CHAPTER 16

SIGNALIZED INTERSECTIONS

CONTENTS

EXHIBITS

I. INTRODUCTION

SCOPE OF THE METHODOLOGY

This chapter contains a methodology for analyzing the capacity and level of service (LOS) of signalized intersections. The analysis must consider a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric characteristics, and details of intersection signalization. The methodology focuses on the determination of LOS for known or projected conditions.

Background and underlying concepts for this chapter are in Chapter 10

The methodology addresses the capacity, LOS, and other performance measures for lane groups and intersection approaches and the LOS for the intersection as a whole. Capacity is evaluated in terms of the ratio of demand flow rate to capacity (v/c ratio), whereas LOS is evaluated on the basis of control delay per vehicle (in seconds per vehicle). Control delay is the portion of the total delay attributed to traffic signal operation for signalized intersections. Control delay includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. Appendix A presents a method for observing intersection control delay in the field. Exhibit 10-9 provides definitions of the basic terms used in this chapter.

Each lane group is analyzed separately. Equations in this chapter use the subscript i to indicate each lane group. The capacity of the intersection as a whole is not addressed because both the design and the signalization of intersections focus on the accommodation of traffic movement on approaches to the intersection.

A lane group is indicated in formulas by the subscript i

The capacity analysis methodology for signalized intersections is based on known or projected signalization plans. Two procedures are available to assist the analyst in establishing signalization plans. The first is the quick estimation method, which produces estimates of the cycle length and green times that can be considered to constitute a reasonable and effective signal timing plan. The quick estimation method requires minimal field data and relies instead on default values for the required traffic and control parameters. It is described and documented in Chapter 10.

See Chapter 10 for description of quick estimation method

A more detailed procedure is provided in Appendix B of this chapter for estimating the timing plan at both pretimed and traffic-actuated signals. The procedure for pretimed signals provides the basis for the design of signal timing plans that equalize the degree of saturation on the critical approaches for each phase of the signal sequence. This procedure does not, however, provide for optimal operation.

The methodology in this chapter is based in part on the results of a National Cooperative Highway Research Program (NCHRP) study (*1, 2*). Critical movement capacity analysis techniques have been developed in the United States (*3–5*), Australia (*6*), Great Britain (*7*), and Sweden (*8*). Background for delay estimation procedures was developed in Great Britain (*7*), Australia (*9, 10*), and the United States (*11*). Updates to the original methodology were developed subsequently (*12–24*).

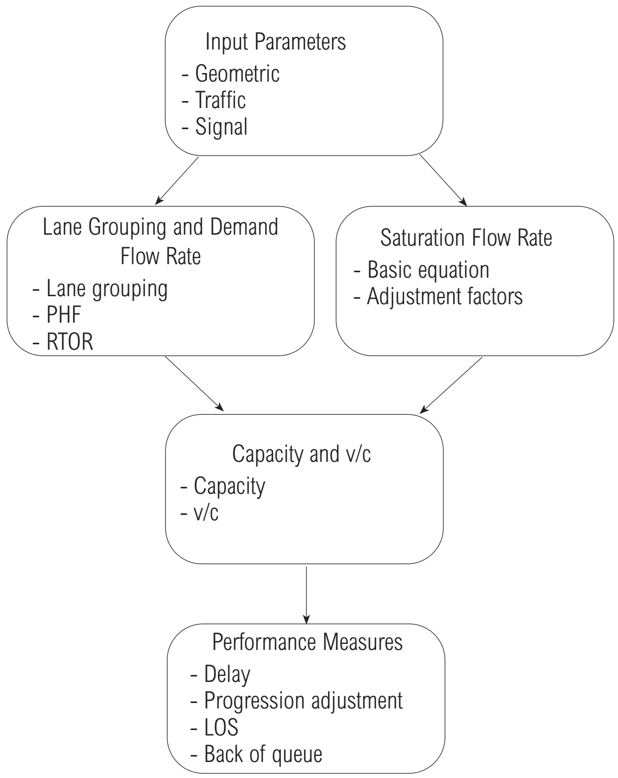
LIMITATIONS TO THE METHODOLOGY

The methodology does not take into account the potential impact of downstream congestion on intersection operation. Nor does the methodology detect and adjust for the impacts of turn-pocket overflows on through traffic and intersection operation.

II. METHODOLOGY

Exhibit 16-1 shows the input and the basic computation order for the method. The primary output of the method is level of service (LOS). This methodology covers a wide range of operational configurations, including combinations of phase plans, lane utilization, and left-turn treatment alternatives. It is important to note that some of these configurations may be considered unacceptable by some operating agencies from a traffic safety point of view. The safety aspect of signalized intersections cannot be ignored, and the provision in this chapter of a capacity and LOS analysis methodology for a specific operational configuration does not imply an endorsement of the suitability for application of such a configuration.

EXHIBIT 16-1. SIGNALIZED INTERSECTION METHODOLOGY



LOS

The average control delay per vehicle is estimated for each lane group and aggregated for each approach and for the intersection as a whole. LOS is directly related to the control delay value. The criteria are listed in Exhibit 16-2.

LOS criteria

EXHIBIT 16-2. LOS CRITERIA FOR SIGNALIZED INTERSECTIONS

LOS	Control Delay per Vehicle (s/veh)
A	≤ 10
B	> 10–20
C	> 20–35
D	> 35–55
E	> 55–80
F	> 80

INPUT PARAMETERS

Exhibit 16-3 provides a summary of the input information required to conduct an operational analysis for signalized intersections. This information forms the basis for selecting computational values and procedures in the modules that follow. The data needed are detailed and varied and fall into three main categories: geometric, traffic, and signalization.

Inputs needed
- *Geometric,*
- *Traffic, and*
- *Signalization*

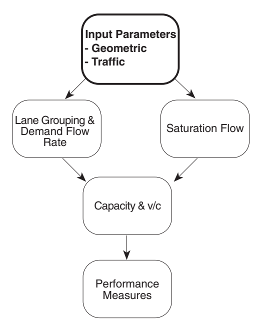
EXHIBIT 16-3. INPUT DATA NEEDS FOR EACH ANALYSIS LANE GROUP

Type of Condition	Parameter
Geometric conditions	Area type Number of lanes, N Average lane width, W (ft) Grade, G (%) Existence of exclusive LT or RT lanes Length of storage bay, LT or RT lane, L_s (ft) Parking
Traffic conditions	Demand volume by movement, V (veh/h) Base saturation flow rate, s_o (pc/h/ln) Peak-hour factor, PHF Percent heavy vehicles, HV (%) Approach pedestrian flow rate, v_{ped} (p/h) Local buses stopping at intersection, N_B (buses/h) Parking activity, N_m (maneuvers/h) Arrival type, AT Proportion of vehicles arriving on green, P Approach speed, S_A (mi/h)
Signalization conditions	Cycle length, C (s) Green time, G (s) Yellow-plus-all-red change-and-clearance interval (intergreen), Y (s) Actuated or pretimed operation Pedestrian push-button Minimum pedestrian green, G_p (s) Phase plan Analysis period, T (h)

Geometric Conditions

Intersection geometry is generally presented in diagrammatic form and must include all of the relevant information, including approach grades, the number and width of lanes, and parking conditions. The existence of exclusive left- or right-turn lanes should be noted, along with the storage lengths of such lanes.

When the specifics of geometry are to be designed, these features must be assumed for the analysis to continue. State or local policies and guidelines should be used in establishing the trial design. When these are not readily available, Chapter 10 contains suggestions for geometric design that may be useful in preparing an assumed preliminary design for analysis.



Traffic Conditions

Traffic volumes (for oversaturated conditions, demand must be used) for the intersection must be specified for each movement on each approach. These volumes are the flow rates in vehicles per hour for the 15-min analysis period, which is the duration of

15-min flow rates can be estimated using hourly volumes and PHFs the typical analysis period (T = 0.25). If the 15-min data are not known, they may be estimated using hourly volumes and peak-hour factors (PHFs). In situations where the v/c is greater than about 0.9, control delay is significantly affected by the length of the analysis period. In these cases, if the 15-min flow rate remains relatively constant for more than 15 min, the length of time the flow is constant should be used as the analysis period, T, in hours.

Study the entire period during which volumes approach and exceed capacity

If v/c exceeds 1.0 during the analysis period, the length of the analysis period should be extended to cover the period of oversaturation in the same fashion, as long as the average flow during that period is relatively constant. If the resulting analysis period is longer than 15 min and different flow rates can be identified during equal-length subperiods within the longer analysis period, a multiple-period analysis using the procedures in Appendix F should be performed using each of these subperiods individually. The length of the subperiods would normally be, but not be limited to, 15 min each.

Heavy vehicles are those having more than four tires on the pavement

Vehicle type distribution is quantified as the percent of heavy vehicles (% HV) in each movement, where heavy vehicles are defined as those with more than four tires touching the pavement. The number of local buses on each approach should also be identified, including only those buses making stops to pick up or discharge passengers at the intersection (on either the approach or departure side). Buses not making such stops are considered to be heavy vehicles.

Pedestrian and bicycle flows that interfere with permitted right or left turns are needed. The pedestrian and bicycle flows used to analyze a given approach are the flows in the crosswalk interfering with right turns from the approach. For example, for a westbound approach, the pedestrian and bicycle flows in the north crosswalk would be used for the analysis.

An important traffic characteristic that must be quantified to complete an operational analysis of a signalized intersection is the quality of the progression. The parameter that describes this characteristic is the arrival type, AT, for each lane group. Six arrival types for the dominant arrival flow are defined in Exhibit 16-4.

EXHIBIT 16-4. ARRIVAL TYPES

Arrival Type	Description
1	Dense platoon containing over 80 percent of the lane group volume, arriving at the start of the red phase. This AT is representative of network links that may experience very poor progression quality as a result of conditions such as overall network signal optimization.
2	Moderately dense platoon arriving in the middle of the red phase or dispersed platoon containing 40 to 80 percent of the lane group volume, arriving throughout the red phase. This AT is representative of unfavorable progression on two-way streets.
3	Random arrivals in which the main platoon contains less than 40 percent of the lane group volume. This AT is representative of operations at isolated and noninterconnected signalized intersections characterized by highly dispersed platoons. It may also be used to represent coordinated operation in which the benefits of progression are minimal.
4	Moderately dense platoon arriving in the middle of the green phase or dispersed platoon containing 40 to 80 percent of the lane group volume, arriving throughout the green phase. This AT is representative of favorable progression on a two-way street.
5	Dense to moderately dense platoon containing over 80 percent of the lane group volume, arriving at the start of the green phase. This AT is representative of highly favorable progression quality, which may occur on routes with low to moderate side-street entries and which receive high-priority treatment in the signal timing plan.
6	This arrival type is reserved for exceptional progression quality on routes with near-ideal progression characteristics. It is representative of very dense platoons progressing over a number of closely spaced intersections with minimal or negligible side-street entries.

The arrival type is best observed in the field but can be approximated by examining time-space diagrams for the street in question. The arrival type should be determined as accurately as possible because it will have a significant impact on delay estimates and LOS determination. Although there are no definitive parameters to precisely quantify arrival type, the platoon ratio is computed by Equation 16-1.

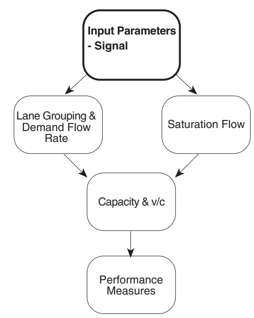
$$R_p = \frac{P}{\frac{g_i}{C}} \qquad (16\text{-}1)$$

where

R_p = platoon ratio,
P = proportion of all vehicles in movement arriving during green phase,
C = cycle length (s), and
g_i = effective green time for movement or lane group (s).

P may be estimated or observed in the field, whereas g_i and C are computed from the signal timing. The value of P may not exceed 1.0.

Signalization Conditions



Complete information regarding signalization is needed to perform an analysis. This information includes a phase diagram illustrating the phase plan, cycle length, green times, and change-and-clearance intervals. Lane groups operating under actuated control must be identified, including the existence of push-button pedestrian-actuated phases.

If pedestrian timing requirements exist, the minimum green time for the phase is indicated and provided for in the signal timing. The minimum green time for a phase is estimated by Equation 16-2 or local practice.

$$\begin{aligned} G_p &= 3.2 + \frac{L}{S_p} + \left(2.7\frac{N_{ped}}{W_E}\right) \quad \text{for } W_E > 10 \text{ ft} \\ G_p &= 3.2 + \frac{L}{S_p} + \left(0.27 N_{ped}\right) \quad \text{for } W_E \le 10 \text{ ft} \end{aligned} \qquad (16\text{-}2)$$

where

G_p = minimum green time (s),
L = crosswalk length (ft),
S_p = average speed of pedestrians (ft/s),
W_E = effective crosswalk width (ft),
3.2 = pedestrian start-up time (s), and
N_{ped} = number of pedestrians crossing during an interval (p).

15th-percentile pedestrian speed is assumed as 4.0 ft/s. Local values can be substituted.

It is assumed that the 15th-percentile walking speed of pedestrians crossing a street is 4.0 ft/s in this computation. This value is intended to accommodate crossing pedestrians who walk at speeds slower than the average. Where local policy uses different criteria for estimating minimum pedestrian crossing requirements, these criteria should be used in lieu of Equation 16-2.

When signal phases are actuated, the cycle length and green times will vary from cycle to cycle in response to demand. To establish values for analysis, the operation of the signal should be observed in the field during the same period that volumes are observed. Average field-measured values of cycle length and green time may then be used.

When signal timing is to be established for analysis, state or local policies and procedures should be applied where appropriate. Appendix B contains suggestions for the design of a trial signal timing. These suggestions should not be construed to be standards or criteria for signal design. A trial signal timing cannot be designed until the volume adjustment and saturation flow rate modules have been completed. In some cases, the computations will be iterative because left-turn adjustments for permitted turns used in the saturation flow rate module depend on signal timing. Appendix B also contains suggestions for estimating the timing of an actuated signal if field observations are unavailable.

Appendix B contains procedure for estimating average cycle lengths under actuated control

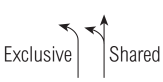
An operational analysis requires the specification of a signal timing plan for the intersection under study. The planning level application presented in Chapter 10 offers a procedure for establishing a reasonable and effective signal timing plan. This procedure is recommended only for the estimation of LOS and not for the design of an implementable signal timing plan. The signal timing design process is more complicated and involves, for example, iterative checks for minimum green-time violations. When phases are traffic actuated, the timing plan will differ for each cycle. The traffic-actuated procedure presented in Appendix B can be used to estimate the average cycle length and phase times under these conditions provided that the signal controller settings are available.

The design of an implementable timing plan is a complex and iterative process that can be carried out with the assistance of computer software. Although the methodology presented here is oriented toward the estimation of delay at traffic signals, it was suggested earlier that the computations can be applied iteratively to develop a signal timing plan. Some of the available signal timing software products employ the methodology of this chapter, at least in part.

There are, however, several aspects of signal timing design that are beyond the scope of this manual. One such aspect is the choice of the timing strategy itself. At intersections with traffic-actuated phases, the signal timing plan is determined on each cycle by the instantaneous traffic demand and the controller settings. When all of the phases are pretimed, a timing plan design must be developed. Timing plan design and estimation are covered in detail in Appendix B.

LANE GROUPING

The methodology for signalized intersections is disaggregate; that is, it is designed to consider individual intersection approaches and individual lane groups within approaches. Segmenting the intersection into lane groups is a relatively simple process that considers both the geometry of the intersection and the distribution of traffic movements. In general, the smallest number of lane groups is used that adequately describes the operation of the intersection. The following guidelines may be applied.



- An exclusive left-turn lane or lanes should normally be designated as a separate lane group unless there is also a shared left-through lane present, in which case the proper lane grouping will depend on the distribution of traffic volume between the movements. The same is true of an exclusive right-turn lane.
- On approaches with exclusive left-turn or right-turn lanes, or both, all other lanes on the approach would generally be included in a single lane group.
- When an approach with more than one lane includes a lane that may be used by both left-turning vehicles and through vehicles, it is necessary to determine whether equilibrium conditions exist or whether there are so many left turns that the lane essentially acts as an exclusive left-turn lane, which is referred to as a de facto left-turn lane.

The analyst should determine if there is a de facto left-turn lane

De facto left-turn lanes cannot be identified effectively until the proportion of left turns in the shared lane has been computed. If the computed proportion of left turns in the shared lane equals 1.0 (i.e., 100 percent), the shared lane must be considered a de facto left-turn lane.

When two or more lanes are included in a lane group for analysis purposes, all subsequent computations treat these lanes as a single entity. Exhibit 16-5 shows some common lane groups used for analysis.

EXHIBIT 16-5. TYPICAL LANE GROUPS FOR ANALYSIS

Number of Lanes	Movements by Lanes	Number of Possible Lane Groups
1	LT + TH + RT	1 (Single-lane approach)
2	EXC LT TH + RT	2
2	LT + TH TH + RT	1 OR 2
3	EXC LT TH TH + RT	2 OR 3

DETERMINING FLOW RATE

Demand volumes are best provided as average flow rates (in vehicles per hour) for the analysis period. Although analysis periods are usually 15 min long, the procedures for this chapter allow for any length of time to be used. However, demand volumes may also be stated for a time that encompasses more than one analysis period, such as an hourly volume. In such cases, peaking factors must be provided that convert these to demand flow rates for each particular analysis period.

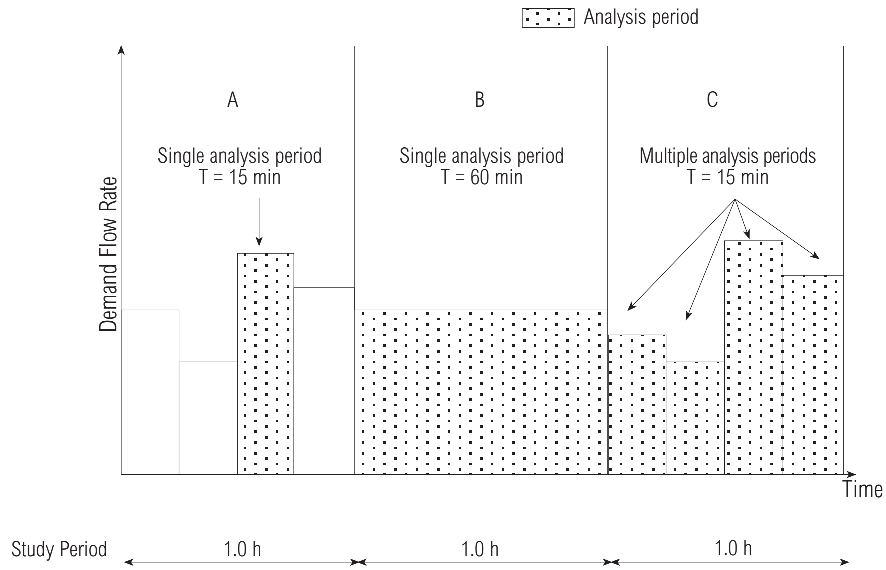
Alternative Study Approaches

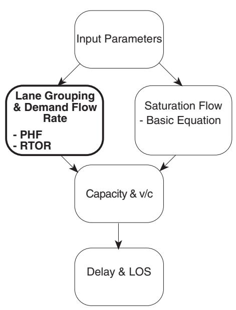
If queue carryover occurs, a multiple-period analysis is best

Two major analytic steps are performed in the volume adjustment module. Movement volumes are adjusted to flow rates for each desired period of analysis, if necessary, and lane groups for analysis are established. Exhibit 16-6 demonstrates three alternative ways in which an analyst might proceed for a given study. Other alternatives exist. Approach A is the one that has traditionally been used in the HCM. The length of the period being analyzed is only 15 min, and the analysis period (T), therefore, is 15 min or 0.25 h. In this case, either a peak 15-min volume is available or one is derived from an hourly volume by use of a PHF. A difficulty with considering only one 15-min period is that a queue may be left at the end of the analysis period because of demand in excess of capacity. In such cases it is possible that the queue carried over to the next period will result in delay to vehicles that arrive in that period beyond that which would have resulted had there not been a queue carryover.

Approach A may involve use of PHF, but Approach C will not

EXHIBIT 16-6. THREE ALTERNATIVE STUDY APPROACHES





Approach B involves a study of an entire hour of operation at the site using an analysis period (T) of 60 min. In this case, the analyst may have included the more critical period of operation, missed under Approach A, but because the volume being used is an hourly one, it implicitly assumes that the arrival of vehicles on the approach is distributed equally across the period of study. Therefore, the effects of peaking within the hour may not be identified, especially if, by the end of the hour, any excess queuing can be dissipated. The analyst therefore runs the risk of underestimating delays during the hour. If a residual queue remains at the end of 60 min, a second 60-min period of analysis can be used (and so on) until the total period ends with no excess queue.

Approach C involves a study of the entire hour but divides it into four 15-min analysis periods (T). The procedures in this chapter allow the analyst to account for queues that carry over to the next analysis period. Therefore, when demand exceeds capacity during the study period, a more accurate representation of delay experienced during the hour can be achieved using this method.

A peak 15-min flow rate is derived from an hourly volume by dividing the movement volumes by an appropriate PHF, which may be defined for the intersection as a whole, for each approach, or for each movement. The flow rate is computed using Equation 16-3.

$$v_p = \frac{V}{PHF} \tag{16-3}$$

where

v_p = flow rate during peak 15-min period (veh/h),
V = hourly volume (veh/h), and
PHF = peak-hour factor.

Use of a single PHF assumes that all movements peak in the same period

The conversion of hourly volumes to peak flow rates using the PHF assumes that all movements peak during the same 15-min period, and somewhat higher estimates of control delay will result. PHF values of 1.0 should be used if 15-min flow rates are entered directly. Because not all intersection movements may peak at the same time, it is valuable to observe 15-min flows directly and select critical periods for analysis. It is particularly conservative if different PHF values are assumed for each movement. It should be noted also that statistically valid surveys of the PHF for individual movements are difficult to obtain during a single peak hour.

Adjustment for Right Turn on Red

Subtract RTOR volume from RT volume

When right turn on red (RTOR) is permitted, the right-turn volume for analysis may be reduced by the volume of right-turning vehicles moving on the red phase. This reduction is generally done on the basis of hourly volumes before the conversion to flow rates.

The number of vehicles able to turn right on a red phase is a function of several factors, including

- Approach lane allocation (shared or exclusive right-turn lane),
- Demand for right-turn movements,
- Sight distance at the intersection approach,
- Degree of saturation of the conflicting through movement,
- Arrival patterns over the signal cycle,
- Left-turn signal phasing on the conflicting street, and
- Conflicts with pedestrians.

For an existing intersection, it is appropriate to consider the RTORs that actually occur. For both the shared lane and the exclusive right-turn lane conditions, the number of RTORs may be subtracted from the right-turn volume before analysis of lane group capacity or LOS. At an existing intersection, the number of RTORs should be determined by field observation.

If field data are not available, ignore RTOR, except in special cases. Remove free-flowing RTs from RT volume.

If the analysis is dealing with future conditions or if the RTOR volume is not known from field data, it is necessary to estimate the number of RTOR vehicles. In the absence of field data, it is preferable for most purposes to utilize the right-turn volumes directly without a reduction for RTOR except when an exclusive right-turn lane movement runs concurrent with a protected left-turn phase from the cross street. In this case the total right-turn volume for analysis can be reduced by the number of shadowed left turners. Free-flowing right turns that are not under signal control should be removed entirely from the analysis.

DETERMINING SATURATION FLOW RATE

A saturation flow rate for each lane group is computed according to Equation 16-4. The saturation flow rate is the flow in vehicles per hour that can be accommodated by the lane group assuming that the green phase were displayed 100 percent of the time (i.e., g/C = 1.0).

$$s = s_o\, N\, f_w\, f_{HV}\, f_g\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{LT}\, f_{RT}\, f_{Lpb}\, f_{Rpb} \tag{16-4}$$

where

See Exhibit 16-7 for formulas. For default values refer to Chapter 10.

s = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h);
s_o = base saturation flow rate per lane (pc/h/ln);
N = number of lanes in lane group;
f_w = adjustment factor for lane width;
f_{HV} = adjustment factor for heavy vehicles in traffic stream;
f_g = adjustment factor for approach grade;
f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;
f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area;
f_a = adjustment factor for area type;
f_{LU} = adjustment factor for lane utilization;
f_{LT} = adjustment factor for left turns in lane group;
f_{RT} = adjustment factor for right turns in lane group;
f_{Lpb} = pedestrian adjustment factor for left-turn movements; and
f_{Rpb} = pedestrian-bicycle adjustment factor for right-turn movements.

Field measurement method for saturation flow is described in Appendix H

Appendix H presents a field measurement method for determining saturation flow rate. Field-measured values of saturation flow rate will produce more accurate results than the estimation procedure described here and can be used directly without further adjustment.

Base Saturation Flow Rate

Computations begin with the selection of a base saturation flow rate, usually 1,900 passenger cars per hour per lane (pc/h/ln). This value is adjusted for a variety of conditions. The adjustment factors are given in Exhibit 16-7.

Adjustment for Lane Width

Do not use width < 8.0 ft for calculations

The lane width adjustment factor, f_w, accounts for the negative impact of narrow lanes on saturation flow rate and allows for an increased flow rate on wide lanes. Standard lane widths are 12 ft. The lane width factor may be calculated with caution for lane widths greater than 16 ft, or an analysis using two narrow lanes may be conducted. Note that use of two narrow lanes will always result in a higher saturation flow rate than a single wide lane, but in either case, the analysis should reflect the way in which the width is actually used or expected to be used. In no case should the lane width factor be calculated for widths less than 8.0 ft.

Adjustment for Heavy Vehicles and Grade

The effects of heavy vehicles and approach grades are treated by separate factors, f_{HV} and f_g, respectively. Their separate treatment recognizes that passenger cars are affected by approach grades, as are heavy vehicles. Heavy vehicles are defined as those with more than four tires touching the pavement. The heavy-vehicle factor accounts for the additional space occupied by these vehicles and for the difference in operating capabilities of heavy vehicles compared with passenger cars. The passenger-car equivalent (E_T) used for each heavy vehicle is 2.0 passenger-car units and is reflected in the formula. The grade factor accounts for the effect of grades on the operation of all vehicles.

Adjustment for Parking

Parking maneuver assumed to block traffic for 18 s. Use practical limit of 180 maneuvers/h.

The parking adjustment factor, f_p, accounts for the frictional effect of a parking lane on flow in an adjacent lane group as well as for the occasional blocking of an adjacent lane by vehicles moving into and out of parking spaces. Each maneuver (either in or out) is assumed to block traffic in the lane next to the parking maneuver for an average of 18 s. The number of parking maneuvers used is the number of maneuvers per hour in parking areas directly adjacent to the lane group and within 250 ft upstream from the stop line. If more than 180 maneuvers per hour exist, a practical limit of 180 should be used. If the parking is adjacent to an exclusive turn lane group, the factor only applies to that lane group. On a one-way street with no exclusive turn lanes, the number of maneuvers used is the total for both sides of the lane group. Note that parking conditions with zero maneuvers have a different impact than a no-parking situation.

Adjustment for Bus Blockage

Applies to bus stops within 250 ft of the stop line and a limit of 250 buses/h

The bus blockage adjustment factor, f_{bb}, accounts for the impacts of local transit buses that stop to discharge or pick up passengers at a near-side or far-side bus stop within 250 ft of the stop line (upstream or downstream). This factor should only be used when stopping buses block traffic flow in the subject lane group. If more than 250 buses per hour exist, a practical limit of 250 should be used. When local transit buses are believed to be a major factor in intersection performance, Chapter 27 may be consulted for more information on this effect. The factor used here assumes an average blockage time of 14.4 s during a green indication.

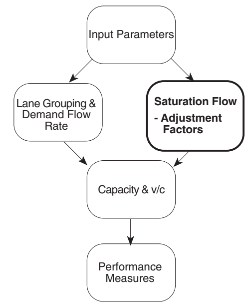


EXHIBIT 16-7. ADJUSTMENT FACTORS FOR SATURATION FLOW RATE[a]

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, a two-lane analysis may be considered
Heavy vehicles	$f_{HV} = \frac{100}{100 + \%HV(E_T - 1)}$	% HV = % heavy vehicles for lane group volume	E_T = 2.0 pc/HV
Grade	$f_g = 1 - \frac{\%G}{200}$	% G = % 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 = 1.000 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 Exhibit C16-1, Appendix C, for nonprotected 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 LT green P_{RT} = proportion of RTs in lane group P_{RTA} = proportion of RT protected green over total RT green	Refer to Appendix D for step-by-step procedure

Note:
See Chapter 10, Exhibit 10-12, for default values of base saturation flow rates and variables used to derive adjustment factors.
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.

The factor reflects increased headways due to regular and frequent interferences

Adjustment for Area Type

The area type adjustment factor, f_a, accounts for the relative inefficiency of intersections in business districts in comparison with those in other locations.

Application of this adjustment factor is typically appropriate in areas that exhibit central business district (CBD) characteristics. These characteristics include narrow street rights-of-way, frequent parking maneuvers, vehicle blockages, taxi and bus activity, small-radius turns, limited use of exclusive turn lanes, high pedestrian activity, dense population, and mid-block curb cuts. Use of this factor should be determined on a case-by-case basis. This factor is not limited to designated CBD areas, nor will it need to be used for all CBD areas. Instead, this factor should be used in areas where the geometric design and the traffic or pedestrian flows, or both, are such that the vehicle headways are significantly increased to the point where the capacity of the intersection is adversely affected.

Adjustment for Lane Utilization

The lane utilization adjustment factor, f_{LU}, accounts for the unequal distribution of traffic among the lanes in a lane group with more than one lane. The factor provides an adjustment to the base saturation flow rate. The adjustment factor is based on the flow in the lane with the highest volume and is calculated by Equation 16-5:

$$f_{LU} = \frac{v_g}{\left(v_{g1}N\right)} \tag{16-5}$$

where

f_{LU} = lane utilization adjustment factor,
v_g = unadjusted demand flow rate for lane group (veh/h),
v_{g1} = unadjusted demand flow rate on single lane with highest volume in lane group (veh/h), and
N = number of lanes in lane group.

This adjustment is normally applied and can be used to account for the variation of traffic flow on the individual lanes in a lane group due to upstream or downstream roadway characteristics such as changes in the number of lanes available or flow characteristics such as the prepositioning of traffic within a lane group for heavy turning movements.

Actual lane volume distributions observed in the field should be used, if known, in the computation of the lane utilization adjustment factor. A lane utilization factor of 1.0 can be used when uniform traffic distribution can be assumed across all lanes in the lane group or when a lane group comprises a single lane. When average conditions exist or traffic distribution in a lane group is not known, the default values summarized in Chapter 10 can be used. Guidance on how to account for impacts of short lane adds or drops is also given in Chapter 10.

The right-turn adjustment factor is 1.0 if the lane group does not include any right turns

Adjustment for Right Turns

The right-turn adjustment factors, f_{RT}, in Exhibit 16-7 are primarily intended to reflect the effect of geometry. A separate pedestrian and bicycle blockage factor is used to reflect the volume of pedestrians and bicycles using the conflicting crosswalk.

The right-turn adjustment factor depends on a number of variables, including

- Whether the right turn is made from an exclusive or shared lane, and
- Proportion of right-turning vehicles in the shared lanes.

The right-turn factor is 1.0 if the lane group does not include any right turns. When RTOR is permitted, the right-turn volume may be reduced as described in the discussion of RTOR.

Adjustment for Left Turns

The left-turn adjustment factor, f_{LT}, is based on variables similar to those for the right-turn adjustment factor, including

- Whether left turns are made from exclusive or shared lanes,
- Type of phasing (protected, permitted, or protected-plus-permitted),
- Proportion of left-turning vehicles using a shared lane group, and
- Opposing flow rate when permitted left turns are made.

The left-turn adjustment factor is 1.0 if the lane group does not include any left turns

An additional factor for pedestrian blockage is provided, based on pedestrian volumes. Left-turn adjustment factors are used for six cases of left-turn phasing, as follows:

- Case 1: Exclusive lane with protected phasing,
- Case 2: Exclusive lane with permitted phasing,
- Case 3: Exclusive lane with protected-plus-permitted phasing,
- Case 4: Shared lane with protected phasing,
- Case 5: Shared lane with permitted phasing, and
- Case 6: Shared lane with protected-plus-permitted phasing.

Phasing	*Left Turn Adjustment Cases*	
	Lane	
	LT Excl	*LT Share*
Protected	*1*	*4*
Permitted	*2*	*5*
Prot/Perm	*3*	*6*

Adjustment for Pedestrians and Bicyclists

The procedure to determine the left-turn pedestrian-bicycle adjustment factor, f_{Lpb}, and the right-turn pedestrian-bicycle adjustment factor, f_{Rpb}, consists of four steps. The first step is to determine average pedestrian occupancy, which only accounts for the pedestrian effect. Then relevant conflict zone occupancy, which accounts for both pedestrian and bicycle effects, is determined. Relevant conflict zone occupancy takes into account whether other traffic is also in conflict (e.g., adjacent bicycle flow for the case of right turns or opposing vehicle flow for the case of left turns). In either case, adjustments to the initial occupancy are made. The proportion of green time in which the conflict zone is occupied is determined as a function of the relevant occupancy and the number of receiving lanes for the turning vehicles.

The proportion of right turns using the protected portion of a protected-plus-permitted phase is also needed. This proportion should be determined by field observation, but a gross estimate can be made from the signal timing by assuming that the proportion of right-turning vehicles using the protected phase is approximately equal to the proportion of the turning phase that is protected. If $P_{RTA} = 1.0$ (that is, the right turn is completely protected from conflicting pedestrians), a pedestrian volume of zero should be used.

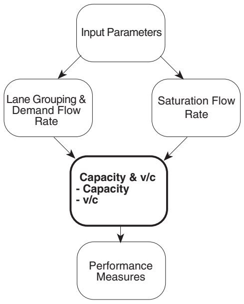
Finally, the saturation flow adjustment factor is calculated from the final occupancy on the basis of the turning movement protection status and the percent of turning traffic in the lane group. A comprehensive step-by-step procedure is provided in Appendix D.

DETERMINING CAPACITY AND v/c RATIO

Capacity

Capacity and flow ratio defined

Capacity at signalized intersections is based on the concept of saturation flow and saturation flow rate. The flow ratio for a given lane group is defined as the ratio of the actual or projected demand flow rate for the lane group (v_i) and the saturation flow rate (s_i). The flow ratio is given the symbol $(v/s)_i$ for lane group i. The capacity of a given lane group may be stated as shown in Equation 16-6:



$$c_i = s_i \frac{g_i}{C} \tag{16-6}$$

where

c_i = capacity of lane group i (veh/h),
s_i = saturation flow rate for lane group i (veh/h), and
g_i/C = effective green ratio for lane group i.

Green ratio defined

Degree of saturation defined

v/c Ratio

The ratio of flow rate to capacity (v/c), often called the volume to capacity ratio, is given the symbol X in intersection analysis. It is typically referred to as degree of saturation. For a given lane group i, X_i is computed using Equation 16-7.

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{s_i\left(\frac{g_i}{C}\right)} = \frac{v_i C}{s_i g_i} \tag{16-7}$$

where

X_i = $(v/c)_i$ = ratio for lane group i,
v_i = actual or projected demand flow rate for lane group i (veh/h),
s_i = saturation flow rate for lane group i (veh/h),
g_i = effective green time for lane group i (s), and
C = cycle length (s).

Sustainable values of X_i range from 1.0 when the flow rate equals capacity to zero when the flow rate is zero. Values above 1.0 indicate an excess of demand over capacity. The capacity of the entire intersection is not a significant concept and is not specifically defined here. Rarely do all movements at an intersection become saturated at the same time of day.

Critical Lane Groups

X_c is v/c for critical movements, assuming green time allocated proportionately to v/s values

Another concept used for analyzing signalized intersections is the critical v/c ratio, X_c. This is the v/c ratio for the intersection as a whole, considering only the lane groups that have the highest flow ratio (v/s) for a given signal phase. For example, with a two-phase signal, opposing lane groups move during the same green time. Generally, one of these two lane groups will require more green time than the other (i.e., it will have a higher flow ratio). This would be the critical lane group for that signal phase. Each signal phase will have a critical lane group that determines the green-time requirements for the phase. When signal phases overlap, the identification of these critical lane groups becomes somewhat complex. The critical v/c ratio for the intersection is determined by using Equation 16-8:

$$X_c = \Sigma\left(\frac{v}{s}\right)_{ci}\left(\frac{C}{C-L}\right) \tag{16-8}$$

where

X_c = critical v/c ratio for intersection;
$\Sigma\left(\frac{v}{s}\right)_{ci}$ = summation of flow ratios for all critical lane groups i;
C = cycle length (s); and
L = total lost time per cycle, computed as lost time, t_L, for critical path of movements (s).

Equation 16-8 is useful in evaluating the overall intersection with respect to the geometrics and total cycle length and also in estimating signal timings when they are unknown or not specified by local policies or procedures. It gives the v/c ratio for all critical movements, assuming that green time has been allocated in proportion to the v/s values. Flow ratios are computed by dividing the adjusted demand flow, v, computed in the volume adjustment module by the adjusted saturation flow rate, s.

If the signal timing is not known, a timing plan will have to be estimated or assumed to make these computations. Appendix B contains suggestions for making these estimates, but state or local policies and guidelines should also be consulted. A quick estimation method also offers a procedure for the synthesis of timing plans based on the concepts presented in Chapter 10.

The v/c ratio for each lane group is computed directly by dividing the adjusted flows by the capacities computed above, as in Equation 16-7. It is possible to have a critical v/c ratio of less than 1.0 and still have individual movements oversaturated within the signal cycle. A critical v/c ratio less than 1.0, however, does indicate that all movements in the intersection can be accommodated within the defined cycle length and phase sequence by proportionally allocating green time.

The X_c value can, however, be misleading when used as an indicator of the overall sufficiency of the intersection geometrics, as is often required in planning applications. The problem is that low flow rates dictate the need for short cycle lengths to minimize delay. Equation 16-8 suggests that shorter cycle lengths produce a higher X_c for a specified level of traffic demand. Furthermore, many signal timing methods, including the quick estimation method described in Appendix A of Chapter 10, are based on a fixed target value of X_c. This tends to make X_c independent of the demand volumes.

To compute X_c, the critical lane groups must be identified

The computation of the critical v/c ratio, X_c, requires that critical lane groups be identified. During each signal phase, green indications are displayed to one or more lane groups. One lane group will have the most intense demand and will be the one that determines the amount of green time needed. This lane group will be the critical lane group for the phase in question.

The normalized measure of demand intensity in any lane group is given by the v/s ratio. With no overlapping phases in the signal design, such as in a simple two-phase signal, the determination of critical lane groups is straightforward. In each discrete phase, the lane group with the highest v/s ratio is critical.

Overlapping phases are more difficult to analyze because various lane groups may have traffic flow in several phases of the signal, and some left-turn movements may operate on a protected-and-permitted basis in various portions of the cycle. In such cases, it is necessary to find the critical path through the signal cycle. The path having the highest sum of v/s ratios is the critical path.

Guidelines for identifying critical lane groups

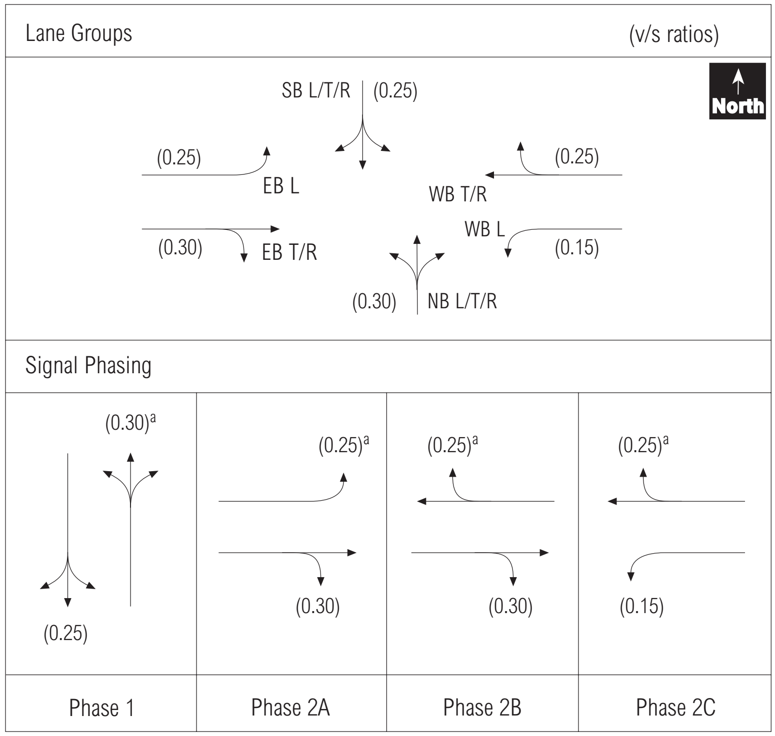
When phases overlap, the critical path must conform to the following rules:

- Excluding lost times, one critical lane group must be moving at all times during the signal cycle;
- At no time in the signal cycle may more than one critical lane group be moving; and
- The critical path has the highest sum of v/s ratios.

These rules are more easily explained by example. Consider the case of a leading and lagging green phase plan on a street with exclusive left-turn lanes, as shown in Exhibit 16-8. Phase 1 is discrete, with NB and SB lane groups moving simultaneously. The critical lane group for Phase 1 is chosen on the basis of the highest v/s ratio, which is 0.30 for the NB lane group.

Phase 2 involves overlapping leading and lagging green phases. There are two possible paths through Phase 2 that conform to the stated rule that (except for lost times) there must be only one critical lane group moving at any time. The EB through and right-turn (T/R) lane group moves through Phases 2A and 2B with a v/s ratio of 0.30. The WB left-turn lane group moves only in Phase 2C with a v/s ratio of 0.15. The total v/s ratio for this path is therefore 0.30 + 0.15, or 0.45. The only alternative path involves the EB left-turn lane group, which moves only in Phase 2A (v/s = 0.25), and the WB T/R lane group, which moves in Phases 2B and 2C (v/s = 0.25). Because the sum of the v/s ratios for this path is 0.25 + 0.25 = 0.50, this is the critical path through Phase 2. Thus, the sum of critical v/s ratios for the cycle is 0.30 for Phase 1 plus 0.50 for Phase 2, for a total of 0.80.

EXHIBIT 16-8. CRITICAL LANE GROUP DETERMINATION WITH PROTECTED LEFT TURNS



Note:
a. Critical v/s.

The solution for X_c also requires that the lost time for the critical path (L) through the signal be determined. Using the general rule that a movement's lost time of t_L is applied when a movement is initiated, the following conclusions are reached:

- The critical NB movement is initiated in Phase 1, and its lost time is applied;
- The critical EB left-turn movement is initiated in Phase 2A, and its lost time is applied;
- The critical WB T/L movement is initiated in Phase 2B, and its lost time is applied;
- No critical movement is initiated in Phase 2C, so no lost time is applied to the critical path here; although the WB left-turn movement is initiated in this phase, it is not a critical movement, and its lost time is not included in L; and
- For this case, $L = 3t_L$, assuming that each movement has the same lost time, t_L.

This problem may be altered significantly by adding a permitted left turn in both directions to Phase 2B, as shown in Exhibit 16-9, with the resulting v/s ratios. Note that in this case, a separate v/s ratio is computed for the protected and permitted portions of the EB and WB left-turn movements. In essence, the protected and permitted portions of these movements are treated as separate lane groups.

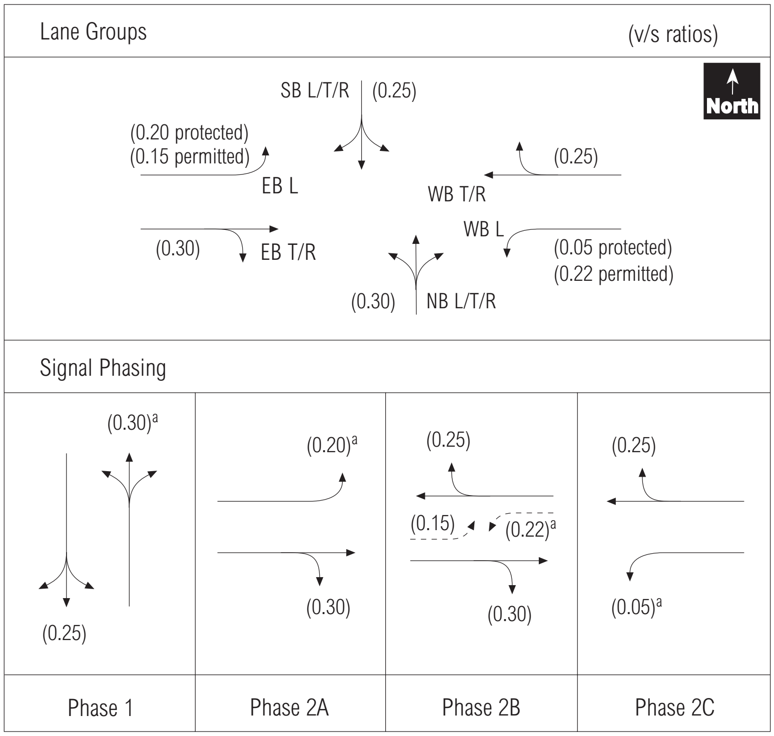
The analysis of Phase 1 does not change, because it is discrete. The NB lane group is still critical, with a v/s ratio of 0.30. There are now four different potential paths through Phase 2 that conform to the rules for determining critical paths:

- WB T/R + EB left turn (protected) = 0.25 + 0.20 = 0.45,
- EB T/R + WB left turn (protected) = 0.30 + 0.05 = 0.35,

• EB left turn (protected) + EB left turn (permitted) + WB left turn (protected) = 0.20 + 0.15 + 0.05 = 0.40, and

• EB left turn (protected) + WB left turn (permitted) + WB left turn (protected) = 0.20 + 0.22 + 0.05 = 0.47.

EXHIBIT 16-9. CRITICAL LANE GROUP DETERMINATION WITH PROTECTED AND PERMITTED LEFT TURNS



Note:
a. Critical v/s.

The critical path through Phase 2 is the alternative with the highest total v/s ratio, in this case, 0.47. When 0.47 is added to the 0.30 for Phase 1, the sum of critical v/s ratios is 0.77.

The lost time for the critical path is determined as follows:

• The NB critical flow begins in Phase 1, and its lost time is applied;

• The critical EB left turn (protected) is initiated in Phase 2A, and its lost time is applied;

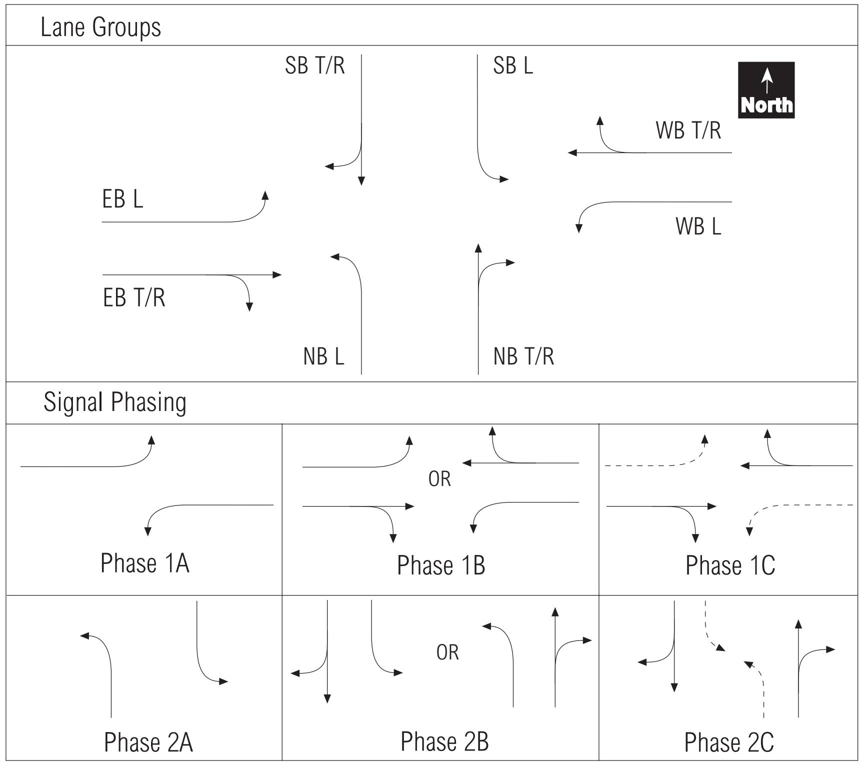
• The critical WB left turn (permitted) is initiated in Phase 2B, and its lost time is applied;

• The critical WB left turn (protected) is a continuation of the WB left turn (permitted); because the left-turn movement is already moving when Phase 2C is initiated, no lost time is applied here; and

• For this case, $L = 3t_L$, assuming that each movement has the same lost time, t_L.

Exhibit 16-10 shows another complex case with actuated control and a typical eight-phase plan. Although eight phases are provided on the controller, the path through the cycle cannot include more than six of these phases, as shown. The leading phases (1B and 2B) will be chosen on the basis of which left-turn movements have higher demands on a cycle-by-cycle basis.

EXHIBIT 16-10. CRITICAL LANE GROUP DETERMINATION FOR MULTIPHASE SIGNAL



The potential critical paths through Phase 1 are as follows:

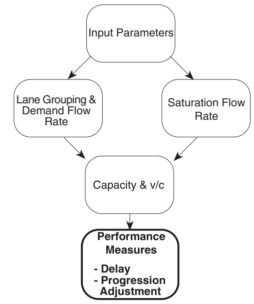
- EB left turn (protected) + EB left turn (permitted),
- EB left turn (protected) + WB left turn (permitted),
- EB left turn (protected) + WB T/R,
- WB left turn (protected) + WB left turn (permitted),
- WB left turn (protected) + EB left turn (permitted), and
- WB left turn (protected) + EB T/R.

The combination with the highest v/s ratio would be chosen as the critical path. A similar set of choices exists for Phase 2, with NB replacing EB and SB replacing WB.

The most interesting aspect of this problem is the number of lost times that must be included in L for each of these paths. The paths involving EB left turn (protected) + EB left turn (permitted) and WB left turn (protected) + WB left turn (permitted) each involve only one application of t_L because the turning movement in question moves continuously throughout the three subphases. All other paths involve two applications of t_L because each critical movement is initiated in a distinct portion of the phase. Note that the left turn that does not continue in Phase 1B or 2B is a discontinuous movement; that is, it moves as a protected turn in Phase 1A or 2A, stops in Phase 1B or 2B, and moves again as a permitted turn in Phase 1C or 2C.

For this complex phasing, the lost time through each major phase could have one or two lost times applied, based on the critical path. Therefore, for the total cycle, which comprises two streets, two to four lost times will be applied, again depending on the critical path. In general terms, up to n lost times are to be applied in the calculation of the total lost time per cycle, where n is the number of movements in the critical path through the signal cycle. For the purposes of determining n, a protected-plus-permitted movement is considered to be one movement if the protected and permitted phases are contiguous.

DETERMINING DELAY



The values derived from the delay calculations represent the average control delay experienced by all vehicles that arrive in the analysis period, including delays incurred beyond the analysis period when the lane group is oversaturated. Control delay includes movements at slower speeds and stops on intersection approaches as vehicles move up in queue position or slow down upstream of an intersection.

The average control delay per vehicle for a given lane group is given by Equation 16-9. Appendix A provides a procedure to measure control delay in the field.

$$d = d_1(PF) + d_2 + d_3 \tag{16-9}$$

where

- d = control delay per vehicle (s/veh);
- d_1 = uniform control delay assuming uniform arrivals (s/veh);
- PF = uniform delay progression adjustment factor, which accounts for effects of signal progression;
- d_2 = incremental delay to account for effect of random arrivals and oversaturation queues, adjusted for duration of analysis period and type of signal control; this delay component assumes that there is no initial queue for lane group at start of analysis period (s/veh); and
- d_3 = initial queue delay, which accounts for delay to all vehicles in analysis period due to initial queue at start of analysis period (s/veh) (detailed in Appendix F of this chapter).

Progression Adjustment Factor

Progression primarily affects uniform delay

Good signal progression will result in a high proportion of vehicles arriving on the green. Poor signal progression will result in a low proportion of vehicles arriving on the green. The progression adjustment factor, PF, applies to all coordinated lane groups, including both pretimed control and nonactuated lane groups in semiactuated control systems. In circumstances where coordinated control is explicitly provided for actuated lane groups, PF may also be applied to these lane groups. Progression primarily affects uniform delay, and for this reason, the adjustment is applied only to d_1. The value of PF may be determined using Equation 16-10.

$$PF = \frac{(1-P)f_{PA}}{1-\left(\frac{g}{C}\right)} \tag{16-10}$$

where

- PF = progression adjustment factor,
- P = proportion of vehicles arriving on green,
- g/C = proportion of green time available, and
- f_{PA} = supplemental adjustment factor for platoon arriving during green.

If PF for Arrival Type 4 calculates to greater than 1.0, set the value to 1.0

The value of P may be measured in the field or estimated from the arrival type. If field measurements are carried out, P should be determined as the proportion of vehicles in the cycle that arrive at the stop line or join the queue (stationary or moving) while the green phase is displayed. The approximate ranges of R_p are related to arrival type as shown in Exhibit 16-11, and default values are suggested for use in subsequent computations in Exhibit 16-12.

See Exhibit 16-4 for definition of arrival types

EXHIBIT 16-11. 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	> 0.50–0.85	0.667	Unfavorable
3	> 0.85–1.15	1.000	Random arrivals
4	> 1.15–1.50	1.333	Favorable
5	> 1.50–2.00	1.667	Highly favorable
6	> 2.00	2.000	Exceptional

EXHIBIT 16-12. PROGRESSION ADJUSTMENT FACTOR FOR UNIFORM DELAY CALCULATION

Green Ratio (g/C)	Arrival Type (AT)					
	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
f_{PA}	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

Notes:
PF = $(1 - P)f_{PA}/(1 - g/C)$.
Tabulation is based on default values of f_{PA} and R_p.
P = R_p * g/C (may not exceed 1.0).
PF may not exceed 1.0 for AT 3 through AT 6.

PF may be computed from measured values of P using the given values for f_{PA}. Alternatively, Exhibit 16-12 may be used to determine PF as a function of the arrival type based on the default values for P (i.e., $R_p g_i/C$) and f_{PA} associated with each arrival type. If PF is estimated by Equation 16-10, its calculated value may exceed 1.0 for Arrival Type 4 with extremely low values of g/C. As a practical matter, PF should be assigned a maximum value of 1.0 for Arrival Type 4.

Use Arrival Type 4 for coordinated lane groups

When delay is estimated for future situations involving coordination, particularly in the analysis of alternatives, it is advisable to assume Arrival Type 4 as a base condition for coordinated lane groups (except left turns). Arrival Type 3 should be assumed for all uncoordinated lane groups.

Use Arrival Type 3 for random arrivals

Movements made from exclusive left-turn lanes on protected phases are not usually provided with good progression. Thus, Arrival Type 3 is usually assumed for coordinated left turns. When the actual arrival type is known, it should be used. When the coordinated left turn is part of a protected-permitted phasing, the effective green for the protected phase should only be used to determine PF since the protected phase is normally the phase associated with platooned coordination. When a lane group contains movements that have different levels of coordination, a flow-weighted average of P should be used in determining the PF.

Uniform Delay

Equation 16-11 gives an estimate of delay assuming uniform arrivals, stable flow, and no initial queue. It is based on the first term of Webster's delay formulation and is widely accepted as an accurate depiction of delay for the idealized case of uniform arrivals (*7*). Note that values of X beyond 1.0 are not used in the computation of d_1.

Appendix E contains discussions of how to compute uniform delay for protected-plus-permitted left-turn operation.

$$d_1 = \frac{0.5C\left(1-\frac{g}{C}\right)^2}{1-\left[\min(1,X)\frac{g}{C}\right]} \qquad (16\text{-}11)$$

where

d_1 = uniform control delay assuming uniform arrivals (s/veh);
C = cycle length (s); cycle length used in pretimed signal control, or average cycle length for actuated control (see Appendix B for signal timing estimation of actuated control parameters);
g = effective green time for lane group (s); green time used in pretimed signal control, or average lane group effective green time for actuated control (see Appendix B for signal timing estimation of actuated control parameters); and
X = v/c ratio or degree of saturation for lane group.

Incremental Delay

Incremental delay reflects nonuniform arrivals and some queue carryover between cycles within the analysis period

Equation 16-12 is used to estimate the incremental delay due to nonuniform arrivals and temporary cycle failures (random delay) as well as delay caused by sustained periods of oversaturation (oversaturation delay). It is sensitive to the degree of saturation of the lane group (X), the duration of the analysis period (T), the capacity of the lane group (c), and the type of signal control, as reflected by the control parameter (k). The equation assumes that there is no unmet demand that causes initial queues at the start of the analysis period (T). Should that not be the case, the analyst should refer to Appendix F for additional procedures that can account for the effect on control delay of a nonzero initial queue. Finally, the incremental delay term is valid for all values of X, including highly oversaturated lane groups.

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right] \qquad (16\text{-}12)$$

where

d_2 = incremental delay to account for effect of random and oversaturation queues, adjusted for duration of analysis period and type of signal control (s/veh); this delay component assumes that there is no initial queue for lane group at start of analysis period;
T = duration of analysis period (h);
k = incremental delay factor that is dependent on controller settings;
I = upstream filtering/metering adjustment factor;
c = lane group capacity (veh/h); and
X = lane group v/c ratio or degree of saturation.

Incremental Delay Calibration Factor

The calibration term (k) is included in Equation 16-12 to incorporate the effect of controller type on delay. For pretimed signals, a value of k = 0.50 is used, which is based on a queuing process with random arrivals and uniform service time equivalent to the lane group capacity. Actuated controllers, on the other hand, have the ability to tailor the green time to traffic demand, thus reducing incremental delay. The delay reduction depends in part on the controller's unit extension and the prevailing v/c ratio. Recent research indicates that lower unit extensions (i.e., snappy intersection operation) result in lower values of k and d_2. However, when v/c approaches 1.0, an actuated controller will tend to behave in a manner similar to a pretimed controller. Thus, the k parameter will converge to the pretimed value of 0.50 when demand equals capacity. The recommended k-values for pretimed and actuated lane groups are given in Exhibit 16-13.

EXHIBIT 16-13. k-VALUES TO ACCOUNT FOR CONTROLLER TYPE

	Degree of Saturation (X)					
Unit Extension (s)	≤ 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[a]	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 unit extension and its k_{min} value at X = 0.5: $k = (1 - 2k_{min})(X - 0.5) + k_{min}$, $k \geq k_{min}$, and $k \leq 0.5$.
a. For unit extension > 5.0, extrapolate to find k, keeping $k \leq 0.5$.

For unit extension values other than those listed in Exhibit 16-13, k-values may be interpolated. If the formula in Exhibit 16-13 is used, the k_{min}-value (the k-value for X = 0.5) should first be interpolated for the given unit extension and then the formula should be used. Exhibit 16-13 may be extrapolated for unit extension values beyond 5.0 s, but in no case should the extrapolated k-value exceed 0.5.

Upstream Filtering or Metering Adjustment Factor

The incremental delay adjustment factor (I) incorporates the effects of metering arrivals from upstream signals, as described in Chapter 15. For a signal analysis of an isolated intersection using the methodology of this chapter, a value of 1.0 for I is used.

Initial Queue Delay

Procedure is described in Appendix F

When a residual queue from a previous time period causes an initial queue to occur at the start of the analysis period (T), additional delay is experienced by vehicles arriving in the period since the initial queue must first clear the intersection. A procedure for determining this initial queue delay is described in detail in Appendix F. This procedure is also extended to analyze delay over multiple time periods, each having a duration T, in which an unmet demand may be carried from one time period to the next. If this is not the case, a value of zero is used for d_3.

Aggregated Delay Estimates

The procedure for delay estimation yields the control delay per vehicle for each lane group. It is often desirable to aggregate these values to provide delay for an intersection approach and for the intersection as a whole. This aggregation is done by computing weighted averages, where the lane group delays are weighted by the adjusted flows in the lane groups.

Thus, the delay for an approach is computed using Equation 16-13:

$$d_A = \frac{\sum d_i v_i}{\sum v_i} \quad (16\text{-}13)$$

where

d_A = delay for Approach A (s/veh),
d_i = delay for lane group i (on Approach A) (s/veh), and
v_i = adjusted flow for lane group i (veh/h).

Control delays on the approaches can be further aggregated using Equation 16-14 to provide the average control delay for the intersection:

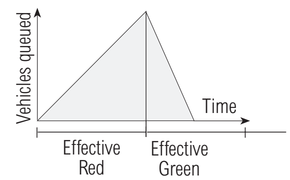
$$d_I = \frac{\sum d_A v_A}{\sum v_A} \tag{16-14}$$

where

d_I = delay per vehicle for intersection (s/veh),
d_A = delay for Approach A (s/veh), and
v_A = adjusted flow for Approach A (veh/h).

Special Procedure for Uniform Delay with Protected-Plus-Permitted Left-Turn Operation from Exclusive Lanes

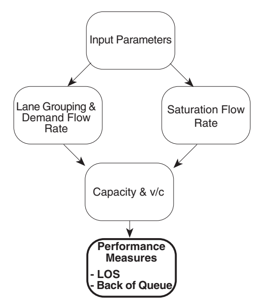
Queue accumulation polygon (QAP): uniform delay = area of triangles



The first term in the delay calculation is easily derived as a function of the area contained within the plot of queue storage as a function of time. With a single green phase per cycle, this plot assumes a triangular shape; that is, the queue size increases linearly on the red phase and decreases linearly on the green. The peak storage occurs at the end of the red phase. The geometry of the triangle depends on the arrival flow rate, the queue discharge rate, and the length of the red and green signal phases.

This simple triangle becomes a more complex polygon when left turns are allowed to proceed on both protected and permitted phases. However, the area of this polygon, which determines the uniform delay, is still relatively easy to compute when the left turns are in an exclusive lane and the proper values for the arrival and discharge rates during the various intervals of the cycle are given along with the interval lengths that determine its shape. The procedure for this analysis is covered in Appendix E.

DETERMINING LEVEL OF SERVICE



Intersection LOS is directly related to the average control delay per vehicle. Once delays have been estimated for each lane group and aggregated for each approach and the intersection as a whole, Exhibit 16-2 is consulted, and the appropriate LOS is determined.

The results of an operational application of this method will yield two key outputs: volume to capacity ratios for each lane group and for all of the critical lane groups within the intersection as a whole, and average control delays for each lane group and approach and for the intersection as a whole along with corresponding LOS.

Any v/c ratio greater than 1.0 is an indication of actual or potential breakdown. In such cases, multiperiod analyses are advised. These analyses encompass all periods in which queue carryover due to oversaturation occurs. When the overall intersection v/c ratio is less than 1.0 but some critical lane groups have v/c ratios greater than 1.0, the green time is generally not appropriately apportioned, and a retiming using the existing phasing should be attempted. Appendix B should be consulted for guidelines.

A critical v/c ratio greater than 1.0 indicates that the overall signal and geometric design provides inadequate capacity for the given flows. Improvements that might be considered include basic changes in intersection geometry (number and use of lanes), increases in the signal cycle length if it is determined to be too short, and changes in the signal phase plan. Chapter 10 and Appendix B contain information on these types of improvements. Existing state and local policies or standards should also be consulted in the development of potential improvements.

Unacceptable delay can occur even if v/c < 1.0, and acceptable delay can occur even if v/c ≥ 1.0

LOS is a measure of the delay incurred by motorists at a signalized intersection. In some cases, delay will be high even when v/c ratios are low. In these situations, poor progression or an inappropriately long cycle length, or both, is generally the cause. Thus, an intersection can have unacceptably high delays without there being a capacity problem. When the v/c approaches or exceeds 1.0, it is possible that delay will remain at acceptable levels. This situation can occur, especially if the time over which high v/c levels occur is short. It can also occur if the analysis is for only a single period and there is queue carryover. In the latter case, conduct of a multiperiod analysis is necessary to gain a true picture of delay. The analysis must consider the results of both the capacity analysis and the LOS analysis to obtain a complete picture of existing or projected intersection operations.

DETERMINING BACK OF QUEUE

Procedure is described in Appendix G

When an estimate of queue length is needed, a procedure to calculate the average back of queue and 70th-, 85th-, 90th-, and 98th-percentile back of queue is presented in Appendix G. The back of queue is the number of vehicles that are queued depending on the arrival patterns of vehicles and on the number of vehicles that do not clear the intersection during a given green phase (overflow). This procedure is also able to analyze back of queue over multiple time periods, each having a duration (T) in which an overflow queue may be carried from one time period to the next.

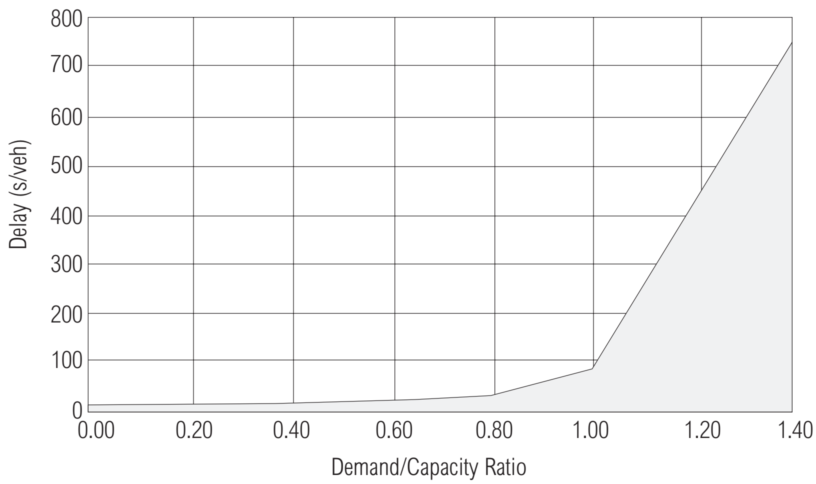
SENSITIVITY OF RESULTS TO INPUT VARIABLES

The methodology is sensitive to the geometric, demand, and control characteristics of the intersection. The predicted delay is highly sensitive to signal control characteristics and the quality of progression. The predicted delay is sensitive to the estimated saturation flow only when demand approaches or exceeds 90 percent of the capacity for a lane group or an intersection approach.

Exhibits 16-14 through 16-17 illustrate the sensitivity of the predicted control delay per vehicle to demand to capacity ratio, g/C, cycle length, and length of the analysis period (T). Delay is relatively insensitive to demand levels until demand exceeds 90 percent of capacity; then it is highly sensitive not only to changes in demand but also to changes in g/C, cycle length, and length of the analysis period. Initial queue delay, d_3, although not shown in Exhibit 16-14, occurs when there is queue spillback.

Sensitivity to demand/capacity

EXHIBIT 16-14. SENSITIVITY OF DELAY TO DEMAND TO CAPACITY RATIO
(SEE FOOTNOTE FOR ASSUMED VALUES)



Note:
Assumptions: cycle length = 100 s , g/C = 0.5 , T = 1 h, k = 0.5, I = 1, s = 1800 veh/h.

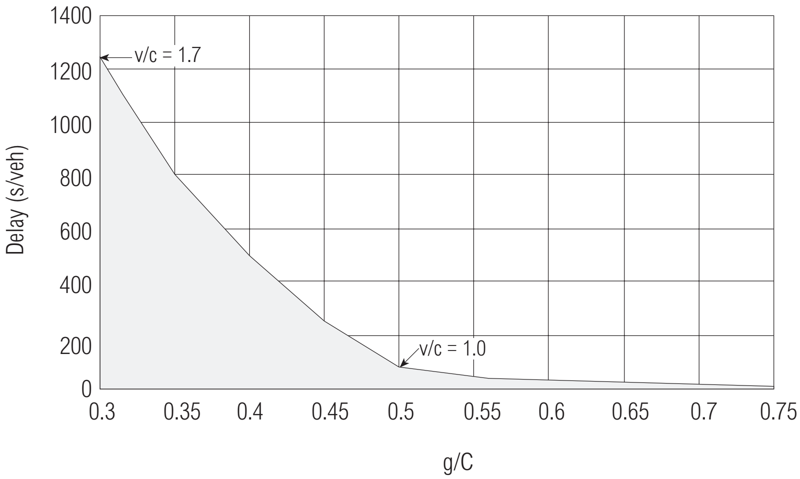
Delay becomes sensitive to signal control parameters (cycle length, g/C, and progression) only at demand levels above 80 percent of capacity. Once demand exceeds 80 percent of capacity, modest increases in demand can cause significant increases in delay. The demand to capacity ratio itself is sensitive to the demand level, the PHF, the saturation flow rate, and the g/C ratio.

Small g/C values that do not provide sufficient capacity to serve the demand cause excessive delays for the movement. Once there is sufficient g/C to serve the movement, little is gained by providing more g/C to the movement (see Exhibit 16-15).

If the cycle length does not allow enough g/C time (which affects capacity) to serve a movement, the delay increases rapidly. Long cycle lengths also increase delay, but not as rapidly as short cycle lengths that provide insufficient capacity to serve the movements at the intersection (see Exhibit 16-16).

Sensitivity to g/C

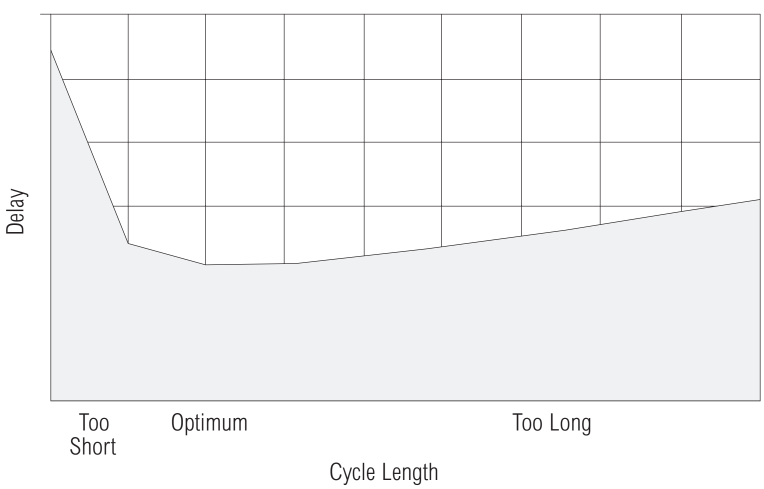
EXHIBIT 16-15. SENSITIVITY OF DELAY TO g/C
(SEE FOOTNOTE FOR ASSUMED VALUES)



Note:
Assumptions: cycle length = 100 s , v/s = 0.5 , T = 1 h, k = 0.5, I = 1, s = 1800 veh/h.

Sensitivity to C

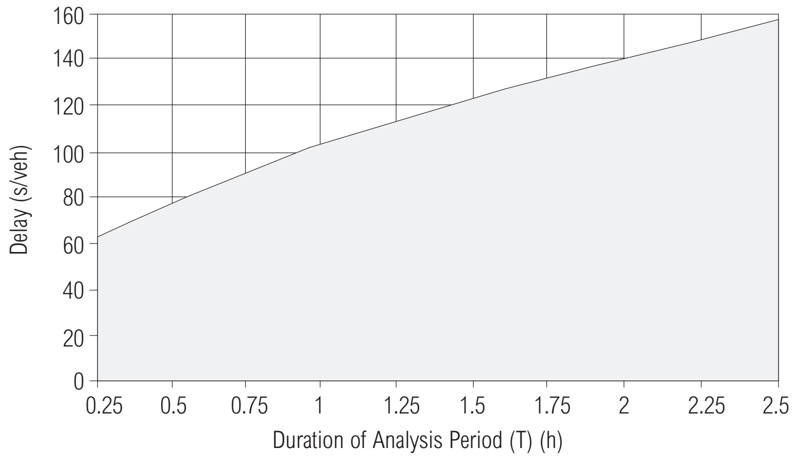
EXHIBIT 16-16. SENSITIVITY OF DELAY TO CYCLE LENGTH



The length of the analysis period (T) determines how long the demand is assumed to be at the specified flow rate. When demand is less than capacity, the length of the analysis period has little influence on the estimated mean delay. However, when demand exceeds capacity, the longer analysis period means that a longer queue is built up and that it takes longer to clear the bottleneck. The result is that mean delay in oversaturated conditions is highly sensitive to the selected length of the analysis period (see Exhibit 16-17).

Sensitivity to T

EXHIBIT 16-17. SENSITIVITY OF DELAY TO ANALYSIS PERIOD (T) (for v/c ≈ 1.0) (SEE FOOTNOTE FOR ASSUMED VALUES)



Note:
Assumptions: cycle length = 100 s , g/C = 0.4, v/s = 0.44 , k = 0.5, I = 1, s = 1800 veh/h.

III. APPLICATIONS

Guidelines on inputs and estimated values are in Chapter 10

The methodology for analyzing signalized intersections considers the details of each of four components: flow rates at the intersection (vehicular, pedestrian, and bicycle), signalization of the intersection, geometric design or characteristics of the intersection, and the delay or LOS that results from these. The methodology is capable of treating any of these four components as an unknown to be determined once the details of the other three are known. Thus the method can be used for each of four operational and design analysis types, each having a target output, with the remaining parameters known or assumed for use as inputs:

- Operational (LOS): Determine LOS when details of intersection flows, signalization, and geometrics are known.
- Design (v_p): Determine allowable service flow rates for selected LOS when the details of signalization and geometrics are known.
- Design (Sig): Determine signal timing (for an assumed phase plan) when the desired LOS, details of flows, and geometrics are known.
- Design (Geom): Determine basic geometrics (number and allocation of lanes) when the desired LOS and details of flows and signalization are known.

Planning analysis is intended for use in sizing the overall geometrics of the intersection or in identifying the general sufficiency of the capacity of an intersection. It is based on the sum of critical lane volumes and requires minimum input information. In this chapter, a quick estimation method is denoted as Planning (X_{cm}) and is explained in Appendix A of Chapter 10.

Planning analysis is a link to operational and design analyses through the same basic computational methodology. However, the level of precision inherent in the operational analysis exceeds the accuracy of the data available in a planning context. The requirement for a complete description of the signal timing plan is also a burden, especially when the method is being applied in transportation planning situations. Therefore, the concept of planning analysis is to apply the required approximations to the input data and not to the computational procedures. For planning purposes, the only site-specific data that should be needed are the traffic volumes and number of lanes for each