ln/c)
v_o = adjusted opposing flow rate (veh/h)
qr_o = opposing queue ratio, that is, the proportion of opposing flow rate originating in opposing queues
= $1 - R_{po}(g/C)$

R_{po} = platoon ratio for the opposing flow, obtained from Table 10.3, based on the arrival type of the opposing flow.

g_o = effective green time for the opposing flow (sec)
$E_{L2} = (1 - P_{THO}^{n})/P_{LTO}$; $E_{L2} \ge 1.0$
P_{LTO} = proportion of left turns in opposing single-lane approach
P_{THO} = proportion of through and right-turning vehicles in opposing single-lane approach computed as $(1 - P_{LTO})$

n _ maximum number of opposing vehicles that can arrive during $(g_{q7A} - g_{f\,7A})$, computed as $(g_{q7A} - g_{f\,7A})/2$ with $n \ge 0$
E_{L1} = through-car equivalent for each left-turning vehicle.

For Case 7B (single-lane approach opposed by a multilane approach), gaps are not opened in the opposing flow by opposing left-turning vehicles blocking opposing through movements. The single-lane model therefore does not apply and the multilane models $f_{\text{LT}} = f_{\text{m5}}$; however, the single-lane model for f_f is used. That is

$$f_{LT7B} = \frac{g_{f7B}}{g} + \frac{g_{f7B}}{g}\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right]$$

where

g = effective green time for the lane group (sec)

g_{f7B} = portion of the effective green time that expires before a left-turning vehicle arrives (sec)

$= G \exp(0.860 LTC^{0.629}) - t_L$

g_{u7B} = portion of the effective green time during which left-turning vehicles filter through the opposing flow (sec)

$= g - g_{q7B}$ when $g_q \geq g_{f7B}$

$= g - g_{f7B}$ when $g_q < g_{f7B}$

Note that the practical minimum value of f_{LT7B} may be estimated as 2(1 + PL)/g.
For Case 7C (multilane approach opposed by a single-lane approach), the single lane model given in previous equ. applies with two revisions. First,

by g_{f7C}, where $g_{f7C} = G \exp(-0.882 LTC^{0.717}) - t_L$

Second, P_{LT} should be replaced by an estimated P_L that accounts for the effect of left turns on the other lanes of the lane group from which left turns are not made. These substitutions give the left-turn factor (f_{LT7C}) as:

$$f_{LT7C} = [f_{m7C} + 0.91(N - 1)/N]$$

$$f_{m7C} = \frac{g_{f7C}}{g} + \frac{g_{\text{diff}}}{g}\left[\frac{1}{1 + P_L(E_{L2} - 1)}\right] + \frac{g_u}{g}\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right]$$

$$P_L = P_{LT}\left[1 + \frac{(N - 1)g}{g_{f7C} + \frac{g_u}{E_{L1}} + 4.24}\right]$$

$$E_{L2} = \frac{(1 - P_{THO}^n)}{P_{LTO}}$$

where

$$g_{q7C} = 4.943 v_{olC}^{0.762} q r_o^{1.061} - t_L, \quad 0.0 \le g_q \le g$$
$$g_{\text{diff}} = \max[(g_{q7C} - g_{f7C}), 0]$$
$$g_u = g - g_{f7C} \text{ (when } g_{q7C} \ge g_{f7C})$$
$$= g - g_{q7C} \text{ (when } g_{q7C} < g_{f7C})$$

g = effective green time (sec)

$$g_{f7C} = G \exp(-0.882 LTC^{0.717}) - t_L$$

G = actual green time for the permitted phase (sec)

LTC = left turns per cycle $= v_{LT}C/3600$

v_{LT} = adjusted left-turn flow rate (veh/h)

C = cycle length (sec)

t_L = lost time per phase (sec)

v_{olc} = adjusted opposing flow rate per lane per cycle (veh/ln/c) $= v_o C/(3600 N_o) f_{Lo}$

P_{LT} = proportion of left turns in the lane group

N = number of lanes in the lane group

f_s = left-turn saturation factor ($f_s \ge 0$) $= (875 - 0.625 v_o)/1000$

v_o = adjusted opposing flow rate (veh/h)

P_{LTO} = proportion of left turns in opposing single-lane approach

P_{THO} = proportion of through and right-turning vehicles in opposing single-lane approach $= (1 - P_{LTO})$

n = maximum number of opposing vehicles that can arrive during $(g_{q7C} - g_{f7C})$ computed as $(g_{q7C} - g_{f7C}) \ne 2, n \geqslant 0$

E_{L1} = through-car equivalent for each left-turning vehicle.

Note, however, that when the subject approach is a dual left-turn lane, $f_{LT7C} = f_{m7C}$,

The worksheets shown in Figures below may be used to compute the left-turn factors for multilane and single-lane opposing approaches, respectively.

SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS OPPOSED BY MULTILANE APPROACH

General Information

Project Description ______________________________

Input

	EB	WB	NB	SB
Cycle length, C (s)				
Total actual green time for LT lane group,[1] G (s)				
Effective permitted green time for LT lane group,[1] g (s)				
Opposing effective green time, g_o (s)				
Number of lanes in LT lane group,[2] N				
Number of lanes in opposing approach, N_o				
Adjusted LT flow rate, v_{LT} (veh/h)				
Proportion of LT volume in LT lane group,[3] P_{LT}				
Adjusted flow rate for opposing approach, v_o (veh/h)				
Lost time for LT lane group, t_L				

Computation

	EB	WB	NB	SB
LT volume per cycle, $LTC = v_{LT}C/3600$				
Opposing lane utilization factor, f_{LUo} (refer to Volume Adjustment and Saturation Flow Rate Worksheet)				
Opposing flow per lane, per cycle $v_{olc} = \frac{v_o C}{3600 N_o f_{LUo}}$ (veh/C/ln)				
$g_f = G[e^{-0.882(LTC^{0.717})}] - t_L$, $g_f \le g$ (except for exclusive left-turn lanes)[1, 4]				
Opposing platoon ratio, R_{po}				
Opposing queue ratio, $qr_o = \max[1 - R_{po}(g_o/C), 0]$				
$g_q = \frac{v_{olc} qr_o}{0.5 - [v_{olc}(1 - qr_o)/g_o]} - t_L$, $v_{olc}(1 - qr_o)/g_o \le 0.49$ (note case-specific parameters)[1]				
$g_u = g - g_q$ if $g_q \ge g_f$, or $g_u = g - g_f$ if $g_q < g_f$				
E_{L1}				
$P_L = P_{LT}\left[1 + \frac{(N-1)g}{(g_f + g_u/E_{L1} + 4.24)}\right]$ (except with multilane subject approach)[5]				
$f_{min} = 2(1 + P_L)/g$				
$f_m = [g_f/g] + [g_u/g]\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right]$, $(f_{min} \le f_m \le 1.00)$				
$f_{LT} = [f_m + 0.91(N - 1)]/N$ (except for permitted left turns)[6]				

Notes

1. Refer to Figure 10.6 for case-specific parameters and adjustment factors.
2. For exclusive left-turn lanes, N is equal to the number of exclusive left-turn lanes. For shared left-turn lanes, N is equal to the sum of the shared left-turn, through, and shared right-turn (if one exists) lanes in that approach.
3. For exclusive left-turn lanes, $P_{LT} = 1$.
4. For exclusive left-turn lanes, $g_f = 0$, and skip the next step. Lost time, t_L, may not be applicable for protected-permitted case.
5. For a multilane subject approach, if $P_L \ge 1$ for a left-turn shared lane, then assume it to be a de facto exclusive left-turn lane and redo the calculation.
6. For permitted left turns with multiple exclusive left-turn lanes $f_{LT} = f_m$.

Figure: Supplemental Worksheet for Permitted Left Turns Where Approach Is Opposed by Multilane Approach.

SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS OPPOSED BY SINGLE-LANE APPROACH

General Information				
Project Description ____				
Input				
	EB	WB	NB	SB
Cycle length, C (s)				
Total actual green time for LT lane group,[1] G (s)				
Effective permitted green time for LT lane group,[1] g (s)				
Opposing effective green time, g_o (s)				
Number of lanes in LT lane group,[2] N				
Adjusted LT flow rate, v_{LT} (veh/h)				
Proportion of LT volume in LT lane group, P_{LT}				
Proportion of LT volume in opposing flow, P_{LTo}				
Adjusted flow rate for opposing approach, v_o (veh/h)				
Lost time for LT lane group, t_L				
Computation				
LT volume per cycle, $LTC = v_{LT}C/3600$				
Opposing flow per lane, per cycle, $v_{olc} = v_oC/3600$ (veh/C/ln)				
Opposing platoon ratio, R_{po}				
$g_f = G[e^{-0.860(LTC^{0.629})}] - t_L$ $\quad g_f \le g$ (except exclusive left-turn lanes)[3]				
Opposing queue ratio, $qr_o = \max[1 - R_{po}(g_o/C), 0]$				
$g_q = 4.943 v_{olc}^{0.762} qr_o^{1.061} - t_L$ $\quad g_q \le g$				
$g_u = g - g_q$ if $g_q \ge g_f$, or $g_u = g - g_f$ if $g_q < g_f$				
$n = \max[(g_q - g_f)/2, 0]$				
$P_{THo} = 1 - P_{LTo}$				
E_{L1} (refer to Exhibit C16-3)				
$E_{L2} = \max[(1 - P_{THo}^{\,n})/P_{LTo}, 1.0]$				
$f_{min} = 2(1 + P_{LT})/g$				
$g_{diff} = \max[g_q - g_f, 0]$ (except when left-turn volume is 0)[4]				
$f_{LT} = f_m = [g_f/g] + \left[\frac{g_u/g}{1 + P_{LT}(E_{L1} - 1)}\right] + \left[\frac{g_{diff}/g}{1 + P_{LT}(E_{L2} - 1)}\right]$ $(f_{min} \le f_m \le 1.00)$				

Notes

1. Refer to Figure 10.6 for case-specific parameters and adjustment factors.
2. For exclusive left-turn lanes, N is equal to the number of exclusive left-turn lanes. For shared left-turn lanes, N is equal to the sum of the shared left-turn, through, and shared right-turn (if one exists) lanes in that approach.
3. For exclusive left-turn lanes, $g_f = 0$, and skip the next step. Lost time, t_L, may not be applicable for protected-permitted case.
4. If the opposing left-turn volume is 0, then $g_{diff} = 0$.

Figure: Supplemental Worksheet for Permitted Left Turns Where Approach Is Opposed by Single-Lane Approach.

To compute the appropriate values for G, g, g_{f}, g_{q}, and g_{u} for protected-plus permitted left-turn phases, one can use the models presented in the previous section when the left turn can move only in a permitted phase. When left turns can be made during protected-plus-permitted phases, the protected portion of the phase is separated from the permitted portion and each portion is treated separately. Two left-turn factors are then determined: one for the protected phase and another for the

permitted phase. The left-turn factor for the protected phase is determined as discussed earlier, and that for the permitted phase is obtained from the appropriate model, but with modified values of G, g, g_{f} and g_{q} (which are denoted as G*, g*, g_{f} *, g_{q}*). Figure below shows the equation for obtaining G*, g*, g_{f}*, and g_{q}* for different cases. For example, Case 2 shows the case with an exclusive left-turn lane and a leading green G which is followed by a period G/Y_1 during which the left-turn movement is given the yellow indication and the through movement is still given the green ball indication. This is then followed by a period G_2, during which both the NB and

SB traffic streams have first the green ball indication and then a full yellow indication, Y_2. This results in an effective green time for the NB permitted phase g* of G_2 + Y_2, and for the SB direction, g* of G_2 + Y_2 - t_{L}. The reason for this is that there is no lost time for the NB traffic during the permitted-left-turn phase, since the movement was initiated during the protected portion of the phase, and the lost time is assessed there.

This results in different effective green times for NB and SB traffic streams. Similarly, if the NB left turns are made from a shared lane, g_{f} would be computed from the total green time of G_1+G/Y_1+G_2, which includes the leading phase green time. However, in computing the appropriate g, only the portion of g_{f} that applies to the permitted phase should be used. This results in g_f^* being g_{f} - G_1-G/Y_1+t_L. Also, in computing the appropriate g_q^* for the NB traffic stream, it is noted that this should be the portion of the NB permitted green phase (g*) that is blocked by the clearance of the opposing queue. However, the permitted NB phase does not account for the lost time, and g_q^* is obtained as g_{q}+ t_{L}. Similar considerations are used to obtain the modified equations for g*, G*, g_f^*, and g_q^* for the different cases shown.

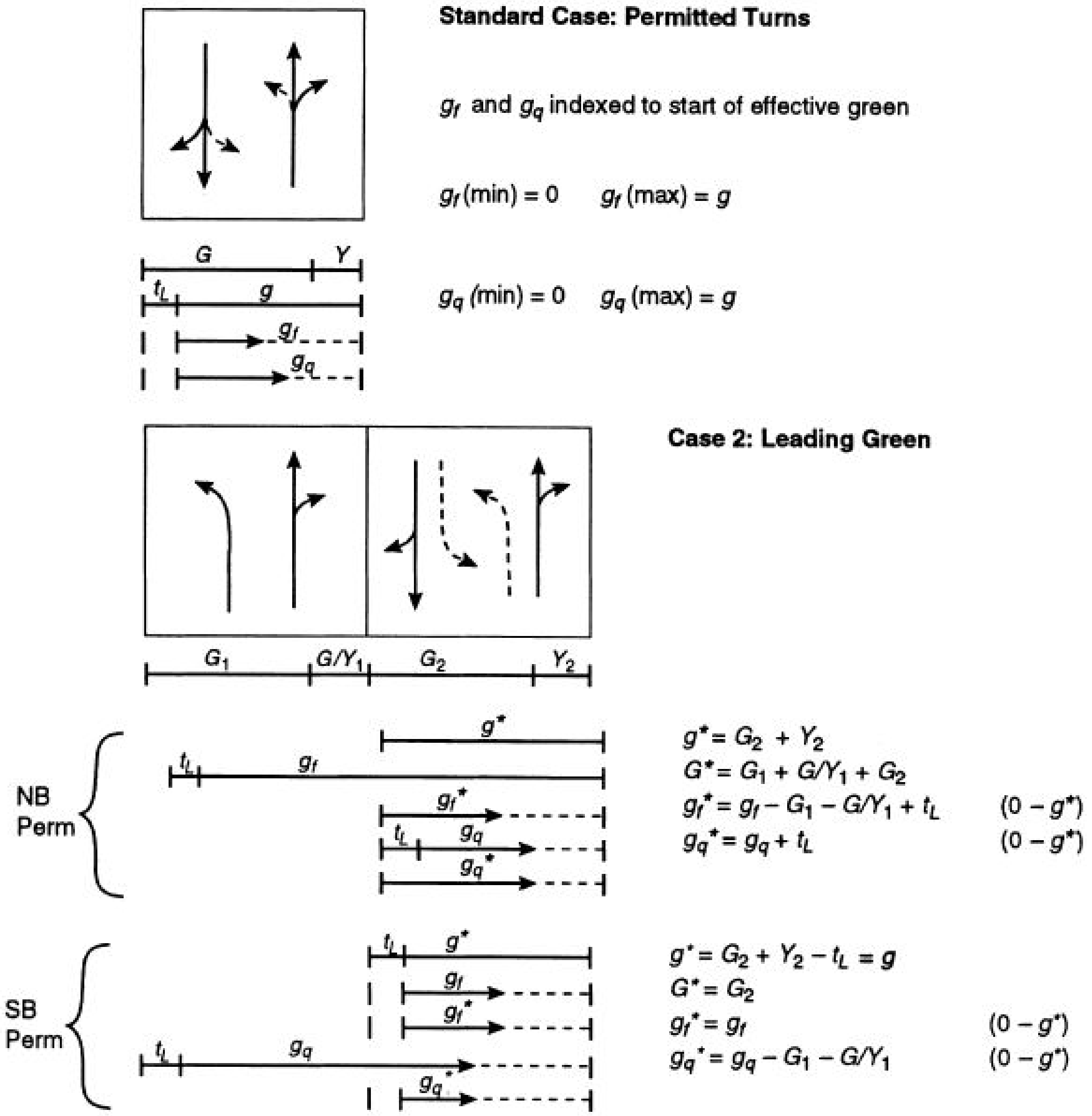


Figure: Green Time Adjustments for Protected-Plus-Permitted Phasing.

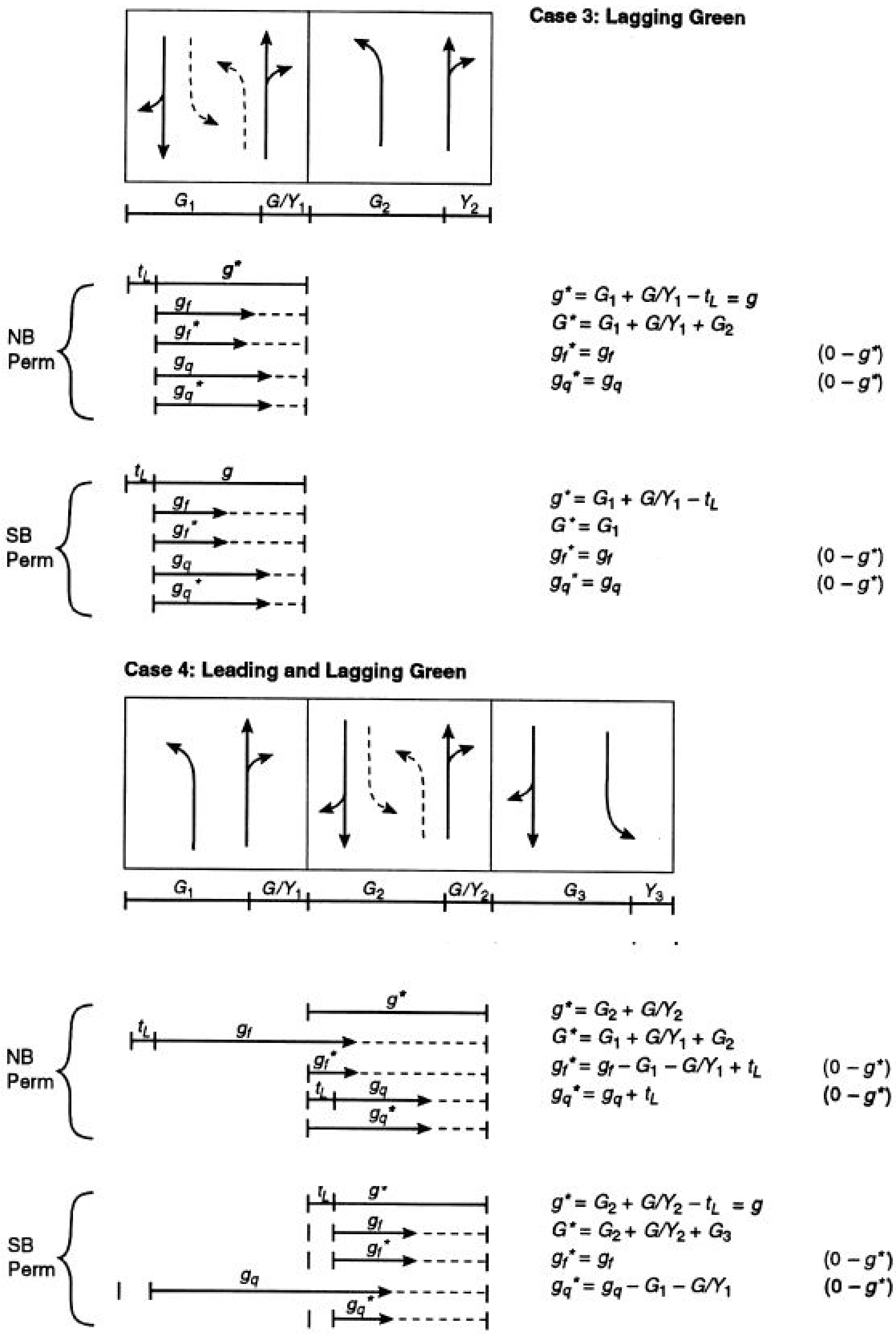


Figure : Green Time Adjustments for Protected-Plus-Permitted Phasing (continued).

Pedestrian and Bicycle Adjustment Factors

These factors are included in the saturation flow equation to account for the reduction in the saturation flow rate resulting from the conflicts between automobiles, pedestrians, and bicycles. The specific zones within the intersection where these conflicts occur are shown in Figure 10.7. The parameters required for the computation for these factors as presented by the HCM are shown in Previous Table. The flow chart shown in Figure below illustrates the procedure. The procedure can be divided into the following four main tasks:

1. Determine average pedestrian occupancy, OCC_{pedg}
2. Determine relevant conflict zone occupancy, OCC_{r}
3. Determine permitted phase pedestrian-bicycle adjustment factors for turning movements A_{pBT}
4. Determine saturation flow adjustment factors for turning movements (for f_{Lpb} left-turn movements and f_{Rpb} for right-turn movements).

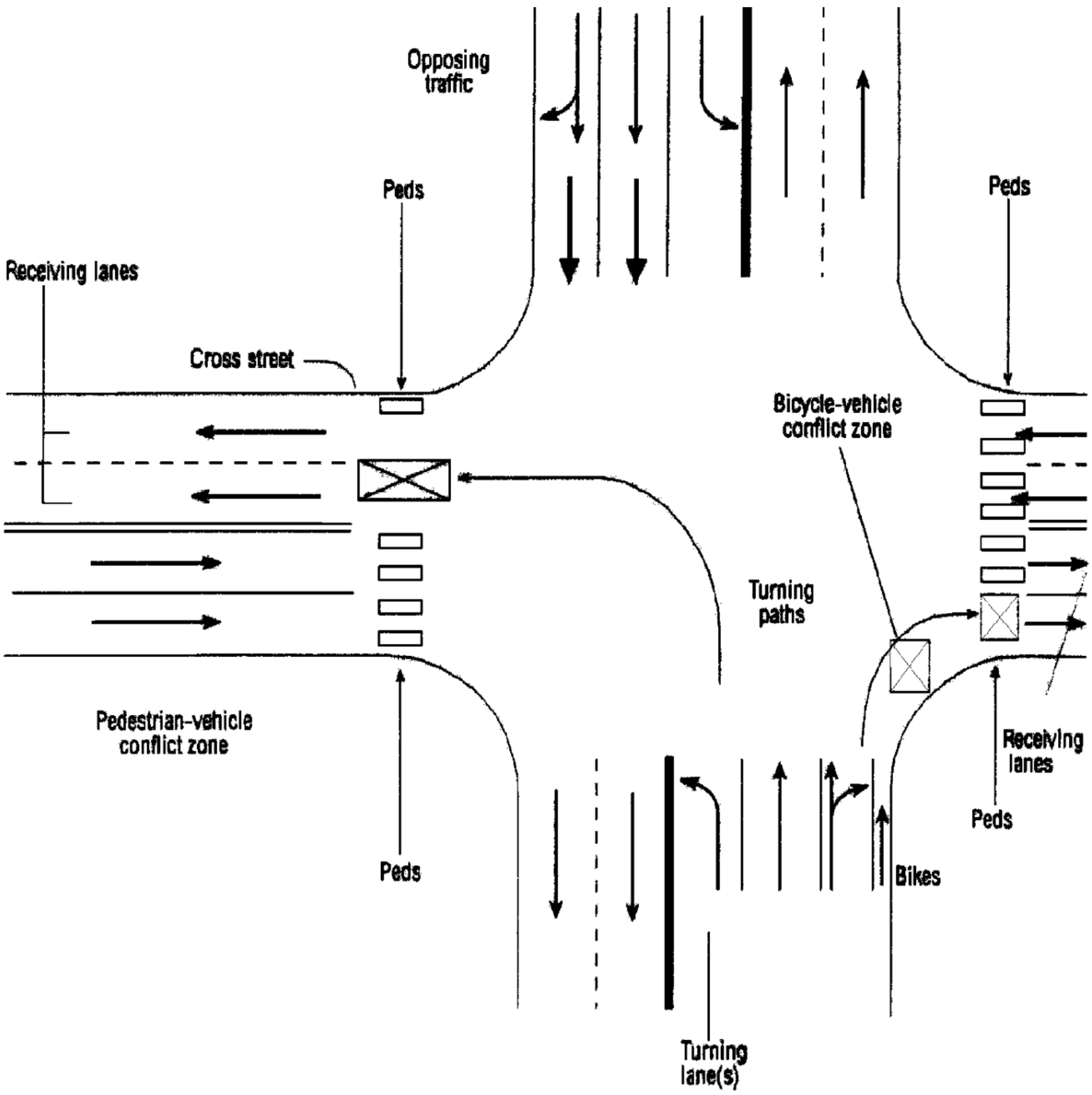


Figure Conflict Zone Locations.

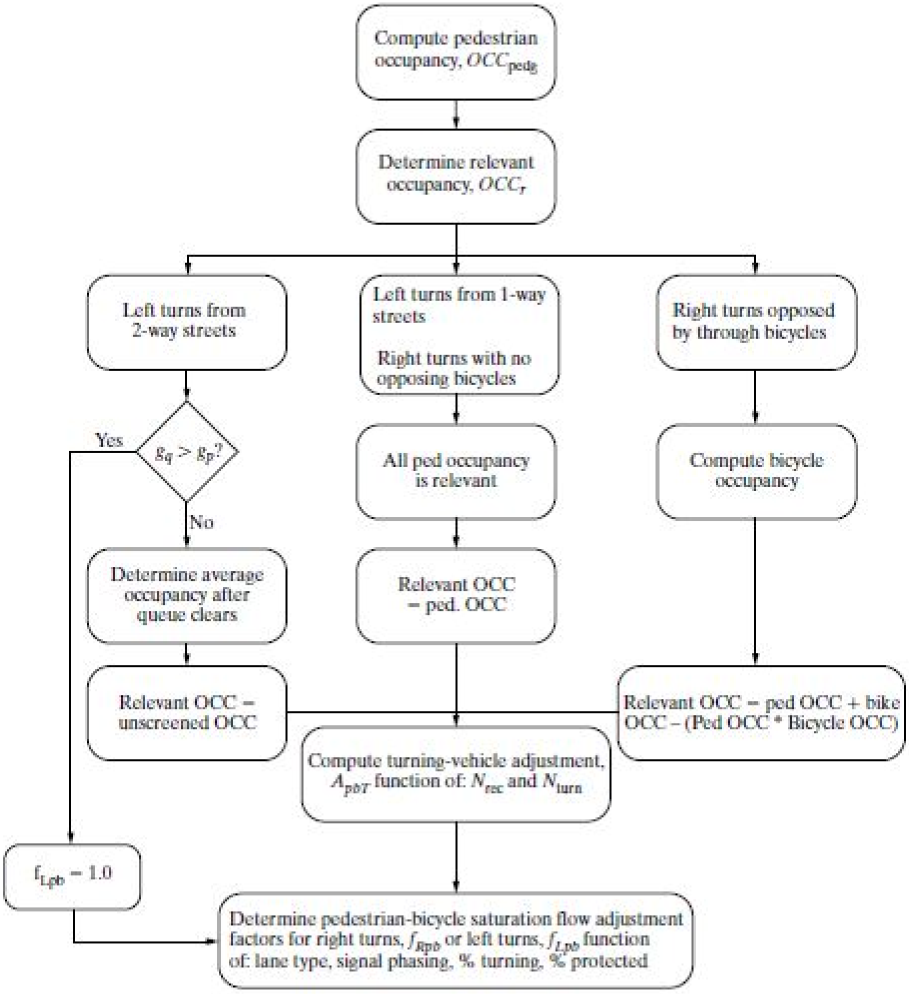


Figure: Outline of Computational Procedure for f_{Rpb} and f_{Lpb}.

Step 1. Determine Average Pedestrian Occupancy (OCC_{pedg}). In this task, the pedestrian flow rate is first computed from the pedestrian volume using Eq. below, and the average pedestrian occupancy is then computed from the pedestrian flow rate using Eqs. below:

$$v_{\text{pedg}} = v_{\text{ped}}\left({}^{C}/_{g_p}\right) \quad (v_{\text{pedg}} \leq 5000)$$

$$OCC_{\text{pedg}} = v_{\text{pedg}}/2000 \quad (v_{\text{pedg}} \leq 1000, \text{ and } OCC_{\text{pedg}} \leq 0.5)$$

$$OCC_{\text{pedg}} = 0.4 + v_{\text{pedg}}/10{,}000 \quad (1000 < v_{\text{pedg}} \leq 5000, \text{ and } 0.5 < OCC_{\text{pedg}} \leq 0.9)$$

where

v_{pedg} = pedestrian flow rate
v_{ped} = pedestrian volume
g_p = pedestrian green walk + flashing don't walk time (sec) (if pedestrian signal timing is unknown, g_p may be assumed to be equal to g (sec))
C = cycle length (sec)

Step 2. Determine Relevant Conflict Zone Occupancy (OCC_{r}). Two conditions influence the computation of this factor. These are
(1) right turning bicycles and automobiles weave to the right before reaching the stop line, and
(2) left-turn movements are made from a one-way street. When the first condition exists, the bicycle/automobile interaction within the intersection is eliminated; the bicycle volume should therefore be ignored and only the impact of pedestrians should be considered. For both of these conditions, the HCM gives the relevant conflict zone occupancy as:

$$OCC_r = OCC_{\text{pedg}}$$

where OCC_{pedg} = average pedestrian occupancy.

When bicycle interaction is also expected within the intersection, the bicycle flow rate (v_{bicg}) is first computed from the bicycle volume using Eq. below, and the bicycle conflict zone occupancy (OCC_{bicg}) is then determined from the bicycle flow rate using Eq. below. The relevant conflict zone occupancy (OCC_{r}) is then computed from the pedestrian occupancy (OCC_{pedg}) and the bicycle occupancy (OCC_{bicg}) using Eq. below.

$$v_{\text{bicg}} = v_{\text{bic}}\,(C/g) \qquad (v_{\text{bicg}} \leq 1900)$$

$$OCC_{\text{bicg}} = 0.02 + (v_{\text{bicg}} / 2700) \qquad (v_{\text{bicg}} < 1900 \text{ and } OCC_{\text{bicg}} \leq 0.72)$$

$$OCC_r = OCC_{\text{pedg}} + OCC_{\text{bicg}} - (OCC_{\text{pedg}})(OCC_{\text{bicg}})$$

where

v_{bicg} = bicycle flow rate (bicycles per hr)
v_{bic} = bicycle volume

As in the previous case, when bicycle flow rates are collected directly in the field, these values should be used and previous Eq. not used.
When left turns are made from an approach on a two-way street, a comparison of the opposing queue clearance (g_{q}) and the pedestrian green time (g_{p}) is first made to determine whether g_{p} is less or greater than g_{p}. If $g_{\mathrm{q}} \geq g_{\mathrm{p}}$, then the pedestrian green time is used entirely by the opposing queue and the pedestrian adjustment factor for left-turn movements (f_{Lpb}) is 1.0. However, if $g_{\mathrm{q}} < g_{\mathrm{p}}$, then the pedestrian occupancy after the opposing queue clears (OCC_{pedu}) is determined from the average pedestrian occupancy (OCC_{pedg}) using Eq. below. The relevant conflict zone occupancy (OCC_{r}) is then determined from OCC_{pedu} using below. Equation is based on the fact that left turning vehicles can go through the intersection only after the opposing queue has cleared, and that accepted gaps in the opposing flow v_{o} must be available for left turning vehicles.

$$OCC_{\text{pedu}} = OCC_{\text{pedg}}[1 - 0.5(g_q/g_p)]$$

$$OCC_r = OCC_{\text{pedu}}[e^{-(5/3600)v_o}]$$

where

OCC_{pedu} = pedestrian occupancy after the opposing queue clears
OCC_{pedg} = average pedestrian occupancy
g_q = opposing queue clearing time (sec)
q_p = pedestrian green walk + flashing DON'T WALK (sec). If the pedestrian signal timing is unknown, g_p may be assumed to be equal to the effective green (g).

Step 3. Determine Permitted Phase Pedestrian-Bicycle Adjustment Factors
for Turning Movement (A_{pbT}). Two conditions are considered in the determination of the A_{pbT}. These are (1) number of turning lanes (N_{turn}) is the same as the number of the cross-street receiving lanes (N_{rec}) and (2) number of turning lanes is less than the number of the cross-street receiving lanes.
When N_{turn} is equal to N_{rec}, the proportion of the time the conflict zone is occupied is the adjustment factor, as it is unlikely that the turning vehicles will be able to move around pedestrians or bicycles. The permitted phase pedestrian-bicycle adjustment factor (A_{pbT}) is therefore obtained from Eq. below:

$$A_{pbT} = 1 - OCC_r \qquad (N_{\text{turn}} = N_{\text{rec}})$$

When N_{turn} is less than the N_{rec}, the impact of pedestrians and bicycles on the saturation flow is reduced as it is more likely that the turning vehicles will be able to move around pedestrians and bicycles. The A_{pbT} in this case is obtained from Eq. below.

$$A_{pbT} = 1 - 0.6(OCC_r) \qquad (N_{\text{turn}} < \text{N}_{\text{rec}})$$

<u>where</u>:
N_{turn} = the number of turning lanes.
N_{rec} = the number of receiving lanes.

It is recommended that actual field observation be carried out to determine the number of turning lanes (N_{turn}) and the number of receiving lanes (N_{rec}). The reason for this is that at some intersections, left turns are illegally made deliberately from an outer lane or the receiving lane is blocked by vehicles that are double-parked, making it difficult for the turning vehicles to make a proper turn. Simply reviewing the intersection plans and noting the striping cannot identify these conditions.

Step 4. Determine Saturation Flow Adjustment Factors for Turning Movements (f_{Lpb} for left-turn movements, and f_{Rpb} for right-turn movements).

These factors depend on the A_{pbT} and the proportion of the turning flow that uses the protected phase. The pedestrian–bicycle adjustment factor for left-turns (f_{Lpb}) is obtained from Eq. below and that for right turns (f_{Rpb}) is obtained from Eq. below.

$$f_{Lpb} = 1.0\, P_{LT}\,(1 - A_{pbT})(1 - P_{LTA})$$

where:

P_{LT} = proportion of left turn volumes (used only for left turns made from a single lane approach or for shared lanes)

A_{pbT} = permitted phase pedestrian-bicycle adjustment factor for turning movements (obtained from Eq. above)

P_{LTA} = the proportion of left turns using protected phase (used only for protected/permissive phases)

$$f_{Rpb} = 1.0\, P_{RT}\,(1 - A_{pbT})(1 - P_{RTA})$$

where:

P_{RT} = proportion of right-turn volume (used only for right turns made from single-lane approach or for shared turning lanes).

A_{pbT} = permitted phase pedestrian-bicycle adjustment factor for turning movements (obtained from previous Eqs.).

P_{RTA} = the proportion of right turns using protected phase (used only for protected/permissive phases).

Figure below shows a supplemental worksheet that can be used to carry out these procedures.

SUPPLEMENTAL WORKSHEET FOR PEDESTRIAN-BICYCLE EFFECTS ON PERMITTED LEFT TURNS AND RIGHT TURNS

General Information

Project Description ______________________________

Permitted Left Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)				
Conflicting pedestrian volume,[1] v_{ped} (p/h)				
$v_{pedg} = v_{ped}\ (C/g_p)$				
$OCC_{pedg} = v_{pedg}/2000$ if $(v_{pedg} \le 1000)$ or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if $(1000 < v_{pedg} \le 5000)$				
Opposing queue clearing green,[3,4] g_q (s)				
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p; if $g_q \ge g_p$ then $f_{Lpb} = 1.0$				
$OCC_{pedu} = OCC_{pedg}\ [1 - 0.5(g_q/g_p)]$				
Opposing flow rate,[3] v_o (veh/h)				
$OCC_r = OCC_{pedu}\ [e^{-(5/3600)v_o}]$				
Number of cross-street receiving lanes,[1] N_{rec}				
Number of turning lanes,[1] N_{turn}				
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$ $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$				
Proportion of left turns,[5] P_{LT}				
Proportion of left turns using protected phase,[6] P_{LTA}				
$f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$				

Permitted Right Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)				
Conflicting pedestrian volume,[1] v_{ped} (p/h)				
Conflicting bicycle volume,[1,7] v_{bic} (bicycles/h)				
$v_{pedg} = v_{ped}(C/g_p)$				
$OCC_{pedg} = v_{pedg}/2000$ if $(v_{pedg} \le 1000)$, or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if $(1000 < v_{pedg} \le 5000)$				
Effective green,[1] g (s)				
$v_{bicg} = v_{bic}(C/g)$				
$OCC_{bicg} = 0.02 + v_{bicg}/2700$				
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg})$				
Number of cross-street receiving lanes,[1] N_{rec}				
Number of turning lanes,[1] N_{turn}				
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$ $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$				
Proportion of right turns,[5] P_{RT}				
Proportion of right turns using protected phase,[8] P_{RTA}				
$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$				

Notes

1. Refer to Input Worksheet.
2. If intersection signal timing is given, use Walk + flashing Don't Walk (use G + Y if no pedestrian signals). If signal timing must be estimated, use (Green Time – Lost Time per Phase) from Quick Estimation Control Delay and LOS Worksheet.
3. Refer to supplemental worksheets for left turns.
4. If unopposed left turn, then $g_q = 0$, $v_o = 0$, and $OCC_r = OCC_{pedu} = OCC_{pedg}$.
5. Refer to Volume Adjustment and Saturation Flow Rate Worksheet.
6. Ideally determined from field data; alternatively, assume it equal to $(1 - \text{permitted phase } f_{LT})/0.95$.
7. If $v_{bic} = 0$ then $v_{bicg} = 0$, $OCC_{bicg} = 0$, and $OCC_r = OCC_{pedg}$.
8. P_{RTA} is the proportion of protected green over the total green, $g_{prot}/(g_{prot} + g_{perm})$. If only permitted right-turn phase exists, then $P_{RTA} = 0$.

Figure: Supplemental Worksheet for Pedestrian-Bicycle Effects.

Field Determination of Saturation Flow

An alternative to the use of adjustment factors is to determine directly the saturation flow in the field. It was shown in Chapter 8 that the saturation flow rate is the maximum discharge flow rate during the green time. This flow rate is usually achieved 10 to 14 seconds after the start of the green phase which is usually the time the fourth, fifth, or sixth passenger car crosses the stop line. Therefore, saturation flow rates are computed starting with the headway after the fourth vehicle in the queue.

Two people are needed to carry out the procedure, with one being the timer equipped with a stopwatch, and the other the recorder equipped with a push-button event recorder or a notebook computer with appropriate software. The form shown in Figure below. It is suggested that the general information section of the form shown in Figure below be completed and other details such as area type, width, and grade of the lane being evaluated be measured and recorded.

An observation point is selected at the intersection such that a clear vision of the traffic signals and the stop line is maintained. A reference point is selected to indicate when a vehicle has entered the intersection. This reference point is usually the stop line such that all vehicles that cross the stop line are considered as having entered the intersection.

The following steps are then carried out for each cycle and for each lane:

Step 1. The timer starts the stopwatch at the beginning of the green phase and notifies the recorder.

Step 2. The recorder immediately notes the last vehicle in the stopped queue and describes it to the timer and also notes which vehicles are heavy vehicles and which vehicles turn left or right.

Step 3. The timer then counts aloud each vehicle in the queue as its rear axle crosses the reference point (that is, “one,” “two,” “three,” and so on). Note that right- or left-turning vehicles that are yielding to either pedestrians
or opposing vehicles are not counted until they have gone through the opposing traffic.

Step 4. The timer calls out the times that the fourth, tenth, and last vehicles in the queue cross the stop line, and these are noted by the recorder.

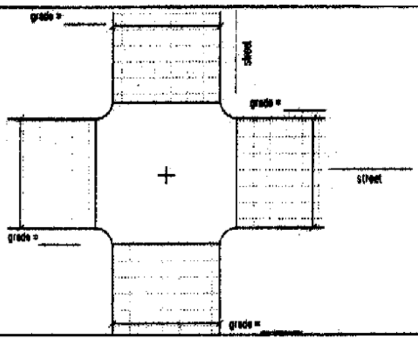
Step 5. In cases where queued vehicles are still entering the intersection at the end of the green phase, the number of the last vehicle at the end of the green phase is identified by the timer and told to the recorder so that number can be recorded.

Step 6. The width of the lane and the slope of the approach are then measured and recorded together with any unusual occurrences that might have affected the saturation flow.

FIELD SATURATION FLOW RATE STUDY WORKSHEET

General Information		Site Information	
Analyst	______	Intersection	______
Agency or Company	______	Area Type	☐ CBD ☐ Other
Date Performed	______	Jurisdiction	______
Analysis Time Period	______	Analysis Year	______

Lane Movement Input



Movements Allowed
- ☐ Through
- ☐ Right turn
- ☐ Left turn

Identify all lane movements and the lane studied

Input Field Measurement

Veh. in queue	Cycle 1			Cycle 2			Cycle 3			Cycle 4			Cycle 5			Cycle 6		
	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		
16																		
17																		
18																		
19																		
20																		
End of saturation																		
End of green																		
No. veh. > 20																		
No. veh. on yellow																		

Glossary and Notes

HV = Heavy vehicles (vehicles with more than 4 tires on pavement)
T = Turning vehicles (L = Left, R = Right)
Pedestrians and buses that block vehicles should be noted with the time that they block traffic, for example,
P12 = Pedestrians blocked traffic for 12 s
B15 = Bus blocked traffic for 15 s

Figure: Field Sheet for Direct Observation of Prevailing Saturation Flow Ratio.

Step 7. Since the flow just after the start of the green phase is less than saturation flow, the time considered for calculating the saturation flow is that between the time the rear axle of the fourth car crosses the reference point (t_4) and the time the rear axle of the last vehicle queued at the beginning of the green crosses the same reference point (t_n). The saturation flow is then determined from Eq. below.

$$\text{Saturation flow} = \frac{3600}{(t_4 - t_n)/(n - 4)}$$

where n is the number of the last vehicle surveyed. The data record on heavy vehicles, turning vehicles, and approach geometrics can be used in the future if adjustment factors are to be applied.

<u>Capacity and v/c Analysis Module</u>

In this module, results of the computations carried out in the previous modules are used to determine the important capacity variables which include:

- Flow ratios for different lane groups
- Capacities for different lane groups
- (v/c) ratios for different lane groups
- The critical (v/c) ratio for the overall intersection

The adjusted demand volume obtained for each lane group in the volume adjustment module is divided by the saturation flow for the appropriate lane group determined in the saturation flow module to obtain the flow ratio (v_i / s_i) for that lane group.

$$c_i = s_i(g_i/C)$$

the volume-to-capacity (v/c) ratio is then computed for each lane group.

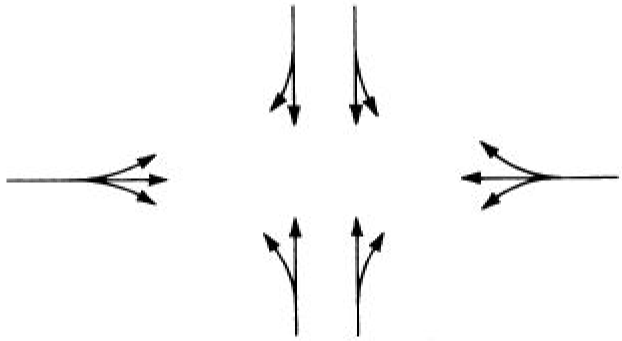
$$X_i = (v_i/c_i)$$

Similarly, using Eq. below, the critical volume-to-capacity ratio X_c is then computed for the intersection.

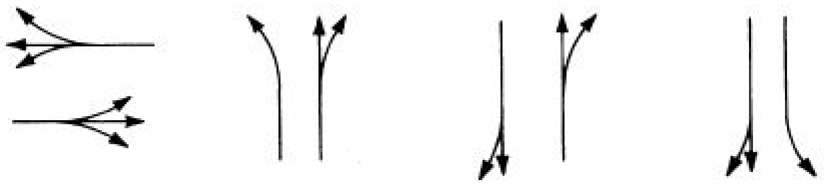
$$X_c = \sum_i (v/s)_{ci} \frac{C}{C - L}$$

The identification of the critical lane group for each green phase is necessary before the critical volume-to-capacity ratio (X_c) can be determined for the intersection.
This identification is relatively simple when there are no overlapping phases because the lane group with the maximum flow ratio (v_i / s_i) during each green phase is the critical lane group for that phase. When the phases overlap, however, identification of the critical lane group is not so simple. The basic principle used in this case is that the critical (v/c) ratio for the

intersection is based on the combinations of lane groups that will use up the largest amount of the capacity available. This is demonstrated by Figure below, which shows a phasing system that provides for exclusive left turn lanes in the south and north approaches and overlapping phases.



(a) Lane grouping



(b) Signal phasing

<u>Figure</u>: Illustrative Example for Determining Critical Lane Group.

There are only two lane groups during Phase 1–that is, EBLT/TH/RT and WBLT/TH/RT. The critical lane group is simply selected as the lane group with the greater (v_i /s_i) ratio. The three other phases, however, include overlapping movements, and the critical lane group is not so straightforward to identify. It can be seen that the NBTH/RT lane group moves during Phases 2A and 2B and therefore overlaps with the SBTH/RT lane group which moves during Phases 2B and 2C, while the NBLT lane group moves only during Phase 2A and the SBLT lane group moves only during
Phase 2C. The NBTH /RT lane group can therefore be critical for the sum of Phases 2A and 2B, whereas the SBLT lane group can be critical for Phase 2C. In determining the critical lane group, any one phase or portion of a phase can have only one critical lane group. If a critical lane group has been determined for the sum of phases i and j, no other lane group can be critical for either phase i or j or for any combination of phases that includes phase i or j. Note also that in determining the signal timing for any intersection–that is, for design of the intersection–the critical lane group is used.

Two possibilities therefore exist for the (v/s) ratios for the overlapping Phases 2A,2B, and 2C.

1. NBTH/RT + SBLT
2. SBTH/RT + NBLT

The critical lane flow ratio for the intersection is therefore the maximum flow ratio of the following:
EBLT/TH/RT + NBTH/RT + SBLT
EBLT/TH/RT + SBTH/RT + NBLT
WBLT/TH/RT + NBTH/RT + SBLT
WBLT/TH/RT+ SBTH/RT+ NBLT

It is also necessary to determine the total lost time L before the critical (v/c) ratio can be computed. The general rule is that it is assumed that a lost time of t_{L} occurs when a movement is initiated. Therefore, the total lost time L for each possible critical movement identified above is 3tL, where t_{L} is the lost time per phase. Thus, in general, $L = nt_L$, where n is the number of movements in the critical path through the signal cycle. Note that the lost time for each phase (t_{L}) is the sum of the start-up lost times (t_{L}) and the yellow-plus and red interval (y) minus the extension time (e).
The computation of the critical volume-to-capacity ratio (X_{c}) completes the definition of the capacity characteristics of the intersection. As stated earlier, these characteristics must be evaluated separately and in conjunction with the delay
and levels of service that are determined in the next module. Following are some key points that should be kept in mind when the capacity characteristics are being evaluated.

1. When the critical (v/c) ratio is greater than 1.00, the geometric and signal characteristics are inadequate for the critical demand flows at the intersection. The operating characteristics at the intersection may be improved by increasing the cycle length, changing the cycle phase, and/or changing the roadway geometrics.
2. When there is a large variation in the (v/c) ratio for the different critical lane groups but the critical (v/c) ratio is acceptable, the green time is not proportionately distributed, and reallocation of the green time should be considered.
3. A protected left-turn phase should be considered when the use of permitted left turns results in drastic reductions of the saturation flow rate for the appropriate lane group.
4. When the critical v/c ratio approaches 1.0, it is quite likely that the existing signal and geometric characteristics will not be adequate for an increased demand flow rate. Consideration therefore should be given to changing either the signal timing and/or the roadway geometrics.
5. If the (v/c) ratios are above acceptable limits and protected turning phases have been included for the turning movements with high flows, then changes in roadway geometrics will be required to reduce the (v/c) ratios.

The computation required for this module may be carried out in the format shown in Figure below. Note that in row 2 of Figure below, "Phase Type," when left turns are made from exclusive lanes during a protected permissive phase, the protected phase should be represented by a separate column. The protected phase is considered to be the primary phase and is designated as "P," the permitted phase is considered as the secondary phase and is designated "S," and the column containing the total flows is designated as "T." However, certain quantities (such as lane group capacity) should be computed as the sum of the primary and secondary phase flows.
It also should be noted that both lane groups with shared left-turn lanes and lane groups with only protected or permitted phasing have only a primary phase.

Performance Measures Module

The results obtained from the volume adjustment, saturation flow rate, and capacity analysis modules are now used in this module to determine the average control time delay per vehicle in each lane group and hence the level of service for each approach and the intersection as a whole. The computation first involves the determination of the uniform, incremental, and residual delays.

Uniform Delay. The uniform delay is that which will occur in a lane group if vehicles arrive with a uniform distribution and if saturation does not occur during any cycle. It is based on the first term of the Webster delay model.
Uniform delay is determined as:

$$d_{li} = 0.50C \frac{(1 - g_i/C)^2}{1 - (g_i/C)[\min(X_i, 1.0)]}$$

where

d_{li} = uniform delay (sec/vehicle) for lane group i
C = cycle length (sec)
g_i = effective green time for lane group i (sec)
X_i = (v/c) ratio for lane group i

Incremental Delay. The incremental delay takes into consideration that the arrivals are not uniform but random and that some cycles will overflow (random delay), as well as delay caused by sustained periods of oversaturation. It is given as

$$d_{2i} = 900T\left[(X_i - 1) + \sqrt{(X_i - 1)^2 + \frac{8k_iI_iX_i}{c_iT}}\right]$$

CAPACITY AND LOS WORKSHEET

General Information

Project Description ______________________________

Capacity Analysis

Phase number												
Phase type												
Lane group												
Adjusted flow rate, v (veh/h)												
Saturation flow rate, s (veh/h)												
Lost time, t_L (s), $t_L = l_1 + Y - e$												
Effective green time, g (s), $g = G + Y - t_L$												
Green ratio, g/C												
Lane group capacity,[1] $c = s(g/C)$, (veh/h)												
v/c ratio, X												
Flow ratio, v/s												
Critical lane group/phase (√)												
Sum of flow ratios for critical lane groups, Y_c $Y_c = \sum$ (critical lane groups, v/s)												
Total lost time per cycle, L (s)												
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$												

Lane Group Capacity, Control Delay, and LOS Determination

	EB	WB	NB	SB
Lane group				
Adjusted flow rate,[2] v (veh/h)				
Lane group capacity,[2] c (veh/h)				
v/c ratio,[2] $X = v/c$				
Total green ratio,[2] g/C				
Uniform delay, $d_1 = \frac{0.50\,C\,[1 - (g/C)]^2}{1 - [\min(1, X)g/C]}$ (s/veh)				
Incremental delay calibration,[3] k				
Incremental delay,[4] d_2 $d_2 = 900T\left[(X-1) + \sqrt{(X-1)^2 + \frac{8klX}{cT}}\right]$ (s/veh)				
Initial queue delay, d_3 (s/veh)				
Uniform delay, d_1 (s/veh)				
Progression adjustment factor, PF				
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)				
LOS by lane group				
Delay by approach, $d_A = \frac{\sum (d)(v)}{\sum v}$ (s/veh)				
LOS by approach				
Approach flow rate, v_A (veh/h)				
Intersection delay, $d_I = \frac{\sum (d_A)(v_A)}{\sum v_A}$ (s/veh)				

Notes

1. For permitted left turns, the minimum capacity is $(1 + P_L)(3600/C)$.
2. Primary and secondary phase parameters are summed to obtain lane group parameters.
3. For pretimed or nonactuated signals, $k = 0.5$. Otherwise, refer to Table 10.7.
4. T = analysis duration (h); typically $T = 0.25$, which is for the analysis duration of 15 min.
   I = upstream filtering metering adjustment factor; I = 1 for isolated intersections.

Figure: Capacity Analysis Worksheet.

where:
d_{2i} = incremental delay (sec/vehicle) for lane group i.
c_i = capacity of lane group i (veh/h).
T = duration of analysis period (hr).
k_i = incremental delay factor that is dependent on controller settings (see Table below).
I_i = upstream filtering metering adjustment factor accounts for the effect of filtered arrivals from upstream signals (for isolated intersections, I=1; for nonisolated intersections see Table below).
X_i = v/c ratio for lane group i.

Table: Recommended k Values for Lane Groups under Actuated and Pretimed Control.

Unit Extension (UE) (sec)	*Degree of Saturation (X)*					
	≤ 0.50	*0.60*	*0.70*	*0.80*	*0.90*	*≥ 1.0*
≤ 2.0	0.04	0.13	0.22	0.32	0.41	0.50
2.5	0.08	0.16	0.25	0.33	0.42	0.50
3.0	0.11	0.19	0.27	0.34	0.42	0.50
3.5	0.13	0.20	0.28	0.35	0.43	0.50
4.0	0.15	0.22	0.29	0.36	0.43	0.50
4.5	0.19	0.25	0.31	0.38	0.44	0.50
5.0[1]	0.23	0.28	0.34	0.39	0.45	0.50
Pretimed or nonactuated movement	0.50	0.50	0.50	0.50	0.50	0.50

Note: For a given UE and its k_{min} value at $X = 0.5$: $k = (1 - 2k_{\min})(X - 0.5) + k_{\min}$, $k \leq k_{\min}$, $k \leq 0.5$.
[1]For UE > 5.0, extrapolate to find k, keeping $k \leq 0.5$.

Table: Recommended I-Values for Lane Groups with Upstream Signals.

	Degree of Saturation at Upstream Intersection, X_u						
	0.40	*0.50*	*0.60*	*0.70*	*0.80*	*0.90*	*≥ 1.0*
I	0.922	0.858	0.769	0.650	0.500	0.314	0.090

Note: $I = 1.0 - 0.91\, X_u^{2.68}$ and $X_u \leq 1.0$.

Residual Demand Delay. This delay occurs as a result of an initial unmet demand Q_b vehicles at the start of the analysis period T. That is, a residual event of length Q_b exists at the start of the analysis period. In computing this residual demand, one of the following five cases will apply.

Case 1: $Q_b = 0$, analysis period is unsaturated
Case 2: $Q_b = 0$, analysis period is saturated
Case 3: $Q_b > 0$ and Q_b can be fully served during analysis period T, i.e., unmet demand Q_b and total demand in period T (qT) should be less than capacity cT, i.e., $Q_b + qT < cT$
Case 4: Q_b=0, but Q_b is decreasing, i.e., demand in time T, (qT) is less than the capacity cT
Case 5: Q_b =0, and demand in time T exceeds capacity cT

Residual demand is obtained as:

$$d_{3i} = \frac{1800 Q_{bi} (1 + u_i) t_i}{c_i T}$$

where

Q_{bi} = initial unmet demand at the start of period T_i vehicles for lane group i
c_i = adjusted lane group capacity veh/h
T = duration of analysis period (h)
t_i = duration of unmet demand in T for lane group i (h)
u_i = delay parameter for lane group i

and

$$t_i = 0 \; if \; Q_b = 0, \text{ else } t_i = \min\left[T, \frac{Q_{bi}}{c_i (1 - \min(1, X_i))}\right]$$

$$u_i = 0 \text{ if } t_i < T, \text{ else } u_i = 1 - \frac{c_i T}{Q_{bi}[1 - \min(1, X_i)]}$$

where X_i is the degree of saturation (v/c) for lane group i.

Total Control Delay

The total control delay for lane group i is given as:

$$d_l = d_{li}PF + d_{2i} + d_{3i}$$

<u>where</u>:

dl = the average control delay per vehicle for a given lane group

PF = uniform delay adjustment factor for quality of progression (See Table below)

d_{li} = uniform control delay component assuming uniform arrival

$d_{2\mathrm{i}}$ =incremental delay component for lane group i, no residual demand at the start of the analysis period

$d_{3\mathrm{i}}$= residual delay for lane group i

Table 10.9

Progression Adjustment Factor (PF) PF = $(1 - P)f_p/(1 - g/C)$ (see Note)

	Arrival Type (AT)					
Green Ratio (g/C)	*AT-1*	*AT-2*	*AT-3*	*AT-4*	*AT-5*	*AT-6*
0.20	1.167	1.007	1.000	1.000	0.833	0.750
0.30	1.286	1.063	1.000	0.986	0.714	0.571
0.40	1.445	1.136	1.000	0.895	0.555	0.333
0.50	1.667	1.240	1.000	0.767	0.333	0.000
0.60	2.001	1.395	1.000	0.576	0.000	0.000
0.70	2.556	1.653	1.000	0.256	0.000	0.000
Default, f_p	1.00	0.93	1.00	1.15	1.00	1.00
Default, R_p	0.333	0.667	1.000	1.333	1.667	2.000

Note: 1. Tabulation is based on default values of f_p and R_p.
2. $P = R_p\, g/C$ (may not exceed 1.0).
3. *PF* may not exceed 1.0 for AT-3 through AT-6.
4. For arrival type see Table 10.3.

It should be noted that while the adjustment factor for controller type (k) accounts for the ability of actuated controllers to adjust timing from cycle to cycle, the delay adjustment factor (PF) accounts for the effect of quality of signal progression at the intersection. PF accounts for the positive effect that good signal progression has on the flow of traffic through the intersection and depends on the arrival type. It has a value of one for isolated intersections (arrival type 3). The six different arrival types were defined earlier under "Specifying Traffic Conditions." Table above gives values for PF. When Q_{b} is greater than zero, which it is for Cases 3, 4, and 5, it is necessary to evaluate the uniform control delay for two periods: (1) the period when oversaturation queue exists (i.e., X ≥ 1) and (2) the period of undersaturation when X < 1. A value of X = 1 is used to determine the portion of the uniform control delay during the oversaturated period (t) using earlier Eq. and the actual value of X is used to find the portion of the uniform control delay during the undersaturated period (T - t). The value of d_1 is then obtained as the sum of the weighted values of the delay for each period as shown in Eq. below:

$$d_1 = \frac{d_s t}{T} + \frac{d_u(PF)(t - T)}{T}$$

<u>where:</u>
d_s = saturated delay (d_1 evaluated for X = 1) hr.
d_u = undersaturated delay (d_1 evaluated for actual X value) hr.
T = analysis period.

It is obvious from above Eq. that when the oversaturated period is as long as the analysis period (Cases 4 and 5, i.e., T = t), the du term drops off and the uniform delay is obtained directly from earlier Eq. using X = 1. However, when left turns are made from exclusive left-turn lanes with a protected-permissive phase, a special procedure described in the following paragraphs is used to estimate d_s and d_u.

Special Procedure for Uniform Delay with Protected-Plus-Permitted Operations.
The uniform delay given by earlier Eq is based on the queue storage as a function of time. When there is only a single green phase per cycle for a given lane group, the variation of the queue storage with time can be represented as a triangle. When left turns are allowed to proceed on both protected and permitted phases, the variation of queue storage is no longer a simple triangle but rather a more complex polygon. In order to determine the area of this complex polygon, representing the uniform delay, it is necessary to determine the proper values of the arrival and discharge rates during the different intervals. If the protected phase is considered the "primary" phase and the permitted phase is considered the secondary phase, then the following quantities must be known to compute the uniform delay.

- The arrival rate q_a (vehicle / sec), assumed to be uniform throughout the cycle
- The saturation flow rate S_p (vehicle / sec) for the primary phase
- The saturation flow rate S_s (vehicle / sec) for the unsaturated portion of the secondary
- phase (The unsaturated portion begins when the queue of opposing vehicles has not been served.).
- The effective green time g (sec) for the primary phase in which the left-turn traffic has the green arrow.
- The green time g_q (sec) during which the permitted left turns cannot be made because they are blocked by the clearance of an opposing queue (This interval begins at the start of the permitted green and ends when the queue of the opposing through vehicles is completely discharged.).
- The green time g_u (sec) during which the permitted left turns can filter through gaps in the opposing flow. This green period starts at the end of g_q.
- The red time r during which the signal is effectively red for the left turns.

The equations for determining these queue lengths depend on whether the phasing system is protected and permitted (leading) or permitted and protected (lagging). Shown below are the three conditions under the leading phase system and two under the lagging phase system.

• Protected and Permitted (Leading Left Turns)

Condition 1. No queue remains at the end of a protected or permitted phase. Note that this does not occur for uniform delays if $v/c < 1$.

Condition 2. A queue remains at the end of the protected phase but not at the end of the permitted phase.

Condition 3. No queue remains at the end of the protected phase, but there is a queue at the end of the permitted phase.

• Permitted and Protected (Lagging Left Turns)

Condition 4. No queue remains at the end of the permitted phase. This means that there will also be no queue at the end of the protected phase, since all left-turning vehicles will be cleared during the protected phase.

Condition 5. A queue remains at the end of the permitted phase. Note also that if $v/c < 1$, this queue will be cleared during the protected phase.

These different queue lengths and the uniform delay for any one of the five conditions are determined from the equations given in Figure below.

SUPPLEMENTAL UNIFORM DELAY WORKSHEET FOR LEFT TURNS FROM EXCLUSIVE LANES WITH PROTECTED AND PERMITTED PHASES

General Information

Project Description ______________________________

v/c Ratio Computation

	EB	WB	NB	SB
Cycle length, C (s)				
Protected phase eff. green interval, g (s)				
Opposing queue effective green interval, g_q (s)				
Unopposed green interval, g_u (s)				
Red time, r (s) $r = C - g - g_q - g_u$				
Arrival rate, q_a (veh/s) $q_a = \frac{v}{3600 * \max[X, 1.0]}$				
Protected phase departure rate, s_p (veh/s) $s_p = \frac{s}{3600}$				
Permitted phase departure rate, s_s (veh/s) $s_s = \frac{s(g_q + g_u)}{(g_u * 3600)}$				
If leading left (protected + permitted) v/c ratio, $X_{perm} = \frac{q_a(g_q + g_u)}{s_s\, g_u}$ If lagging left (permitted + protected) v/c ratio, $X_{perm} = \frac{q_a(r + g_q + g_u)}{s_s\, g_u}$				
If leading left (protected + permitted) v/c ratio, $X_{prot} = \frac{q_a(r + g)}{s_p\, g}$ If lagging left (permitted + protected) v/c ratio, X_{prot} is N/A				

Uniform Queue Size and Delay Computations

	EB	WB	NB	SB
Queue at beginning of green arrow, Q_a				
Queue at beginning of unsaturated green, Q_u				
Residual queue, Q_r				
Uniform delay, d_1				

Uniform Queue Size and Delay Equations

	Case	Q_a	Q_u	Q_r	d_1
If $X_{perm} \le 1.0$ & $X_{prot} \le 1.0$	1	$q_a r$	$q_a g_q$	0	$[0.50/(q_a C)][rQ_a + Q_a^2/(s_p - q_a) + g_q Q_u + Q_u^2/(s_s - q_a)]$
If $X_{perm} \le 1.0$ & $X_{prot} > 1.0$	2	$q_a r$	$Q_r + q_a g_q$	$Q_a - g(s_p - q_a)$	$[0.50/(q_a C)][rQ_a + g(Q_a + Q_r) + g_q(Q_r + Q_u) + Q_u^2/(s_s - q_a)]$
If $X_{perm} > 1.0$ & $X_{prot} \le 1.0$	3	$Q_r + q_a r$	$q_a g_q$	$Q_u - g_u(s_s - q_a)$	$[0.50/(q_a C)][g_q Q_u + g_u(Q_u + Q_r) + r(Q_r + Q_a) + Q_a^2/(s_p - q_a)]$
If $X_{perm} \le 1.0$ (lagging lefts)	4	0	$q_a(r + g_q)$	0	$[0.50/(q_a C)][(r + g_q)Q_u + Q_u^2/(s_s - q_a)]$
If $X_{perm} > 1.0$ (lagging lefts)	5	$Q_u - g_u(s_s - q_a)$	$q_a(r + g_q)$	0	$[0.50/(q_a C)][(r + g_q)Q_u + g_u(Q_u + Q_a) + Q_a^2/(s_p - q_a)]$

Figure: Supplemental Uniform Delay Worksheet for Left Turns from Exclusive Lanes with Primary and Secondary Phases.

Approach Delay

Having determined the average stopped delay for each lane group, we can now determine the average stopped delay for any approach as the weighted average of the stopped delays of all lane groups on that approach. The approach delay is given as:

$$d_A = \frac{\sum_{i=1}^{n_A} (d_{ia} v_i)}{\sum_{i=1}^{n_A} v_i}$$

where

d_A = delay for approach A (sec/veh).

d_{ia} = adjusted delay for lane group on approach A (sec/veh).

v_i = adjusted flow rate for lane group i (veh/h).

n_A = number of lane groups on approach A.

The level of service of approach A then can be determined from LOS Table.

Intersection Delay

The average intersection stopped delay is found in a manner similar to the approach delay. In this case, the weighted average of the delays at all approaches is the average stopped delay at the intersection. The average intersection delay is therefore given as:

$$d_I = \sum_{A=1}^{A_n} \frac{d_A v_A}{\sum_{A}^{A_n} v_A}$$

where:

d_I = average stopped delay for the intersection (sec/veh).

d_A = adjusted delay for approach A (sec/veh).

v_A= adjusted flow rate for approach A (veh/h).

A_n = number of approaches at the intersection.

The level of service for the intersection is then determined from earlier Table for LOS using the average control delay for the intersection. The computation required to determine the different levels of service may be organized in the format shown in previous Figure. It should be emphasized again that short or acceptable delays do not automatically indicate adequate capacity. Both the capacity and the delay should be considered in the evaluation of any intersection. Where long and unacceptable delays are determined, it is necessary to find the cause of the delay. For example, if (v/c) ratios are low and delay is long, the most probable cause for the long delay is that the cycle length is too long and/or the progression (arrival type) is unfavorable. Delay therefore can be reduced by improving the arrival type, by coordinating the intersection signal with the signals at adjacent intersections, and/or by reducing the cycle length at the intersection.

When delay is long but arrival types are favorable, the most probable cause is that the intersection geometrics are inadequate and/or the signal timing is improperly designed.