CHAPTER 16

SIGNALIZED INTERSECTIONS

CONTENTS

EXHIBITS

I. INTRODUCTION

SCOPE OF THE METHODOLOGY

Background and underlying concepts for this chapter are in Chapter 10

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.

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.

A lane group is indicated in formulas by the subscript i

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.

See Chapter 10 for description of quick estimation method

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.

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*).

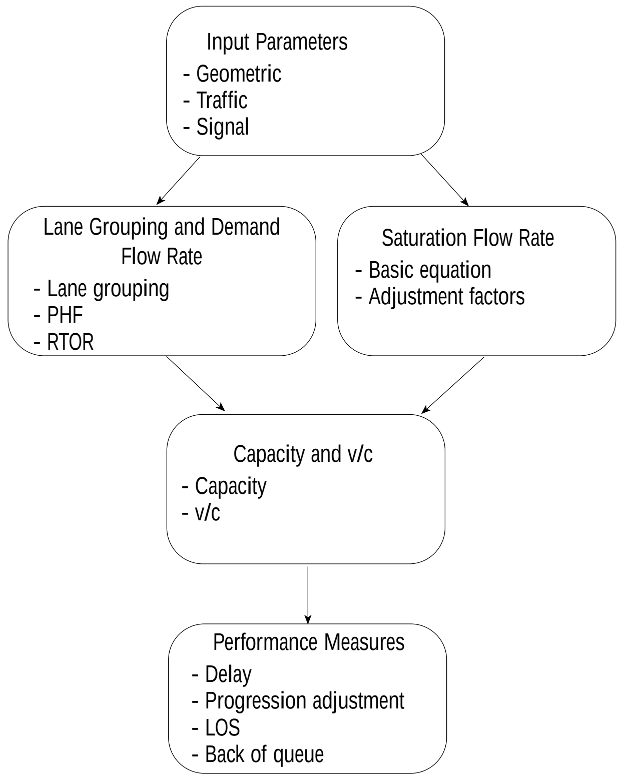
LIMITATIONS TO THE METHODOLOGY

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

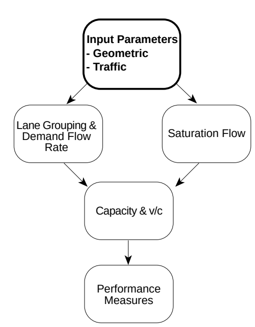
Inputs needed
- Geometric,
- Traffic, and
- Signalization

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.

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 (m) Grade, G (%) Existence of exclusive LT or RT lanes Length of storage bay, LT or RT lane, L_s (m) 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 (km/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

15-min flow rates can be estimated using hourly volumes and PHFs

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 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.

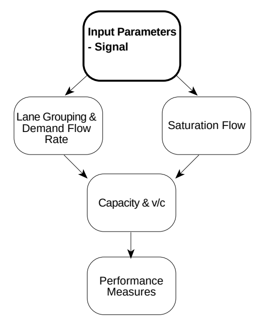
$$R_p = \frac{P}{\frac{g_i}{C}} \tag{16-1}$$

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.

$$G_p = 3.2 + \frac{L}{S_p} + \left(0.81\frac{N_{ped}}{W_E}\right) \quad \text{for } W_E > 3.0 \text{ m}$$
$$G_p = 3.2 + \frac{L}{S_p} + \left(0.27 N_{ped}\right) \quad \text{for } W_E \le 3.0 \text{ m} \tag{16-2}$$

where

G_p = minimum green time (s),
L = crosswalk length (m),
S_p = average speed of pedestrians (m/s),
W_E = effective crosswalk width (m),
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 1.2 m/s. Local values can be substituted.

It is assumed that the 15th-percentile walking speed of pedestrians crossing a street is 1.2 m/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

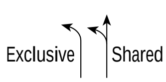
An operational analysis requires the specification of a signal timing plan for the intersection under study. The planning level application presented in Chapter 10 offers a procedure for establishing a reasonable and effective signal timing plan. This procedure is recommended only for the estimation 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.

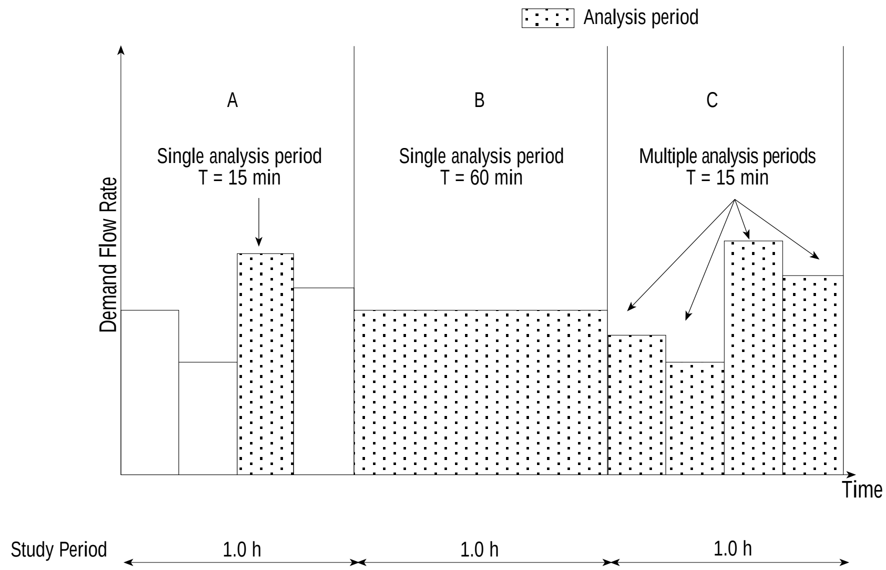
Alternative Study Approaches

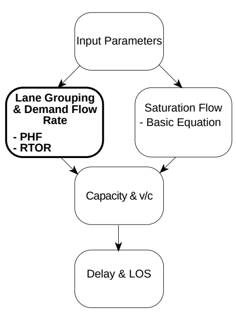
Two major analytic steps are performed in the volume adjustment module. Movement volumes are adjusted to flow rates for each desired period of analysis, if necessary, and lane groups for analysis are established. Exhibit 16-6 demonstrates three alternative ways in which an analyst might proceed for a 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.

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

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.

$$v_p = \frac{V}{PHF} \qquad (16\text{-}3)$$

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

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.

Subtract RTOR volume from RT volume

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).

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

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 < 2.4 m 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 3.6 m. The lane width factor may be calculated with caution for lane widths greater than 4.8 m, 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 2.4 m.

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 75 m 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

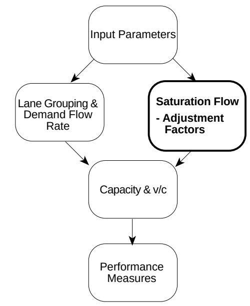
Applies to bus stops within 75 m of the stop line and a limit of 250 buses/h

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 75 m 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	$f_w = 1 + \frac{(W - 3.6)}{9}$	W = lane width (m)	$W \geq 2.4$ If W > 4.8, 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.

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.

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

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.

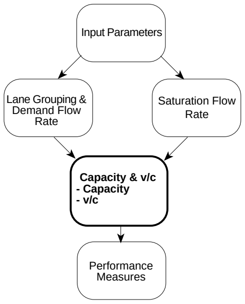
Finally, the saturation flow adjustment factor is calculated from the final occupancy on the basis of the turning movement protection status and the percent of turning traffic in the lane group. A comprehensive step-by-step procedure is provided in Appendix D.

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:



Green ratio defined

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

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.

v/c Ratio

Degree of saturation defined

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:

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

where

X_c = critical v/c ratio for intersection;
$\sum\left(\frac{v}{s}\right)_{ci}$ = 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

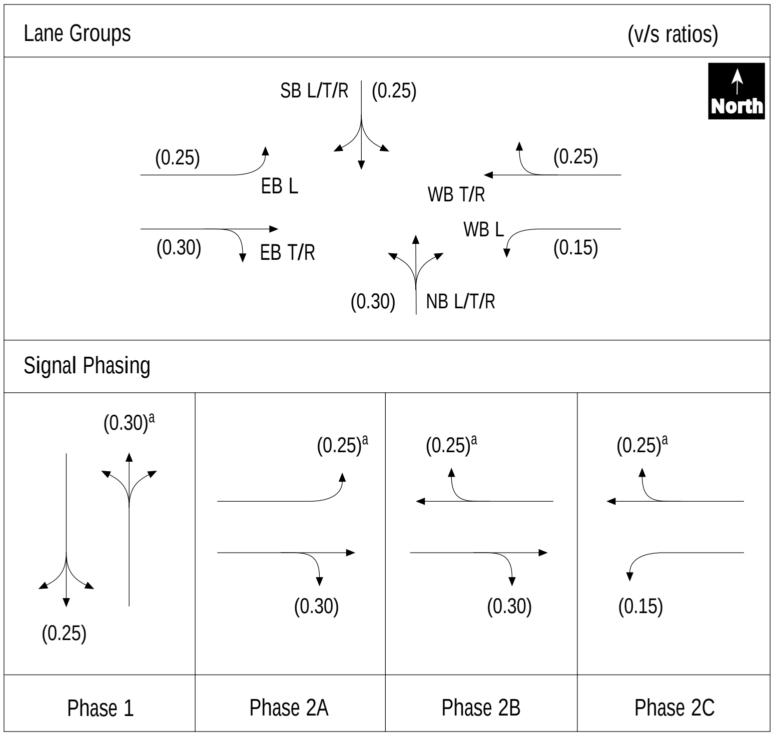
When phases overlap, the critical path must conform to the following rules:

- 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.

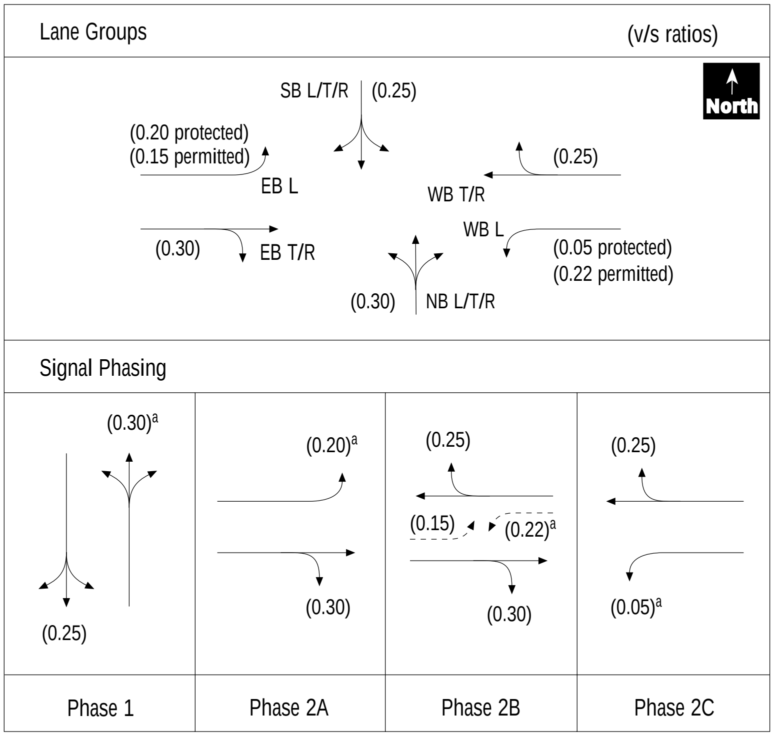
The analysis of Phase 1 does not change, because it is discrete. The NB lane group is still critical, with a v/s ratio of 0.30. There are now four different potential paths through Phase 2 that conform to the rules for determining critical paths:

- 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;

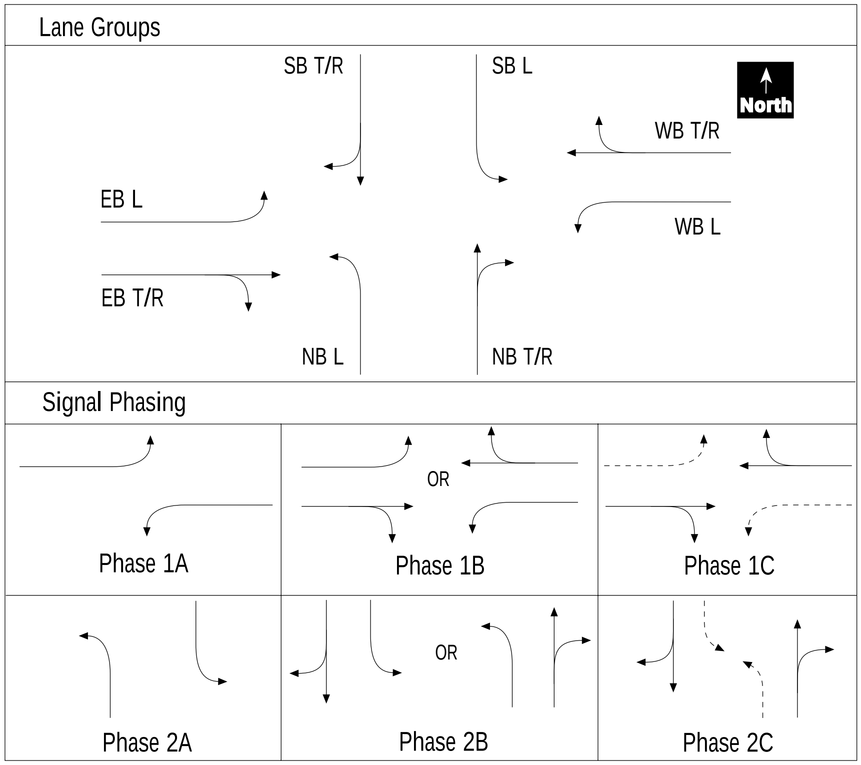
• The critical WB left turn (permitted) is initiated in Phase 2B, and its lost time is applied;

• 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:

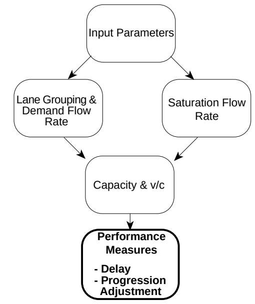
- EB left turn (protected) + EB left turn (permitted),
- EB left turn (protected) + WB left turn (permitted),
- EB left turn (protected) + WB T/R,
- WB left turn (protected) + WB left turn (permitted),
- WB left turn (protected) + EB left turn (permitted), and
- WB left turn (protected) + EB T/R.

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.

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

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.

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

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

Unit Extension (s)	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[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:

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

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:

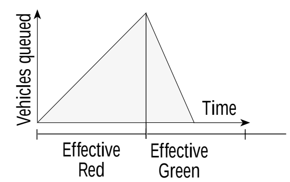
$$d_I = \frac{\sum d_A v_A}{\sum v_A} \quad (16\text{-}14)$$

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

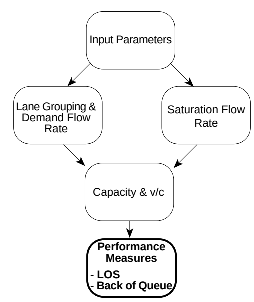
Queue accumulation polygon (QAP): uniform delay = area of triangles



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.

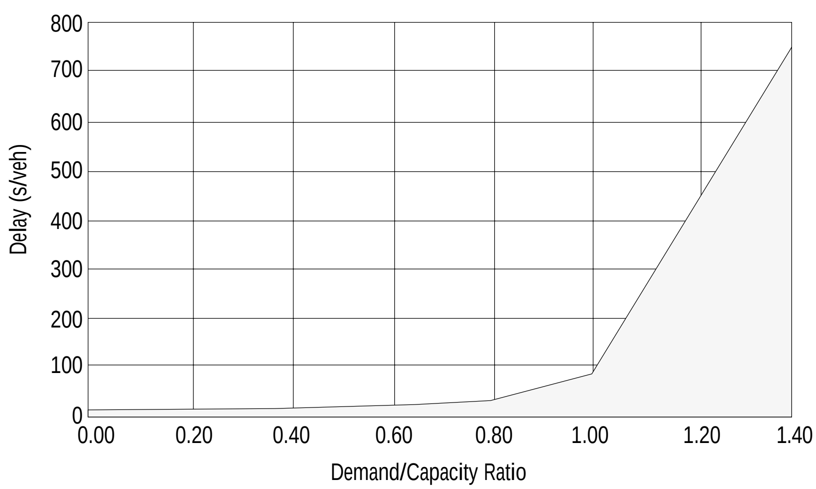
SENSITIVITY OF RESULTS TO INPUT VARIABLES

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.

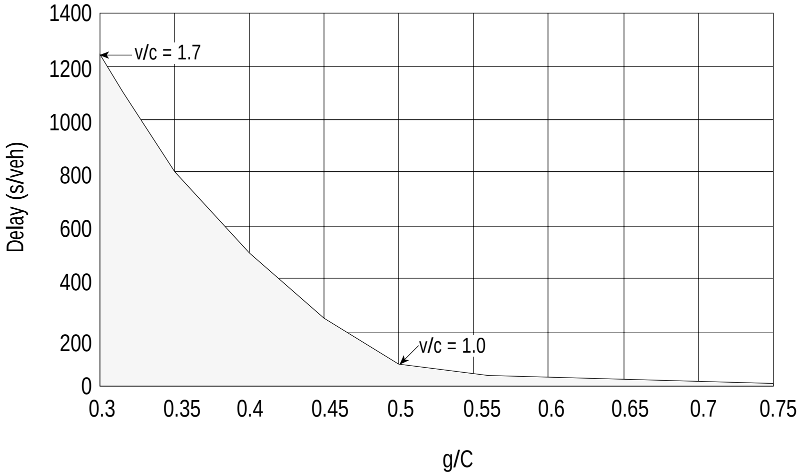
Delay becomes sensitive to signal control parameters (cycle length, g/C, and progression) only at demand levels above 80 percent of capacity. Once demand exceeds 80 percent of capacity, modest increases in demand can cause significant increases in delay. The demand to capacity ratio itself is sensitive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

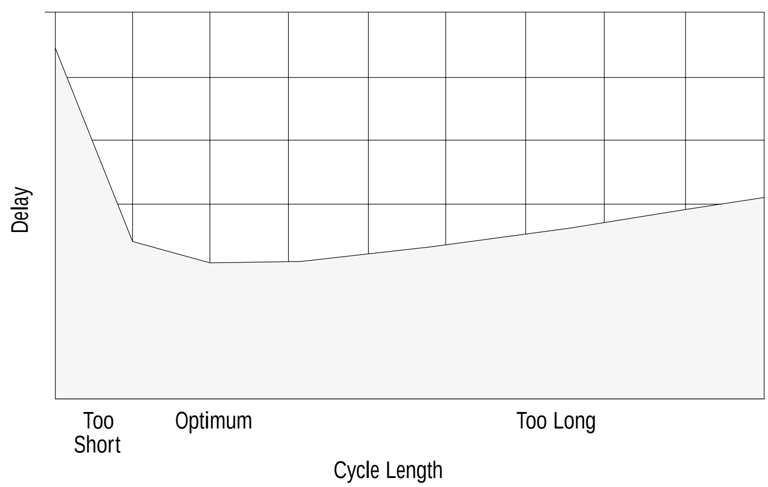
EXHIBIT 16-15. SENSITIVITY OF DELAY TO g/C
(SEE FOOTNOTE FOR ASSUMED VALUES)



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

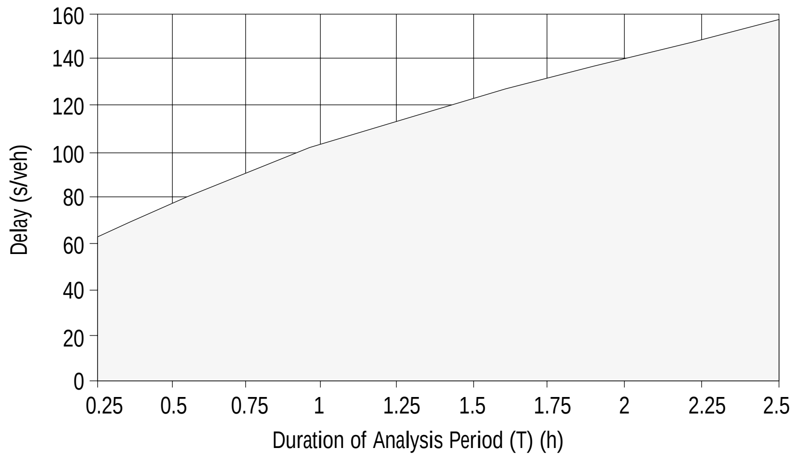
EXHIBIT 16-16. SENSITIVITY OF DELAY TO CYCLE LENGTH



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 movement together with a minimal description of the signal design and related operating parameters.

COMPUTATIONAL STEPS

Exhibit 16-18 gives the five types of analysis. Although the methodology is capable of computations for all five, the specific procedures and worksheets are designed for the first of these (i.e., a solution for LOS). In the development of alternative signal and geometric designs, it is often necessary to consider changes simultaneously in both. Rarely can signalization be considered in isolation from geometric design and vice versa. Thus, the most frequent type of analysis would consider such alternatives on a trial-and-error basis and would not attempt to hold one constant and solve for the other.

EXHIBIT 16-18. TYPES OF ANALYSIS COMMONLY PERFORMED

Type	Given	Objective	Comments/Assumptions Required
Operational (LOS)	Volume Signal timing Geometrics	LOS	None
Design (v_p)	Signal timing Geometrics Desired LOS	Maximum service flow rate	Requires iterative procedure, with complex interactions possible
Design (Sig)	Volume Geometrics Desired LOS	Signal timing	Initial estimate of signal timing needed to perform iterative solution
Design (Geom)	Volume Signal timing Desired LOS	Geometrics	Initial assumptions on geometrics needed to perform iterative solution
Planning (X_{cm})	Volume Signal timing	X_{cm}	Most inputs are estimates

Operational analysis is divided into five modules: input, volume adjustment, saturation flow rate, capacity analysis, and LOS. The computations for each of these modules are conducted or summarized on the appropriate worksheet.

In addition to the module-related worksheets, supplementary worksheets are provided to handle computations that are more complex. An overview of the information flow among all worksheets is presented in Exhibit 16-19, which also shows the proper treatment of all combinations of left-turn lanes and phasing. A given lane group may have

- Left turns from an exclusive lane,
- Left turns from a shared lane, or
- No left turns at all.

When left turns are present, the signal phasing may provide

- Permitted left-turn operation,
- Protected left-turn operation, or
- A combination of protected and permitted left turns.

There are six different possibilities, each of which must be handled in a slightly different manner using the worksheets.

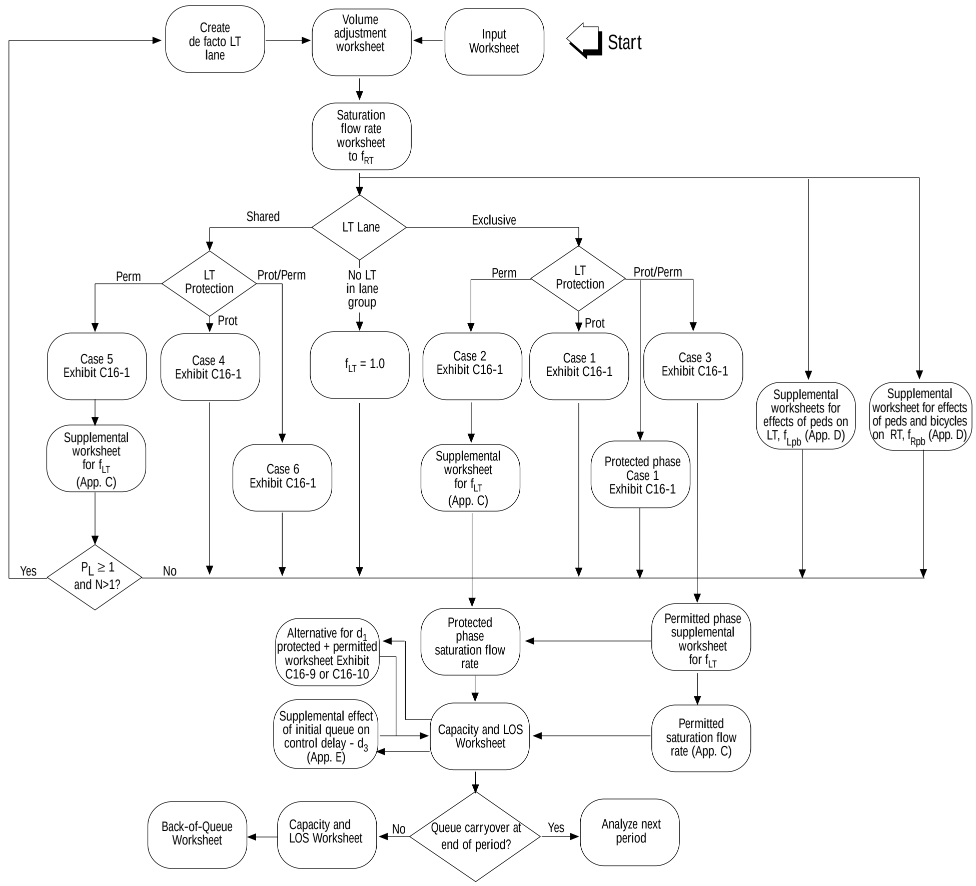
Phasing	Left-Turn Adjustment Cases — Lane: LT Excl	LT Share
Protected	1	4
Permitted	2	5
Prot/Perm	3	6

Input Parameters

One set of worksheets is used for each analysis period

The input parameters are the geometric, volume, and signalization characteristics needed to perform computations. When an existing intersection is under study, most of these data will be obtained from field studies. When future conditions are under consideration, traffic data will be forecast and geometric and signal designs will be based on existing conditions or will be proposed. The Input Worksheet is shown in Exhibit 16-20.

EXHIBIT 16-19. FLOW OF WORKSHEETS AND APPENDICES TO PERFORM OPERATIONAL ANALYSIS



The upper third of the worksheet contains a schematic intersection drawing on which basic volume and geometric data are recorded. The details of lane geometrics should be shown within the intersection diagram. Details should include

- Number of lanes,
- Lane widths,
- Grade (plus sign is upgrade),
- Traffic movements using each lane (shown by arrows),
- Existence and location of curb parking lanes,
- Existence and location of bus stops,
- Existence and length of storage bays, and
- Other features such as channelization.

When geometric conditions are not known, a design should be proposed based on state or local practice. Chapter 10 may be consulted for assistance in establishing a design for analysis. When separate left-turn lanes exist, the procedures assume that the storage length is adequate. This assumption should be checked against the criteria in Chapter 10.

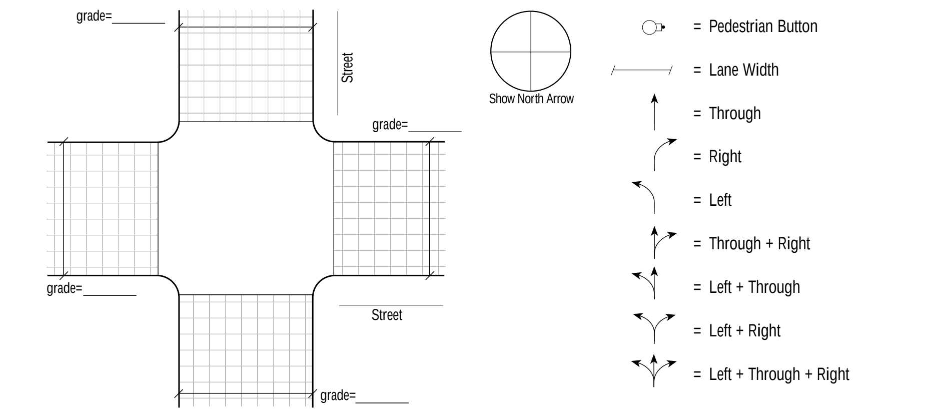
The middle portion of the worksheet consists of a tabulation of volume and timing data for each movement. Hourly volume or 15-min flow rate and volume-related parameters are entered into the tabulation in the middle of the worksheet. Separate entries are required for each approach and for each movement if appropriate. Note that RTOR volume should be removed from the total RT volume before the RT volume is entered on the worksheet.

EXHIBIT 16-20. INPUT WORKSHEET

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

DIAGRAM	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
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 16-2.

For percent heavy vehicles and peak-hour factor, an average value for an entire approach can be used for all movements. For arrival type, either a value for P or the designation of type (1 to 6) is entered. Each approach movement is identified as pretimed (P) or actuated (A), and start-up lost time (l_1) and extension time (e) for each approach movement are entered. For pedestrians and bicycles, the volumes are those occurring in the crosswalk that conflicts with right turns from the subject approach.

When data for some of these variables are not available or forecasts cannot be adequately established, default values may be used. Guidelines on the use of defaults are presented in Chapter 10 of this manual.

The sequence of signal phases is diagrammed in the eight boxes at the bottom of the input worksheet. Each box is used to show a single phase or subphase during which the allowable movements remain constant. For each phase, the actual green time (G) and the actual yellow-plus-all-red time (Y) are shown. For most cases, an assumed or existing signal design will be used. For some analyses, however, the signal timing and phasing will not be known. Setting timing and phasing for the purposes of analysis will influence the determination of lane groups. This portion of the signal design may be established on the basis of state or local practice. For additional suggestions on establishing the type of control and phase sequence, Appendix B should be consulted.

Appendix B provides recommendations for establishing signal design

The timing of the signal will not be known when signal timing design is to be established. It may or may not be known when actuated signals are in place, depending on whether average phase durations were observed in the field. Estimates, however, cannot be computed until the first half of the capacity analysis module is complete. Other computations may proceed without this information. Appendix B contains recommendations for establishing phase times based on an assumed signal type and phase sequence and for estimating the average phase lengths of actuated signals when observations are not available.

The establishment of signal timing will usually involve iterative computations. It is preferable, therefore, to specify a complete signal timing for analysis using trial-and-error computations to determine an appropriate final timing. As an alternative, the timing plan may be synthesized using the planning application presented in Chapter 10 of this manual. If a fully implementable timing plan is required, a variety of signal timing optimization models are commercially available.

Volume Adjustment and Saturation Flow Rate

The second major analysis module focuses on adjustment of hourly movement volumes to flow rates for a peak 15-min period within the hour and on establishment of lane groups. A worksheet for designating lane groups, making volume adjustments, and computing saturation flow rate is shown in Exhibit 16-21. For this worksheet, the hourly volume (V) for each approach and movement is taken from the Input Worksheet and the flow rate, v_p, is computed. Lane groups are established and associated flow rates and turn proportions are noted. For saturation flow rate, adjustment factors are identified and an adjusted saturation flow is computed for each lane group.

Capacity Analysis

In the Capacity and LOS Worksheet (Exhibit 16-22), information and computational results from the input, volume adjustment, and saturation flow rate modules are combined to compute the capacity and v/c of each lane group and the delay and LOS for each lane group and approach and for the intersection as a whole.

The top portion of the worksheet is used to compute capacities. Phase numbers and types are entered in the first two rows. Phase type is included to accommodate left turns that have both protected and permitted phases. In this case, the protected phase will be the primary phase and the permitted phase will be the secondary phase. The primary and secondary phases must be represented by separate column entries on this worksheet, and certain quantities, such as lane group capacity, must be computed as the sum of the primary and secondary phase values in the lower portion of the worksheet. Primary phase entries should be designated P in this row. Secondary phase entries should be designated S.

The flow rate for each lane group is obtained from the Volume Adjustment and Saturation Flow Rate Worksheet and entered in this worksheet. In the case of lane groups with both protected and permitted phases, for computation of the critical v/c ratio, X_c, it is necessary to apportion the total flow rate between the primary and secondary phases. It is appropriate to consider whichever phase is displayed first to be fully saturated by left-turn traffic and to apply any residual flow to the phase that is displayed second.

EXHIBIT 16-21. VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET

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 (see Exhibit 16-7 to determine adjustment factors)												
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.

EXHIBIT 16-22. CAPACITY AND LOS WORKSHEET

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 $Y_c = \Sigma$ (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 $d_2 = 900T[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}\,]$ (s/veh)												
Initial queue delay, d_3 (s/veh) (Appendix F)												
Uniform delay, d_1 (s/veh) (Appendix F)												
Progression adjustment factor, PF												
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)												
LOS by lane group (Exhibit 16-2)												
Delay by approach, $d_A = \frac{\Sigma(d)(v)}{\Sigma v}$ (s/veh)												
LOS by approach (Exhibit 16-2)												
Approach flow rate, v_A (veh/h)												
Intersection delay, $d_I = \frac{\Sigma(d_A)(v_A)}{\Sigma v_A}$ (s/veh)				Intersection LOS (Exhibit 16-2)								
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 Exhibit 16-13. 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.												

The adjusted saturation flow rate for each lane group is obtained directly from the Volume Adjustment and Saturation Flow Rate Worksheet. The flow ratio for each lane group is computed as v/s and entered in columns representing primary and secondary phases.

Effective green time defined

The next step is to calculate movement lost time for all primary and secondary phases using the start-up lost time (l_1), extension time (e), and yellow-plus-all-red time

(Y). Effective green times are calculated using actual green time (G) and movement lost time (t_L). Then the g/C ratio for each lane group, the effective green time divided by the cycle length, is computed and used to compute lane group capacity. Effective green times can be taken as equal to the actual green time plus the change-and-clearance interval minus the lost time for the movement.

Minimum capacity for permitted left turns

The capacity of each lane group is computed from Equation 16-6 as the saturation flow rate times the green ratio. A minimum capacity value based on sneakers per cycle must be imposed as a practical matter for all permitted left-turning movements. This value may be computed as indicated on the worksheet.

The v/c ratio (X) for each lane group is computed. At this point in the computations, critical lane groups and lost time per cycle may be identified according to the guidelines discussed. A critical lane group is defined as the lane group with the highest flow ratio in each phase or set of phases. When overlapping phases exist, all possible combinations of critical lane groups must be examined for the combination producing the highest sum of flow ratios. Critical lane groups are identified by a check mark. The flow ratios for critical lane groups are summed. The critical v/c ratio, X_c, which indicates the degree of saturation associated with the geometrics, volumes, and signal phasing, is then computed.

Delay and Level of Service

The LOS module combines the results of the volume adjustment, saturation flow rate, and capacity analysis modules to find the average control delay per vehicle in each lane group. LOS is directly related to delay and is found from Exhibit 16-2. The worksheet for this module is also shown in Exhibit 16-22. Lane group capacities and flow rates are the sum of primary and secondary phases from the capacity analysis section.

Delay is found from Equations 16-9 through 16-14. The worksheet is designed for computation of the uniform and incremental delay terms separately. The uniform delay is multiplied by the progression adjustment factor (PF) to account for the impact of progression. The value of PF is obtained from Exhibit 16-12. If a measured or estimated value of P was used in lieu of the arrival type in the computation of PF, the arrival type may be determined from Exhibit 16-11. In this case, the platoon ratio, R_p, must first be estimated by $R_p = PC/g$.

For an analysis of an initial queue, see Appendix F

When no residual queue exists from a previous time period, the initial queue delay term, d_3, is equal to zero. When an initial queue of vehicles exists at the start of the analysis period (observed at the beginning of the red phase), the procedures in Appendix F are used to modify the calculation of d_1, to calculate d_3, and to determine delay and LOS.

See Appendix E for analysis of protected-plus-permitted phasing

For exclusive left-turn lane groups with both protected and permitted phases, the supplemental worksheet presented in Exhibit 16-23 is used for calculation of uniform delay. Discussions of this procedure are also included in Appendix E.

The second term of the delay equation accounts for the incremental delay, that is, the delay over and above uniform delay due to random arrivals rather than uniform arrivals and those due to cycles that fail. It is based on the v/c ratio (X) and the capacity for the lane group (c). The capacity for each lane group is taken from the top part of the Capacity and LOS Worksheet.

The incremental delay calibration factor (k) is obtained from Exhibit 16-13. This value is a function of the controller type and degree of saturation. For isolated intersections, the upstream filtering or metering adjustment factor (I) is set equal to 1.0. Refer to Chapter 15 for further information on this factor. The second-term delay is computed using Equation 16-12.

EXHIBIT 16-23. SUPPLEMENTAL UNIFORM DELAY WORKSHEET FOR LEFT TURNS FROM EXCLUSIVE LANES WITH PROTECTED AND PERMITTED PHASES

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

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 $X_{perm} \leq 1.0$ & $X_{prot} \leq 1.0$	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 $X_{perm} \leq 1.0$ & $X_{prot} > 1.0$	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 $X_{perm} > 1.0$ & $X_{prot} \leq 1.0$	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 $X_{perm} \leq 1.0$ (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	$[0.50/(q_a C)][(r + g_q)Q_u + g_u(Q_u + Q_a) + Q_a^2/(s_p - q_a)]$

Delay and LOS are found by multiplying the uniform delay by the progression factor and adding the result to the incremental delay and initial queue delay, in accordance with Equation 16-9. The LOS corresponding to this delay, taken from Exhibit 16-2, is entered on the Capacity and LOS Worksheet.

In the event that the analysis period is oversaturated or when there is a final residual queue at the end of the analysis period, additional analysis periods should be studied until the residual queue no longer occurs.

The average delay per vehicle is found for each approach by adding the product of the lane group flow rate and the delay for each lane group on the approach and dividing by the total approach flow rate. LOS is determined from Exhibit 16-2.

The average control delay per vehicle for the intersection as a whole is found by adding the product of the approach flow rate and the approach delay for all approaches and dividing the sum by the total intersection flow rate. This weighted-average delay is entered at the bottom of the Capacity and LOS Worksheet. The overall intersection LOS is found from Exhibit 16-2.

INTERPRETATION OF RESULTS

The computations discussed in the previous section result in an estimation of the average delay per vehicle in each lane group for each approach and for the intersection as a whole. LOS is directly related to delay values and is assigned on that basis. LOS is a measure of the acceptability of delay levels to motorists at a given intersection. When delays are unacceptable, the causes of delay should be carefully examined. Although the discussion below is clearly not exhaustive, some of the more common situations are as follows.

1. LOS is an indication of the general acceptability of delay to drivers. It should be noted that this is somewhat subjective: what might be acceptable in a large city is not necessarily acceptable in a smaller city or rural area.

2. When delay levels are acceptable for the intersection as a whole but are unacceptable for certain lane groups, the phase plan, allocation of green time, or both might be examined to provide for more efficient handling of the disadvantaged movement or movements.

3. When delay levels are unacceptable but v/c ratios are relatively low, the cycle length may be too long for prevailing conditions, the phase plan may be inefficient, or both. It should be noted, however, that when signals are part of a coordinated system, the cycle length at individual intersections is determined by system considerations, and alterations at isolated locations may not be practical.

4. When both delay levels and v/c ratios are unacceptable, the situation is critical. Delay is already high, and demand is near or over capacity. In such situations, the delay may increase rapidly with small changes in demand. The full range of potential geometric and signal design improvements should be considered in the search for improvements.

The following point must be emphasized: unacceptable delay can exist where capacity is a problem as well as in cases in which it is adequate. Further, acceptable delay levels do not automatically ensure that capacity is sufficient. Delay and LOS, like capacity, are complex variables influenced by a wide range of traffic, roadway, and signalization conditions. The operational analysis techniques presented here are useful in estimating the performance characteristics of the intersection and in providing basic insights into probable causal factors.

The determination of LOS is based on average control delay. It is possible, however, for average delay to decrease with increasing volumes if the volume increases occur in movements with less than the average delay. Even with increases in more than one movement on an approach, the net effect can still be a decrease in average delay if the movements with less than average delay increase sufficiently.

One way to avoid this anomaly is to consider the change in mean delay on a lane-group-by-lane-group basis rather than by averaging delay over the entire intersection. Adding traffic to a particular lane group will always increase the delay for that lane group (as long as all other factors remain unchanged).

These procedures do not, however, account for all possible conditions. The influences of such characteristics as specific curb-corner radii, intersection angle, combinations of grades on various approaches, odd geometric features (offset intersections, narrowing on the departure lanes, etc.), and other unusual site-specific conditions are not addressed in the methodology. Field studies may be conducted in such cases to determine delay directly (see Appendix A) and or to calibrate the prevailing saturation flow rate (see Appendix H).

At the completion of a capacity calculation, the characteristics of the intersection have been defined. These characteristics must be evaluated in their own right as well as in conjunction with the delays and LOS resulting from the delay and LOS calculation. Some key factors to consider when the results of capacity computations are assessed are identified in the following text. A critical v/c ratio of greater than 1.0 indicates that the signal and geometric design cannot accommodate the combination of critical flows at the intersection. The given demand in these movements exceeds the capacity of the intersection to handle them. The condition may be ameliorated by increased cycle length, changes in the phasing plan, and basic changes in geometrics.

The analysis requires use of demand volumes, not departure volumes, to make a proper estimate

Computations should be conducted using arrival volumes. When the v/c ratios are less than 1.0, arrival and departure volumes are the same. When v/c ratios are greater than 1.0, either for an individual lane group or for the overall intersection, departure volumes are less than arrival volumes. By definition, future volume forecasts are also arrival volumes. When counts of actual departure volumes are used in analysis, the actual v/c ratio cannot be greater than 1.0. If v/c ratios greater than 1.0 persist for actual departure volumes, it is an indication that the intersection operates more efficiently than anticipated by these computational techniques or that the saturation flow rates used in the calculations are lower than those actually experienced in the field.

When the critical v/c ratio is acceptable but the v/c ratios for critical lane groups vary widely, the green-time allocation should be reexamined, because disproportionate distribution of available green is indicated. If permitted left turns result in extreme reductions in saturation flow rate for applicable lane groups, protected phasing might be considered. If the critical v/c ratio exceeds 1.0, it is unlikely that the existing geometric and signal design can accommodate the demand. Changes in either or both should be considered. When v/c ratios are unacceptable and signal phasing already includes protective phasing for significant turning movements, it is probable that geometric changes will be required to ameliorate the condition.

The capacity of an intersection is a complex variable depending on a large number of prevailing traffic, roadway, and signalization conditions. Suggestions on interpretation are not meant to be exhaustive or complete but merely to point out some of the more common problems that can be identified from the Capacity and LOS Worksheet results.

ANALYSIS TOOLS

The worksheets shown in Exhibits 16-20, 16-21, and 16-22 and provided in Appendix I can be used to perform Operational (LOS), Design (v_p), Design (Sig), and Design (Geom) analysis types. The worksheets shown in Chapter 10, Appendix B, can be used to perform Planning (X_{cm}) applications.

IV. EXAMPLE PROBLEMS

Problem No	Description	Application
1	Find LOS of signalized intersection	Operational (LOS)
2	Find LOS of signalized intersection	Operational (LOS)
3	Find LOS of signalized intersection	Operational (LOS)
4	Find LOS of proposed improvements on existing intersection	Planning (X_{cm})
5	Find capacity of intersection with projected volumes	Planning (X_{cm})
6	Find maximum service volume of approach for desired LOS	Design (v_p)

EXAMPLE PROBLEM 1

The Intersection The intersection of Third Avenue (NB/SB) and Main Street (EB/WB) is located in the central business district (CBD) of a small urban area. Intersection geometry and flow characteristics are shown on the Input Worksheet.

The Question What are the delay and peak-hour LOS of this intersection?

The Facts

√ EB and WB HV = 5 percent,
√ NB and SB HV = 8 percent,
√ PHF = 0.90,
√ Two-phase signal,
√ NB-SB green = 36 s,
√ EB-WB green = 26 s,
√ Yellow = 4 s,
√ Third Avenue has two lanes, one in each direction,
√ Main Street has four lanes, two in each direction,
√ No parking at intersection,
√ Pedestrian volume = 100 p/h, all approaches,
√ Bicycle volume = 20 bicycles/h, all approaches,
√ Movement lost time = 4 s, and
√ Level terrain.

Comments

√ Assume crosswalk width = 3.0 m for all approaches,
√ Assume base saturation flow rate = 1,900 pc/h/ln,
√ Assume E_T = 2.0,
√ No buses, and
√ 70.0-s cycle length, with green times given.

Steps

1. Pedestrians/cycle.	$100 \frac{p}{h} * \frac{1 h}{3,600 s} * 70 s = 1.944 p$
2. Minimum effective green time required for pedestrians (use Equation 16-2).	$G_p = 3.2 + \frac{L}{1.2} + 0.27 N_{ped}$ G_p (Main) = $3.2 + \frac{9.0}{1.2} + 0.27(1.944) = 11.2$ s G_p (Third) = $3.2 + \frac{13.2}{1.2} + 0.27(1.944) = 14.7$ s
3. Compare minimum effective green time required for pedestrians with actual effective green.	G_p (Main) = 26 s, which is >11.2 s G_p (Third) = 36 s, which is >14.7 s
4. Proportions of left and right turns.	Proportions of left- and right-turn traffic are found by dividing the appropriate turning volumes by the total lane group volume. $P_{LT}(EB) = \frac{65}{65 + 620 + 35} = 0.090$
5. Lane width adjustment factor (use Exhibit 16-7).	$f_w = 1 + \frac{(W - 3.6)}{9}$ $f_w(EB) = 1 + \frac{(3.3 - 3.6)}{9} = 0.967$
6. Heavy-vehicle adjustment factor (use Exhibit 16-7).	$f_{HV} = \frac{100}{100 + \% HV(E_T - 1)}$ $f_{HV}(EB) = \frac{100}{100 + 5(2.0 - 1)} = 0.952$
7. Percent grade adjustment factor (use Exhibit 16-7).	0% grade, f_g = 1.000

8. Parking adjustment factor (use Exhibit 16-7).	No parking maneuvers, f_p = 1.000
9. Bus blockage adjustment factor (use Exhibit 16-7).	No buses stopping, f_{bb} = 1.000
10. Area type adjustment factor (use Exhibit 16-7).	For CBD, f_a = 0.900
11. Lane utilization adjustment factor (use Exhibit 16-7).	Refer to Exhibit 10-23. This factor is applied to establish the conditions in the worst lane within each lane group. Otherwise, the results would reflect the average of all lanes of the defined lane groups. Use f_{LU} = 0.950 for EB and WB approaches, and f_{LU} = 1.000 for NB and SB approaches.
12. Left-turn adjustment factor.	The left turn is permitted, hence a special procedure is needed. The EB and WB left turns are opposed by multilane approaches. The supplemental worksheet for multilane approaches is used. The NB and SB left turns are opposed by single-lane approaches. The supplemental worksheet for a single-lane approach is used.
13. Right-turn adjustment factor (use Exhibit 16-7).	For NB and SB single-lane approaches: $f_{RT} = 1.0 - 0.135P_{RT}$ For EB and WB shared-lane approaches: $f_{RT} = 1.0 - 0.150P_{RT}$
14. Left-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effects is used.
15. Right-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effects is used.
16. Saturation flow.	$s = s_o N f_w f_{HV} f_g f_p f_{bb} f_{LU} f_a f_{LT} f_{RT} f_{Lpb} f_{Rpb}$ s(EB) = 1900 * 2 * 0.967 * 0.952 * 1.000 * 1.000 * 1.000 * 0.900 * 0.950 * 0.716 * 0.993 * 0.997 * 0.992 = 2103 veh/h
17. Lane group capacity.	c = s(g/C) c(EB) = 2103(0.371) = 780 veh/h
18. v/c ratio.	$v/c(EB) = \frac{800}{780} = 1.026$
19. Determine critical lane group in each timing phase.	The lane group with the highest v/c ratio in a phase is considered the critical lane group. In this case, EB and SB lane groups are critical in Phases 1 and 2, respectively.
20. Flow ratio of critical lane groups.	$v/s(EB) = \frac{800}{2103} = 0.380$ $v/s(SB) = \frac{667}{1625} = 0.410$
21. Sum of critical flow ratios.	$Y_c = 0.380 + 0.410 = 0.790$
22. Critical flow rate to capacity ratio.	$X_c = \frac{Y_c * C}{C - L}$ $X_c = \frac{0.790(70.0)}{70.0 - 8} = 0.892$

23. Uniform delay.	$d_1 = \dfrac{0.50C\left(1-\dfrac{g}{C}\right)^2}{1-\left[\min(1,X)\dfrac{g}{C}\right]}$ $d_1(EB) = \dfrac{0.50(70.0)(1-0.371)^2}{1-0.371(1.0)} = 22.015$ s/veh
24. Incremental delay.	$d_2 = 900T\left[(X-1)+\sqrt{(\dots)}\right]$ $d_2(EB) = 900(0.25)\left[(1.026-1)+\sqrt{(\dots)}\right] =$ 39.011 s/veh
25. Progression adjustment factor (use Exhibit 16-12).	PF (EB) = 0.926
26. Lane group delay.	$d = d_1PF + d_2 + d_3$ d (EB) = 22.015(0.926) + 39.011 + 0 = 59.4 s/veh
27. Intersection delay.	$d_I = \dfrac{\sum(d_A)(v_A)}{\sum v_A}$ $d_I = \dfrac{(59.4*800)+(31.0*833)+(14.4*466)+(21.9*667)}{(800+833+466+667)}$ = 34.2 s/veh
28. LOS by lane group, approach, and intersection.	LOS (EB lane group) = E LOS (EB approach) = E LOS Intersection = C

The calculation results are summarized as follows,

Direction/ LnGrp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Fact PF	Lane Grp Cap	Cal Term k	Incr Delay d_2	Lane Grp Delay	Lane Grp LOS	Delay by App	LOS by App
EB/LTR	1.026	0.371	22.015	0.926	780	0.5	39.011	59.4	E	59.4	E
WB/LTR	0.842	0.371	20.138	1.111	989	0.5	8.647	31.0	C	31.0	C
NB/LTR	0.561	0.514	11.617	1.000	830	0.5	2.734	14.4	B	14.4	B
SB/LTR	0.799	0.514	14.028	1.000	835	0.5	7.882	21.9	C	21.9	C
Intersection Delay = 34.2 s/veh						Intersection LOS = C					

Alternatives

Two alternatives are considered: a new lane utilization adjustment factor and new signal timing.

The purpose of the lane utilization adjustment factor (f_{LU}) is to account for uneven distribution of traffic in multilane roadways, and it is reflected in saturation flow rates.

Typically, traffic volume is evenly distributed between lanes at high v/c ratios, and the lane utilization adjustment factor is close to 1.000. In this analysis, f_{LU} is only applicable to Main Street because it has multiple lanes. v/c ratios of 1.026 and 0.842 are considered high, and it is assumed that traffic volume is evenly distributed, with f_{LU} = 1.000.

The performance is reassessed using f_{LU} = 1.000 and the results are summarized as follows.

Direction/ Ln Grp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Factor PF	Ln Grp Capacity	Cal Term k	Incr Delay d_2	Ln Grp Delay	Ln Grp LOS	Delay by App	LOS by App
EB/LTR	0.945	0.371	21.323	0.926	847	0.5	20.134	39.9	D	39.9	D
WB/LTR	0.798	0.371	19.671	1.111	1044	0.5	6.365	28.2	C	28.2	C
NB/LTR	0.561	0.514	11.617	1.000	830	0.5	2.734	14.4	B	14.4	B
SB/LTR	0.799	0.514	14.028	1.000	835	0.5	7.882	21.9	C	21.9	C
Intersection Delay = 27.7 s/veh						Intersection LOS = C					

The assumption of f_{LU} = 1.000 has reduced the delay from 34.2 s/veh to 27.7 s/veh.

The other alternative is to optimize the operation by reallocating green times without changing f_{LU}.

As shown in the calculation results, currently the v/c ratios between critical lane groups are not balanced. The v/c ratio of the EB lane group is much higher than that of the SB lane group. This imbalance results in much higher delay experienced by one critical lane group than by the other.

A new signal timing is introduced by reallocating 1.0 s to the east-west phase from the north-south phase. The resulting signal timing is 27.0 s for the east-west phase and 35.0 s for the north-south phase.

The intersection operation is reassessed with the new timing, and the results are summarized as follows.

Direction/ Ln Grp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Factor PF	Ln Grp Cap	Cal Term k	Incr Delay d_2	Ln Grp Delay	Ln Grp LOS	Delay by App	LOS by App
EB/LTR	0.972	0.386	21.118	0.926	823	0.5	25.265	44.8	D	47.9	47.9
WB/LTR	0.807	0.386	19.165	1.111	1032	0.5	6.766	28.1	C	28.1	28.1
NB/LTR	0.578	0.500	12.307	1.000	806	0.5	3.011	15.3	B	17.3	17.3
SB/LTR	0.821	0.500	14.843	1.000	812	0.5	9.132	24.0	C	34.0	34.0
Intersection Delay = 29.8 s/veh							Intersection LOS = C				

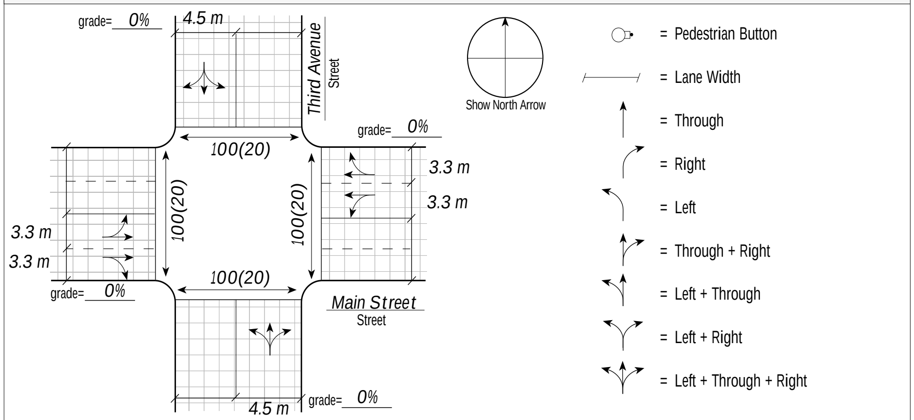
After reallocation of green times, v/c ratios for critical lane groups are more balanced, and the overall intersection performance (in terms of delay) has improved from 34.2 s/veh to 29.8 s/veh.

Example Problem 1

INPUT WORKSHEET

General Information		Site Information	
Analyst	WLL	Intersection	Third Avenue/Main Street
Agency or Company	CEI	Area Type	☒ CBD ❑ Other
Date Performed	4/12/99	Jurisdiction	
Analysis Time Period	4-6 PM	Analysis Year	1999

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)	65	620	35	30	700	20	30	370	20	40	510	50
% heavy vehicles, % HV	5	5	5	5	5	5	8	8	8	8	8	8
Peak-hour factor, PHF		0.90			0.90			0.90			0.90	
Pretimed (P) or actuated (A)		P			P			P			P	
Start-up lost time, l_1 (s)												
Extension of effective green time, e (s)												
Arrival type, AT		4			2			3			3	
Approach pedestrian volume,[2] v_{ped} (p/h)		100			100			100			100	
Approach bicycle volume,[2] v_{bic} (bicycles/h)		20			20			20			20	
Parking (Y or N)		N			N			N			N	
Parking maneuvers, N_m (maneuvers/h)		0			0			0			0	
Bus stopping, N_B (buses/h)		0			0			0			0	
Min. timing for pedestrians,[3] G_p (s)		11.2			11.2			14.7			14.7	

Signal Phasing Plan

	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
DIAGRAM								
Timing	G = 26.0 Y = 4.0	G = 36.0 Y = 4.0	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =

Protected turns	Permitted turns / Pedestrian	Cycle length, C = 70.0 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 16-2.

Example Problem 1

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET

General Information

Project Description *Example Problem 1*

Volume Adjustment

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	65	620	35	30	700	20	30	370	20	40	510	50
Peak-hour factor, PHF		0.90			0.90			0.90			0.90	
Adjusted flow rate, v_p = V/PHF (veh/h)	72	689	39	33	778	22	33	411	22	44	567	56
Lane group												
Adjusted flow rate in lane group, v (veh/h)		800			833			466			667	
Proportion[1] of LT or RT (P_{LT} or P_{RT})	0.090	-	0.049	0.040	-	0.027	0.071	-	0.048	0.067	-	0.083
Saturation Flow Rate (see Exhibit 16-7 to determine adjustment factors)												
Base saturation flow, s_o (pc/h/ln)		1900			1900			1900			1900	
Number of lanes, N		2			2			1			1	
Lane width adjustment factor, f_w		0.967			0.967			1.100			1.100	
Heavy-vehicle adjustment factor, f_{HV}		0.952			0.952			0.926			0.926	
Grade adjustment factor, f_g		1.000			1.000			1.000			1.000	
Parking adjustment factor, f_p		1.000			1.000			1.000			1.000	
Bus blockage adjustment factor, f_{bb}		1.000			1.000			1.000			1.000	
Area type adjustment factor, f_a		0.900			0.900			0.900			0.900	
Lane utilization adjustment factor, f_{LU}		0.950			0.950			1.000			1.000	
Left-turn adjustment factor, f_{LT}		0.716			0.901			0.937			0.951	
Right-turn adjustment factor, f_{RT}		0.993			0.996			0.994			0.989	
Left-turn ped/bike adjustment factor, f_{Lpb}		0.997			0.998			0.999			0.998	
Right-turn ped/bike adjustment factor, f_{Rpb}		0.992			0.995			0.996			0.994	
Adjusted saturation flow, s (veh/h) $s = s_o N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$		2103			2665			1614			1625	

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.

Example Problem 1

SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS OPPOSED BY SINGLE-LANE APPROACH				
General Information				
Project Description *Example Problem 1*				
Input				
	EB	WB	NB	SB
Cycle length, C (s)		70.0		
Total actual green time for LT lane group,[1] G (s)			36.0	36.0
Effective permitted green time for LT lane group,[1] g (s)			36.0	36.0
Opposing effective green time, g_o (s)			36.0	36.0
Number of lanes in LT lane group,[2] N			1	1
Adjusted LT flow rate, v_{LT} (veh/h)			33	44
Proportion of LT volume in LT lane group, P_{LT}			0.071	0.067
Proportion of LT volume in opposing flow, P_{LTo}			0.067	0.071
Adjusted flow rate for opposing approach, v_o (veh/h)			667	466
Lost time for LT lane group, t_L			4	4
Computation				
LT volume per cycle, $LTC = v_{LT}C/3600$			0.642	0.856
Opposing flow per lane, per cycle, $v_{olc} = v_oC/3600$ (veh/C/ln)			12.969	9.061
Opposing platoon ratio, R_{po} (refer to Exhibit 16-11)			1.00	1.00
$g_f = G[e^{-0.860(LTC^{0.629})}] - t_L$ $g_f \le g$ (except exclusive left-turn lanes)[3]			14.779	12.505
Opposing queue ratio, $qr_o = \max[1 - R_{po}(g_o/C), 0]$			0.486	0.486
$g_q = 4.943 v_{olc}^{0.762} qr_o^{1.061} - t_L$ $g_q \le g$			12.201	8.328
$g_u = g - g_q$ if $g_q \ge g_f$, or $g_u = g - g_f$ if $g_q < g_f$			21.221	23.495
$n = \max[(g_q - g_f)/2, 0]$			0	0
$P_{THo} = 1 - P_{LTo}$			0.933	0.929
E_{L1} (refer to Exhibit C16-3)			2.7	2.2
$E_{L2} = \max[(1 - P_{THo}^{\,n})/P_{LTo}, 1.0]$			1.0	1.0
$f_{min} = 2(1 + P_{LT})/g$			0.060	0.059
$g_{diff} = \max[g_q - g_f, 0]$ (except when left-turn volume is 0)[4]			0	0
$f_{LT} = f_m = [g_f/g] + \left[\frac{g_u/g}{1 + P_{LT}(E_{L1} - 1)}\right] + \left[\frac{g_{diff}/g}{1 + P_{LT}(E_{L2} - 1)}\right]$ $(f_{min} \le f_m \le 1.00)$			0.937	0.951

Notes

1. Refer to Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 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 $g_{diff} = 0$.

Example Problem 1

SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS OPPOSED BY MULTILANE APPROACH

General Information

Project Description *Example Problem 1*

Input

	EB	WB	NB	SB
Cycle length, C (s)	70.0			
Total actual green time for LT lane group,[1] G (s)	26.0	26.0		
Effective permitted green time for LT lane group,[1] g (s)	26.0	26.0		
Opposing effective green time, g_o (s)	26.0	26.0		
Number of lanes in LT lane group,[2] N	2	2		
Number of lanes in opposing approach, N_o	2	2		
Adjusted LT flow rate, v_{LT} (veh/h)	72	33		
Proportion of LT volume in LT lane group,[3] P_{LT}	0.090	0.040		
Adjusted flow rate for opposing approach, v_o (veh/h)	833	800		
Lost time for LT lane group, t_L	4	4		

Computation

	EB	WB	NB	SB
LT volume per cycle, $LTC = v_{LT}C/3600$	1.400	0.642		
Opposing lane utilization factor, f_{LUo} (refer to Volume Adjustment and Saturation Flow Rate Worksheet)	0.950	0.950		
Opposing flow per lane, per cycle $v_{olc} = \frac{v_oC}{3600N_of_{LUo}}$ (veh/C/ln)	8.525	8.187		
$g_f = G[e^{-0.882(LTC^{0.717})}] - t_L$ $g_f \le g$ (except for exclusive left-turn lanes)[1, 4]	4.461	9.684		
Opposing platoon ratio, R_{po} (refer to Exhibit 16-11)	0.67	1.33		
Opposing queue ratio, $qr_o = \max[1 - R_{po}(g_o/C), 0]$	0.751	0.506		
$g_q = \frac{v_{olc}qr_o}{0.5 - [v_{olc}(1 - qr_o)/g_o]} - t_L$, $v_{olc}(1 - qr_o)/g_o \le 0.49$ (note case-specific parameters)[1]	11.303	8.027		
$g_u = g - g_q$ if $g_q \ge g_f$, or $g_u = g - g_f$ if $g_q < g_f$	14.697	16.316		
E_{L1} (refer to Exhibit C16-3)	3.3	3.2		
$P_L = P_{LT}\left[1 + \frac{(N-1)g}{(g_f + g_u/E_{L1} + 4.24)}\right]$ (except with multilane subject approach)[5]	0.268	0.095		
$f_{min} = 2(1 + P_L)/g$	0.098	0.084		
$f_m = [g_f/g] + [g_u/g]\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right]$, $(f_{min} \le f_m \le 1.00)$	0.521	0.892		
$f_{LT} = [f_m + 0.91(N - 1)]/N$ (except for permitted left turns)[6]	0.716	0.901		

Notes

1. Refer to Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 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 $f_{LT} = f_m$.

Example Problem 1

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

General Information

Project Description *Example Problem 1*

Permitted Left Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)	26.0	26.0	36.0	36.0
Conflicting pedestrian volume,[1] v_{ped} (p/h)	100	100	100	100
$v_{pedg} = v_{ped}$ (C/g_p)	269	269	194	194
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \le 1000$) or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if ($1000 < v_{pedg} \le 5000$)	0.135	0.135	0.097	0.097
Opposing queue clearing green,[3,4] g_q (s)	11.303	8.027	12.201	8.328
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p; if $g_q \ge g_p$ then $f_{Lpb} = 1.0$	0.435	0.309	0.339	0.231
$OCC_{pedu} = OCC_{pedg}\,[1 - 0.5(g_q/g_p)]$	0.106	0.114	0.081	0.086
Opposing flow rate,[3] v_o (veh/h)	833	800	667	466
$OCC_r = OCC_{pedu}\,[e^{-(5/3600)v_o}]$	0.033	0.038	0.032	0.045
Number of cross-street receiving lanes,[1] N_{rec}	1	1	2	2
Number of turning lanes,[1] N_{turn}	1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$; $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$	0.967	0.962	0.981	0.973
Proportion of left turns,[5] P_{LT}	0.090	0.040	0.071	0.067
Proportion of left turns using protected phase,[6] P_{LTA}	0	0	0	0
$f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$	0.997	0.998	0.999	0.998

Permitted Right Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)	26.0	26.0	36.0	36.0
Conflicting pedestrian volume,[1] v_{ped} (p/h)	100	100	100	100
Conflicting bicycle volume,[1,7] v_{bic} (bicycles/h)	20	20	20	20
$v_{pedg} = v_{ped}(C/g_p)$	269	269	194	194
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \le 1000$), or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if ($1000 < v_{pedg} \le 5000$)	0.135	0.135	0.097	0.097
Effective green,[1] g (s)	26.0	26.0	36.0	36.0
$v_{bicg} = v_{bic}(C/g)$	54	54	39	39
$OCC_{bicg} = 0.02 + v_{bicg}/2700$	0.040	0.040	0.034	0.034
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg})$	0.170	0.170	0.128	0.128
Number of cross-street receiving lanes,[1] N_{rec}	1	1	2	2
Number of turning lanes,[1] N_{turn}	1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$; $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$	0.830	0.830	0.923	0.923
Proportion of right turns,[5] P_{RT}	0.049	0.027	0.048	0.083
Proportion of right turns using protected phase,[8] P_{RTA}	0	0	0	0
$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$	0.992	0.995	0.996	0.994

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 – 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 $P_{RTA} = 0$.

Example Problem 1

CAPACITY AND LOS WORKSHEET

General information

Project Description *Example Problem 1*

Capacity Analysis

Phase number	*1*	*1*	*2*	*2*								
Phase type	*P*	*P*	*P*	*P*								
Lane group												
Adjusted flow rate, v (veh/h)	*800*	*833*	*466*	*667*								
Saturation flow rate, s (veh/h)	*2103*	*2665*	*1614*	*1625*								
Lost time, t_L (s), $t_L = I_1 + Y - e$	*4*	*4*	*4*	*4*								
Effective green time, g (s), $g = G + Y - t_L$	*26.0*	*26.0*	*36.0*	*36.0*								
Green ratio, g/C	*0.371*	*0.371*	*0.514*	*0.514*								
Lane group capacity,[1] c = s(g/C), (veh/h)	*780*	*989*	*830*	*835*								
v/c ratio, X	*1.026*	*0.842*	*0.561*	*0.799*								
Flow ratio, v/s	*0.380*			*0.410*								
Critical lane group/phase (√)	√			√								
Sum of flow ratios for critical lane groups, Y_c $Y_c = \sum$ (critical lane groups, v/s)	*0.790*											
Total lost time per cycle, L (s)	*8*											
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$	*0.892*											

Lane Group Capacity, Control Delay, and LOS Determination

	EB	WB	NB	SB
Lane group				
Adjusted flow rate,[2] v (veh/h)	*800*	*833*	*466*	*667*
Lane group capacity,[2] c (veh/h)	*780*	*989*	*830*	*835*
v/c ratio,[2] X = v/c	*1.026*	*0.842*	*0.561*	*0.799*
Total green ratio,[2] g/C	*0.371*	*0.371*	*0.514*	*0.514*
Uniform delay, $d_1 = \frac{0.50\ C\ [1 - (g/C)]^2}{1 - [\min(1, X)g/C]}$ (s/veh)	*22.015*	*20.138*	*11.617*	*14.028*
Incremental delay calibration,[3] k	*0.5*	*0.5*	*0.5*	*0.5*
Incremental delay,[4] d_2 $d_2 = 900T[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}]$(s/veh)	*39.011*	*8.647*	*2.734*	*7.882*
Initial queue delay, d_3 (s/veh) (Appendix F)	*0*	*0*	*0*	*0*
Uniform delay, d_1 (s/veh) (Appendix F)				
Progression adjustment factor, PF	*0.926*	*1.111*	*1.000*	*1.000*
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	*59.4*	*31.0*	*14.4*	*21.9*
LOS by lane group (Exhibit 16-2)	*E*	*C*	*B*	*C*
Delay by approach, $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	*59.4*	*31.0*	*14.4*	*21.9*
LOS by approach (Exhibit 16-2)	*E*	*C*	*B*	*C*
Approach flow rate, v_A (veh/h)	*800*	*833*	*466*	*667*
Intersection delay, $d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ (s/veh)	*34.2*	Intersection LOS (Exhibit 16-2)		*C*

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 Exhibit 16-13.
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.

EXAMPLE PROBLEM 2

The Intersection The intersection of Sixth Street (NB) and Western Boulevard (EB/WB) is located in an outlying area. Intersection geometry and flow characteristics are shown on the Input Worksheet.

The Question What are the delay and peak-hour LOS for this intersection?

The Facts

- √ EB and WB HV = 10 percent,
- √ NB HV = 5 percent,
- √ PHF = 0.95,
- √ No parking EB/WB,
- √ Arrival Type 3,
- √ Movement lost time = 4 s, each phase
- √ Sixth Street NB grade = -2 percent
- √ Three-phase signal,
- √ Western Blvd. bus stopping = 20 b/h,
- √ Pedestrian volume = 50 p/h, all approaches,
- √ All lane widths are 3.6 m,
- √ Western Boulevard has two lanes in each direction plus an added left-turn lane for EB,
- √ Sixth Street is a NB one-way street with two lanes,
- √ Bicycle volume = 20 bicycles/h, all approaches,
- √ NB parking = 20 maneuvers/h, and
- √ Peak-hour volume data, by approach and movement, are shown on the Input Worksheet.

Comments

- √ Assume a range of cycle lengths of 70 s to 100 s. This range relates to default values given in Chapter 10,
- √ Assume crosswalk width = 3.0 m for all approaches,
- √ Assume N/S crosswalks are 18.0 m long and E/W crosswalks are 13.2 m long,
- √ Assume base saturation flow rate = 1,900 pc/h/ln,
- √ Assume E_T = 2.0, and
- √ Signal timing and cycle length are not given, therefore the quick estimation method is required to determine these two parameters.

Steps

1.	The quick estimation method is used to determine the signal phasing and cycle length. Known or assumed input data are entered on the Input Worksheet.	
2.	In the Left-Turn Treatment Worksheet, EBLT is treated as a protected phase and NBLT as an unopposed phase. EBLT is treated as a protected phase because $(v_L)(v_o)$ exceeds 90,000 with two opposing through lanes.	
3.	For each intersection approach, a Lane Volume Worksheet is completed and critical lane volumes are determined.	
4.	Using the computed critical lane volumes, cycle length is computed as 31.9 s (see Quick Estimation Control Delay and LOS Worksheet). However, we use 70.0 s since this was set by the analyst as the "minimum." Green times are computed for EB/WB and NB phases. The Quick Estimation Control Delay and LOS Worksheet is used. For example, for Phase 1: $g = (70 - 12)\left(\frac{126}{1014}\right) + t_L = 7.2\text{ s} + 4.0 = 11\,2\text{ s}$ These computed green times are now used as input for the next steps.	
5.	The timing for Phases 1, 2, and 3 is 11.2 s, 27.4 s, and 31.4 s, respectively. After yellow time is subtracted, the effective green is 7.2 s, 23.4 s, and 27.4 s, respectively.	
6.	Pedestrians/cycle.	$50\frac{p}{h} * \frac{1\text{h}}{3,600\text{ s}} * 70\text{ s} = 0.972$ p/cycle

7. Minimum effective green time required for pedestrians (use Equation 16-2).	$G_p = 3.2 + \frac{L}{1.2} + 0.27N_{ped}$ $G_p(\text{Western}) = 3.2 + \frac{18.0}{1.2} + 0.27(0.972) = 18.5$ s $G_p(\text{Sixth}) = 3.2 + \frac{13.2}{1.2} + 0.27(0.972) = 14.5$ s
8. Compare minimum effective green time required for pedestrians with actual effective green.	g(Western) = 23.4 s, which is >18.5 s g(Sixth) = 27.4 s, which is >14.5 s
9. Proportion of left turns and right turns.	Proportions of left- and right-turn traffic are found by dividing the appropriate turning volumes by the total lane group volume. P_{LT} (EB) = 1.000 for the exclusive left-turn lane
10. Lane width adjustment factor (use Exhibit 16-7).	$f_w = 1 + \frac{(W - 3.6)}{9}$ $f_w(EB) = 1 + \frac{(3.6 - 3.6)}{9} = 1.000$
11. Heavy-vehicle adjustment factor (use Exhibit 16-7).	$f_{HV} = \frac{100}{100 + \%HV(E_T - 1)}$ $f_{HV}(EB) = \frac{100}{100 + 10(2.0 - 1)} = 0.909$
12. Percent grade adjustment factor (use Exhibit 16-7).	0% grade, f_g = 1.000
13. Parking adjustment factor (use Exhibit 16-7).	No parking maneuvers (EB and WB), f_p = 1.000 20 parking maneuvers/hour (NB), f_p = 0.900
14. Bus blockage adjustment factor (use Exhibit 16-7).	No bus stopping (NB), f_{bb} = 1.0 20 buses/hour stopping (EB and WB), f_{bb} = 0.960
15. Area type adjustment factor (use Exhibit 16-7).	For outlying areas, f_a = 1.000
16. Lane utilization adjustment factor (use Exhibit 16-7).	Refer to Exhibit 10-23. This factor is applied to establish the conditions in the worst lane within each lane group. Otherwise, the results would reflect the average of all lanes of the defined lane groups. Use f_{LU} = 0.950 for all through movements.
17. Left-turn adjustment factor (use Exhibit 16-7).	f_{LT} applies to the EB left turn. For protected LT, f_{LT} = 0.950. For permitted LT, the supplemental worksheet for multilane approaches is used. f_{LT} (permitted) = 0.149. NB left turn is treated as a shared lane protected left turn because it has no opposing flow. f_{LT} (NB) = 0.998 (use equation in Exhibit 16-7).
18. Right-turn adjustment factor (use Exhibit 16-7).	f_{RT} is applied to NB and WB right turns. $f_{RT} = 1.0 – 0.150P_{RT}$
19. Left-turn pedestrian/bicycle adjustment factor (use Exhibit 16-7).	Supplemental worksheet for pedestrian-bicycle effects is used.
20. Right-turn pedestrian/bicycle adjustment factor (use Exhibit 16-7).	Supplemental worksheet for pedestrian-bicycle effects is used.

21. Saturation flow (use Equation 16-4).	$s = s_o\ N\ f_w\ f_{HV}\ f_g\ f_p\ f_{bb}\ f_{LU}\ f_a\ f_{LT}\ f_{RT}\ f_{Lpb}\ f_{Rpb}$ s(EBLT prot) = 1900 * 1 * 1.000 * 0.909 * 1.000 * 1.000 * 1.000 * 1.000 * 1.000 * 0.950 * 1.000 * 1.000 * 1.000 = 1641 veh/h
22. Lane group capacity (use Equation 16-6).	c = s(g/C) c(EBLT prot) = 1641(0.103) = 169 veh/h
23. v/c ratio.	$v/c(EB) = \frac{126}{169} = 0.746$
24. Determine critical lane group in each timing phase.	The lane group with the highest v/c ratio in a phase is considered the critical lane group. The critical lane groups are EBLT (protected), WB through, and NB through.
25. Flow ratio of critical lane group.	$v/s(EBLT\ Prot) = \frac{126}{1641} = 0.077$
26. Sum of critical lane group v/s ratios.	$Y_c = 0.077 + 0.275 + 0.289 = 0.641$
27. Critical flow rate to capacity ratio.	$X_c = \frac{Y_c * C}{C - L}$ $X_c = \frac{0.641(70.0)}{70.0 - 12} = 0.774$
28. Uniform delay.	Supplemental uniform delay worksheet is needed to compute d_1 for the EBLT because it has both protected and permitted phases. d_1 (EBLT) = 11.811 s/veh.
29. Incremental delay (use Equation 16-13).	$d_2 = 900T\left[(X-1) + \sqrt{(\ldots)}\right]$ $d_2(EBLT) = 900(0.250)\left[(0.468-1) + \sqrt{(\ldots)}\right] = 5.748$ s/veh
30. Progression adjustment factor (use Exhibit 16-12).	PF = 1.000 for arrival type 3
31. Lane group delay (use Equation 16-10).	$d = d_1PF + d_2 + d_3$ d (EBLT) = 11.811(1.000) + 5.748 + 0 = 17.6 s/veh
32. Approach delay (use Equation 16-14).	$d_A = \frac{\sum(d)(v)}{\sum v}$ $d_A = \frac{(17.6 * 126) + (15.6 * 1{,}032)}{126 + 1{,}032} = 15.8$ s/veh
33. Intersection delay (use Equation 16-15).	$d_I = \frac{\sum(d_A)(V_A)}{\sum V_A}$ $d_I(EB) = \frac{(15.8 * 1{,}158) + (28.8 * 842) + (22.4 * 894)}{(1{,}158 + 842 + 894)}$ = 21.6 s/veh
34. LOS by lane group, approach, and intersection (use Exhibit 16-2).	LOS (EBLT) = B LOS (EB) = B LOS intersection = C

The calculation results are summarized as follows.

Direc/ Ln Grp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Factor PF	Ln Grp Cap	Cal Term k	Incr Delay d_2	Ln Grp Delay	Ln Grp LOS	Delay by App	LOS by App
EB/L	0.468	0.494	11.811	1.000	269	0.500	5.748	17.6	B		
EB/T	0.663	0.494	13.326	1.000	1556	0.500	2.243	15.6	B	15.8	B
WB/TR	0.822	0.334	21.400	1.000	1024	0.500	7.429	28.8	C	28.8	C
NB/LTR	0.740	0.391	18.266	1.000	1208	0.5000	4.097	22.4	C	22.4	C
Intersection Delay = 21.6 s/veh							Intersection LOS = C				

Alternatives

Two alternatives are considered for the EB left-turn treatment: protected only and permitted-plus-protected. The first alternative is to assess the impact of eliminating the EB permitted left-turn phase. The left-turn volume is low and is below the capacity of the protected phase. Hence, queue spillovers will not occur.

The supplemental worksheet for permitted left turns is not needed because there are no permitted left turns. The supplemental uniform delay worksheet is also not needed because the left turn is contained in a single phase. The intersection performance is reassessed and the results are as follows.

Direction/ LnGrp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Fact PF	Lane Grp Cap	Cal Term k	Incr Delay d_2	Lane Grp Delay	Lane Grp LOS	Delay by App	LOS by App
EB/L	0.746	0.103	30.505	1.000	169	0.500	25.564	56.1	E		
EB/T	0.663	0.494	13.326	1.000	1556	0.500	2.243	15.6	B	20.0	B
WB/TR	0.822	0.334	21.400	1.000	1024	0.500	7.429	28.8	C	28.3	C
NB/LTR	0.740	0.391	18.266	1.000	1208	0.500	4.097	22.4	C	22.5	C
Intersection Delay = 23.3 s/veh							Intersection LOS = C				

The elimination of the permitted phase has caused a drop in the left-turn capacity from 269 veh/h to 169 veh/h. Correspondingly, the delay has increased from 17.6 s/veh to 56.1 s/veh. The overall intersection delay has also increased by 1.7 s/veh to 23.3 s/veh.

From the operational standpoint, the protected-only phase is undesirable because it induces additional delay. However, from the safety standpoint, the phase may be desirable because safety may be enhanced by reducing the number of accidents caused by turning vehicles.

The second alternative is to reverse the EB left-turn treatment from protected-plus-permitted to permitted-plus-protected. The cycle length and phase times are kept the same. The intersection performance is reassessed and the results are as follows.

Direc/ Ln Grp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Factor PF	Ln Grp Cap	Cal Term k	Incr Delay d_2	Lane Grp Delay	Lane Grp LOS	Delay by App	LOS by App
EB/L	0.346	0.494	15.626	1.000	364	0.500	2.593	18.2	B		
EB/T	0.663	0.494	13.326	1.000	1556	0.500	2.243	15.6	B	15.9	B
WB/TR	0.822	0.334	21.400	1.000	1024	0.500	7.429	28.8	C	28.3	C
NB/LTR	0.740	0.391	18.266	1.000	1208	0.500	4.097	22.4	C	22.5	C
Intersection Delay = 21.7 s/veh							Intersection LOS = C				

Reversing the EB left-turn phase results in a slight increase in delay from 17.6 s/veh to 18.2 s/veh. The intersection delay also increases by 0.1 s/veh to 21.7 s/veh.

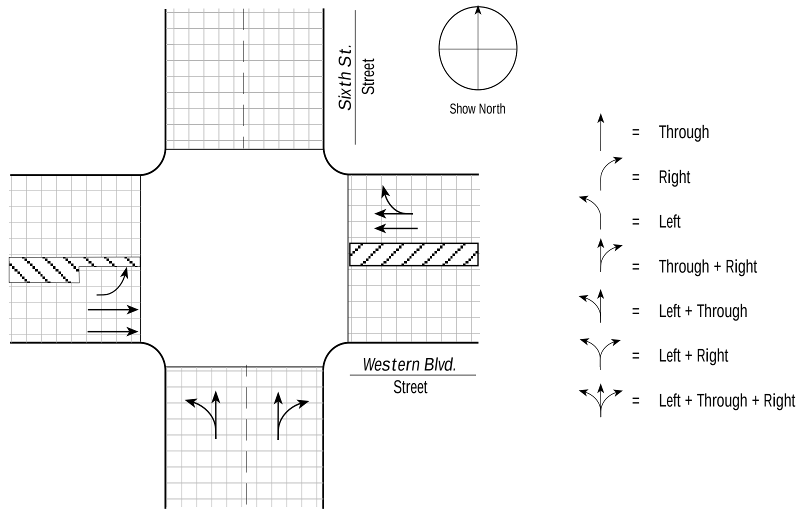
In conclusion, solely on the basis of operations, the first option (protected-plus-permitted EB left turn) appears to be the most desirable.

Example Problem 2

QUICK ESTIMATION INPUT WORKSHEET

General Information		Site Information	
Analyst	*WLL*	Intersection	*Sixth St./Western Blvd.*
Agency or Company	*CEI*	Area Type	❑ CBD ☒ Other
Date Performed	*4/21/99*	Jurisdiction	
Analysis Time Period		Analysis Year	*1999*

Intersection Geometry



Volume and Signal 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)	*120*	*980*			*700*	*100*	*40*	*785*	*25*			
Proportion of LT or RT (P_{LT} or P_{RT})[2]		-			-			-			-	
Parking (Yes/No)		*No*			*No*			*Yes*				
Left-turn treatment (permitted, protected, not opposed) (if known)								*Not Opposed*				

Peak-hour factor, PHF *0.95*

Cycle length Minimum, C_{min} *70* s Maximum, C_{max} *100* s or Given, C ____ s

Lost time/phase *4* s

Notes

1. RT volumes, as shown, exclude RTOR.
2. 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.

Example Problem 2

LEFT-TURN TREATMENT WORKSHEET

General Information

Description *Example Problem 2*

Check #1. Left-Turn Lane Check

Approach	EB	WB	NB	SB
Number of left-turn lanes	*1*			
Protect left turn (Y or N)?	*N*			

If the number of left-turn lanes on any approach exceeds 1, then it is recommended that the left turns on that approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Check #2. Minimum Volume Check

Approach	EB	WB	NB	SB
Left-turn volume	*120*			
Protect left turn (Y or N)?	*N*			

If left-turn volume on any approach exceeds 240 veh/h, then it is recommended that the left turns on that approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Check #3. Minimum Cross-Product Check

Approach	EB	WB	NB	SB
Left-turn volume, V_L (veh/h)	*120*			
Opposing mainline volume, V_o (veh/h)	*800*			
Cross-product (V_L * V_o)	*96,000*			
Opposing through lanes	*2*			
Protected left turn (Y or N)?	*Y*			

Minimum Cross-Product Values for Recommending Left-Turn Protection

Number of Through Lanes	Minimum Cross-Product
1	50,000
2	90,000
3	110,000

If the cross-product on any approach exceeds the above values, then it is recommended that the left turns on that approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Check #4. Sneaker Check

Approach	EB	WB	NB	SB
Left-turn volume, V_L (veh/h)				
Sneaker capacity = 7200/C				
Left-turn equivalence, E_{L1} (Exhibit C16-3)				
Protected left turn (Y or N)?				

If the left-turn equivalence factor is 3.5 or higher (computed in Exhibit A10-4, quick estimation lane volume worksheet) and the unadjusted left turn is greater than the sneaker capacity, then it is recommended that the left turns on that approach be protected.

Notes

1. If any approach is recommended for left-turn protection but the analyst wishes to analyze it as permitted, the planning application may give overly optimistic results. The analyst should instead use the more robust method described in Chapter 16, Signalized Intersections.
2. All volumes used in this worksheet are unadjusted hourly volumes.

Example Problem 2

QUICK ESTIMATION LANE VOLUME WORKSHEET

General Information

Description/Approach *Example Problem 2, EB*

Right-Turn Movement

	Exclusive RT Lane	Shared RT Lane
RT volume, V_R (veh/h)		
Number of exclusive RT lanes, N_{RT}		use 1
RT adjustment factor,[1] f_{RT}		
RT volume per lane, V_{RT} (veh/h/ln) $V_{RT} = \frac{V_R}{(N_{RT} \times f_{RT})}$		

Left-Turn Movement

LT volume, V_L (veh/h)	120
Opposing mainline volume, V_o (veh/h)	800
Number of exclusive LT lanes, N_{LT}	1
LT adjustment factor,[2] f_{LT}	0.95
LT volume per lane,[3] V_{LT} (veh/h/ln) $V_{LT} = \frac{V_L}{(N_{LT} \times f_{LT})}$	Permitted LT use 0 Protected LT 126 Not Opposed LT ______

Through Movement

	Permitted LT	Protected LT	Not Opposed LT
Through volume, V_T (veh/h)		980	
Parking adjustment factor, f_p		1.000	
Number of through lanes, N_{TH}		2	
Total approach volume,[4] V_{tot} (veh/h) $V_{tot} = \frac{[V_{RT}(\text{shared}) + V_T + V_{LT\,(\text{not opp})}]}{f_p}$		980	

Through Movement with Exclusive LT Lane

Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{tot}}{N_{TH}}$		490	
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{LT}, V_{RT} (exclusive), V_{TH}]		490	

Through Movement with Shared LT Lane

Proportion of left turns, P_{LT}		Does not apply	Does not apply
LT equivalence, E_{L1} (Exhibit C16-3)		Does not apply	Does not apply
LT adjustment, f_{DL} (Exhibit A10-6)			use 1.0
Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{tot}}{(N_{TH} \times f_{DL})}$			
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{RT}(exclusive), V_{TH}]			

Notes

1. For RT shared or single lanes, use 0.85. For RT double lanes, use 0.75.
2. For LT single lanes, use 0.95. For LT double lanes, use 0.92. For a one-way street or T-intersection, use 0.85 for one lane and 0.75 for two lanes.
3. For unopposed LT shared lanes, N_{LT} = 1.
4. For exclusive RT lanes, V_{RT} (shared) = 0. If not opposed, add V_{LT} to V_T and set $V_{LT\,(\text{not opp})} = 0$.
5. V_{LT} is included only if LT is unopposed. V_{RT} (exclusive) is included only if RT is exclusive.

Example Problem 2

QUICK ESTIMATION LANE VOLUME WORKSHEET

General Information

Description/Approach *Example Problem 2, WB*

Right-Turn Movement

	Exclusive RT Lane	Shared RT Lane
RT volume, V_R (veh/h)		*100*
Number of exclusive RT lanes, N_{RT}		use 1
RT adjustment factor,[1] f_{RT}		*0.850*
RT volume per lane, V_{RT} (veh/h/ln) $V_{RT} = \frac{V_R}{(N_{RT} \times f_{RT})}$		*118*

Left-Turn Movement

LT volume, V_L (veh/h)	
Opposing mainline volume, V_o (veh/h)	
Number of exclusive LT lanes, N_{LT}	
LT adjustment factor,[2] f_{LT}	
LT volume per lane,[3] V_{LT} (veh/h/ln) $V_{LT} = \frac{V_L}{(N_{LT} \times f_{LT})}$	Permitted LT use 0 Protected LT ____ Not Opposed LT ____

Through Movement

	Permitted LT	Protected LT	Not Opposed LT
Through volume, V_T (veh/h)	*700*		
Parking adjustment factor, f_p	*1.000*		
Number of through lanes, N_{TH}	*2*		
Total approach volume,[4] V_{tot} (veh/h) $V_{tot} = \frac{[V_{RT}(\text{shared}) + V_T + V_{LT\ (\text{not opp})}]}{f_p}$	*818*		

Through Movement with Exclusive LT Lane

Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{tot}}{N_{TH}}$	*409*		
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{LT}, V_{RT} (exclusive), V_{TH}]	*409*		

Through Movement with Shared LT Lane

Proportion of left turns, P_{LT}		Does not apply	Does not apply
LT equivalence, E_{L1} (Exhibit C16-3)		Does not apply	Does not apply
LT adjustment, f_{DL} (Exhibit A10-6)			use 1.0
Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{tot}}{(N_{TH} \times f_{DL})}$			
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{RT}(exclusive), V_{TH}]			

Notes

1. For RT shared or single lanes, use 0.85. For RT double lanes, use 0.75.
2. For LT single lanes, use 0.95. For LT double lanes, use 0.92. For a one-way street or T-intersection, use 0.85 for one lane and 0.75 for two lanes.
3. For unopposed LT shared lanes, N_{LT} = 1.
4. For exclusive RT lanes, V_{RT} (shared) = 0. If not opposed, add V_{LT} to V_T and set $V_{LT\ (\text{not opp})}$ = 0.
5. V_{LT} is included only if LT is unopposed. V_{RT} (exclusive) is included only if RT is exclusive.

Example Problem 2

QUICK ESTIMATION LANE VOLUME WORKSHEET

General Information

Description/Approach *Example Problem 2, NB*

Right-Turn Movement

	Exclusive RT Lane	Shared RT Lane
RT volume, V_R (veh/h)		*25*
Number of exclusive RT lanes, N_{RT}		use 1
RT adjustment factor,[1] f_{RT}		*0.85*
RT volume per lane, V_{RT} (veh/h/ln) $V_{RT} = \frac{V_R}{(N_{RT} \times f_{RT})}$		*29*

Left-Turn Movement

LT volume, V_L (veh/h)	*40*
Opposing mainline volume, V_o (veh/h)	*0*
Number of exclusive LT lanes, N_{LT}	*0*
LT adjustment factor,[2] f_{LT}	*0.85*
LT volume per lane,[3] V_{LT} (veh/h/ln) $V_{LT} = \frac{V_L}{(N_{LT} \times f_{LT})}$	Permitted LT use 0 Protected LT _______ Not Opposed LT *47*

Through Movement

	Permitted LT	Protected LT	Not Opposed LT
Through volume, V_T (veh/h)			*785*
Parking adjustment factor, f_p			*0.900*
Number of through lanes, N_{TH}			*2*
Total approach volume,[4] V_{tot} (veh/h) $V_{tot} = \frac{[V_{RT}(\text{shared}) + V_T + V_{LT\,(\text{not opp})}]}{f_p}$			*957*

Through Movement with Exclusive LT Lane

Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{tot}}{N_{TH}}$			
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{LT}, V_{RT} (exclusive), V_{TH}]			

Through Movement with Shared LT Lane

Proportion of left turns, P_{LT}		Does not apply	Does not apply
LT equivalence, E_{L1} (Exhibit C16-3)		Does not apply	Does not apply
LT adjustment, f_{DL} (Exhibit A10-6)			use 1.0
Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{tot}}{(N_{TH} \times f_{DL})}$			*479*
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{RT}(exclusive), V_{TH}]			*479*

Notes

1. For RT shared or single lanes, use 0.85. For RT double lanes, use 0.75.
2. For LT single lanes, use 0.95. For LT double lanes, use 0.92. For a one-way street or T-intersection, use 0.85 for one lane and 0.75 for two lanes.
3. For unopposed LT shared lanes, $N_{LT} = 1$.
4. For exclusive RT lanes, V_{RT} (shared) = 0. If not opposed, add V_{LT} to V_T and set $V_{LT\,(\text{not opp})} = 0$.
5. V_{LT} is included only if LT is unopposed. V_{RT} (exclusive) is included only if RT is exclusive.

Example Problem 2

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET

General Information

Description *Example Problem 2*

East-West Phasing Plan

Selected plan (Exhibit A10-8) *2b*	Phase No. 1	Phase No. 2	Phase No. 3
Movement codes	*EB TH LT*	*EB WB TH*	
Critical phase volume, CV (veh/h)	*126*	*409*	
Lost time/phase, t_L (s)	*4*	*4*	

North-South Phasing Plan

Selected plan (Exhibit A10-8) *1*	Phase No. 1	Phase No. 2	Phase No. 3
Movement codes	*NB TH LT*		
Critical phase volume, CV (veh/h)	*479*		
Lost time/phase, t_L (s)	*4*		

Intersection Status Computation

Critical sum, CS (veh/h) $CS = \sum CV$	*1014*
Lost time/cycle, L (s) $L = \sum t_L$	*12*
Reference sum flow rate RS (veh/h)[1]	*1625*
Cycle length, C (s) $C_{min} \le C \le C_{max}$ $C = \dfrac{L}{1 - \left[\dfrac{\min(CS, RS)}{RS}\right]}$	*31.9 < 70, use 70*
Critical v/c ratio, X_{cm} $X_{cm} = \dfrac{CS}{RS\left(1 - \dfrac{L}{C}\right)}$	*0.753*
Intersection status (Exhibit A10-9)	*under capacity*

Green Time Calculation

East-West Phasing	Phase No. 1	Phase No. 2	Phase No. 3
Green time, g (s) $g = \left[(C - L)\left(\dfrac{CV}{CS}\right) + t_L\right]$	*11.2*	*27.4*	
North-South Phasing	**Phase No. 1**	**Phase No. 2**	**Phase No. 3**
Green time, g (s) $g = \left[(C - L)\left(\dfrac{CV}{CS}\right) + t_L\right]$	*31.4*		

Control Delay and LOS

	EB	WB	NB	SB
Lane group				
Lane group adjusted volume from lane volume worksheet, V (veh/h)				
Green ratio, g/C				
Lane group saturation flow rate, s (veh/h) s = RS * number of lanes in lane group				
v/c ratio, X $X = \dfrac{V/s}{g/C}$				
Lane group capacity, c (veh/h) $c = \dfrac{V}{X}$				
Progression adjustment factor, PF (Exhibit 16-12)				
Uniform delay, d_1 (s/veh) (Equation 16-11)				
Incremental delay, d_2 (s/veh) (Equation 16-12)				
Initial queue delay, d_3 (s/veh) (Appendix F, Ch. 16)				
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)				
Delay by approach, d_A (s/veh) $\dfrac{\sum(d)(V)}{\sum V}$				
Approach flow rate, V_A (veh/h)				
Intersection delay, d_I (s/veh) $d_I = \dfrac{\sum(d_A)(V_A)}{\sum V_A}$		Intersection LOS (Exhibit 16-2)		

Notes

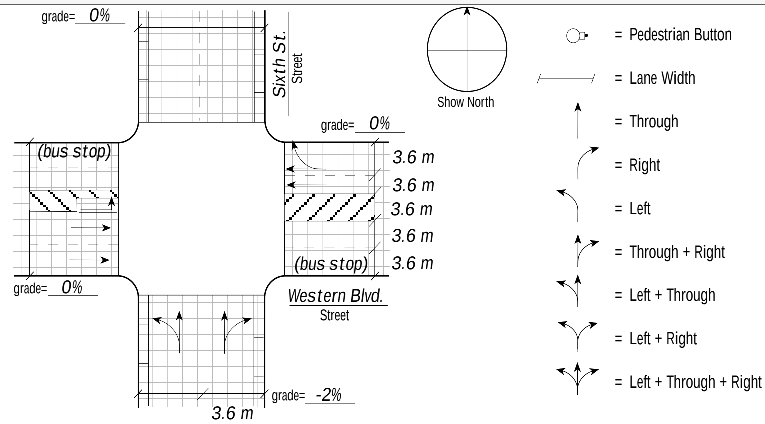
1. $RS = 1710(PHF)(f_a)$, where f_a is area adjustment factor (0.90 for CBD and 1.0 for all others).

Example Problem 2

INPUT WORKSHEET

General Information		**Site Information**	
Analyst	*WLL*	Intersection	*Sixth St./Western Blvd.*
Agency or Company	*CEI*	Area Type	❑ CBD ☒ Other
Date Performed	*4/21/99*	Jurisdiction	
Analysis Time Period		Analysis Year	*1999*

Intersection Geometry

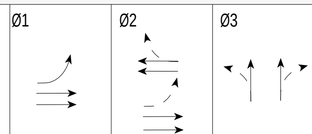


Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]
Volume, V (veh/h)	120	980			700	100	40	785	25			
% heavy vehicles, % HV		10			10			5				
Peak-hour factor, PHF		0.95			0.95			0.95				
Pretimed (P) or actuated (A)												
Start-up lost time, l_1 (s)												
Extension of effective green time, e (s)												
Arrival type, AT		3			3			3				
Approach pedestrian volume,[2] v_{ped} (p/h)		50			50			50				
Approach bicycle volume,[2] v_{bic} (bicycles/h)		20			20			20				
Parking (Y or N)		N			N			Y				
Parking maneuvers, N_m (maneuvers/h)		-			-			20				
Bus stopping, N_B (buses/h)		20			20			-				
Min. timing for pedestrians,[3] G_p (s)		18.5			18.5			14.5				

Signal Phasing Plan

DIAGRAM



	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
Timing	G = 7.2 Y = 4	G = 23.4 Y = 4	G = 27.4 Y = 4	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =

Protected turns — Permitted turns — Pedestrian — Cycle length, C = 70 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 16-2.

Example Problem 2

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET

General Information

Project Description *Example Problem 2*

Volume Adjustment

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	120	980			700	100	40	785	25			
Peak-hour factor, PHF		0.95			0.95			0.95				
Adjusted flow rate, v_p = V/PHF (veh/h)	126	1032			737	105	42	826	26			
Lane group												
Adjusted flow rate in lane group, v (veh/h)	126		1032		842			894				
Proportion[1] of LT or RT (P_{LT} or P_{RT})	1.000	-			-	0.125	0.047	-	0.029		-	

Saturation Flow Rate (see Exhibit 16-7 to determine adjustment factors)

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Base saturation flow, s_o (pc/h/ln)	1900	1900	1900		1900			1900				
Number of lanes, N	1	1	2		2			2				
Lane width adjustment factor, f_w	1.000	1.000	1.000		1.000			1.000				
Heavy-vehicle adjustment factor, f_{HV}	0.909	0.909	0.909		0.909			0.952				
Grade adjustment factor, f_g	1.000	1.000	1.000		1.000			1.010				
Parking adjustment factor, f_p	1.000	1.000	1.000		1.000			0.900				
Bus blockage adjustment factor, f_{bb}	1.000	1.000	0.960		0.960			1.000				
Area type adjustment factor, f_a	1.000	1.000	1.000		1.000			1.000				
Lane utilization adjustment factor, f_{LU}	1.000	1.000	0.950		0.950			0.950				
Left-turn adjustment factor, f_{LT}	0.950	0.149	1.000		1.000			0.998				
Right-turn adjustment factor, f_{RT}	1.000	1.000	1.000		0.981			0.996				
Left-turn ped/bike adjustment factor, f_{Lpb}	1.000	0.999	1.000		1.000			0.998				
Right-turn ped/bike adjustment factor, f_{Rpb}	1.000	1.000	1.000		0.992			0.998				
Adjusted saturation flow, s (veh/h) $s = s_o N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$	1641	257	3150		3066			3074				

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.

Example Problem 2

SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS OPPOSED BY MULTILANE APPROACH

General Information

Project Description *Example Problem 2*

Input

	EB	WB	NB	SB
Cycle length, C (s)	70			
Total actual green time for LT lane group,[1] G (s)	34.6			
Effective permitted green time for LT lane group,[1] g (s)	27.4			
Opposing effective green time, g_o (s)	23.4			
Number of lanes in LT lane group,[2] N	1			
Number of lanes in opposing approach, N_o	2			
Adjusted LT flow rate, v_{LT} (veh/h)	126			
Proportion of LT volume in LT lane group,[3] P_{LT}	1.000			
Adjusted flow rate for opposing approach, v_o (veh/h)	842			
Lost time for LT lane group, t_L	0			

Computation

	EB	WB	NB	SB
LT volume per cycle, $LTC = v_{LT}C/3600$	2.450			
Opposing lane utilization factor, f_{LUo} (refer to Volume Adjustment and Saturation Flow Rate Worksheet)	0.950			
Opposing flow per lane, per cycle $v_{olc} = \frac{v_oC}{3600N_of_{LUo}}$ (veh/C/ln)	8.617			
$g_f = G[e^{-0.882(LTC^{0.717})}] - t_L$ $g_f \le g$ (except for exclusive left-turn lanes)[1, 4]	0			
Opposing platoon ratio, R_{po} (refer to Exhibit 16-11)	1			
Opposing queue ratio, $qr_o = \max[1 - R_{po}(g_o/C), 0]$	0.666			
$g_q = \frac{v_{olc}qr_o}{0.5 - [v_{olc}(1 - qr_o)/g_o]} - t_L$, $v_{olc}(1 - qr_o)/g_o \le 0.49$ (note case-specific parameters)[1]	15.222			
$g_u = g - g_q$ if $g_q \ge g_f$, or $g_u = g - g_f$ if $g_q < g_f$	12.178			
E_{L1} (refer to Exhibit C16-3)	3.0			
$P_L = P_{LT}\left[1 + \frac{(N-1)g}{(g_f + g_u/E_{L1} + 4.24)}\right]$ (except with multilane subject approach)[5]	1			
$f_{min} = 2(1 + P_L)/g$	0.146			
$f_m = [g_f/g] + [g_u/g]\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right]$, $(f_{min} \le f_m \le 1.00)$	0.149			
$f_{LT} = [f_m + 0.91(N - 1)]/N$ (except for permitted left turns)[6]	0.149			

Notes

1. Refer to Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 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 $f_{LT} = f_m$.

Example Problem 2

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

General Information

Project Description *Example Problem 2*

Permitted Left Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)	*27.4*		*27.4*	
Conflicting pedestrian volume,[1] v_{ped} (p/h)	*50*		*50*	
$v_{pedg} = v_{ped}$ (C/g_p)	*128*		*128*	
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \le 1000$) or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if ($1000 < v_{pedg} \le 5000$)	*0.064*		*0.064*	
Opposing queue clearing green,[3,4] g_q (s)	*15.222*		*0*	
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p; if $g_q \ge g_p$ then $f_{Lpb} = 1.0$	*0.556*		*0*	
$OCC_{pedu} = OCC_{pedg}\,[1 - 0.5(g_q/g_p)]$	*0.046*		*0.064*	
Opposing flow rate,[3] v_o (veh/h)	*842*		*0*	
$OCC_r = OCC_{pedu}\,[e^{-(5/3600)v_o}]$	*0.014*		*0.064*	
Number of cross-street receiving lanes,[1] N_{rec}	*2*		*2*	
Number of turning lanes,[1] N_{turn}	*1*		*1*	
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$; $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$	*0.992*		*0.962*	
Proportion of left turns,[5] P_{LT}	*1.000*		*0.047*	
Proportion of left turns using protected phase,[6] P_{LTA}	*0.896*		*0*	
$f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$	*0.999*		*0.998*	

Permitted Right Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)		*27.4*	*27.4*	
Conflicting pedestrian volume,[1] v_{ped} (p/h)		*50*	*50*	
Conflicting bicycle volume,[1,7] v_{bic} (bicycles/h)		*20*	*20*	
$v_{pedg} = v_{ped}(C/g_p)$		*150*	*128*	
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \le 1000$), or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if ($1000 < v_{pedg} \le 5000$)		*0.075*	*0.064*	
Effective green,[1] g (s)		*23.4*	*27.4*	
$v_{bicg} = v_{bic}(C/g)$		*60*	*51*	
$OCC_{bicg} = 0.02 + v_{bicg}/2700$		*0.042*	*0.039*	
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg})$		*0.114*	*0.101*	
Number of cross-street receiving lanes,[1] N_{rec}		*2*	*2*	
Number of turning lanes,[1] N_{turn}		*1*	*1*	
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$; $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$		*0.932*	*0.939*	
Proportion of right turns,[5] P_{RT}		*0.125*	*0.029*	
Proportion of right turns using protected phase,[8] P_{RTA}		*0*	*0*	
$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$		*0.992*	*0.998*	

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 – 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 $P_{RTA} = 0$.

Example Problem 2

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

General Information

Project Description *Example Problem 2*

v/c Ratio Computation

	EB	WB	NB	SB
Cycle length, C (s)	*70.0*			
Protected phase eff. green interval, g (s)	*7.2*			
Opposing queue effective green interval, g_q (s)	*15.222*			
Unopposed green interval, g_u (s)	*12.178*			
Red time, r (s) $r = C - g - g_q - g_u$	*35.400*			
Arrival rate, q_a (veh/s) $q_a = \frac{v}{3600 * \max[X, 1.0]}$	*0.035*			
Protected phase departure rate, s_p (veh/s) $s_p = \frac{s}{3600}$	*0.456*			
Permitted phase departure rate, s_s (veh/s) $s_s = \frac{s(g_q + g_u)}{(g_u * 3600)}$	*0.161*			
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, $X_{perm} = \frac{q_a(r + g_q + g_u)}{s_s\, g_u}$	*0.489*			
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	*0.454*			

Uniform Queue Size and Delay Computations

	EB	WB	NB	SB
Queue at beginning of green arrow, Q_a	*1.239*			
Queue at beginning of unsaturated green, Q_u	*0.533*			
Residual queue, Q_r	*0*			
Uniform delay, d_1	*11.811*			

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	$[0.50/(q_a C)][(r + g_q)Q_u + g_u(Q_u + Q_a) + Q_a^2/(s_p - q_a)]$

Example Problem 2

CAPACITY AND LOS WORKSHEET

General information

Project Description *Example Problem 2*

Capacity Analysis

Phase number	1	2	1/2	2	3
Phase type	P	S	P	P	P
Lane group					
Adjusted flow rate, v (veh/h)	126	0	1032	842	894
Saturation flow rate, s (veh/h)	1641	257	3150	3066	3090
Lost time, t_L (s), $t_L = I_1 + Y - e$	4	0	4	4	4
Effective green time, g (s), $g = G + Y - t_L$	7.2	27.4	34.6	23.4	27.4
Green ratio, g/C	0.103	0.391	0.494	0.334	0.391
Lane group capacity,[1] c = s(g/C), (veh/h)	169	100	1556	1024	1208
v/c ratio, X	0.746	0.000	0.663	0.822	0.740
Flow ratio, v/s	0.077	0.000	0.328	0.275	0.289
Critical lane group/phase (√)	√			√	√
Sum of flow ratios for critical lane groups, Y_c $Y_c = \Sigma$ (critical lane groups, v/s)	0.641				
Total lost time per cycle, L (s)	12				
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$	0.774				

Lane Group Capacity, Control Delay, and LOS Determination

	EB		WB	NB	SB
Lane group					
Adjusted flow rate,[2] v (veh/h)	126	1032	842	894	
Lane group capacity,[2] c (veh/h)	269	1556	1024	1208	
v/c ratio,[2] X = v/c	0.468	0.663	0.822	0.740	
Total green ratio,[2] g/C	0.494	0.494	0.334	0.391	
Uniform delay, $d_1 = \frac{0.50\, C\, [1 - (g/C)]^2}{1 - [\min(1, X)g/C]}$ (s/veh)	11.811	13.326	21.400	18.266	
Incremental delay calibration,[3] k	0.500	0.500	0.500	0.500	
Incremental delay,[4] d_2 $d_2 = 900T\left[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}\right]$ (s/veh)	5.748	2.243	7.429	4.097	
Initial queue delay, d_3 (s/veh) (Appendix F)	0	0	0	0	
Uniform delay, d_1 (s/veh) (Appendix F)					
Progression adjustment factor, PF	1.000	1.000	1.000	1.000	
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	17.6	15.6	28.8	22.4	
LOS by lane group (Exhibit 16-2)	B	B	C	C	
Delay by approach, $d_A = \frac{\Sigma(d)(v)}{\Sigma v}$ (s/veh)	15.8		28.8	22.4	
LOS by approach (Exhibit 16-2)	B		C	C	
Approach flow rate, v_A (veh/h)	1158		842	894	
Intersection delay, $d_I = \frac{\Sigma(d_A)(v_A)}{\Sigma v_A}$ (s/veh)	21.6		Intersection LOS (Exhibit 16-2)		C

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 Exhibit 16-13.
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.

EXAMPLE PROBLEM 3

The Intersection The intersection of Fifth Avenue (NB/SB) and Twelfth Street (EB/WB) is a major CBD junction of two urban streets.

The Question What are the delay and LOS during the peak hour for lane groups, approaches, and the intersection as a whole?

The Facts

√ Twelfth Street HV = 5 percent,
√ Fifth Avenue HV = 2 percent,
√ Twelfth Street PHF = 0.85,
√ Fifth Avenue PHF = 0.90,
√ Actuated signal,
√ Yellow = 4.0 s,
√ Level terrain,
√ 3.0-m lane widths for EB/WB,
√ Pedestrian signals exist,
√ 3.6-m lane widths for NB/SB,
√ Fifth Avenue is a four-lane street,
√ Twelfth Street is a four-lane street,
√ Twelfth Street parking, 5 maneuvers/h,
√ Twelfth Street pedestrian volume = 120 p/h,
√ Fifth Avenue pedestrian volume = 40 p/h,
√ Movement lost time = 4 s,
√ Arrival Type 3,
√ No bicycles, and
√ No buses.

Comments

√ Assume crosswalk width = 3.0 m for all approaches,
√ Assume base saturation flow rate = 1,900 pc/h/ln,
√ Assume E_T = 2.0,
√ No overlaps in signal phasing,
√ 90.0-s cycle length, with green times given, and
√ Assume a unit extension of 2.5 s for all phases.

Steps

1. Pedestrians/cycle.	$120\,\frac{p}{h} * \frac{1\,h}{3{,}600\,s} * 90.0\,s = 3$ p (12th St.) $40\,\frac{p}{h} * \frac{1\,h}{3{,}600\,s} * 90.0\,s = 1$ p (5th Ave.)
2. Minimum effective green time required for pedestrians (use Equation 16-2).	$G_p = 3.2 + \frac{L}{S_p} + 0.27N_{ped}$ (for $W_E \leq 3.0$ m) G_p (12th) = $3.2 + \frac{18.0}{1.2} + 0.27(3) = 19.0$ s G_p (5th) = $3.2 + \frac{21.0}{1.2} + 0.27(1) = 21.0$ s
3. Compare minimum effective green time required for pedestrians with actual effective green.	g(12th) = 19.2 s, which is > 19.0 s g(5th) = 50.7 s, which is > 21.0 s
4. Proportion of left turns and right turns.	Proportions of left- and right-turn traffic are found by dividing the appropriate turning flow rates by the total lane group flow rate. P_{LT} for exclusive LT lane is 1.000
5. Lane width adjustment factor (use Exhibit 16-7).	$f_w = 1 + \frac{(W - 3.6)}{9}$ $f_w(NB/SB) = 1 + \frac{(3.6 - 3.6)}{9} = 1.000$ $f_w(EB/WB) = 1 + \frac{(3.0 - 3.6)}{9} = 0.933$

6. Heavy-vehicle adjustment factor (use Exhibit 16-7).	$f_{HV} = \frac{100}{100 + \% HV(E_T - 1)}$ $f_{HV}(NB / SB) = \frac{100}{100 + 2(2.0 - 1)} = 0.980$ $f_{HV}(EB / WB) = \frac{100}{100 + 5(2.0 - 1)} = 0.952$
7. Percent grade adjustment factor (use Exhibit 16-7).	0% grade, f_g = 1.000
8. Parking adjustment factor (use Exhibit 16-7).	$f_p = \frac{N - 0.1 - \frac{18N_m}{3600}}{N}$ f_p = 0.938 for EB and WB through/right lane groups
9. Bus blockage adjustment factor (use Exhibit 16-7).	No bus stopping, f_{bb} = 1.000
10. Area type adjustment factor (use Exhibit 16-7).	For CBD, f_a = 0.900
11. Lane utilization adjustment factor (use Exhibit 10-23).	No specific data are given. Use default of f_{LU} = 1.000 for exclusive LT. Use 0.950 for shared LT.
12. Left-turn adjustment factor.	The left turn is permitted. A special procedure is used. All approaches are opposed by multilane approaches. The supplemental worksheet for multilane approaches is used to determine the factor.
13. Right-turn adjustment factor.	For all shared-lane approaches: $f_{RT} = 1.0 - 0.150P_{RT}$ Where P_{RT} is the proportion of right turns in lane group, f_{RT} (EB) = 1.0 – 0.150(0.250) = 0.963
14. Left-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effects is used to determine the factor.
15. Right-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effects is used to determine the factor.
16. Saturation flow.	$s = s_o\ N\ f_w\ f_{HV}\ f_g\ f_p\ f_{bb}\ f_{LU}\ f_a\ f_{LT}\ f_{RT}\ f_{Lpb}\ f_{Rpb}$ s(EBTHRT) = 1900 * 2 * 0.933 * 0.952 * 1.000 * 0.938 * 1.000 * 0.900 * 0.950 * 1.000 * 0.963 * 1.000 * 0.958 = 2497 veh/h
17. Lane group capacity.	c = s(g/C) c(EBTHRT) = 2497(0.213) = 532 veh/h
18. v/c ratio.	v/c(EB) = $\frac{424}{532}$ = 0.797
19. Determine critical lane group in each timing phase.	Critical lane groups: Phase 1: SB protected left turn Phase 2: NB through + right Phase 3: WB left turn
20. Flow ratio of critical lane groups.	v/s(SBLT) = $\frac{143}{1592}$ = 0.090 v/s(NBTHRT) = $\frac{1733}{3155}$ = 0.549 v/s (WBLT) = $\frac{118}{480}$ = 0.246
21. Sum of critical lane group v/s ratios.	Y_c = 0.090 + 0.549 + 0.246 = 0.885

22. Critical flow rate to capacity ratio.	$X_c = \frac{Y_c * C}{C - L}$ $X_c = \frac{0.885(90.0)}{90.0 - 12} = 1.021$
23. Uniform delay.	$d_1 = \frac{0.50C\left(1 - \frac{g}{C}\right)^2}{1 - \left[\min(1, X)\frac{g}{C}\right]}$ $d_1(EBLT) = \frac{0.50(90.0)(1 - 0.213)^2}{1 - (0.213)(1.0)} = 35.415$ s/veh Since NB and SB left turns are contained in two phases, a supplemental uniform delay worksheet is used.
24. Incremental delay.	$d_2 = 900T\left[(X - 1) + \sqrt{(\ldots)}\right]$ $d_2(EBLT) = 900(0.25)\left[(1.109 - 1) + \sqrt{(\ldots)}\right] =$ 145.509 s/veh
25. Progression adjustment factor (use Exhibit 16-12).	PF = 1.000
26. Lane group delay.	$d = d_1PF + d_2 + d_3$ (d_3 is assumed to be 0 for the first iteration) d (EBLT) = 34.415(1.000) + 145.509 = 180.9 s/veh
27. Approach delay.	$d_A = \frac{\sum(d)(v)}{\sum v}$ $d_A(EB) = \frac{(180.9 * 71) + (41.6 * 424)}{(71 + 424)} = 61.6$ s/veh
28. Intersection delay.	$d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ $d_I = \frac{(495 * 61.6) + (742 * 113.0) + (1866 * 33.0) + (1205 * 20.6)}{(495 + 742 + 1866 + 1205)} =$ 46.6 s/veh
29. LOS by lane group, approach, and intersection.	LOS (EBLT) = F LOS (EB) = E LOS Intersection = D

The calculation results are summarized as follows.

Direction/ LnGrp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Factor PF	Lane Grp Cap	Cal Term k	Incr Delay d_2	Lane Grp Delay	Lane Grp LOS	Delay by App	LOS by App
EB/L	1.109	0.213	35.415	1.000	64	0.500	145.509	180.9	F		E
EB/TR	0.797	0.213	33.571	1.000	532	0.329	8.034	41.6	D	61.6	
WB/L	1.157	0.213	33.415	1.000	102	0.500	137.481	172.9	F		
WB/TR	1.095	0.213	35.415	1.000	570	0.500	66.241	101.7	F	113.0	F
NB/L	0.383	0.698	6.582	1.000	347	0.080	0.514	7.1	A		
NB/TR	0.976	0.563	19.075	1.000	1776	0.480	15.966	35.0	C	33.0	C
SB/L	0.894	0.698	25.957	1.000	217	0.411	33.699	59.7	E		
SB/TR	0.572	0.563	12.676	1.000	1768	0.140	0.380	13.1	B	20.6	C
Intersection Delay = 46.6 s/veh							Intersection LOS = D				

Alternatives

As shown in the results, the v/c ratios for critical groups are not balanced. As a result, certain lane groups experience high delay, whereas others experience little delay. Reallocation of green times is needed.

Volume to capacity ratios for EB and WB lane groups are greater than those for NB and SB lane groups. The result is higher delay for EB and WB. The performance of EB and WB lane groups could be improved by assigning more green time, so 4.0 s is reallocated to the east-west phase from the north-south through phase. The resulting phase times are as follows:

- Phase 1 (NB/SB LT): 8.1 s,
- Phase 4 (NB/SB TH+RT): 46.7 s, and
- Phase 5 (EB/WB TH+RT): 23.2 s.

The intersection performance is reassessed, and the results are as follows.

Direction/ LnGrp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Factor PF	Lane Grp Cap	Cal Term k	Incr Delay d_2	Lane Grp Delay	Lane Grp LOS	Delay by App	LOS by App
EB/L	0.855	0.258	31.787	1.000	83	0.500	64.372	96.2	F		
EB/TR	0.653	0.258	29.795	1.000	649	0.329	3.362	33.2	C	42.2	D
WB/L	0.843	0.258	31.662	1.000	140	0.500	42.939	74.6	E		
WB/TR	0.903	0.258	32.301	1.000	691	0.500	17.352	49.7	D	53.7	D
NB/L	0.422	0.653	8.447	1.000	315	0.080	0.666	9.1	A		
NB/TR	1.059	0.519	21.645	1.000	1637	0.480	39.338	61.0	E	57.3	E
SB/L	0.894	0.653	25.886	1.000	217	0.411	33.699	59.6	E		
SB/TR	0.620	0.519	15.351	1.000	1630	0.140	0.504	15.9	B	22.9	C
Intersection Delay = 45.3 s/veh							Intersection LOS = D				

The intersection performance has improved, with delay reduced from 46.6 s/veh to 45.3 s/veh.

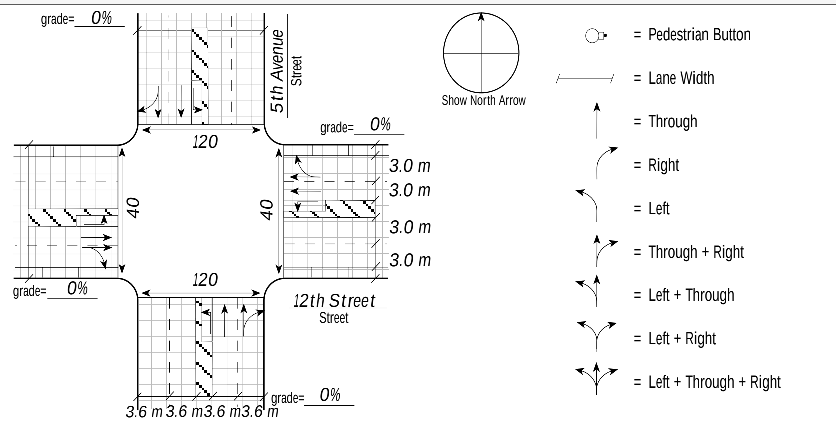
Volume to capacity ratios for critical lane groups are high. Although the intersection performance could still be improved by reallocating green times, delay reduction will be minimal because the critical elements are close to capacity. Consideration should be given to physical improvements to further optimize intersection operation.

Example Problem 3

INPUT WORKSHEET

General Information		Site Information	
Analyst	RPR	Intersection	5th Avenue/12th Streeet
Agency or Company	PTU	Area Type	☒ CBD ❑ Other
Date Performed	4/12/99	Jurisdiction	
Analysis Time Period	5-6 PM	Analysis Year	1999

Intersection Geometry



Volume and Timing Input

	EB LT	EB TH	EB RT[1]	WB LT	WB TH	WB RT[1]	NB LT	NB TH	NB RT[1]	SB LT	SB TH	SB RT[1]
Volume, V (veh/h)	60	270	90	100	510	20	120	1480	80	175	840	70
% heavy vehicles, % HV		5			5			2			2	
Peak-hour factor, PHF		0.85			0.85			0.90			0.90	
Pretimed (P) or actuated (A)		A			A			A			A	
Start-up lost time, l_1 (s)												
Extension of effective green time, e (s)												
Arrival type, AT		3			3			3			3	
Approach pedestrian volume,[2] v_{ped} (p/h)		120			120			40			40	
Approach bicycle volume,[2] v_{bic} (bicycles/h)		0			0			0			0	
Parking (Y or N)		Y			Y			N			N	
Parking maneuvers, N_m (maneuvers/h)		5			5			0			0	
Bus stopping, N_B (buses/h)		0			0			0			0	
Min. timing for pedestrians,[3] G_p (s)		19.0			19.0			21.0			21.0	

Signal Phasing Plan

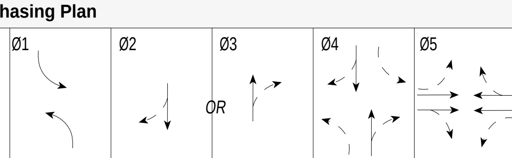


DIAGRAM	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
Timing	G = 8.1 Y = 4.0	G = 0.0 Y = 0.0	G = 0.0 Y = 0.0	G = 50.7 Y = 4.0	G = 19.2 Y = 4.0	G = Y =	G = Y =	G = Y =

Protected turns — Permitted turns — Pedestrian — Cycle length, C = 90.0 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 16-2.

Example Problem 3

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET

General Information

Project Description *Example Problem 3*

Volume Adjustment

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	60	270	90	100	510	20	120	1480	80	175	840	70
Peak-hour factor, PHF		0.85			0.85			0.90			0.90	
Adjusted flow rate, v_p = V/PHF (veh/h)	71	318	106	118	600	24	133	1644	89	194	933	78
Lane group												
Adjusted flow rate in lane group, v (veh/h)	71	424		118	624		133		1733	194		1011
Proportion[1] of LT or RT (P_{LT} or P_{RT})	1.000	-	0.250	1.000	-	0.038	1.000	-	0.051	1.000	-	0.077

Saturation Flow Rate (see Exhibit 16-7 to determine adjustment factors)

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Base saturation flow, s_o (pc/h/ln)	1900	1900		1900	1900		1900	1900	1900	1900	1900	1900
Number of lanes, N	1	2		1	2		1	1	2	1	1	2
Lane width adjustment factor, f_w	0.933	0.933		0.933	0.933		1.000	1.000	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment factor, f_{HV}	0.952	0.952		0.952	0.952		0.980	0.980	0.980	0.980	0.980	0.980
Grade adjustment factor, f_g	1.000	1.000		1.000	1.000		1.000	1.000	1.000	1.000	1.000	1.000
Parking adjustment factor, f_p	1.000	0.938		1.000	0.938		1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment factor, f_{bb}	1.000	1.000		1.000	1.000		1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment factor, f_a	0.900	0.900		0.900	0.900		0.900	0.900	0.900	0.900	0.900	0.900
Lane utilization adjustment factor, f_{LU}	1.000	0.950		1.000	0.950		1.000	1.000	0.950	1.000	1.000	0.950
Left-turn adjustment factor, f_{LT}	0.208	1.000		0.343	1.000		0.950	0.200	1.000	0.950	0.073	1.000
Right-turn adjustment factor, f_{RT}	1.000	0.963		1.000	0.994		1.000	1.000	0.992	1.000	1.000	0.988
Left-turn ped/bike adjustment factor, f_{Lpb}	0.951	1.000		0.921	1.000		1.000	0.999	1.000	1.000	1.000	1.000
Right-turn ped/bike adjustment factor, f_{Rpb}	1.000	0.958		1.000	0.994		1.000	1.000	0.999	1.000	1.000	0.998
Adjusted saturation flow, s (veh/h) $s = s_o N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$	300	2497		480	2675		1592	335	3155	1592	122	3140

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.

Example Problem 3

SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS OPPOSED BY MULTILANE APPROACH

General Information

Project Description *Example Problem 3*

Input

	EB	WB	NB	SB
Cycle length, C (s)	90.0			
Total actual green time for LT lane group,[1] G (s)	19.2	19.2	62.8	62.8
Effective permitted green time for LT lane group,[1] g (s)	19.2	19.2	54.7	54.7
Opposing effective green time, g_o (s)	19.2	19.2	50.7	50.7
Number of lanes in LT lane group,[2] N	1	1	1	1
Number of lanes in opposing approach, N_o	2	2	2	2
Adjusted LT flow rate, v_{LT} (veh/h)	71	118	133	194
Proportion of LT volume in LT lane group,[3] P_{LT}	1.000	1.000	1.000	1.000
Adjusted flow rate for opposing approach, v_o (veh/h)	624	424	1011	1733
Lost time for LT lane group, t_L	4	4	0	0
Computation				
LT volume per cycle, LTC = $v_{LT}C/3600$	1.775	2.950	3.325	4.850
Opposing lane utilization factor, f_{LUo} (refer to Volume Adjustment and Saturation Flow Rate Worksheet)	0.950	0.950	0.950	0.950
Opposing flow per lane, per cycle $v_{olc} = \frac{v_o C}{3600 N_o f_{LUo}}$ (veh/C/ln)	8.211	5.579	13.303	22.803
$g_f = G[e^{-0.882(LTC^{0.717})}] - t_L$ $g_f \leq g$ (except for exclusive left-turn lanes)[1, 4]	0	0	0	0
Opposing platoon ratio, R_{po} (refer to Exhibit 16-11)	1.00	1.00	1.00	1.00
Opposing queue ratio, $qr_o = \max[1 - R_{po}(g_o/C), 0]$	0.787	0.787	0.437	0.437
$g_q = \frac{v_{olc} qr_o}{0.5 - [v_{olc}(1 - qr_o)/g_o]} - t_L$, $v_{olc}(1 - qr_o)/g_o \leq 0.49$ (note case-specific parameters)[1]	11.803	6.022	16.502	40.379
$g_u = g - g_q$ if $g_q \geq g_f$, or $g_u = g - g_f$ if $g_q < g_f$	7.397	13.178	38.198	14.321
E_{L1} (refer to Exhibit C16-3)	2.4	2.0	3.5	7.314
$P_L = P_{LT}\left[1 + \frac{(N-1)g}{(g_f + g_u/E_{L1} + 4.24)}\right]$ (except with multilane subject approach)[5]	1.000	1.000	1.000	1.000
$f_{min} = 2(1 + P_L)/g$	0.208	0.208	0.073	0.073
$f_m = [g_f/g] + [g_u/g]\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right]$, $(f_{min} \leq f_m \leq 1.00)$	0.208	0.343	0.200	0.073
$f_{LT} = [f_m + 0.91(N - 1)]/N$ (except for permitted left turns)[6]	0.208	0.343	0.200	0.073

Notes

1. Refer to Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 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 $P_L \geq 1$ 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 $f_{LT} = f_m$.

Example Problem 3

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

General Information

Project Description *Example Problem 3*

Permitted Left Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)	19.2	19.2	50.7	50.7
Conflicting pedestrian volume,[1] v_{ped} (p/h)	120	120	40	40
$v_{pedg} = v_{ped}$ (C/g_p)	563	563	71	71
$OCC_{pedg} = v_{pedg}/2000$ if $(v_{pedg} \le 1000)$ or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if $(1000 < v_{pedg} \le 5000)$	0.282	0.282	0.036	0.036
Opposing queue clearing green,[3,4] g_q (s)	11.803	6.022	16.502	40.379
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p; if $g_q \ge g_p$ then $f_{Lpb} = 1.0$	0.615	0.314	0.325	0.796
$OCC_{pedu} = OCC_{pedg}\,[1 - 0.5(g_q/g_p)]$	0.195	0.238	0.030	0.022
Opposing flow rate,[3] v_o (veh/h)	624	424	1011	1733
$OCC_r = OCC_{pedu}\,[e^{-(5/3600)v_o}]$	0.082	0.132	0.007	0.002
Number of cross-street receiving lanes,[1] N_{rec}	2	2	2	2
Number of turning lanes,[1] N_{turn}	1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$ $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$	0.951	0.921	0.996	0.999
Proportion of left turns,[5] P_{LT}	1.000	1.000	1.000	1.000
Proportion of left turns using protected phase,[6] P_{LTA}	0	0	0.842	0.976
$f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$	0.951	0.921	0.999	1.000

Permitted Right Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)	19.2	19.2	50.7	50.7
Conflicting pedestrian volume,[1] v_{ped} (p/h)	120	120	40	40
Conflicting bicycle volume,[1,7] v_{bic} (bicycles/h)	0	0	0	0
$v_{pedg} = v_{ped}(C/g_p)$	563	563	71	71
$OCC_{pedg} = v_{pedg}/2000$ if $(v_{pedg} \le 1000)$, or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if $(1000 < v_{pedg} \le 5000)$	0.282	0.282	0.036	0.036
Effective green,[1] g (s)	19.2	19.2	50.7	50.7
$v_{bicg} = v_{bic}(C/g)$	0	0	0	0
$OCC_{bicg} = 0.02 + v_{bicg}/2700$	0	0	0	0
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg})$	0.282	0.282	0.036	0.036
Number of cross-street receiving lanes,[1] N_{rec}	2	2	2	2
Number of turning lanes,[1] N_{turn}	1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$ $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$	0.831	0.831	0.978	0.978
Proportion of right turns,[5] P_{RT}	0.250	0.038	0.051	0.077
Proportion of right turns using protected phase,[8] P_{RTA}	0	0	0	0
$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$	0.958	0.994	0.999	0.998

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 – 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 $P_{RTA} = 0$.

Example Problem 3

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

General Information

Project Description *Example Problem 3*

v/c Ratio Computation

	EB	WB	NB	SB
Cycle length, C (s)	*90.0*			
Protected phase eff. green interval, g (s)			*8.1*	*8.1*
Opposing queue effective green interval, g_q (s)			*16.502*	*40.379*
Unopposed green interval, g_u (s)			*38.198*	*14.321*
Red time, r (s) $r = C - g - g_q - g_u$			*27.200*	*27.200*
Arrival rate, q_a (veh/s) $q_a = \frac{v}{3600 * \max[X, 1.0]}$			*0.037*	*0.054*
Protected phase departure rate, s_p (veh/s) $s_p = \frac{s}{3600}$			*0.442*	*0.442*
Permitted phase departure rate, s_s (veh/s) $s_s = \frac{s(g_q + g_u)}{(g_u * 3600)}$			*0.133*	*0.129*
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, $X_{perm} = \frac{q_a(r + g_q + g_u)}{s_s\, g_u}$			*0.398*	*1.599*
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			*0.365*	*0.532*

Uniform Queue Size and Delay Computations

	EB	WB	NB	SB
Queue at beginning of green arrow, Q_a			*1.006*	*2.575*
Queue at beginning of unsaturated green, Q_u			*0.611*	*2.180*
Residual queue, Q_r			*0*	*1.106*
Uniform delay, d_1			*6.582*	*25.957*

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	$[0.50/(q_a C)][(r + g_q)Q_u + g_u(Q_u + Q_a) + Q_a^2/(s_p - q_a)]$

Example Problem 3

CAPACITY AND LOS WORKSHEET

General information

Project Description *Example Problem 3*

Capacity Analysis

Phase number	1	1	4	4	4	4	5	5	5	5		
Phase type												
Lane group												
Adjusted flow rate, v (veh/h)	133	143	0	1733	51	1011	71	424	118	624		
Saturation flow rate, s (veh/h)	1592	1592	335	3155	122	3140	300	2497	480	2675		
Lost time, t_L (s), $t_L = I_1 + Y - e$	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0		
Effective green time, g (s), $g = G + Y - t_L$	8.1	8.1	54.7	50.7	54.7	50.7	19.2	19.2	19.2	19.2		
Green ratio, g/C	0.090	0.090	0.608	0.563	0.608	0.563	0.213	0.213	0.213	0.213		
Lane group capacity,[1] c = s(g/C), (veh/h)	143	143	204	1776	74	1768	64	532	102	570		
v/c ratio, X	0.930	1.000	0.000	0.976	0.689	0.572	1.109	0.797	1.157	1.095		
Flow ratio, v/s		0.090		0.549					0.246			
Critical lane group/phase (√)		√		√					√			
Sum of flow ratios for critical lane groups, Y_c $Y_c = \sum$ (critical lane groups, v/s)	0.885											
Total lost time per cycle, L (s)	12.0											
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$	1.021											

Lane Group Capacity, Control Delay, and LOS Determination

		EB			WB			NB			SB	
Lane group												
Adjusted flow rate,[2] v (veh/h)	71	424		118	624		133	1733		194	1011	
Lane group capacity,[2] c (veh/h)	64	532		102	570		347	1776		217	1768	
v/c ratio,[2] X = v/c	1.109	0.797		1.157	1.095		0.383	0.976		0.894	0.572	
Total green ratio,[2] g/C	0.213	0.213		0.213	0.213		0.698	0.563		0.698	0.563	
Uniform delay, $d_1 = \frac{0.50\, C\, [1 - (g/C)]^2}{1 - [\min(1, X)g/C]}$ (s/veh)	35.415	33.571		33.415	35.415		6.582	19.075		25.957	12.676	
Incremental delay calibration,[3] k	0.500	0.329		0.500	0.500		0.080	0.480		0.411	0.140	
Incremental delay,[4] d_2 $d_2 = 900T\left[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}\right]$ (s/veh)	145.509	8.034		137.481	66.241		0.514	15.966		33.699	0.380	
Initial queue delay, d_3 (s/veh) (Appendix F)	0	0		0	0		0	0		0	0	
Uniform delay, d_1 (s/veh) (Appendix F)												
Progression adjustment factor, PF	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	180.9	41.6		172.9	101.7		7.1	35.0		59.7	13.1	
LOS by lane group (Exhibit 16-2)	F	D		F	F		A	C		E	B	
Delay by approach, $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	61.6			113.0			33.0			20.6		
LOS by approach (Exhibit 16-2)	E			F			C			C		
Approach flow rate, v_A (veh/h)	495			742			1866			1205		
Intersection delay, $d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ (s/veh)	46.6			Intersection LOS (Exhibit 16-2)						D		

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 Exhibit 16-13.
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.

EXAMPLE PROBLEM 4

The Intersection Tenth Avenue (EB/WB) and First Street (NB/SB) are two-lane streets located in an area with high economic growth. In 20 years, the existing intersection of these two streets is projected to be inadequate as a result of major developments. A proposed geometric improvement and projected volumes are shown on the Input Worksheet.

The Question Is the proposed improvement adequate? If not, what additional improvements are needed?

The Facts
√ PHF = 0.90,
√ Cycle length = 90.0 s to 120.0 s, and
√ Movement lost time = 4 s.

Comments
√ High left-turn and opposing volumes, therefore protected treatment for left turns is used; and
√ Protected-plus-permitted treatment is not favorable because of safety concerns and the operation of adjacent intersections.

Steps

1. Lane volume and signal operations worksheets are used.
2. In this analysis, the main interest is to assess the intersection status. The results show the intersection status to be over capacity, with a critical v/c ratio of 1.023. The estimated cycle length is 120.0 s. This result could be interpreted as an uncertain indication that the demand might exceed the capacity, especially since the projections are for 20 years. Long-term projections are often based on coarse assumptions and approximations, and the end results are often not particularly accurate.
3. According to Chapter 10, per-lane volumes are suggested to be kept to 450 veh/h or less in intersection design. Currently, the eastbound approach violates that suggestion.
4. An exclusive right-turn lane is provided for the eastbound approach because of its high volume. The per-lane volumes are brought to below 450 veh/h. The planning method is again used to evaluate the intersection performance.
5. According to the analysis results, the eastbound right turn is now the critical movement, and the intersection v/c ratio has been reduced from 1.023 to 0.999.
6. The signal timing plan synthesized by the planning method for the westbound left turn violates the minimum green time requirement. The violation is overcome by eliminating the eastbound through and left-turn phase and reassigning the green time to the left-turn phase. The new signal timing plan is as follows.

Movement	Phase Time (s)
EB WB LT	12.2
EB WB HT	36.4
NB SB LT	17.7
NB HT LT	4.1
NB SB HT	29.6

7. The operational analysis is performed using default values, and the analysis results are summarized in an exhibit. The intersection operates at LOS D, and the v/c ratios are well balanced.

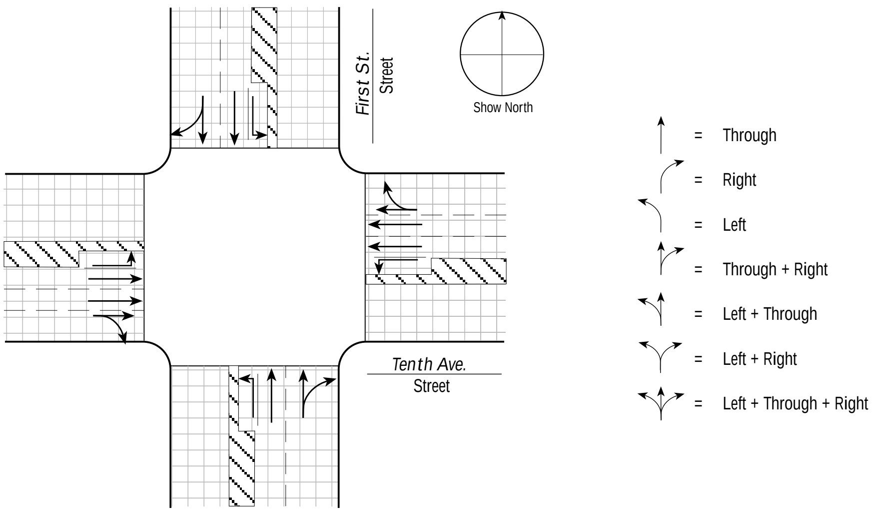
Results

The intersection performance is adequate assuming that the improvements are implemented.

Example Problem 4

QUICK ESTIMATION INPUT WORKSHEET			
General Information		**Site Information**	
Analyst	CJM	Intersection	Tenth Ave. and First St.
Agency or Company	CEI	Area Type	❑ CBD ☒ Other
Date Performed	5/13/99	Jurisdiction	Trenton, NJ
Analysis Time Period	4:45 - 5:45 pm	Analysis Year	1999

Intersection Geometry



Volume and Signal 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)	120	1300	460	80	1200	100	260	700	180	200	550	100
Proportion of LT or RT (P_{LT} or P_{RT})[2]		-			-			-			-	
Parking (Yes/No)		No			No			No			No	
Left-turn treatment (permitted, protected, not opposed) (if known)		Protected			Protected			Protected			Protected	

Peak-hour factor, PHF 0.90

Cycle length Minimum, C_{min} 90 s Maximum, C_{max} 120 s or Given, C ____ s

Lost time/phase 4 s

Notes

1. RT volumes, as shown, exclude RTOR.
2. 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.

Example Problem 4

QUICK ESTIMATION LANE VOLUME WORKSHEET (RUN 1)

	EB	WB	NB	SB
Right-Turn Movement				
Lane type	Shared	Shared	Shared	Shared
RT volume, RV	460	100	180	100
Number of exclusive RT lanes, N	1	1	1	1
RT adjustment factor, f_{RT}	0.85	0.85	0.85	0.85
RT volume per lane, VT	541	118	212	118
Left-Turn Movement				
Lane type	Exclusive	Exclusive	Exclusive	Exclusive
LT treatment	Prot	Prot	Prot	Prot
LT volume, VS	120	80	260	200
Opposing volume, V_o	1300	1760	650	880
Cross-product, V_L V_o	156000	140800	169000	176000
Number of exclusive LT lanes, NT	1	1	1	1
LT adjustment factor, f_{LT}	0.95	0.95	0.95	0.95
LT volume per lane, VT	126	84	274	211
Through Movement				
Through volume, V_T	1300	1200	700	550
Parking adjustment factor, f_p	1.000	1.000	1.000	1.000
Number of through lanes, HT	3	3	2	2
Total approach volume, Tot	1841	1318	912	668
Through volume per lane, HT	614	439	456	334
Critical lane volume, V_{CL}	614	439	456	334
Sneaker Left-Turn Check				
Permitted left sneaker capacity, c_{LS}	N/A	N/A	N/A	N/A

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET (RUN 1)

	Phase 1	Phase 2	Phase 3
East-West Phasing Plan			
Selected plan 3a			
Movement codes	EBWBLT	EBTHLT	EBWBTH
Critical phase volume, CV	84	42	572
Lost time/phase, t_L	4	0	4
North-South Phasing Plan			
Selected plan 3a			
Movement codes	NBSBLT	NBTHLT	NBSBTH
Critical phase volume, CV	211	63	393
Lost time/phase, t_L	4	0	4
Intersection Status Computation			
Critical sum, CS	1365		
Lost time/cycle, L	16		
Reference sum flow rate, RS	1539		
Cycle length, C	120.0		
Critical v/c ratio, X_{cm}	1.023		
Intersection status	Over capacity		
Green-Time Calculation			
East-west phasing	Phase 1	Phase 2	Phase 3
Green time, g	10.4	3.2	47.6
North-south phasing	Phase 1	Phase 2	Phase 3
Green time, g	20.1	4.8	33.9

Example Problem 4

QUICK ESTIMATION LANE VOLUME WORKSHEET (RUN 2)

	EB	WB	NB	SB
Right-Turn Movement				
Lane type	Exclusive	Shared	Shared	Shared
RT volume, RV	460	100	180	100
Number of exclusive RT lanes, N_{RT}	1	1	1	1
RT adjustment factor, f_{RT}	0.85	0.85	0.85	0.85
RT volume per lane, V_{RT}	541	118	212	118
Left-Turn Movement				
Lane type	Exclusive	Exclusive	Exclusive	Exclusive
LT treatment	Prot	Prot	Prot	Prot
LT volume, V_L	120	80	260	200
Opposing volume, V_o	1300	1760	650	880
Cross-product, $V_L V_o$	156000	140800	169000	176000
Number of exclusive LT lanes, N_{LT}	1	1	1	1
LT adjustment factor, f_{LT}	0.95	0.95	0.95	0.95
LT volume per lane, V_{LT}	126	84	274	211
Through Movement				
Through volume, V_T	1300	1200	700	550
Parking adjustment factor, f_p	1.000	1.000	1.000	1.000
Number of through lanes, N_{HT}	3	3	2	2
Total approach volume, V_{tot}	1300	1318	912	668
Through volume per lane, V_{HT}	433	439	456	334
Critical lane volume, V_{CL}	541	439	456	334
Sneaker Left-Turn Check				
Permitted left sneaker capacity, c_{LS}	N/A	N/A	N/A	N/A

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET (RUN 2)

East-West Phasing Plan			
	Phase 1	Phase 2	Phase 3
Selected plan 3a			
Movement codes	EBWBLT	EBTHLT	EBWBTH
Critical phase volume, CV	84	42	499
Lost time/phase, t_L	4	0	4
North-South Phasing Plan			
Selected plan 3a			
Movement codes	NBSBLT	NBTHLT	NBSBTH
Critical phase volume, CV	211	63	393
Lost time/phase, t_L	4	0	4
Intersection Status Computation			
Critical sum, CS	1292		
Lost time/cycle, L	16		
Reference sum flow rate, RS	1539		
Cycle length, C	100.0		
Critical v/c ratio, X_{cm}	0.999		
Intersection status	At capacity		
Green-Time Calculation			
East-west phasing	Phase 1	Phase 2	Phase 3
Green time, g	9.5	2.7	36.4
North-south phasing	Phase 1	Phase 2	Phase 3
Green time, g	17.7	4.1	29.6

Example Problem 4

LOS MODULE WORKSHEET (RUN 2)

Direction/ LnGrp	v/c Ratio	g/C Ratio	Unif Delay d_1	Progr Factor PF	Lane Grp Cap	Cal Term k	Incr Delay d_2	Lane Grp Delay	Lane Grp LOS	Delay by App	LOS by App
EB/L	0.917	0.082	45.562	1.000	145	0.500	55.293	100.9	F		
EB/T	0.876	0.324	31.904	1.000	1648	0.500	6.870	38.8	D		
EB/R	0.996	0.324	33.735	1.000	513	0.500	38.767	72.5	E	51.0	D
WB/L	0.614	0.082	44.370	1.000	145	0.500	17.901	62.3	E		
WB/TR	0.886	0.324	32.049	1.000	1629	0.500	7.493	39.5	D	40.8	D
NB/L	0.917	0.178	40.374	1.000	315	0.500	33.351	73.7	E		
NB/TR	0.960	0.297	34.566	1.000	1019	0.500	20.053	54.6	D	59.0	E
SB/L	0.917	0.137	42.589	1.000	242	0.500	39.789	82.4	F		
SB/TR	0.816	0.256	34.985	1.000	885	0.500	8.452	43.4	D	52.6	D
Intersection Delay = 50.3 s/veh							Intersection LOS = D				

EXAMPLE PROBLEM 5

The Intersection The intersection of Eighth Avenue (EB/WB) and Main Street (NB/SB) is located in a rapidly growing semirural community. Eighth Avenue is a four-lane roadway, and Main Street is a two-lane roadway. No turning lanes exist. The existing intersection geometry and projected volumes are shown on the Input Worksheet.

The Question Will projected demand exceed the existing intersection capacity? If so, what countermeasures should be implemented?

The Facts
√ PHF = 0.90,
√ Cycle length = 80 s to 120 s,
√ Movement lost time = 4 s, and
√ Non-CBD.

Steps

1. Run 1—Exclusive left turn for westbound approach: For the westbound approach, one lane is assigned as an exclusive left-turn lane, and the other is assigned as a shared lane. This assignment is appropriate for the volumes. The initial solution for signal phasing is to use protected-only left-turn treatment for the westbound approach and permitted treatment for all other approaches.

The quick estimation method is used to assess traffic operations. The results show that critical lane volumes are high, and the critical v/c ratio is computed as 1.077, indicating operations over capacity.

2. Run 2—Right-turn lane added on eastbound approach: The eastbound through and right-turn movement is identified as the critical lane group largely because of the high right-turn volume. An exclusive right-turn lane is added to the eastbound approach as a countermeasure. The traffic operation is reassessed, and the results show that the critical v/c ratio is reduced to 0.962. The intersection operates at capacity.

3. Run 3—Split phase operation for northbound and southbound approaches: Although the intersection status is satisfied, northbound and southbound left turns merit further consideration. Significant left-turn volumes in a pair of opposing single-lane approaches should be avoided. Split phase signal phasing is introduced to provide a complete directional separation between the northbound and southbound traffic.

The results show that the critical v/c ratio is increased to 1.138, which is over capacity. The split phasing is not appropriate. Although northbound and southbound per-lane volumes have been reduced, the critical sum is increased as the reduced volumes are added into the critical sum because northbound and southbound movements operate in different phases.

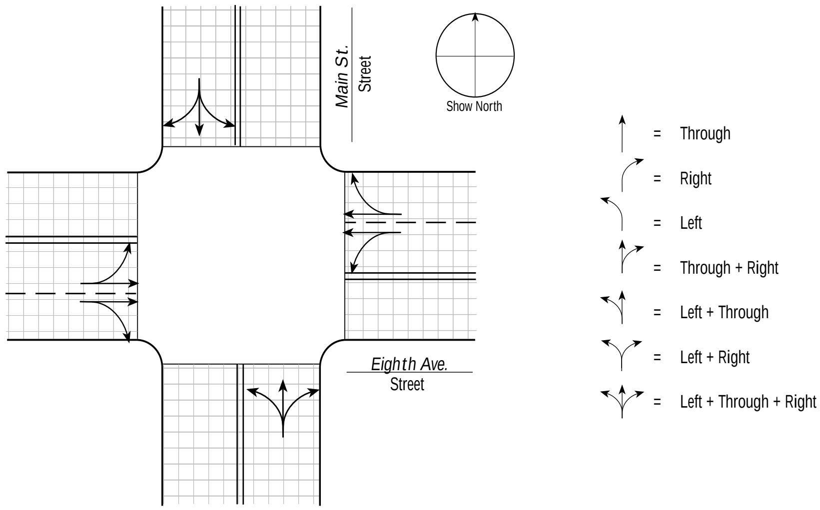
4. Run 4—Exclusive left-turn lane added on northbound and southbound approaches: In order to provide a protected phase for the northbound and southbound left turns while satisfying the operational requirement, an exclusive left-turn lane is added. As shown in the results, northbound and southbound per-lane volumes decrease substantially. The critical v/c ratio is computed as 0.933, which is near capacity.

Example Problem 5

QUICK ESTIMATION INPUT WORKSHEET

General Information		**Site Information**	
Analyst	*CJM*	Intersection	*Eighth Ave. and Main St.*
Agency or Company	*CEI*	Area Type	❑ CBD ☒ Other
Date Performed	*5/13/99*	Jurisdiction	*Fairfax, VA*
Analysis Time Period	*4:30 - 5:30 pm*	Analysis Year	*1999*

Intersection Geometry



Volume and Signal 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)	*120*	*690*	*280*	*170*	*360*	*110*	*80*	*150*	*60*	*120*	*230*	*170*
Proportion of LT or RT (P_{LT} or P_{RT})[2]		-			-			-			-	
Parking (Yes/No)		*No*			*No*			*No*			*No*	
Left-turn treatment (permitted, protected, not opposed) (if known)												

Peak-hour factor, PHF *0.90*

Cycle length Minimum, C_{min} *80* s Maximum, C_{max} *120* s or Given, C ______ s

Lost time/phase *4* s

Notes

1. RT volumes, as shown, exclude RTOR.
2. 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.

Example Problem 5

QUICK ESTIMATION LANE VOLUME WORKSHEET (RUN 1)

	EB	WB	NB	SB
Right-Turn Movement				
Lane type	Shared	Shared	Shared	Shared
RT volume, RV	280	110	60	170
Number of exclusive RT lanes, N_{RT}	1	1	1	1
RT adjustment factor, f_{RT}	0.85	0.85	0.85	0.85
RT volume per lane, VT	329	129	71	200
Left-Turn Movement				
Lane type	Shared	Exclusive	Shared	Shared
LT treatment	Perm	Prot	Perm	Perm
LT volume, VS	120	170	80	120
Opposing volume, V_o	470	1090	400	210
Cross-product, V_L V_o	56400	185300	32000	25200
Number of exclusive LT lanes, NT	0	1	0	0
LT adjustment factor, f_{LT}	0.95	0.95	0.95	0.95
LT volume per lane, VT	0	179	0	0
Through Movement				
Through volume, V_T	690	360	150	230
Parking adjustment factor, f_p	1.000	1.000	1.000	1.000
Number of through lanes, N_{HT}	2	1	1	1
Total approach volume, V_{tot}	1019	489	221	430
Through volume per lane, V_{HT}	721	489	287	592
Critical lane volume, V_{CL}	721	489	287	592
Sneaker Left-Turn Check				
Permitted left sneaker capacity, c_{LS}	N/A	N/A	N/A	N/A

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET (RUN 1)

East-West Phasing Plan			
	Phase 1	Phase 2	Phase 3
Selected plan 2a			
Movement codes	WBTHLT	EBWBTH	
Critical phase volume, CV	179	721	
Lost time/phase, t_L	4	4	
North-South Phasing Plan			
Selected plan 1			
Movement codes	NBSBLT		
Critical phase volume, CV	592		
Lost time/phase, t_L	4		
Intersection Status Computation			
Critical sum, CS	1492		
Lost time/cycle, L	12		
Reference sum flow rate, RS	1539		
Cycle length, C	120.0		
Critical v/c ratio, X_{cm}	1.077		
Intersection status	Over capacity		
Green-Time Calculation			
East-west phasing	Phase 1	Phase 2	Phase 3
Green time, g	17.0	56.2	
North-south phasing	Phase 1	Phase 2	Phase 3
Green time, g	46.9		

Example Problem 5

QUICK ESTIMATION LANE VOLUME WORKSHEET (RUN 2)

	EB	WB	NB	SB
Right-Turn Movement				
Lane type	Exclusive	Shared	Shared	Shared
RT volume, V_R	280	110	60	170
Number of exclusive RT lanes, N_{RT}	1	1	1	1
RT adjustment factor, f_{RT}	0.85	0.85	0.85	0.85
RT volume per lane, V_{RT}	329	129	71	200
Left-Turn Movement				
Lane type	Shared	Exclusive	Shared	Shared
LT treatment	Perm	Prot	Perm	Perm
LT volume, V_L	120	170	80	120
Opposing volume, V_o	470	1090	400	210
Cross-product, $V_L V_o$	56400	185300	32000	25200
Number of exclusive LT lanes, N_{LT}	0	1	0	0
LT adjustment factor, f_{LT}	0.95	0.95	0.95	0.95
LT volume per lane, V_{LT}	0	179	0	0
Through Movement				
Through volume, V_T	690	360	150	230
Parking adjustment factor, f_p	1.000	1.000	1.000	1.000
Number of through lanes, N_{HT}	2	1	1	1
Total approach volume, V_{tot}	690	489	221	430
Through volume per lane, V_{HT}	488	489	287	592
Critical lane volume, V_{CL}	488	489	287	592
Sneaker Left-Turn Check				
Permitted left sneaker capacity, c_{LS}	N/A	N/A	N/A	N/A

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET (RUN 2)

East-West Phasing Plan			
	Phase 1	Phase 2	Phase 3
Selected plan 2a			
Movement codes	WBTHLT	EBWBTH	
Critical phase volume, CV	179	488	
Lost time/phase, t_L	4	4	
North-South Phasing Plan			
Selected plan 1			
Movement codes	NBSBLT		
Critical phase volume, CV	592		
Lost time/phase, t_L	4		
Intersection Status Computation			
Critical sum, CS	1259		
Lost time/cycle, L	12		
Reference sum flow rate, RS	1539		
Cycle length, C	80.0		
Critical v/c ratio, X_{cm}	0.962		
Intersection status	At capacity		
Green-Time Calculation			
East-west phasing	Phase 1	Phase 2	Phase 3
Green time, g	13.7	30.4	
North-south phasing	Phase 1	Phase 2	Phase 3
Green time, g	36.0		

Example Problem 5

QUICK ESTIMATION LANE VOLUME WORKSHEET (RUN 3)

	EB	WB	NB	SB
Right-Turn Movement				
Lane type	Exclusive	Shared	Shared	Shared
RT volume, V_R	280	110	60	170
Number of exclusive RT lanes, N_{RT}	1	1	1	1
RT adjustment factor, f_{RT}	0.85	0.85	0.85	0.85
RT volume per lane, V_{RT}	329	129	71	200
Left-Turn Movement				
Lane type	Shared	Exclusive	Shared	Shared
LT treatment	Perm	Prot	Nopp	Nopp
LT volume, V_L	120	170	80	120
Opposing volume, V_o	470	1090	0	0
Cross-product, $V_L V_o$	56400	185300	0	0
Number of exclusive LT lanes, N_{LT}	0	1	0	0
LT adjustment factor, f_{LT}	0.95	0.95	0.95	0.95
LT volume per lane, V_{LT}	0	179	80	120
Through Movement				
Through volume, V_T	690	360	150	230
Parking adjustment factor, f_p	1.000	1.000	1.000	1.000
Number of through lanes, N_{HT}	2	1	1	1
Total approach volume, V_{tot}	690	489	301	550
Through volume per lane, V_{HT}	488	489	301	550
Critical lane volume, V_{CL}	488	489	301	550
Sneaker Left-Turn Check				
Permitted left sneaker capacity, c_{LS}	N/A	N/A	N/A	N/A

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET (RUN 3)

East-West Phasing Plan			
	Phase 1	Phase 2	Phase 3
Selected plan 2a			
Movement codes	WBTHLT	EBWBTH	
Critical phase volume, CV	179	488	
Lost time/phase, t_L	4	4	
North-South Phasing Plan			
Selected plan 4			
Movement codes	NBTHLT	SBTHLT	
Critical phase volume, CV	301	550	
Lost time/phase, t_L	4	4	
Intersection Status Computation			
Critical sum, CS	1518		
Lost time/cycle, L	16		
Reference sum flow rate, RS	1539		
Cycle length, C	120.0		
Critical v/c ratio, X_{cm}	1.138		
Intersection status	Over capacity		
Green-Time Calculation			
East-west phasing	Phase 1	Phase 2	Phase 3
Green time, g	16.3	37.4	
North-south phasing	Phase 1	Phase 2	Phase 3
Green time, g	24.6	41.7	

Example Problem 5

QUICK ESTIMATION LANE VOLUME WORKSHEET (RUN 4)

	EB	WB	NB	SB
Right-Turn Movement				
Lane type	Exclusive	Shared	Shared	Shared
RT volume, V_R	280	110	60	170
Number of exclusive RT lanes, N_{RT}	1	1	1	1
RT adjustment factor, f_{RT}	0.85	0.85	0.85	0.85
RT volume per lane, V_{RT}	329	129	71	200
Left-Turn Movement				
Lane type	Shared	Exclusive	Exclusive	Exclusive
LT treatment	Perm	Prot	Prot	Prot
LT volume, V_L	120	170	80	120
Opposing volume, V_o	470	1090	400	210
Cross-product, V_LV_o	56400	185300	32000	25200
Number of exclusive LT lanes, N_{LT}	0	1	1	1
LT adjustment factor, f_{LT}	0.95	0.95	0.95	0.95
LT volume per lane, V_{LT}	0	179	84	126
Through Movement				
Through volume, V_T	690	360	150	230
Parking adjustment factor, f_p	1.000	1.000	1.000	1.000
Number of through lanes, N_{HT}	2	1	1	1
Total approach volume, V_{tot}	690	489	221	430
Through volume per lane, V_{HT}	488	489	221	430
Critical lane volume, V_{CL}	488	489	221	430
Sneaker Left-Turn Check				
Permitted left sneaker capacity, c_{LS}	N/A	N/A	N/A	N/A

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET (RUN 4)

East-West Phasing Plan			
	Phase 1	Phase 2	Phase 3
Selected plan 2a			
Movement codes	WBTHLT	EBWBTH	
Critical phase volume, CV	179	488	
Lost time/phase, t_L	4	4	
North-South Phasing Plan			
Selected plan 3b			
Movement codes	NBSBLT	SBTHLT	NBSBTH
Critical phase volume, CV	84	42	388
Lost time/phase, t_L	4	0	4
Intersection Status Computation			
Critical sum, CS	1181		
Lost time/cycle, L	16		
Reference sum flow rate, RS	1539		
Cycle length, C	90.0		
Critical v/c ratio, X_{cm}	0.933		
Intersection status	Near capacity		
Green-Time Calculation			
East-west phasing	Phase 1	Phase 2	Phase 3
Green time, g	15.2	34.6	
North-south phasing	Phase 1	Phase 2	Phase 3
Green time, g	9.3	2.6	28.3

EXAMPLE PROBLEM 6

The Intersection A two-lane through movement at one approach to a signalized intersection has a cycle length of 90 s with a g/C ratio of 0.50. The arrival type is currently 3 (random), but this could be improved by altering the progression.

The Question What is the maximum service flow rate that could be accommodated at LOS B (20 s/veh delay) on this approach?

The Facts

√ Cycle length = 90 s,
√ g/C = 0.50, and
√ s = 3,200 veh/h.

Comments This calculation is intended to illustrate the potential for alternative computational sequences using the basic operational analysis format. Only one lane group is addressed. The computations become far more complex when multiple lane groups are addressed simultaneously. Nevertheless, the procedure is capable of determining service flow rates or geometric or signal parameters based on a desired LOS.

Steps

1. Delay is a function of the v/c ratio, X; the green ratio, g/C; the cycle length, C; the lane group capacity, c; and the progression factor, PF. The lane group capacity is the product of a saturation flow rate, s, and a g/C ratio.

$$c = s * g/C = 3200 * 0.50 = 1600 \text{ veh/h}$$

2. At the LOS B threshold of 20.0 s/veh, the delay equation is expressed as follows:

$$20.0 = d_1PF + d_2 + d_3$$

where

$$d_1 = \frac{0.5(90)(1-0.50)^2}{(1-0.50X)}$$

$$d_2 = 225\left[(X-1)+\sqrt{(X-1)^2+\left(\frac{X}{100}\right)}\right]$$

$$d_3 = 0$$

3. Two tables are generated based on the equations above. The first table provides delay as a function of arrival types and v/c ratios, X. The second table provides v/c ratios and service flow rates as a function of delay and arrival types. In this problem, the second table is more appropriate because service flows are the direct output.

4. Service flow rates, SF, are computed as X * c, where c = 1,600 veh/h. Thus, at LOS B, the approach can carry a maximum service flow rate of 1,126 veh/h at existing Arrival Type 3. The maximum flow rate increases to 1,491 veh/h at Arrival Type 5 and to 1,571 at Arrival Type 6.

Example Problem 6

SERVICE FLOW RATE SOLUTIONS FOR EXAMPLE PROBLEM 6

X	Flow Rate	d_1	d_2	Delay AT 1	Delay AT 2	Delay AT 3	Delay AT 4	Delay AT 5	Delay AT 6
0.0	0	11.250	0.000	18.8	14.0	11.3	8.6	3.7	0.0
0.1	160	11.842	0.125	19.9	14.8	12.0	9.2	4.1	0.1
0.2	320	12.500	0.281	21.1	15.8	12.8	9.9	4.4	0.3
0.3	480	13.235	0.481	22.5	16.9	13.7	10.6	4.9	0.5
0.4	640	14.063	0.748	24.2	18.2	14.8	11.5	5.4	0.7
0.5	800	15.000	1.119	26.1	19.7	16.1	12.6	6.1	1.1
0.6	960	16.071	1.672	28.5	21.6	17.7	14.0	7.0	1.7
0.7	1120	17.308	2.576	31.4	24.0	19.9	15.9	8.3	2.6
0.8	1280	18.750	4.295	35.6	27.5	23.0	18.7	10.5	4.3
0.9	1440	20.455	8.514	42.6	33.9	29.0	24.2	15.3	8.5
1.0	1600	22.500	22.500	60.0	50.4	45.0	39.8	30.0	22.5

LOS	Max Delay	AT 1 SF_{max}	AT 1 X	AT 2 SF_{max}	AT 2 X	AT 3 SF_{max}	AT 3 X	AT 4 SF_{max}	AT 4 X	AT 5 SF_{max}	AT 5 X	AT 6 SF_{max}	AT 6 X
A	5									513	0.32	1307	0.82
	10							348	0.22	1241	0.78	1457	0.91
	15			191	0.12	663	0.41	1046	0.65	1429	0.89	1514	0.95
B	20	177	0.11	824	0.51	1126	0.70	1318	0.82	1491	0.93	1571	0.98
	25	707	0.44	1164	0.73	1333	0.83	1448	0.91	1546	0.97		
	30	1043	0.65	1342	0.84	1450	0.91	1500	0.94				
C	35	1259	0.79	1451	0.91	1500	0.94	1551	0.97				
	40	1381	0.86	1499	0.94	1550	0.97						
	45	1462	0.91	1548	0.97	1600	1.00						
	50	1508	0.94	1596	1.00								
D	55	1554	0.97										
	60	1600	1.00										
	65												
	70												
	75												
E	80												

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APPENDIX A. FIELD MEASUREMENT OF INTERSECTION CONTROL DELAY

GENERAL NOTES

Equipment and personnel requirements

As an alternative to the estimation of control delay per vehicle using Equation 16-9 and the progression adjustment factor, delay at existing locations may be measured directly. There are a number of techniques for making this measurement, including the use of test-car observations, path tracing of individual vehicles, and the recording of arrival and departure volumes on a cycle-by-cycle basis. The method summarized here is based on direct observation of vehicle-in-queue counts at the intersection and normally requires two field personnel per lane group surveyed, unless the volume is light. Also needed is a multifunction digital watch that includes a countdown-repeat timer, with the countdown interval in seconds, plus a volume-count board with at least two tally counters. As an alternative, a laptop computer can be programmed to emit audio count markers at user-selected intervals, take volume counts, and execute real-time delay computations, thus simplifying data reduction.

In general, this method is applicable to all undersaturated signalized intersections. For oversaturated conditions, queue buildup normally makes the method impractical. Under such conditions, more personnel will be required to complete the field study, and other methods may be considered, such as an input-output technique or a zoned-survey technique.

In the input-output technique, different observers count arrivals separately from departures and vehicles in queue are calculated as the accumulated difference, subject to in-process checks for vehicles leaving the queue before they reach the stop line. The zoned-survey technique requires subdividing the approach into manageable segments to which the observers are assigned; they then count queued vehicles in their assigned zone. Both of these techniques require more personnel and are more complicated in setup and execution.

The method described here is applicable to situations in which the average maximum queue per cycle is no more than about 20 to 25 veh/ln. When queues are long or the demand to capacity ratio is near 1.0, care must be taken to continue the vehicle-in-queue count past the end of the arrival count period, as detailed below. This requirement is for consistency with the analytic delay equation used in the chapter text.

Delay during deceleration is not directly measured

The method does not directly measure delay during deceleration and during a portion of acceleration, which are very difficult to measure without sophisticated tracking equipment. However, this method has been shown to yield a reasonable estimate of control delay. The method includes an adjustment for errors that may occur when this type of sampling technique is used, as well as an acceleration-deceleration delay correction factor. The acceleration-deceleration factor is a function of the typical number of vehicles in queue during each cycle and the normal free-flow speed when vehicles are unimpeded by the signal.

Exhibit A16-1 is a worksheet that can be used for recording observations and computation of average time-in-queue delay. Before beginning the detailed survey, the observers need to make an estimate of the average free-flow speed during the study period. Free-flow speed is the speed at which vehicles would pass unimpeded through the intersection if the signal were green for an extended period. This speed may be obtained by driving through the intersection a few times when the signal is green and there is no queue and recording the speed at a location least affected by signal control. Typically, the recording location should be upstream about midblock.

EXHIBIT A16-1. INTERSECTION CONTROL DELAY WORKSHEET

INTERSECTION CONTROL DELAY WORKSHEET

General Information		**Site Information**		
Analyst	______________	Intersection	______________	
Agency or Company	______________	Area Type	❑ CBD	❑ Other
Date Performed	______________	Jurisdiction	______________	
Analysis Time Period	______________	Analysis Year	______________	

Input Initial Parameters

Number of lanes, N	______________	Total vehicles arriving, V_{tot}	______________
Free-flow speed, FFS (km/h)	______________	Stopped-vehicle count, V_{stop}	______________
Survey count interval, I_s (s)	______________	Cycle length, C (s)	______________

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue Count Interval 1	2	3	4	5	6	7	8	9	10
Total											

Computations

Total vehicles in queue, ΣV_{iq} =	______________	Number of cycles surveyed, N_c =	______________
Time-in-queue per vehicle, $d_{vq} = (I_s * \frac{\Sigma V_{iq}}{V_{tot}}) * 0.9$	______________ s	Fraction of vehicles stopping, FVS = $\frac{V_{stop}}{V_{tot}}$	______________
No. of vehicles stopping per lane each cycle = $\frac{V_{stop}}{(N_c \times N)}$	______________	Accel/Decel correction delay, d_{ad} = FVS * CF	______________ s
Accel/Decel correction factor, CF (Ex. A16-2)	______________	Control delay/vehicle, $d = d_{vq} + d_{ad}$	______________ s

MEASUREMENT TECHNIQUE

Field procedure

The survey should begin at the start of the red phase of the lane group, ideally when there is no overflow queue from the previous green phase. There is a need for consistency with the analytic delay equation, which is based on delay to vehicles that arrive during the study period, not before. If the survey does start with an overflow queue, the overflow vehicles need to be excluded from subsequent queue counts.

Observer 1 performs the following tasks during the field study.

1. Keeps track of the end of standing queues for each cycle in the survey period by observing the last vehicle in each lane that stops because of the signal. This count includes vehicles arriving when the signal is actually green but stopped because vehicles in front have not yet started moving. For purposes of the survey, a vehicle is considered as having joined the queue when it approaches within one car length of a stopped vehicle and is itself about to stop. This definition is used because of the difficulty of keeping precise track of the moment when a vehicle comes to a stop. All vehicles that join a queue are then included in the vehicle-in-queue counts until they cross the stop line.

2. At regular intervals of between 10 and 20 s, records the number of vehicles in queue (e.g., using the countdown-repeat timer on a digital watch to signal the count time). The regular intervals should not be an integral divisor of the cycle length (e.g., if the cycle length is 120 s, 14-s or 16-s count intervals should be used, not 15-s intervals). Vehicles in queue are those that are included in the queue of stopping vehicles as defined in Step 1 and have not yet exited the intersection. For through vehicles, exiting the intersection can be considered to occur when the rear axle of a vehicle crosses the stop line. For turning vehicles, exiting the intersection occurs the instant a vehicle clears opposing through traffic or pedestrians to which it must yield and begins accelerating back to free-flow speed. Note that the vehicle-in-queue count often includes some vehicles that have regained speed but have not yet exited the intersection.

3. Enters the vehicle-in-queue counts in the appropriate box on the worksheet. Cycles of the survey period are listed in the second column of the sheet, after the column to record clock time every five cycles, and interval count identifiers are listed as column headings. For ease in conducting the study, the survey period is most conveniently defined as an integer number of cycles, though a precisely defined time length for the survey period (e.g., 15 min) can be used. The key point is that the end of the survey period must be clearly defined in advance since the last arriving vehicle or vehicles that stop in the period must be identified and counted until they exit the intersection, per the next step.

4. At the end of the survey period, continues taking vehicle-in-queue counts for all vehicles that arrived during the survey period until all of them have exited the intersection. This step requires mentally noting the last stopping vehicle that arrived during the survey period in each lane of the lane group and continuing the vehicle-in-queue counts until the last stopping vehicle or vehicles, plus all vehicles in front of the last stopping vehicles, exit the intersection. Stopping vehicles that arrive after the end of the survey period are not included in the final vehicle-in-queue counts.

Observer 2 performs the following study task.

1. During the entire survey period, maintains separate volume counts of total vehicles arriving during the survey period and total vehicles arriving during the survey period that stop one or more times. A vehicle stopping multiple times is counted only once as a stopping vehicle. Enters these volumes in the appropriate boxes on the worksheet.

Data reduction is accomplished with the following steps.

1. Sum each column of vehicle-in-queue counts, then sum the column totals for the entire survey period.

2. A vehicle recorded as part of a vehicle-in-queue count is in queue, on average, for the time interval between counts. The average time-in-queue per vehicle arriving during the survey period is estimated using Equation A16-1.

$$\textit{Time-in-queue per vehicle} = \left(I_s * \frac{\sum V_{iq}}{V_{tot}} \right) * 0.9 \qquad \text{(A16-1)}$$

where

I_s = interval between vehicle-in-queue counts (s),
$\sum V_{iq}$ = sum of vehicle-in-queue counts (veh),

V_{tot} = total number of vehicles arriving during the survey period (veh), and
0.9 = empirical adjustment factor.

The 0.9 adjustment factor accounts for the errors that may occur when this type of sampling technique is used to derive actual delay values, normally resulting in an overestimate of delay. Research has shown the correction required to be fairly consistent over a variety of conditions.

3. Compute the fraction of vehicles stopping and the average number of vehicles stopping per lane in each signal cycle, as indicated on the worksheet.

4. Using Exhibit A16-2, look up a correction factor appropriate to the lane group free-flow speed and the average number of vehicles stopping per lane in each cycle. This factor adds an adjustment for deceleration and acceleration delay, which cannot be measured directly with manual techniques.

EXHIBIT A16-2. ACCELERATION-DECELERATION DELAY CORRECTION FACTOR, CF (s)

Free-Flow Speed	≤ 7 Vehicles	8 - 19 Vehicles	20 - 30 Vehicles[a]
≤ 60 km/h	+ 5	+ 2	- 1
> 60–71 km/h	+ 7	+ 4	+ 2
> 71 km/h	+ 9	+ 7	+ 5

Note:
a. Vehicle-in-queue counts in excess of about 30 vehicles per lane are typically unreliable.

5. Multiply the correction factor by the fraction of vehicles stopping, then add this product to the time-in-queue value of Step 2 to obtain the final estimate of control delay per vehicle.

Exhibit A16-3 presents a sample computation for a study site over a 15-min period, operating with a 115-s cycle over almost eight cycles. The exhibit is annotated to clarify the procedure. The 15-s count interval is not an integral divisor of the cycle length, thus eliminating potential survey bias due to queue buildup in a regular, cyclic pattern.

Exhibit A16-4 shows how the field study would have been finished if a queue still remained at the end of the 15-min study period. Only the vehicles that arrived during the 15-min period would be counted.

If the study site is an actuated signal with varying cycle and phase lengths, the count interval may be chosen as the most convenient value for conducting the field survey on the basis of volume and vantage point considerations.

EXHIBIT A16-3. EXAMPLE APPLICATION

INTERSECTION CONTROL DELAY WORKSHEET

General Information		**Site Information**	
Analyst		Intersection	*Cicero & Belmont*
Agency or Company		Area Type	❑ CBD ☒ Other
Date Performed		Jurisdiction	
Analysis Time Period	*PM*	Analysis Year	*1999*

Input Initial Parameters

Number of lanes, N	*2*	Total vehicles arriving, V_{tot}	*530*
Free-flow speed, FFS (km/h)	*65*	Stopped vehicles count, V_{stop}	*223*
Survey count interval, I_s (s)	*15*	Cycle length, C (s)	

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue / Count Interval 1	2	3	4	5	6	7	8	9	10
4:34	*1*	*3*	*8*	*11*	*15*	*12*	*2*	*0*	*2*		
	2	*6*	*12*	*15*	*16*	*6*	*0*	*0*	*2*		
	3	*7*	*11*	*14*	*14*	*2*	*0*	*0*			
	4	*5*	*7*	*10*	*13*	*13*	*2*	*0*	*1*		
4:42	*5*	*4*	*6*	*10*	*12*	*3*	*0*	*0*	*1*		
	6	*5*	*7*	*9*	*13*	*4*	*0*	*0*			
	7	*3*	*6*	*8*	*12*	*12*	*0*	*0*	*0*		
4:47	*8*	*4*	*7*	*11*	*16*	*9*	*0*				
Total		*37*	*64*	*88*	*111*	*61*	*4*	*0*	*6*		

Computations

Total vehicles in queue, $\sum V_{iq}$ =	*371*	Number of cycles surveyed, N_c =	*7.8*
Time-in-queue per vehicle, $d_{vq} = (I_s * \frac{\sum V_{iq}}{V_{tot}}) * 0.9$	*9.5* s	Fraction of vehicles stopping, FVS = $\frac{V_{stop}}{V_{tot}}$	*0.42*
No. of vehicles stopping per lane each cycle = $\frac{V_{stop}}{(N_c \times N)}$	*14*	Accel/Decel correction delay, d_{ad} = FVS * CF	*1.7* s
Accel/Decel correction factor, CF (Ex. A16-2)	*4*	Control delay/vehicle, $d = d_{vq} + d_{ad}$	*11.2* s

EXHIBIT A16-4. EXAMPLE APPLICATION WITH RESIDUAL QUEUE AT END

INTERSECTION CONTROL DELAY WORKSHEET

General Information		**Site Information**	
Analyst		Intersection	*Cicero & Belmont*
Agency or Company		Area Type	❑ CBD ☒ Other
Date Performed		Jurisdiction	
Analysis Time Period	*PM*	Analysis Year	*1999*

Input Initial Parameters

Number of lanes, N	*2*	Total vehicles arriving, V_{tot}	
Free-flow speed, FFS (km/h)	*65*	Stopped-vehicle count, V_{stop}	
Survey count interval, I_s (s)	*15*	Cycle length, C (s)	

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue – Count Interval 1	2	3	4	5	6	7	8	9	10
4:47	*8*	*4*	*7*	*11*	*16*	*9*	*0*	*Queue count in previous example*			
4:47	*8*	*4*	*4**	*4**	*4**	*0*	*First four in queue have cleared by now*				
			say 15 min. survey period ends here								
			4 - last stopping vehicles in survey period; count only until they clear.*								
Total		*37*	*61*	*81*	*99*	*52*	*4*	*0*	*6*		

Computations

Total vehicles in queue, ΣV_{iq} =		Number of cycles surveyed, N_c =	
Time-in-queue per vehicle, $d_{vq} = (I_s * \frac{\Sigma V_{iq}}{V_{tot}}) * 0.9$	s	Fraction of vehicles stopping, $FVS = \frac{V_{stop}}{V_{tot}}$	
No. of vehicles stopping per lane each cycle = $\frac{V_{stop}}{(N_c \times N)}$		Accel/Decel correction delay, d_{ad} = FVS * CF	s
Accel/Decel correction factor, CF (Ex. A16-2)		Control delay/vehicle, $d = d_{vq} + d_{ad}$	s

APPENDIX B. SIGNAL TIMING DESIGN

The design for the operation of a traffic signal is a complex process involving three important decisions: type of signal controller to be used, phase plan to be adopted, and allocation of green time among the various phases.

Each of these three decisions is heavily influenced by state and local policies, guidelines, and standards. This appendix presents the alternatives available to the analyst along with a general discussion of the range in which they are employed. These discussions are intended only to assist the analyst in establishing an initial signal design for study and do not represent established standards or guidelines.

TYPE OF SIGNAL CONTROLLER

Three types of controllers defined:
- fully actuated
- semiactuated
- pretimed

Traffic engineering reference books describe three types of traffic signal controllers: pretimed, fully actuated, and semiactuated. Pretimed controllers have a preset sequence of phases displayed in repetitive order. Each phase has a fixed green time and change interval that are repeated in each cycle to produce a constant cycle length.

Fully actuated controllers operate with timing on all approaches being influenced by vehicle detectors. Each phase is subject to a minimum and maximum green time, and some phases may be skipped if no demand is detected. The cycle length for fully actuated control will vary from cycle to cycle.

Semiactuated controllers operate with some approaches (typically on the minor street) having detectors. The earliest form of semiactuated control was designed to maintain the green on the major street in the absence of a minor-street actuation. Once actuated, the minor-street green is displayed for a period just long enough to accommodate the traffic demand.

Although these equipment-based definitions have persisted in traffic engineering terminology, the evolution of traffic control technology has complicated their function from the analyst's perspective. For purposes of capacity and LOS analysis, it is no longer sufficient to consider the controller type as a global descriptor of the intersection operation. Instead, an expanded set of these definitions must be applied individually to each lane group.

Each lane group may be served by a phase that is either actuated or nonactuated. Nonactuated phases may be coordinated with neighboring signals on the same route, or they may function in an isolated mode without influence from other signals. Nonactuated phases generally operate with fixed minimum green times, which can be extended by reassigning unused green time from actuated phases with low demand.

Actuated phases, on the other hand, may be used at intersections at which other phases are coordinated, but they may not, for purposes of this chapter, be coordinated themselves. Actuated phases are subject to being shortened on cycles with low demand. On cycles with no demand, they may be skipped entirely, or they may be displayed for their minimum duration. With systems in which the nonactuated phases are coordinated, the actuated phases are also subject to early termination (force off) to accommodate the progression design for the system.

If all of the phases at an intersection are nonactuated, the length of each phase, and consequently the cycle length, will be fixed for purposes of analysis. This arrangement denotes the condition of pretimed operation. In current practice, one or more phases under this type of control will usually be coordinated. In general, if the intersection is sufficiently removed from its neighbors to operate in an isolated mode, actuated operation can produce lower delays and a better level of service. The methodology in this chapter will indicate the degree to which vehicle delay may be reduced by actuated control.

If all of the phases at an intersection are actuated, the length of each phase, and consequently the cycle length, will vary with each cycle. This arrangement denotes the condition of fully actuated operation. No coordination with neighboring signals is possible under this control mode. Fully actuated signals are generally used only at intersections where distances are such that coordination would not be expected to be beneficial or where, for administrative or cost reasons, it would not be expected to be implemented. The analysis procedures prescribed previously in this chapter will support an evaluation of the comparative benefits of coordinated operation versus actuated operation.

The semiactuated control mode includes all of the cases that do not fit into either the pretimed or fully actuated categories. The majority of coordinated signal systems must be treated as systems of semiactuated controllers with coordinated nonactuated phases serving the major-street approaches and isolated actuated phases serving the cross-street approaches. The cycle length is constant at coordinated semiactuated intersections and variable at isolated semiactuated intersections.

Analysis procedures in this chapter are based on fixed lengths of intervals and sequences of phases. Other types of control are approximated.

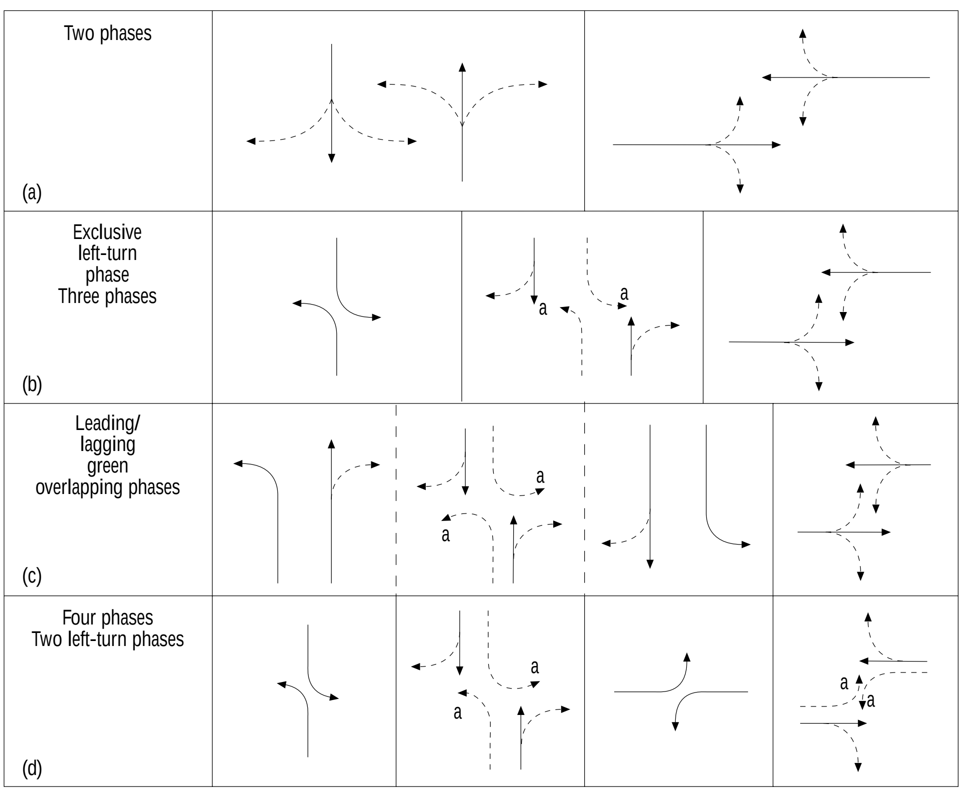
The analysis procedures in this chapter are based on the assumption of a fixed sequence of phases, each of which is displayed for a predictable time. In the case of pretimed control (i.e., no actuated phases), the length of each phase is assumed to be fixed and constant from cycle to cycle. Actuated phases must be approximated for analysis purposes by their average green time, recognizing that the actual time may differ from cycle to cycle. For a given timing plan (i.e., constant or average green times), the differences between actuated and nonactuated phases are recognized by the parameters used in the incremental term (d_2) of the delay equation.

PHASE PLANS

The most critical aspect of any design of signal timing is the selection of an appropriate phase plan. The phase plan comprises the number of phases to be used and the sequence in which they are implemented. As a general guideline, simple two-phase control should be used unless conditions dictate the need for additional phases. Because the change interval between phases contributes to lost time in the cycle, as the number of phases increases, the percentage of the cycle made up of lost time generally also increases.

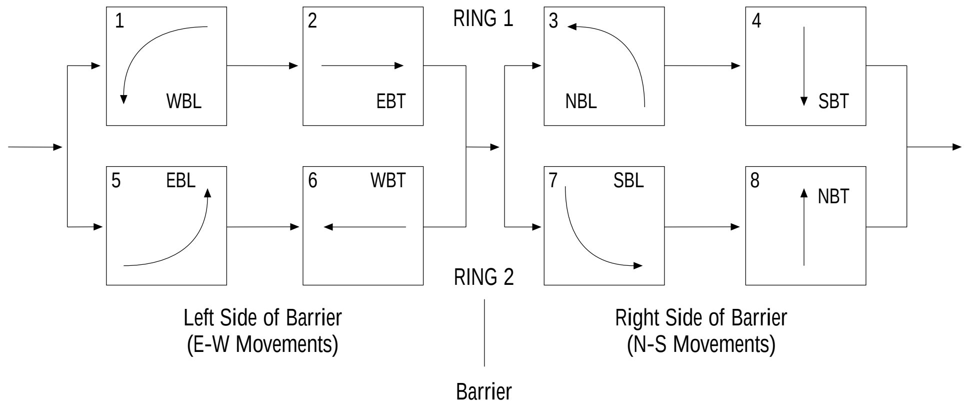
Exhibit B16-1 shows a number of common phase plans that may be used with either pretimed or actuated controllers. Exhibit B16-2 illustrates an optional phasing scheme that typically can be implemented only with actuated controllers.

EXHIBIT B16-1. PHASE PLANS FOR PRETIMED AND TRAFFIC-ACTUATED CONTROL



Note:
a. Optional movement.

EXHIBIT B16-2. DUAL-RING CONCURRENT PHASING SCHEME WITH ASSIGNED MOVEMENTS



Two-Phase Control

Two-phase control is the simplest of the available phase plans. Each of two intersecting streets is given a green phase during which all movements on the street are allowed to proceed. All left and right turns are made on a permitted basis against an opposing vehicle flow, pedestrian flow, or both. The two-phase plan is shown in Exhibit B16-1(a). This phase plan is generally used unless turn volumes require protected phasing.

Multiphase Control

Multiphase control is adopted at any intersection where one or more left or right turns require protected phasing. It is generally the left-turn movement that requires a partially or fully protected phase. Local policy and practice are critical determinants of this need. Most agencies have guidelines for when left turns require protected phasing. Protected left-turn phasing is also considered when the speed of opposing traffic is greater than 65 km/h.

Multiphase control can be provided in a variety of ways, depending on the number of turns requiring protected phasing and the sequence and overlaps used. Exhibit B16-1 presents three common plans for multiphase control. Exhibit B16-1(b) shows a three-phase plan in which an exclusive left-turn phase is provided for both left-turn movements on the major street. It is followed by a through phase for both directions of the major street, during which left turns in both directions may be permitted on an optional basis.

The use of a permitted left-turn phase following protected left-turn phases is very much a matter of local practice. The phasing illustrated in Exhibit B16-1(b) can be used either for protected or protected-permitted operation in either mode. Note that a few agencies use permitted-plus-protected phasing. Exclusive left-turn phases provide for simultaneous movement of opposing left turns and are most efficient when the opposing left-turn volumes are nearly equal. When volumes are unequal, or in cases in which only one left turn requires protected phasing, other phase plans are more efficient.

It is common practice to provide exclusive lanes for left or right turns with protected phases

The three-phase plan may be expanded to a four-phase sequence if both streets require left-turn phases. Such a sequence is shown in Exhibit B16-1(d). Left turns may be continued on a permitted basis concurrent with the through phases. It is common practice to provide exclusive lanes for left or right turns with protected phases.

Exhibit B16-1(c) shows what is commonly referred to as leading and lagging green phasing. The initial phase is a through plus left-turn phase for one direction of the major street, followed by a through phase for both directions of the major street, during which left turns in both directions may be permitted on an optional basis. Note that many operating agencies do not, as a matter of policy, use the optional permitted left turn with this type of phasing because of safety considerations. The direction of flow started in the first phase is then stopped, providing the opposing direction with a through-plus-left-turn phase. The final phase accommodates all movements on the minor street.

Such phasing is extremely flexible. When only one left turn requires a protected phase, a leading green can be provided without a lagging green phase. When left-turn volumes are unequal, the lengths of the leading and lagging green can be adjusted to avoid excessive green time for one or both left-turn movements. Leading or lagging green phase, or both, can even be used where no left-turn exists as long as turns are permitted to continue during the through phase. The phasing of Exhibit B16-1(c) may also be expanded to incorporate leading or lagging green phases on both streets.

All of the phase plans discussed to this point can be implemented with pretimed or actuated controllers. The only difference in operation would be the manner in which green time is allocated to the various phases. For pretimed controllers, green times are preset, whereas for actuated controllers, green times vary on the basis of detector actuations.

At this point, it is necessary to recognize the differences in the way that modern traffic-actuated controllers actually implement the phase plan. Exhibit B16-1 depicts a single-ring, sequential representation of the phase plan, in which a signal phase is used to indicate the combination of all movements that are proceeding at a given point in time. Modern traffic-actuated controllers do not use this scheme. Instead, they implement a dual-ring concurrent phasing in which each phase controls only one movement, but two phases are generally being displayed concurrently.

The dual-ring concurrent concept is illustrated in Exhibit B16-2. Note that eight phases are shown, each of which accommodates one of the through or left-turning movements. A barrier separates the north-south phases from the east-west phases. Any phase in the top group (Ring 1) may be displayed with any phase in the bottom group (Ring 2) on the same side of the barrier without introducing traffic conflicts. For simplicity, the right turns are omitted and assumed to proceed with the through movements.

The definition of a phase as presented in Exhibit B16-2 is not consistent with that in Exhibit B16-1 nor with the definition given in Chapter 10. It is, however, a definition that is universally applied in the traffic control industry. It is the responsibility of the analyst to recognize which definition is applicable to any given situation. For purposes of the capacity and delay analysis procedures presented in the body of this chapter, each lane group is considered to be controlled separately by a phase with specified red, green, yellow, and all-red times, so either definition could apply. The examples shown throughout the chapter are based on the single-ring sequential concept. However, the dual-ring definition must be used for estimating the timing plan at traffic-actuated intersections using the procedure presented in this appendix.

The dual-ring phases that accommodate left turns will only be used if the left turns are protected. Left turns with compound protection will proceed with their concurrent through movements. For example, none of the left-turn phases would be used by a dual-ring controller to implement the two-phase plan shown in Exhibit B16-1(a). All of the other phase plan examples shown in Exhibit B16-1 may be created by selectively omitting left-turn phases and by reversing the order in which the through and left-turn phases are displayed in either ring.

The advantage of the dual-ring concept is that it is able to generate the optimal phase plan for each cycle in response to the traffic demand. Pretimed controllers, and earlier versions of traffic-actuated controllers, are more constrained in this regard. The maximum flexibility is provided by allowing the first (usually left-turn) phases in Rings 1 and 2 to terminate independently after their respective demands have been satisfied.

It is also possible to constrain these phases to terminate simultaneously to emulate the older, less efficient equipment. For example, simultaneous termination of the northbound and southbound left-turn phases in Exhibit B16-2 would produce the phasing example shown in Exhibit B16-1(b). Independent termination of the two left turns would

Careful selection of a phase plan is necessary to achieve efficient operation

introduce an overlap phase between the left-turn phase and the through-movement phase in Exhibit B16-1(b). The overlap phase would accommodate the heavier volume of the two left turns together with the concurrent through movement, thereby making more effective use of the cycle length. The degree of benefit obtained from phase overlaps of this nature depends on the degree of difference in the opposing left-turn volumes.

The establishment of a phase plan is the most creative part of signal design and deserves the careful attention of the analyst. A good phase plan can achieve efficiency in the use of available space and time, whereas an inappropriate plan can cause inefficiency. The phase plans presented and discussed in this appendix represent a sampling of the more common forms used. They may be combined in a number of innovative ways on various approaches of an intersection.

Again, local practice is an important determinant in the selection of a phase plan. Phasing throughout an area should be relatively uniform. The introduction of the protected-plus-permitted phasing at one location in an area where left turns are generally handled in exclusive left-turn phases, for example, may confuse drivers. Thus, system considerations should also be evaluated when phase plans are established.

ALLOCATION OF GREEN TIME

The allocation of green time is an important input to the methodology presented earlier in this chapter for the estimation of delay. The average cycle length and effective green time for each lane group must be defined. The most desirable way to obtain these values is by field measurement; however, there are many cases in which field measurement is not possible. For example, the comparison of hypothetical alternatives precludes field measurements. Even for the evaluation of existing conditions, the required data collection is beyond the resources of many agencies.

A procedure for estimating the signal timing characteristics is therefore an important traffic analysis tool. Such a procedure is also useful in designing timing plans that will optimize some aspect of the signal operation. In this respect, pretimed and actuated control must be treated differently because the design and analysis objectives are different. For pretimed control, the objective is to design an implementable timing plan as an end product. In traffic-actuated control, the timing plan is generated by the controller itself on the basis of operating parameters that are established for each phase. This operation creates two separate objectives for traffic-actuated control. The first is to determine how the controller will respond to a specified combination of operating parameters and traffic conditions. The second is to provide some indication of the optimal values for the key operating parameters.

TIMING PLAN DESIGN FOR PRETIMED CONTROL

The design of an implementable timing plan is a complex and iterative process that is generally carried out with the assistance of computer software. Several software products are available for this purpose, some of which employ, at least in part, the methodology of this chapter.

Design Strategies

There are 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. Three basic strategies are commonly used for pretimed signals.

Equalizing the v/c ratios for critical lane groups is the simplest strategy and the only one that may be calculated without excessive iteration

Equalizing the v/c ratios for critical lane groups is the simplest strategy and the only one that may be calculated without excessive iteration. It will be described briefly in this appendix. It is also employed in the timing plan synthesis procedures of the planning procedure presented in Chapter 10. Under this strategy, the green time is allocated among the various signal phases in proportion to the flow ratio of the critical lane group for each phase.

Minimizing the total delay to all vehicles is generally proposed as the optimal solution to the signal timing problem, often in combination with other measures such as stops and fuel consumption. Many signal timing models offer this optimization feature. Some use a delay estimation procedure identical to the methodology in this chapter, whereas others employ minor departures from this method.

Balancing the LOS for all critical lane groups promotes an LOS on all approaches that is consistent with the overall intersection LOS. Both of the other strategies tend to produce a higher delay per vehicle, and therefore a less favorable LOS, for the minor movements at an intersection. This lack of balance in LOS for critical lane groups causes some difficulty in representing the overall intersection LOS.

Minimizing the total delay to all vehicles is generally proposed as the optimal solution to the signal timing problem, often in combination with other measures such as stops and fuel consumption

Balancing the LOS for all critical lane groups promotes an LOS on all approaches that is consistent with the overall intersection LOS

Procedure for Equalizing Degree of Saturation

Once a phase plan and signal type have been established, signal timing may be estimated using Equations B16-1, B16-2, B16-3, and B16-4.

$$X_i = \frac{v_i C}{s_i g_i} \tag{B16-1}$$

$$X_c = \sum_i \left(\frac{v}{s}\right)_{ci} \left(\frac{C}{C - L}\right) \tag{B16-2}$$

$$C = \frac{L X_c}{\left[X_c - \sum_i \left(\frac{v}{s}\right)_{ci}\right]} \tag{B16-3}$$

$$g_i = \frac{v_i C}{s_i X_i} = \left(\frac{v}{s}\right)_i \left(\frac{C}{X_i}\right) \tag{B16-4}$$

where

- C = cycle length (s);
- L = lost time per cycle (s);
- X_c = critical v/c ratio for the intersection;
- X_i = v/c ratio for lane group i (note that target v/c ratio is a user-specified input with respect to this procedure; default value suggested is 0.90);
- $(v/s)_i$ = flow ratio for lane group i;
- s_i = saturation flow rate for lane group i (veh/h); and
- g_i = effective green time for lane group i (s).

Cycle lengths and green times may be estimated using these relationships, computed flow ratios, and desired v/c ratios.

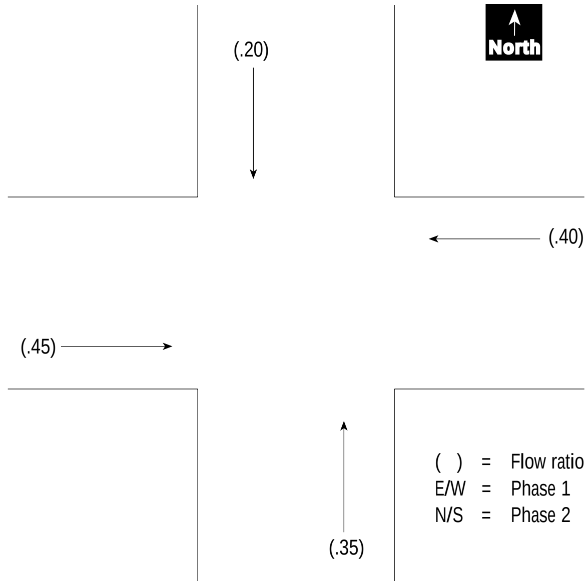
For pretimed signals, fixed green times and cycle lengths may be estimated using Equations B16-3 and B16-4. The procedure will be illustrated using a sample calculation. Consider the two-phase signal shown in Exhibit B16-3. Flow ratios are shown, and it is assumed that lost times equal the change-and-clearance intervals, which are 4 s for each phase or 8 s/cycle.

The cycle length is computed from Equation B16-3 for the desired v/c ratio X_c, which must be selected by the analyst. The shortest cycle length that will avoid oversaturation may be computed by Equation B16-5 using X_c = 1.00:

$$C\ (minimum) = \frac{L X_c}{\left[X_c - \sum_i \left(\frac{v}{s}\right)_{ci}\right]} \tag{B16-5}$$

$$C\ (minimum) = \frac{8(1.0)}{[1.0 - (0.45 + 0.35)]} = \frac{8}{0.2} = 40 \text{ s}$$

EXHIBIT B16-3. SAMPLE TWO-PHASE SIGNAL



This cycle length has no direct value in the design of implementable timing plans; however, it is commonly used as a departure point for iterative procedures that seek to minimize or equalize delay among lane groups.

If a v/c ratio of no more than 0.8 were desired, the computation would become

$$C = \frac{8(0.80)}{[0.8 - (0.45 + 0.35)]} = \frac{6.4}{0} = \textit{infinity}$$

This computation indicates that a critical v/c ratio of 0.8 cannot be provided for a 40-s cycle and the demand levels existing at the intersection. Any cycle length greater than 40 s may be selected. For purposes of illustration, assume a cycle length of 60 s. In all cases, the cycle length assumed would be rounded to the nearest 5 s for values between 30 and 90 s and to the nearest 10 s for higher values.

The actual critical v/c ratio provided by a 60-s cycle is given by Equation B16-6:

$$X_c = \frac{\sum_i \left(\frac{v}{s}\right)_i C}{(C - L)} \tag{B16-6}$$

$$X_c = \frac{(0.45 + 0.35)(60)}{(60 - 8)} = 0.923$$

A number of different policies may be employed in allocating the available green time. A common policy for two-phase signals is to allocate the green such that the v/c ratios for critical movements in each phase are equal. Thus, for the example problem, the v/c ratio for each phase would be 0.923, and green times are computed using Equation B16-7.

$$g_i = \left(\frac{v}{s}\right)_i * \left(\frac{C}{X_i}\right) \tag{B16-7}$$

$$g_1 = 0.45(60/0.923) = 29.3 \text{ s}$$
$$g_2 = 0.35(60/0.923) = \underline{22.7 \text{ s}}$$
$$52.0 \text{ s}$$
$$\text{Lost time} = \underline{8.0 \text{ s}}$$
$$60.0 \text{ s}$$

Another common policy would be to allocate the minimum required green time to the minor approach and assign all remaining green to the major approach. In this case, the v/c ratio for Phase 2 would be 1.0, and

$$g_2 = 0.35(60/1.0) = 21.0 \text{ s}$$
$$g_1 = 60 - 8 - 21 = \underline{31.0 \text{ s}}$$
$$52.0 \text{ s}$$
$$\text{Lost time} = \underline{8.0 \text{ s}}$$
$$60.0 \text{ s}$$

Note that in both cases the entire 60-s cycle is fully allocated among the green times and lost time.

The procedure for timing may be summarized as follows:

- Estimate the cycle length for full saturation using Equation B16-3 and $X_c = 1.0$.
- Estimate the cycle length for the desired critical v/c ratio, X_c, using Equation B16-3.
- From the results of the first two calculations, select an appropriate cycle length for the signal. When system constraints determine the cycle length, the first step and this step may be eliminated.
- Estimate the green times using Equation B16-4 and v/c ratios, X_i, appropriate to the proportioning policy adopted.
- Check the timing to ensure that the sum of green times and the lost time equals the cycle length. Include overlapping green times only once in this summation.

TIMING PLAN ESTIMATION FOR TRAFFIC-ACTUATED CONTROL

This procedure encompasses both a traffic-actuated control model and an analytical structure for implementation of the model.

Functional Requirements of Model

A practical traffic-actuated control model must be functionally capable of providing reasonable estimates of the operating characteristics of traffic-actuated controllers under the normal range of design configurations at both isolated and coordinated intersections. It must also be sensitive to common variations in design parameters. Examples of design parameters include

- Traffic-actuated controller settings (initial interval, allowable gap, maximum green time),
- Conventional actuated versus volume-density control strategies,
- Detector configuration (length and setback),
- Pedestrian timing (Walk and flashing Don't Walk),
- Left-turn treatment (permitted, protected, permitted and protected, not opposed), and
- Left-turn phase position (leading or lagging).

Data Requirements

The information that is already required by this chapter methodology is used to the extent possible to avoid the need for new data. Most of the additional data items relate to the operation of the controller itself. The model structure is based on the standard eight-phase dual-ring control scheme that was illustrated in Exhibit B16-2. This scheme is more or less universally applied in the United States.

For purposes of this discussion, the scheme for assignment of movements to phases presented in Exhibit B16-2 will be adopted. This procedure will greatly simplify the illustration of all modeling procedures without affecting the generality of the results.

The process is highly iterative, and the productive application of the manual worksheets is not practical. Only the input data worksheet will be discussed in detail in this appendix. This worksheet, presented in Exhibit B16-4, gathers together all data required by the analytical model.

Approach-Specific Data

The top portion of the worksheet summarizes the approach-specific information. A separate column is used for each of the four approaches. The logic of the model requires that the left-turn treatment be identified explicitly. The codes used here are consistent with the definitions presented in the body of this chapter.

The term simple left-turn protection refers to left turns moving only on the protected phase. The term compound left-turn protection will be used to denote either protected-plus-permitted or permitted-plus-protected treatments.

Position Codes

Position codes are required to distinguish between leading and lagging left-turn protection. The terms leading and lagging apply equally to the cases of simple and compound left-turn protection. These codes do not apply if the left turn is not protected. The worksheet offers a simple choice of leading, lagging, or N/A. The definition is very simple: leading left turns precede the movement of the opposing through traffic and lagging left turns follow it.

Sneakers

The term sneaker describes the number of left turns per cycle that may be discharged at the end of a permitted phase. An implicit default of two per cycle is built into the supplemental permitted left-turn worksheets for purposes of determining the minimum saturation flow rate. Since any vehicles that rest in the detection zone will extend their respective phases (assuming that the phase has not already extended to a preset maximum), a more detailed treatment of sneakers will be required for traffic-actuated control.

Free Queue

The pretimed model assumes that the first permitted left turn at the stop line will block a shared lane. However, through vehicles in the shared lane are often able to squeeze around one or more left-turning vehicles, which define the free queue. The lack of a free-queue parameter is a deficiency of the pretimed model, but it is especially critical with traffic-actuated control because vehicles in the free queue do not occupy the detector and therefore do not extend the green phase. A permitted left turn stopped on the detector must be treated entirely differently in the modeling process than one that has stopped beyond the detector.

Approach Speed, S_A

The speed of vehicles on a signalized intersection approach is required in the analysis of traffic-actuated operation. It determines the passage time between the detector and the stop line as well as the portion of intervehicle headways during which a presence detector is occupied. In modeling the operation of vehicles at a traffic signal, it is typical to assume a single value for speed that applies throughout the cycle.

EXHIBIT B16-4. TRAFFIC-ACTUATED CONTROL INPUT DATA WORKSHEET

TRAFFIC-ACTUATED CONTROL INPUT DATA WORKSHEET

General Information		**Site Information**	
Analyst	______________	Intersection	______________
Agency or Company	______________	Area Type	❑ CBD ❑ Other
Date Performed	______________	Jurisdiction	______________
Analysis Time Period	______________	Analysis Year	______________

Approach-Specific Data

	EB	WB	NB	SB
Left-turn treatment code[1]				
Left-turn position (leading, lagging, or N/A)				
Left-turn max sneakers, v_{LS} (veh)				
Left-turn free queue, Q_f (veh)				
Approach speed, S_A (km/h)				
Ring 1 and 2 termination (simultaneous, independent, or N/A)				

Phase-Specific Data

	Ring 1				Ring 2			
	1. WBLT	2. EBTH	3. NBLT	4. SBTH	5. EBLT	6. WBTH	7. SBLT	8. NBTH
Phase type[2] (L, T, G, N, X)								
Phase reversal (Yes or No)								
Detector length, DL (m)								
Detector setback, DS (m)								
Max initial interval, MxI (s)								
Added initial per actuation, AI (s)								
Min allowable gap, MnA (s)								
Gap reduction rate, GR								
Ped Walk + Don't Walk, WDW (s)								
Maximum green, MxG (s)								
Intergreen time, Y (s)								
Recall mode (min, max, ped, none)								
Min veh phase time, MnV (s)								

Notes

1. (0) Does not exist (1) Permitted (2) Protected (3) Protected + Permitted (4) Not opposed
2. (L) Protected left turn on a green arrow
   (T) Through and right-turning traffic only
   (G) Permitted left turns and compound left-turn protection
   (N) In addition to other movements, left turns are not opposed
   (X) Inactive phases

Terminating of Rings 1 and 2

The nature of dual-ring control requires that the second phases of Rings 1 and 2 terminate simultaneously because they yield control to approaches with conflicting traffic. However, control may pass from the first phase to the second phase of either ring without causing conflict. Independent termination of the first phases improves efficiency in the allocation of time among competing movements and is generally exploited for this reason. The type of operation created by independent termination is sometimes referred to as phase overlap.

It is, however, not essential that the phases terminate independently. Older single-ring controller operation may be approximated by requiring that the first phases of each ring (i.e., Phases 1 and 5, or 3 and 7) terminate simultaneously. In some situations involving coordination of controllers on major routes, it is common to force both rings out of their first phase simultaneously. The model considers simultaneous or independent termination as legitimate alternatives.

It is possible that one or more of the first phases will not be used, because their associated left turn is not protected. In this case, the question of simultaneous or independent termination will not apply. This is another multiple-choice entry on the worksheet. The alternatives are simultaneous, independent, and N/A.

Phasing and Detector Design Parameters

The bottom portion of the worksheet includes all of the data items that are specific to each of the eight phases represented in Exhibit B16-2. A separate column is provided on the worksheet for each phase. The first group includes the design parameters relating to phasing and detector placement that will affect the operation. The following data items are included.

Phase Type

Convention for designating movements accommodated on a phase

Phase type is the first of several phase-specific inputs that are required. A phase that is not active will not be recognized in any of the subsequent computations. Inactive phases are indicated by an X in the appropriate column of the worksheet. A left-turn phase will be considered active only if it accommodates a protected left turn. A through phase will be considered active if it accommodates any traffic at all—through, left, or right. Active phases will be designated as follows:

- L if the phase accommodates a protected left turn on a green arrow.
- T if the phase accommodates through and right-turning traffic only. In this case, all left turns are accommodated entirely on another phase (i.e., simple left-turn protection).
- G if any left turns are accommodated on this phase and opposed by oncoming traffic. This case will occur on phases with permitted left turns and those with compound left-turn protection.
- N if the phase accommodates, in addition to other movements, left turns that are not opposed at any time in the phase sequence. This case will happen at T-intersections, on one-way streets, and in cases in which the phasing completely separates all movements on opposing approaches.

Note that right turns are not referenced specifically in these designations. Right turns are assumed to proceed concurrently with the through traffic.

Phase Reversal

Normally the first phase in each ring on each side of the barrier (odd-numbered phase) handles protected left turns and the second (even-numbered) phase handles the remaining traffic. This operation creates a condition of leading left-turn protection. When lagging left-turn protection is desired, the movements in the first and second phases are interchanged. Most controllers provide an internal function to specify phase reversal. For purposes of this methodology, two phases may only be interchanged if both phases are active.

Detector Length, DL

Detector length is the effective distance, measured parallel to the direction of travel, through which a vehicle will occupy the detector. This design parameter is user-specified and influenced by local practice. The detector length influences the choice of other parameters, such as the allowable gap in traffic that will terminate the phase.

Detector Setback, DS

Detector setback is the distance between the downstream edge of the detector and the stop line.

Controller Settings

The controller itself has several operating parameters that must be specified for each phase. Collectively, these will be referred to as the controller settings because they must be physically set in the controller with switches, keypads, or some other electrical means. The following settings will exert a significant influence on the operation of the intersection and must therefore be recognized by the analysis methodology.

Maximum Initial Interval, MxI

Maximum initial interval is used only when the initial interval is extended under volume-density control. It must be long enough to ensure that a queue of vehicles released at the beginning of the green will be in motion at the detector before the green phase terminates.

Added Initial per Actuation, AI

When the initial interval is extended under volume-density control, the added initial interval per actuation will depend on the number of approach lanes. It should be long enough to ensure that each vehicle crossing the approach detector on the red will add an appropriate increment of time to the initial interval.

Minimum Allowable Gap, MnA

MnA is a user-specified controller parameter, the effect of which will be illustrated later as the analytical model is exercised. It is typically set in the range of 2 to 3 s. It establishes the threshold for the length of the gap in traffic that will cause the phase to terminate.

Difference between time headway and time gap

It is important to distinguish between the time gap and the time headway between vehicles. The time headway indicates the elapsed time between the successive arrival of two consecutive vehicles at a detector. The time gap indicates the elapsed time between the departure of the first vehicle from the detector and the arrival of the second. The time gap is what is left of the headway after the detector occupancy time has been subtracted. A traffic-actuated controller using presence detectors views the passage of traffic in terms of gaps, not headways.

Gap Reduction Rate, GR

The gap reduction rate determines the speed at which the allowable gap is reduced in volume-density controllers as the green display continues. There are subtle differences in the definition of the gap reduction rate among controllers. For purposes of this project, a linear reduction rate (seconds reduction of gap per second of elapsed green time) will be assumed.

Pedestrian Walk + Don’t Walk, WDW

WDW is the minimum time given to each phase when pedestrian demand is registered or the pedestrian recall is active. It includes both the pedestrian Walk and flashing Don’t Walk intervals. These intervals are actually entered into the controller as two separate parameters but will be combined for purposes of this analysis. If the pedestrian timing function is not implemented in a particular phase, the WDW value should be entered as zero.

Maximum Green, MxG

MxG is a user-specified parameter, the effect of which will be discussed later. Local practice often plays an important part in the determination of maximum green times.

Intergreen Time, Y

The intergreen interval consists of a yellow change interval followed by an all-red clearance interval. These two intervals are entered separately into the controller but will be combined here to simplify the analysis.

Recall Mode

Recall mode determines how a phase will be treated in the absence of demand on the previous red phase. The options are as follows:

- None: the phase will not be displayed.
- Max: the phase will be displayed to its specified maximum length.
- Min: the phase will always be displayed to its specified minimum length but may be extended up to its specified maximum length by vehicle actuations.
- Ped: the phase will be given the full Walk plus flashing Don't Walk intervals and may be extended further, up to its specified maximum by vehicle actuations.
- Coord: a coordinated phase on the major street; this phase will always be displayed for its nominal design time, which may be increased by reassigning unused green time from actuated phases.

The recall function will have a significant effect on the operation of the controller. For example, the maximum recall option will have the effect of creating a nonactuated phase.

Minimum Vehicle Phase Time, MnV

MnV is actually a traffic engineering input that specifies the minimum time for which a phase must be displayed unless it is skipped because of lack of demand. It is implemented in a conventional traffic-actuated controller as the sum of three intervals: the initial interval, the minimum allowable gap (MnA), and the intergreen time (Y). As a matter of design, it is important that the controller settings be compatible with the minimum phase times determined by traffic engineering considerations.

This parameter did not appear earlier in this chapter because the chapter methodology procedure does not offer the ability to compute timing plans. Nevertheless, it is not possible to deal realistically with traffic-actuated control without recognizing the existence of a minimum phase time. For compatibility with other signal timing programs, the phase times include all intervals, including green, yellow, and all-red. For the purposes of the worksheet, the minimum phase time must be replaced by the maximum phase time (MxG + Y) if the recall-to-maximum mode is in effect.

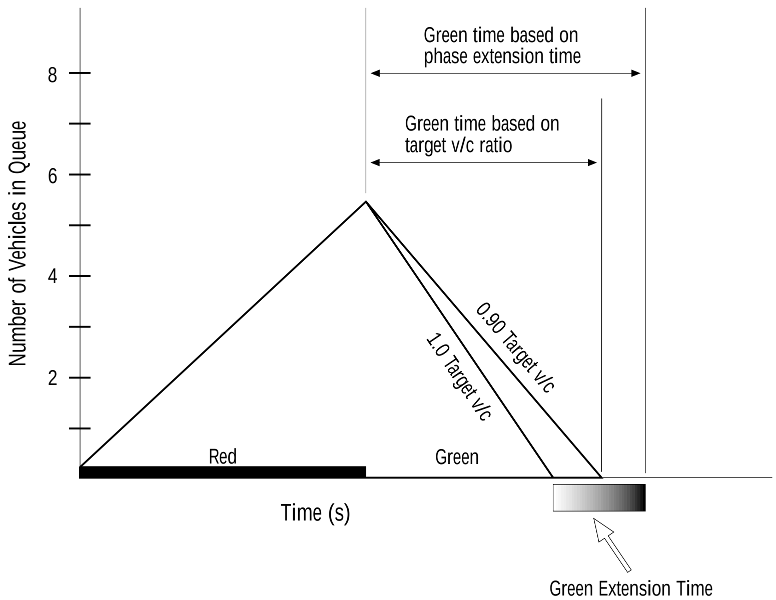
Green-Time Estimation Model

The discussion of pretimed operation presented in this appendix indicates that the determination of required green time is a relatively straightforward process when the cycle length is given. However, traffic-actuated controllers do not recognize specified cycle lengths. Instead they determine, by a mechanical analogy, the required green time given the length of the previous red period and the arrival rate. They accomplish this by holding the right-of-way until the accumulated queue has been served.

Queue accumulation polygon is a basic tool for analysis of signalized intersections

The basic principle underlying all signal timing analysis is the queue accumulation polygon (QAP), which plots the number of vehicles queued at the stop line over the duration of the cycle. The QAP for a simple protected movement is illustrated in Exhibit B16-5. The queue accumulation takes place on the left side of the triangle (i.e., effective red) and the discharge takes place on the right side of the triangle (i.e., effective green). More complex polygons are generated when permitted movements occur when a movement proceeds on more than one phase. The body of this chapter and Appendix E include a discussion on this subject.

EXHIBIT B16-5. QUEUE ACCUMULATION POLYGON ILLUSTRATING TWO METHODS OF GREEN-TIME COMPUTATION



Two methods of determining the required green time given the length of the previous red are depicted in Exhibit B16-5. The first employs a target v/c approach. This method is the basis for the planning procedure described in Chapter 10 and for the discussion on timing plan design for pretimed control presented earlier in this appendix. Under this approach, the green-time requirement is determined by the slope of the line representing the target v/c of 0.9. If the phase ends when the queue has dissipated under these conditions, the target v/c will be achieved.

The second method recognizes the way a traffic-actuated controller really works. It does not deal explicitly with v/c ratios; in fact, it has no way of determining the v/c ratio. Instead it terminates each phase when a gap of a particular length is encountered at the detector. Good practice dictates that the gap threshold must be longer than the gap that would be encountered when the queue is being served. Assuming that gaps large enough to terminate the phase can only occur after the queue service interval (based on v/c = 1.0), the average green time may be estimated as the sum of the queue service time and the phase extension time as shown in Exhibit B16-5.

Queue Service Time

The queue service time, g_s, can be estimated using Equation B16-8.

$$g_s = f_q \frac{q_r r}{(s - q_g)} \qquad \text{(B16-8)}$$

where

g_s	=	queue service time (s);
q_r, q_g	=	red arrival rate (veh/s) and green arrival rate (veh/s), respectively;
r	=	effective red time (s);
s	=	saturation flow rate (veh/s); and
f_q	=	1.08 – 0.1 (actual green time/maximum green time)^2.

The queue calibration factor, f_q, accounts for randomness in arrivals (*1*).

Assumptions regarding headway distribution

Green Extension Time

To estimate the extension time analytically for a particular phase, it is necessary to determine the expected waiting time for a gap of a specific length, given the average intervehicular headways and some assumptions about the headway distribution. An analytical model for this purpose was developed (*1, 2*) on the basis of earlier work (*3, 4*). The average green extension time, g_e, is estimated from Equation B16-9, which is based on the use of a bunched exponential distribution of arrival headways. Equations B16-10, B16-11, and B16-12 are used to compute specific factors used in Equation B16-9.

$$g_e = \frac{e^{\lambda(e_0 + t_0 - \Delta)}}{\varphi q} - \frac{1}{\lambda} \qquad \text{(B16-9)}$$

where

- g_e = green extension time (s),
- q = vehicle arrival rate throughout cycle (veh/s),
- e_0 = unit extension time setting (s), and
- t_0 = time during which detector is occupied by a passing vehicle (s).

$$t_0 = \frac{3.6(L_d + L_v)}{S_A} \qquad \text{(B16-10)}$$

where

- L_v = vehicle length, assumed to be 5.5 m,
- L_d = detector length, DL (m),
- S_A = vehicle approach speed (km/h),
- Δ = minimum arrival (intra-bunch) headway (s),
- φ = proportion of free (unbunched) vehicles, and
- λ = parameter calculated as

$$\lambda = \frac{\varphi q}{1 - \Delta q} \qquad \text{(B16-11)}$$

where λ is in vehicles per second for all lane groups that actuate the phase under consideration.

A detailed discussion of the bunched exponential model and the results of its calibration are given elsewhere (*5, 6*). The following relationship can be used for estimating the proportion of free (unbunched) vehicles in the traffic stream (φ):

$$\varphi = e^{-b\Delta q} \qquad \text{(B16-12)}$$

Note that b is a bunching factor. The recommended parameter values based on the calibration of the bunched exponential model using real-life and simulation data are summarized in Exhibit B16-6.

An iterative process is required to arrive at average times

Computational Structure for Green-Time Estimation

This green-time estimation model is not difficult to implement, but it does not lead directly to the determination of an average cycle length or green time because the green time required for each phase is dependent on the green time required by the other phases. Thus, a circular dependency is established that requires an iterative process to solve. With each iteration, the green time required by each phase, given the green times required by the other phases, can be determined.

The logical starting point for the iterative process involves the minimum times specified for each phase. If these times turn out to be adequate for all phases, the cycle length will simply be the sum of the minimum phase times for the critical phases. If a particular phase demands more than its minimum time, more time should be given to that phase. Thus, a longer red time must be imposed on all of the other phases. This, in turn, will increase the green time required for the subject phase.

EXHIBIT B16-6. RECOMMENDED PARAMETER VALUES

Case	Δ (s)	b
Single-lane	1.5	0.6
Multilane		
2 lanes	0.5	0.5
3 or more lanes	0.5	0.8

Simple Two-Phase Example

The circular dependency mentioned earlier will converge quite reliably through a series of repeated iterations. The convergence may be demonstrated by using a simple example. More complex examples will be introduced later to examine the effects of controller settings and traffic volumes in a practical situation.

Consider an intersection of two streets with a single lane in each direction. Each approach has identical characteristics and carries 675 veh/h with no left or right turns. The average headway is 2.0 s per vehicle and the lost time per phase is 3.0 s. Detectors are 9.1 m long with no setback from the stop line. The actuated controller settings are as follows:

Setting	Time (s)
Initial interval	10
Unit extension	3
Maximum green	46
Intergreen	4

The maximum phase time for each phase will be (46 + 4) = 50 s. The minimum phase time will be 10 + 3 + 4 = 17 s, which will be the starting point for the timing computations. The first iteration will be used with a 34-s cycle with 17 s of green time on each approach. Allowing for lost time, the effective red time will be 20 s, and the effective green time will be 14 s for each phase.

The total lost time is the sum of two components, including the start-up lost time and the clearance lost time. In the methodology of this chapter, all of the lost time is assumed to be concentrated at the beginning of the green. This approximation is valid for delay estimation because the lost time is only used in the computation of effective green time, and its position in the phase is irrelevant.

Guidelines on lost-time assumptions

However, for purposes of traffic-actuated timing estimation, the distribution of lost time throughout the phase will have a definite influence on the results. The lost time at the beginning of the phase will influence the length of the phase. The lost time at the end of the phase will influence the delay, but it will have no effect on the phase duration. It is recommended (*7*) that, for a specified lost time of n seconds, 1 s be assigned to the end of the phase and n – 1 s be assigned to the beginning. Thus, for this example, the start-up lost time (l_1) will be 2.0 s.

The computational process may be described as follows:

1. Compute the arrival rate throughout the cycle, q:

$$q = 675/3600 = 0.188 \text{ veh/s}$$

2. Compute the net departure rate (saturation flow rate – arrival rate):

$$(s - q) = \frac{1800}{3600} - 0.188 = 0.312 \text{ veh/s}$$

3. Compute the queue at the end of 20 s of effective red time:

$$q_r r = 20 * (0.188) = 3.760 \text{ veh}$$

4. Compute the queue calibration factor, f_q:

$$f_q = 1.08 - 0.1\,(13/46)^2 = 1.072$$

5. Compute the time required to serve the queue, g_s:

$$g_s = 1.072\,(3.760/0.312) = 12.919 \text{ s}$$

After 12.919 s of effective green time, the queue will have been served and gaps will start to be observed at the detector. The start-up lost time ($l_1 = 2$) must be added to the queue service time for purposes of determining the total phase time requirements. The question now is how long one would expect to wait for a gap of 3.0 s.

6. Determine the parameters of Equation B16-12 as follows:

$$\Delta = 1.5 \text{ and } b = 0.6, \text{ from Exhibit B16-6}$$

$$\varphi = e^{-b\Delta q}$$

$$= e^{-(0.6 * 1.5 * 0.188)} = 0.844$$

$$\lambda = \frac{\varphi q}{1-\Delta q} = \frac{(0.844)(0.188)}{1-(1.5)(0.188)} = 0.221$$

7. Determine the occupancy time of the detector for a vehicle length of 5.5 m, a detector length of 9.1 m, and an approach speed of 50 km/h:

$$t_0 = \frac{3.6(9.1+5.5)}{50} = 1.051 \text{ s}$$

This is assuming that the detector sensor is operating in the presence mode.

8. Apply Equation B16-9 to determine the expected green extension time, g_e:

$$g_e = \frac{e^{\lambda(e_o+t_o-\Delta)}}{\varphi q} - \frac{1}{\lambda}$$

$$= \frac{e^{0.221(3.0+1.051-1.5)}}{(0.844)(0.188)} - \frac{1}{0.221}$$

$$= 6.550 \text{ s}$$

9. Compute the total phase time:

$$G = l_1 + g_s + g_e + Y$$

$$G = 2.0 + 12.919 + 6.550 + 4.0 = 25.469 \text{ s}$$

10. Compute the phase time deficiency as the difference between the trial phase time and the computed phase time, or 25.469 – 17.0 = 8.469 s.

This computation indicates that the trial phase time was not adequate to satisfy the rules under which the controller operates. It also suggests a new trial green time of 25.469 s and a cycle length of 50.938 s for the next iteration.

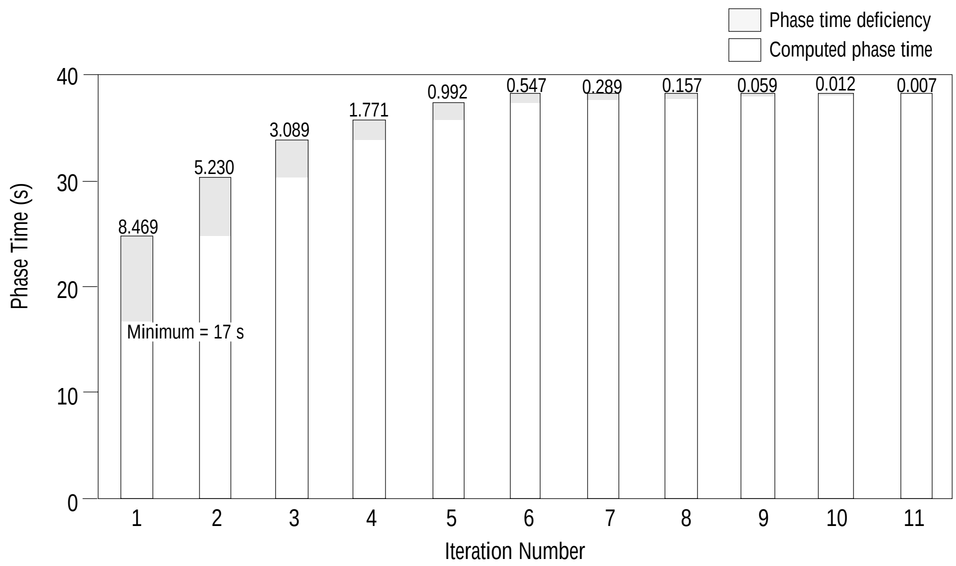
Guideline for termination of iterations

The next iteration will still produce a green-time deficiency because the red time has been increased. However, this deficiency will be smaller. Successive iterations will produce successively smaller green time deficiencies until eventually the solution will converge. This process is illustrated in Exhibit B16-7. The solution converged (i.e., the green-time deficiency became negligible) at a phase time of 37.710 s, producing a cycle length of 75.420 s. This convergence was based on a threshold of 0.1 s difference in the computed cycle length between iterations. In other words, the process terminated when the cycle lengths on two successive iterations fell within 0.1 s of each other.

As a matter of interest, consider the effect of reducing the unit extension time, e_0, from 3.0 s to 2.0 s. This reduction would be expected to reduce the green extension time, g_e, for both phases and to shorten the resulting cycle length. The extent of the reduction may be estimated by repeating all of the steps described above with the new value for g_e. In the first iteration, the queue service time will remain the same, but the green extension time will be reduced from the value of 6.550 s computed above to 4.354 s. Repeated iterations with this lower unit extension time would converge to a cycle length of 65.618 s.

In this example, the green time for both phases was determined by the sum of the queue service time and the extension time. Phase times will also be constrained by their specified maximum and minimum times.

EXHIBIT B16-7. CONVERGENCE OF GREEN-TIME COMPUTATION BY ELIMINATION OF GREEN-TIME DEFICIENCY



Minimum Phase Times

The specified minimum green-time constraints are valid only for pretimed phases and phases that are set to recall to the minimum time regardless of demand. The significance of the minimum time for an actuated phase is that the phase must be displayed for its specified minimum unless it is skipped because of lack of demand. This situation may be addressed analytically by determining the probability of zero arrivals on the previous cycle. Assuming a Poisson arrival distribution, Equation B16-13 may be used.

$$P_{0v} = e^{-qC} \qquad \text{(B16-13)}$$

where

P_{0v} = minimum phase time (s),
q = vehicle arrival rate (veh/s), and
C = cycle length for the current iteration (s).

Assuming that the phase will be displayed for the minimum time, except when no vehicles have arrived, the adjusted minimum phase time is computed using Equation B16-14:

$$AVM = MnV\ (1 - P_{0v}) \qquad \text{(B16-14)}$$

where

AVM = adjusted vehicle minimum time (s), and
MnV = specified minimum green time from worksheet in Exhibit B16-4 (s).

This relationship also has circular dependencies because as the adjusted minimums become shorter, the probability of zero arrivals also becomes higher, which further reduces the adjusted minimums. Fortunately, the solution fits well into the iterative scheme that was just described.

The use of adjusted minimum green times offers a practical method for dealing with phases that are not displayed on each cycle but may have their minimum durations determined by agency policy. The concept applies equally well to pedestrian minimum times.

Multiphase Operation

Three important tools have been introduced for estimating the timing plan at traffic-actuated signals: a model for predicting the green time for any phase given the length of the previous red period, an iterative computational structure that converges to a stable value for the average cycle length and green times, and a procedure to account for minimum green times with low volumes. These concepts were illustrated in a simple example, but fortunately they are robust enough to deal with the practical complexities of traffic-actuated control. These complexities include multiphase operation (both single- and dual-ring), permitted left turns (both exclusive and shared lanes), and compound left-turn protection (both leading and lagging).

Extension of the methodology to
- Account for different arrival and departure times at points in cycle
- Synthesize dual-ring into equivalent single-ring sequence

Two extensions to the methodology presented to this point are required to deal with more complex situations. The first is the extension of the QAP from its simple triangular shape to a more complex shape that represents different arrival and departure times at different points in the cycle. The second is a procedure to synthesize a complete single-ring equivalent sequence by combining critical phases in the dual-ring operation. The QAP extensions will be considered first.

Exhibit B16-5 presented the triangular QAP for a protected movement from an exclusive lane. There are four other cases to be considered, including permitted left turns from an exclusive lane, permitted left turns from a shared lane, protected-plus-permitted left turns, and permitted-plus-protected left turns. The QAP shapes for these cases are shown in Exhibits B16-8 through B16-11. Each of the exhibits conforms to a common terminology with respect to its labeling. Intervals are illustrated along the horizontal axis as follows:

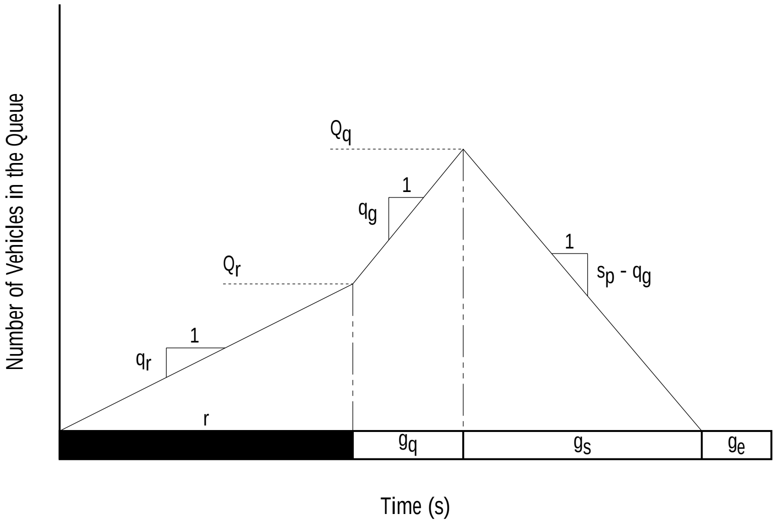
r = effective red time,
g_q = portion of permitted green time blocked by a queue of opposing vehicles,
g_u = portion of permitted green time not blocked by a queue of opposing vehicles,
g_s = portion of protected green time required to serve queue of vehicles that accumulated on previous phases,
g_e = extension to protected green time that occurs while controller waits for a gap in arriving traffic long enough to terminate the phase, and
g_f = portion of green time in which a through vehicle in a shared lane would not be blocked by a left-turn vehicle waiting for opposed movement to clear. (This condition occurs only at the beginning of the permitted green when one or more through vehicles are at the front of the queue.)

Note that, in each case, the phases are arranged so that the protected phase is the last to occur. The length of this phase will be determined by its detector actuations. The actual length will be the sum of the time required to serve the queue that exists at the beginning of the phase plus the extension time.

Points in the cycle at which the queue size is important to the computations are also identified as follows:

Q_r = queue size at end of effective red;
Q_q = queue size at end of interval g_q;
Q_p = queue size at end of permitted green period;
Q'_p = queue size at end of permitted green period, adjusted for sneakers;
Q_{ga} = queue size at beginning of protected green (green arrow) period; and
Q_f = queue size at end of interval g_f.

EXHIBIT B16-8. QUEUE ACCUMULATION POLYGON FOR PERMITTED LEFT TURNS FROM EXCLUSIVE LANE



Note:
s_p is saturation flow under permitted operation.

EXHIBIT B16-9. QUEUE ACCUMULATION POLYGON FOR PERMITTED LEFT TURNS FROM SHARED LANE

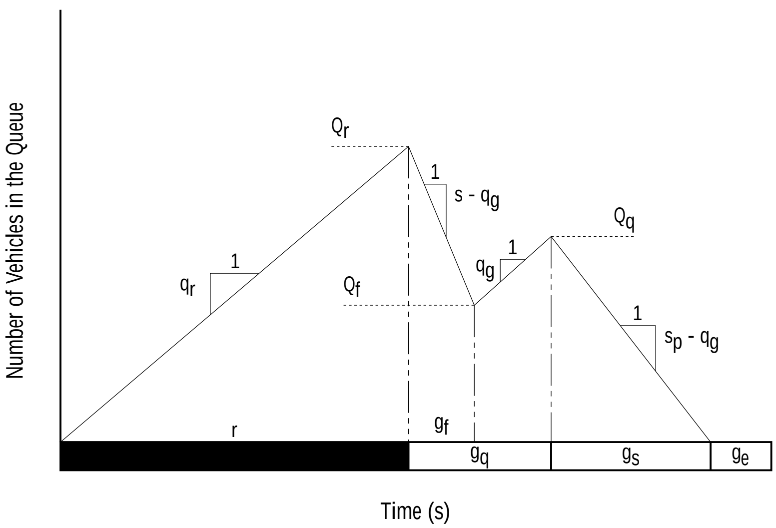


EXHIBIT B16-10. QUEUE ACCUMULATION POLYGON FOR PROTECTED-PLUS-PERMITTED LEFT-TURN PHASING WITH EXCLUSIVE LEFT-TURN LANE

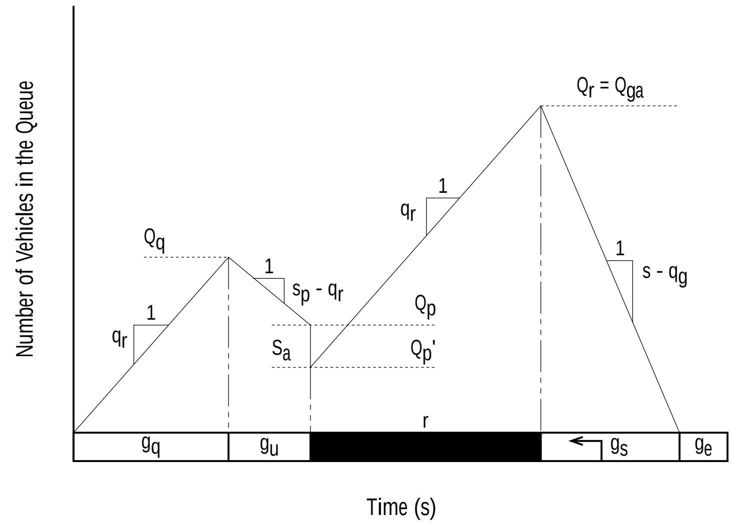
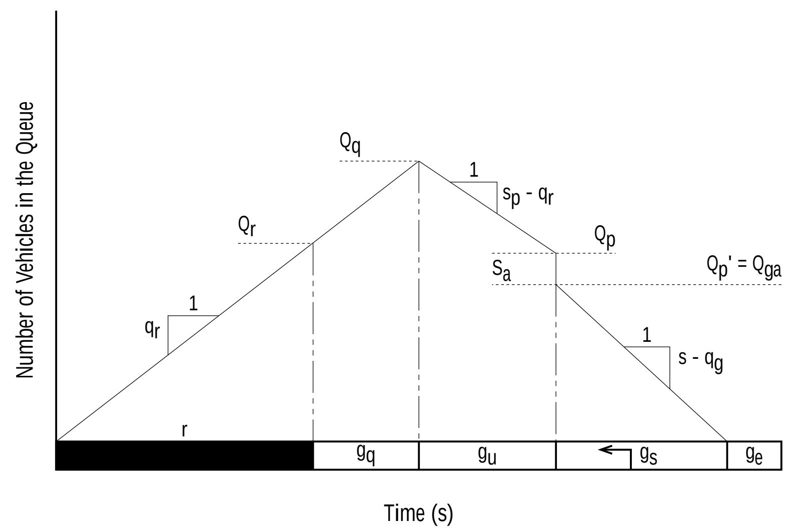


EXHIBIT B16-11. QUEUE ACCUMULATION POLYGON FOR PERMITTED-PLUS-PROTECTED LEFT-TURN PHASING WITH EXCLUSIVE LEFT-TURN LANE



The shape of each QAP is based on termination by a gap that exceeds the unit extension, allowing the full extension time, g_e, to be displayed. When a phase terminates on the maximum green time, the extension time may be reduced or eliminated. If a permitted left-turn phase terminates before the queue has been served, a maximum of two sneakers will be discharged from the queue at that point.

These extensions to the QAP analysis will accommodate all of the practical conditions covered by the methodology presented in the body of this chapter. The remaining issue to be dealt with is the synthesis of the complete cycle by combining critical phases in a dual-ring operation. This procedure may be carried out using the worksheet shown in Exhibit B16-12. The structure of this worksheet is compatible with the dual-ring concurrent phasing depicted in Exhibit B16-2.

EXHIBIT B16-12. TRAFFIC-ACTUATED TIMING COMPUTATIONS

	East-West Movements					North-South Movements				
	A		B		Total	A		B		Total
Ring 1: Phases reversed?	No	Yes	No	Yes		No	Yes	No	Yes	
Movements	WBL	EBT	EBT	WBL		SBL	NBT	NBT	SBL	
Phase times										
Ring 2: Phases reversed?	No	Yes	No	Yes		No	Yes	No	Yes	
Movements	WBL	EBT	EBT	WBL		SBL	NBT	NBT	SBL	
Phase times										
Difference: ABS (Ring 1 - Ring 2)										
Cycle time components: Independent termination										
Simultaneous termination										

The east-west movements (left side of the barrier) are shown in the first three columns. The first column, labeled A, represents the first phase (1 or 5) in each ring. The second column represents the second phase (2 or 6). The third column will contain the total of the phase times. The same format is repeated in the next three columns for the north-south movements (right side of the barrier).

There are three rows for each of the two rings. The first row indicates whether the phase pair is reversed. This information was entered on the input data worksheet presented earlier in this appendix. The next two rows give the movements and phase times for their respective phases. If the phases are not reversed, the assignments will be shown in the dual-ring configuration of Exhibit B16-2. If they are reversed, the through movements will appear in Column A and the left turns in Column B. Note that the movements in a given phase pair cannot be interchanged if the left turn is not protected. The order of the phases in the pair does not affect the total phase time entered in the Total column.

The next row contains the absolute value of the phase time difference between the two rings. Values are entered for each of the six columns.

The components of the cycle time must now be determined and entered in the cycle time components row. The procedure will depend on whether the first phase termination is simultaneous or independent. For simultaneous termination, enter the maximum value of each phase in the A and B columns (Ring 1 or 2, whichever time is greater). For independent termination, enter the maximum value of the total time (A + B) from Ring 1 or 2. So for each side of the barrier, either A and B columns or the Total column will have an entry, but not all three columns. This procedure should be carried out for both sides of the barrier. Remember that the termination treatment may be different on either side. The cycle length may now be determined as the sum of all the entries in the cycle time components row.

If the computed cycle length agrees with the cycle length determined on the previous iteration, no further action is necessary. If not, this timing plan will serve as the starting point for the next iteration.

COORDINATED SEMIACTUATED OPERATION

It is possible that nonactuated phases under semiactuated control may be coordinated with neighboring intersections. In the most common coordination scheme, a background cycle length is imposed. The actuated phases receive their allotment of green time in the usual manner, except that their maximum green times are controlled externally to ensure conformance with the specified cycle length. If the actuated phases require all of their nominal green-time allotment, the intersection operates in a more or less pretimed manner. If they do not, the unused time is reassigned to the coordinated phase.

The analysis of coordinated operation requires another iterative loop, which executes the procedure described in this appendix, adding more green time incrementally to the coordinated phases until the design cycle length has been reached. The result is a timing plan that approximates the operation of the controller in the field.

Iterative procedure

The procedure for timing plan estimation in coordinated systems requires that a design timing plan be established first, with phase splits that add up to the design cycle length. This plan becomes the starting point for the iterative procedure that involves the following steps.

1. Set up the controller timing parameters for the initial timing plan computations. The coordinated major-street phases (usually 2 and 6) should be set for recall to maximum. The maximum green times for all phases should be determined by their respective splits in the pretimed timing plan. No recall modes should be specified for any of the actuated phases.

2. Perform the timing computations to determine the resulting cycle length. If the maximum green times have been specified correctly in Step 1, the computed cycle length will not exceed the specified cycle length.

3. If the computed cycle length is equal to the specified cycle length, there is no green time available for reassignment. In this case the procedure will be complete and the final timing plan will be produced.

4. If the computed cycle length is lower than the specified cycle length, some time should be reassigned to the major-street phases by increasing the maximum green times for the coordinated phases. The recommended procedure is to assign one-half of the difference between the computed cycle and the specified cycle to the coordinated phases. This procedure provides a reasonable speed of convergence without overshooting the specified cycle length.

5. Repeat Steps 2 through 4 iteratively until the computed and specified cycle lengths converge.

MULTIPHASE EXAMPLE

The complete timing estimation procedure described in this appendix will now be applied using a multiphase example. In the discussion of this example presented previously in this appendix, an initial timing plan was developed using the planning procedure. The green times were then modified by trial and error to arrive at a signal timing plan for analyzing capacity, delay, and LOS.

The intersection layout for this example is shown in Exhibit B16-13. Note that all left turns take place from exclusive lanes. The northbound and southbound left turns have protected-plus-permitted phasing. The eastbound and westbound left turns are permitted, with no protected phases.

The timing plan design is as follows:

Phase 1: NB and SB left turns	11 s
Phase 2: NB and SB green	54 s
Phase 3: EB and WB green	30 s
Cycle length	95 s

This timing plan was shown to accommodate all movements with no oversaturation. The average delay per vehicle for all approaches combined was 23 s.

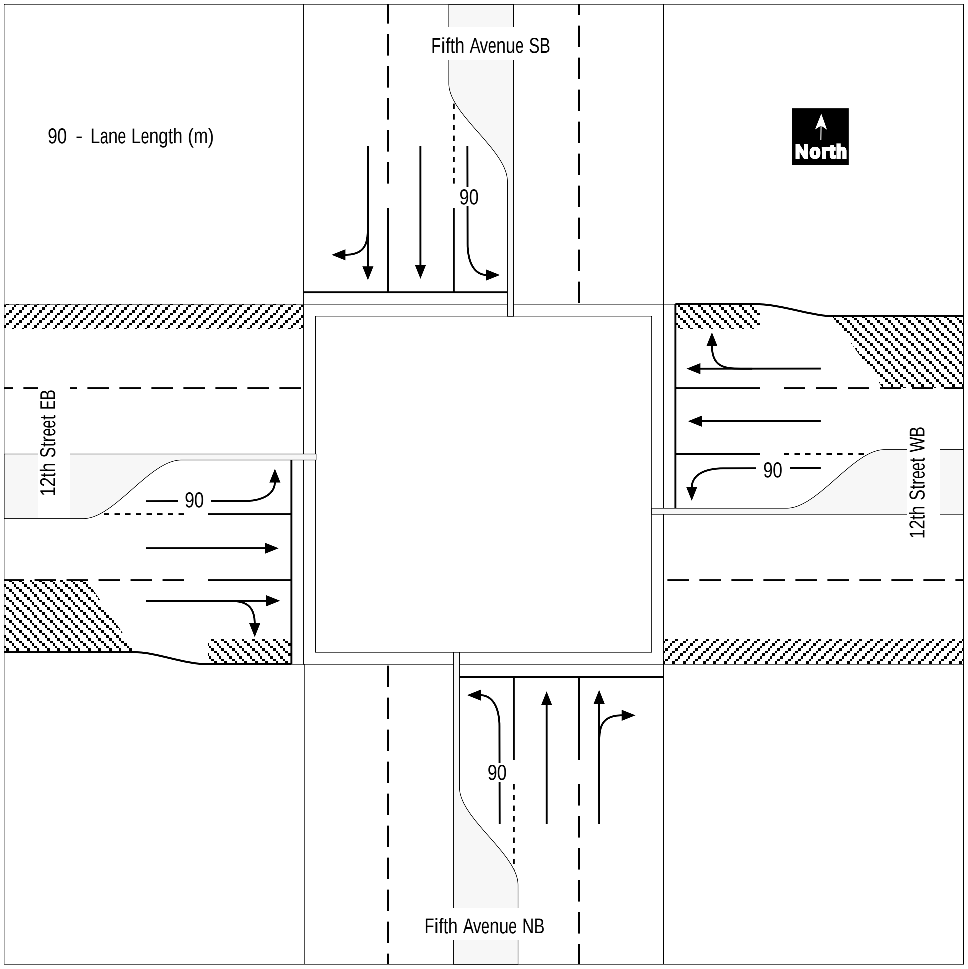
The timing estimation methodology will now be exercised for this example using several different values for some of the actuated controller settings to observe the effect on the results. The detector configuration will use loop detectors 9.1 m long and positioned at the stop line with no setback. A 50-km/h speed will be assumed for all approaches. Both isolated and coordinated operation will be explored. Different values will be used for the unit extension and maximum green settings. Three unit extension settings will be used:

- Short values of 1.5 s and 2.0 s for two-lane and one-lane approaches, respectively, representing a condition sometimes referred to as snappy operation.

• Medium values of 2.5 s and 3.5 s for two-lane approaches and one-lane approaches, respectively, the standard condition for most intersections.

• Long values of 3.5 s and 4.5 s for two-lane approaches and one-lane approaches, respectively, representing a condition sometimes referred to as sluggish operation.

EXHIBIT B16-13. INTERSECTION LAYOUT FOR MULTIPHASE EXAMPLE



These values represent the actual gap between vehicles that will cause a phase to terminate. With the assumed approach speed and detector configuration, each vehicle (assumed length is 5.5 m) passing over the loop will occupy the detector for an additional 1.08 s.

Three different values for maximum green times will also be investigated. The first will use very long maximum times (120 s for each phase) to determine how the intersection would operate if most phases terminated on the unit extension. The second will use maximum times that are proportional to the design times for each phase. It has been proposed that the maximum extensions be set at 125 and 150 percent of the design green times. A more complex scheme has been proposed that results in maximum times in the range of 150 to 200 percent of the design times. For purposes of this exercise, the maximum times will be set at 150 percent of the design times, representing a value somewhere in the middle of the range suggested in the literature. The third value will use the actual design green times as maximums to constrain the operation to its original design.

Recognizing that much of the benefit of traffic-actuated control is derived from the ability of the controller to respond to fluctuations in traffic volumes throughout the day, the operation will also be examined by using volume levels of 70 percent of the peak-hour levels reflected in the original data. Coordinated operation will also be examined at the reduced volume levels to observe the reassignment of unused green time from the actuated phases to the nonactuated phases to reduce the delay to arterial traffic.

The first conditions to be analyzed involve 100 percent volume levels, 120-s maximum green time on each phase, and short unit extensions. The input data worksheet for this example is shown in Exhibit B16-14. The resulting cycle length is 258.2 s. Because of the dual-ring operation, an overlap phase for the north-south approaches appears in the results. The estimated average phase times are as follows:

NB and SB left	17.2 s
SB through and left	15.0 s
NB and SB green	144.0 s
EB and WB green	82.0 s
Cycle length	258.2 s

EXHIBIT B16-14. TRAFFIC-ACTUATED CONTROL DATA FOR MULTIPHASE EXAMPLE

Traffic-Actuated Control Input Data Worksheet
Example Problem 3: Fifth Avenue at 12th Street

Approach Data	Northbound	Southbound	Eastbound	Westbound
LT treatment	3	3	1	1
LT position	Lead	Lead	N/A	N/A
Sneakers	2.0	2.0	2.0	2.0
Free queue	0.00	0.00	0.00	0.00
Speed	50	50	50	50
Termination	--Independent--		--N/A--	

	Ring 1				Ring 2			
Phase Data	1 (WBL)	2 (EBT)	3 (NBL)	4 (SBT)	5 (EBL)	6 (WBT)	7 (SBL)	8 (NBT)
Phase type	X	G	L	G	X	G	L	G
PH reversal?		No	No	No		No	No	No
Det length		30	30	30		30	30	30
Setback		0	0	0		0	0	0
Max initial		16	4	16		16	4	16
Min gap		1.5	2.0	1.5		1.5	2.0	1.5
Walk + FDW		22	0	22		22	0	22
Max green		120	120	120		120	120	120
Change + clr		4.0	4.0	4.0		4.0	4.0	4.0
Recall mode		N	N	N		N	N	N
Veh min		22	10	22		22	10	22
Min initial		16.5	4.0	16.5		16.5	4.0	16.5
Max PH time		124	124	124		124	124	124

This timing plan, when analyzed by the procedure described in the body of this chapter, produces results shown on the LOS worksheet in Exhibit B16-15. Note that the average delay per vehicle is 47.4 s, which is considerably higher than the 23-s/veh delay associated with the timing plan developed in Example Problem 3. The logical conclusion here is that the peak-hour volumes cannot be handled in an optimal manner by a fully actuated controller without some influence being exerted on the timing plan through maximum green constraints.

EXHIBIT B16-15. LOS RESULTS FOR MULTIPHASE EXAMPLE

LOS Worksheet											
Direct/ Ln Grp	v/c Ratio	g/C Ratio	Delay d_1	Del Adj Fact	Lane Group Cap	Calib d_2	Delay d_2	Lane Grp Delay	Lane Grp LOS	Delay by App	LOS by App
NB											
L	0.735	0.118	20.6	0.850	181	16	9.6	27.1	D	58.6	E
TR	1.026	0.508	44.5	0.850	1689	16	23.2	61.0	F		
SB											
L	1.866	0.256	66.3	0.850	224	16	19.3	75.6	F	29.4	D
TR	0.535	0.571	23.9	0.850	1890	16	0.2	20.6	C		
EB											
L	0.884	0.332	57.1	0.850	80	16	43.1	91.7	F	48.4	E
TR	0.478	0.332	48.0	0.850	888	16	0.3	41.1	E		
WB											
L	0.773	0.332	54.3	0.850	153	16	14.1	60.3	F	47.7	E
TR	0.666	0.332	51.8	0.850	937	16	1.3	45.3	E		
Intersection Delay = 47.4 s/veh							Intersection LOS = D				

A total of 10 alternatives, similar in concept to the one just described, were analyzed using combinations of these conditions. For each analysis, the average phase times and cycle length were recorded along with the average delay per vehicle and any movements that were oversaturated. The results are summarized in Exhibit B16-16.

Ten alternatives were analyzed (see Exhibit B16-16)

EXHIBIT B16-16. COMPARISON OF TRAFFIC-ACTUATED CONTROLLER SETTINGS FOR MULTIPHASE EXAMPLE

Volume Level	Maximum Green Time (s)	Gap (s)	Estimated Phase Times (s): NSL	SBT + L	NSG	EWG	Cycle	Average Delay (s/veh)	Comments
100%	Set to 120 s for all phases to eliminate maximum green constraints	Short	17.2	15.0	124.0	82.0	238.2	47.4	NB v/c = 1.02
		Med	19.2	16.3	124.0	85.3	244.8	51.6	NB v/c = 1.04
		Long	20.8	17.4	124.0	88.7	250.9	56.6	NB v/c = 1.07
	Set to 150% of the design splits indicated in Example Problem 3	Med	13.4	1.6	79.0	43.0	137.0	28.9	EB v/c = 1.07
	Set to 100% of the design splits indicated in Example Problem 3	Med	10.2	0.8	54.0	30.0	95	22.9	Same as in Example Problem 3 results
70%	Set to 120 s for all phases to eliminate maximum green constraints	Short	6.8	1.8	31.7	22.0	62.3	10.2	
		Med	7.5	2.1	35.4	22.0	67.0	10.2	
		Long	8.4	2.4	41.7	24.1	76.6	10.6	
	Set to 150% of the design splits indicated in Example Problem 3 (no overlap phase)	Med	10.3		60.0	25.2	95.5	11.9	North-south phases coordinated; east-west phases coordinated
		Med	12.4		45.4	36.0	93.8	14.4	

It is clearly essential that some maximum green times must be imposed to control the apportionment of time between the competing phases

The interpretation is that when traffic volumes are close to capacity, as they are in this example, the maximum green times must be used to apportion the time among the competing phases

In the first three alternatives, only the unit extensions (short, medium, and long) were changed. Note that the cycle length increased with the unit extensions from 238 s (short) to 251 s (long). In all cases, the northbound green phase reached its maximum of 124 s (i.e., 120 s green plus 4 s intergreen). Because the northbound phase was already at its maximum length, it lost time proportionally as the other phases increased and therefore became more oversaturated. It is clearly essential that some maximum green times must be imposed to control the apportionment of time between the competing phases.

The next two analyses used the medium extension times and evaluated both of the strategies for setting the maximum green intervals in proportion to their design values. The average delay was reduced to 29 s/veh for the 150 percent strategy and to 23 s/veh for the 100 percent strategy. The 150 percent strategy produced slightly oversaturated conditions for the eastbound approach. Note that the results of the 100 percent strategy were identical to the results reported in the discussion of the design timing plan for Example Problem 3 in the body of this chapter. The interpretation is that when traffic volumes are close to capacity, as they are in this example, the maximum green times must be used to apportion the time among the competing phases. The gap termination tactic does not ensure a satisfactory distribution of green times.

One of the major benefits of traffic-actuated control is its ability to respond to short-term and daily fluctuations in traffic volumes. To illustrate this principle, volumes are reduced across the board to 70 percent of their peak values. The analysis using 120-s maximum green times on all phases is repeated with the three levels of unit extension. The results indicate that the cycle length varies from 62.3 to 76.6 s throughout the full range of unit extension settings. The average delays are almost identical for these three cases, varying from 10.2 to 10.6 s/veh. This result indicates that when the traffic volumes drop below their saturation levels, it is no longer necessary to control the distribution of green times using the maximum green settings. An important observation here is that the same controller settings would be able to control both the full and reduced volume settings effectively provided that the maximum greens are optimized for the high-volume conditions.

The final two cases deal with coordinated operation. Each of the intersecting routes is assumed to be coordinated in separate cases. The design timing plan, based on a 95-s cycle, is used to establish nominal splits for the coordinated operation. The medium unit extensions are used in combination with maximum green times that reflect 150 percent of the design timing plan. Because the design timing plan includes no overlap, the input data values were adjusted to require simultaneous termination of the first (i.e., left-turn) phases that accommodate the northbound and southbound traffic as opposed to independent termination.

The results indicate that the cycle length established by the iterative computations falls within 1.0 s of the design cycle length. In the case involving east-west coordination, the east-west phases receive more than their nominal allotment of time at the expense of the north-south movements. The reverse is true when coordination is established for the north-south approaches. The delay is also reduced on the approaches that are coordinated and increased on those that are not.

LIMITATIONS OF TRAFFIC-ACTUATED TIMING ESTIMATION PROCEDURE

Appendix B provides a reasonable approximation of traffic-actuated operation

The traffic-actuated estimation procedure described in this appendix provides a reasonable approximation of the operation of a traffic-actuated controller for nearly all of the conditions encountered in practice. The results obtained from this procedure have correlated well with extensive simulation data and with limited field studies (*7*). However, the procedure involves a deterministic analytical representation of an extremely complex stochastic process and therefore has some limitations that must be recognized.

Some of the limitations result from unique situations that cannot be modeled analytically in a satisfactory manner. One example is the case of compound left-turn protection with opposing shared lanes for left turns and through movements. The chapter methodology deals with this as a separate case and applies an empirical treatment to determine the saturation flow adjustment factor for left turns. Simulation provides the only effective way to estimate the timing plan parameters for this case.

The sample problem presented in this appendix demonstrates the sensitivity of the procedure to the unit extension times set in the controller. As expected, longer unit extension times produce longer average green times except when constrained by the maximum green-time settings. Shorter extension times have the opposite effect. There is, however, a lower limit to the range of unit extension times that can be modeled realistically. It is well known in practice that when the unit extension times are too short, premature terminations of a phase may result because of anomalies in the departure headways created primarily by lapses in driver attention. The traffic-actuated control model described in this appendix assumes a constant departure headway and therefore does not reflect this phenomenon. Simulation models introduce a stochastic element into the departure headways based on a theoretical distribution. They are therefore able to invoke premature phase terminations to some extent, but they do not deal with anomalous driver behavior.

As a practical matter, unit extensions should reflect headways at least 50 percent longer than the expected departure headways. For example, assuming a 2-s average departure headway, unit extensions should accommodate up to a 3-s departure headway without terminating the phase. Assuming a detector occupancy time of 1 s, this implies a 2-s gap. The minimum practical value for the unit extension is 2 s. Smaller values may be appropriate in multiple-lane cases in which the average departure headways are shorter.

The analysis of permitted left turns from shared lanes always poses special problems. The semi-empirical treatment prescribed for shared-lane permitted left turns in the body of the chapter does not lend itself to the iterative timing estimation procedure described in this appendix. An analytical approximation of the shared-lane model was therefore substituted to ensure stable convergence of the solution. This produces timing plans that agree well with simulation results; however, the analysis of delay resulting from the timing plan will not always agree with the results of the chapter method. It appears that an iterative method of achieving equilibrium between the shared lane and the adjacent through lanes in the chapter methodology is a prerequisite to the development of a satisfactory timing estimation procedure.

When traffic volumes are extremely low, the timing plan becomes somewhat of an obstruction unless the recall function is used for each phase. In the absence of demand, the green indication rests on the phase that received the latest demand and may do so for several minutes. This operation implies that very long red times will be displayed on some phases; however, no delay will be associated with these red times because no vehicles will be affected. The procedure described in this appendix will compute very short equivalent red times for these phases in an attempt to provide a signal timing plan that will produce realistic delays.

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APPENDIX C. LEFT-TURN ADJUSTMENT FACTORS FOR PERMITTED PHASING

The left-turn adjustment factors in Exhibit C16-1 reflect different cases under which turns may be made.

EXHIBIT C16-1. ADJUSTMENT FACTORS FOR LEFT TURNS (f_{LT})

Case	Type of Lane Group	Left-Turn Factor, f_{LT}
1	Exclusive LT lane; protected phasing	0.95
2	Exclusive LT lane; permitted phasing	Special procedure; see worksheet in Exhibit C16-9 or C16-10
3	Exclusive LT lane; protected-plus-permitted phasing	Apply Case 1 to protected phase; apply Case 2 to permitted phase
4	Shared LT lane; protected phasing	$f_{LT} = 1.0/(1.0 + 0.05\ P_{LT})$
5	Shared LT lane; permitted phasing	Special procedure; see worksheet in Exhibit C16-9 or C16-10
6	Shared LT lane; protected-plus-permitted phasing	Apply Case 4 to protected phase; apply Case 5 to permitted phase

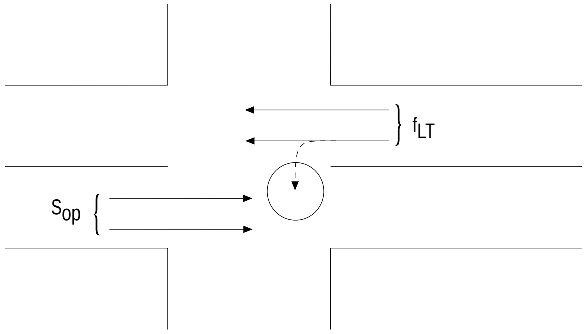
When permitted left turns exist, either from shared lanes or from exclusive lanes, their impact on intersection operations is complex. The procedure outlined in Appendix C is applied to Cases 2, 3, 5, and 6 discussed earlier in this chapter. The basic case for which this model was developed is one in which there are simple permitted left turns from either exclusive or shared lanes. This case does not consider the complications of protected-plus-permitted phasing nor cases in which an opposing leading left phase may exist.

Consider Exhibit C16-2, which shows a permitted left turn being made from a shared lane group. When the green is initiated, the opposing queue begins to move. While the opposing queue clears, left turns from the subject direction are effectively blocked. The portion of effective green blocked by the clearance of an opposing queue of vehicles is designated g_q. During this time, the shared lane from which subject left turns are made is blocked when the left-turning vehicle arrives. Until the first left-turning vehicle arrives, however, the shared lane is unaffected by left turners. The portion of the effective green until the arrival of the first left-turning vehicle is designated g_f.

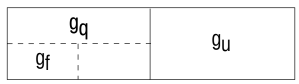
The opposing queue must clear before left turns can begin filtering through

Once the opposing queue of vehicles clears, subject left-turning vehicles filter through an unsaturated opposing flow at a rate affected by the magnitude of the opposing flow. The portion of the effective green during which left turns filter through the opposing flow is designated g_u.

EXHIBIT C16-2. PERMITTED LEFT TURN FROM SHARED LANE



Portions of the Green Phase



This division of the effective green phase for permitted left turns creates up to three distinct periods for which the impact of left turns on a shared or exclusive left-turn lane must be considered.

- g_f: Until the arrival of the first left-turning vehicle, a shared lane is unaffected by left turns. During this period of time, the effective left-turn adjustment factor is logically 1.0 because no left turns are present. By definition, g_f = 0.0 for exclusive permitted left-turn lanes because it is assumed that a queue of left turners is present at the beginning of the phase.
- $g_q - g_f$: If the first left-turning vehicle arrives before the opposing queue clears, it waits until the opposing queue clears, blocking the shared lane, and then seeks a gap in the unsaturated opposing flow that follows. During this period of time, there is effectively no movement in the shared lane, and the left-turn adjustment factor (f_{LT}) applied to the shared lane is logically 0.0.

When the first left-turning vehicle arrives after the opposing queue clears, this period of time does not exist; that is, $g_q - g_f$ has a practical minimum value of zero. The value of g_q has a practical range of 0.0 to g.

- g_u: After the opposing queue clears, left-turning vehicles select gaps through the unsaturated opposing flow. These gaps occur at a reduced rate because of the interference of opposing vehicles and the effect of this interference on other vehicles in the shared lane from which left turns are made. For this period, Exhibit C16-3 is used to assign E_{L1} through-car equivalents for each left-turning vehicle. Then an adjustment factor can be computed for this period by using Equation C16-1:

$$\frac{1}{\left[1.0 + P_L(E_{L1} - 1)\right]} \tag{C16-1}$$

where

P_L = proportion of left-turning vehicles in shared lane, and
E_{L1} = through-car equivalent for permitted left turns (veh/h/ln).

For exclusive permitted left-turn lanes, P_L = 1.0. On the basis of this conception of permitted left-turn operations, the left-turn adjustment factor for the lane from which permitted left turns are made can be computed by Equation C16-2.

$$f_m = \left(\frac{g_f}{g}\right)(1.0) + \left[\frac{g_q - g_f}{g}\right](0.0) + \left(\frac{g_u}{g}\right)\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right] \tag{C16-2}$$

EXHIBIT C16-3. THROUGH-CAR EQUIVALENTS, E_{L1}, FOR PERMITTED LEFT TURNS

Type of Left-Turn Lane	Effective Opposing Flow, $v_{oe} = v_o/f_{LUo}$						
	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.

$E_{L1} = s_{HT}/s_{LT} - 1$ (shared)

$E_{L1} = s_{HT}/s_{LT}$ (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)

A reduced form is given as Equation C16-3.

$$f_m = \left(\frac{g_f}{g}\right) + \left(\frac{g_u}{g}\right)\left[\frac{1}{1 + P_L (E_{L1} - 1)}\right] \quad \text{(C16-3)}$$

Sneakers are considered to be part of the left-turn count for the corresponding green interval, even though they may enter and depart the intersection during the yellow or red intervals

Note that there is no term in this formulation to account for sneakers, that is, for drivers completing left turns during the effective red portion of the clearance-and-change interval. This term is missing because in saturation flow rate measurements, vehicles are counted when they enter the intersection, not when they leave it. However, there is a practical minimum number of left turns that will be made on any phase, defined by sneakers.

To account for this situation, a practical minimum value must be imposed on f_m. One sneaker per cycle may be assumed as a minimum. The probability that a second sneaker will be in position at the end of the green phase will be equal to the proportion of left turns in the shared lane, P_L. The estimated number of sneakers per cycle may therefore be computed as $(1 + P_L)$. Assuming an approximate average headway of 2 s per vehicle in an exclusive lane on a protected phase, the practical minimum value of f_m may be estimated as $2(1 + P_L)/g$.

MULTILANE APPROACH WITH OPPOSING MULTILANE APPROACHES

Multilane approaches are legs of intersections with more than one approach lane

For multilane approaches, the impact of left turns on a shared lane must be extended to include their impact on the entire lane group. One might simply assume that the factor for the shared lane is f_m and that the factor for each other lane in the group is 1.0. In this assumption, however, left turns affect only the lane from which they are made. This is incorrect because vehicles typically maneuver from lane to lane to avoid left-turn congestion. Equation C16-4 provides the relationship for computing realistic f_{LT} values.

$$f_{LT} = \frac{f_m + 0.91(N - 1)}{N} \quad \text{(C16-4)}$$

where

f_{LT} = left-turn adjustment factor applied to a total lane group from which left turns are made, and

f_m = left-turn adjustment factor applied only to lane from which left turns are made.

Regression equations are used to estimate how effective green for a phase will be divided into its three parts

When a single (or double) exclusive-permitted left-turn lane is involved, $f_{LT} = f_m$.

To implement this model, it is necessary to estimate the subportions of the effective green phase, g_f, g_q, and g_u. Regression relationships have been developed to permit this estimation, as follows:

1. Compute g_f:

$$g_f = Ge^{-0.882\,LTC^{0.717}} - t_L \quad \textit{(shared permitted left-turn lanes) } 0 \le g_f \le g \qquad \text{(C16-5)}$$

$$g_f = 0.0 \textit{ (exclusive-permitted left-turn lanes)}$$

where

G	=	actual green time for permitted phase (s);
LTC	=	left turns per cycle, computed as $v_{LT}C/3600$;
v_{LT}	=	adjusted left-turn flow rate (veh/h);
C	=	cycle length (s); and
t_L	=	lost time for subject left-turn lane group (s).

2. Compute g_q:

$$g_q = \frac{v_{olc} qr_o}{0.5 - \dfrac{v_{olc}(1 - qr_o)}{g_o}} - t_L \qquad \text{(C16-6)}$$

$$\frac{v_{olc}(1 - qr_o)}{g_o} \le 0.49$$

$$0.0 \le g_q \le g$$

where

v_{olc}	=	adjusted opposing flow rate per lane per cycle, computed as $v_oC/(3600N_o\, f_{LUo})$;
v_o	=	adjusted opposing flow rate (veh/h);
f_{LUo}	=	lane utilization factor for opposing flow;
N_o	=	number of opposing lanes;
qr_o	=	opposing queue ratio, that is, proportion of opposing flow rate originating in opposing queues, computed as $1 - R_{po}(g_o/C)$; $qr_o \ge 0$;
R_{po}	=	platoon ratio for opposing flow, obtained from Exhibit 16-12 based on opposing arrival type;
g_o	=	effective green for opposing flow (s); and
t_L	=	lost time for opposing lane group (s).

3. Compute g_u:

$$g_u = g - g_q \;\; \textit{when } g_q \ge g_f$$

$$g_u = g - g_f \;\; \textit{when } g_q < g_f$$

where

g	=	effective green time for subject permitted left turn (s).

Note that when $g_q < g_f$, that is, when the first left-turning vehicle does not arrive until after the opposing queue clears, an effective adjustment factor of 1.0 is applied throughout g_f and a factor based on E_{L1} thereafter.

4. Select the appropriate value of E_{L1} from Exhibit C16-3 on the basis of the opposing flow rate, v_o, and the lane utilization adjustment factor of the opposing flow, f_{LUo}. For the purposes of determining v_o, opposing right and left turns from exclusive lanes are not included in v_o.

5. Compute P_L (proportion of left turns in shared lane):

$$P_L = P_{LT}\left[1 + \frac{(N-1)g}{g_f + \frac{g_u}{E_{L1}} + 4.24}\right] \quad \text{(C16-7)}$$

where

P_{LT} = proportion of left turns in lane group, and
N = number of lanes in lane group.

Note that when an exclusive-permitted left-turn lane is involved, $P_L = P_{LT} = 1.0$.

6. Compute f_m using Equation C16-3.
7. Compute f_{LT} using Equation C16-4.

SINGLE-LANE APPROACH OPPOSED BY SINGLE-LANE APPROACH

The case of a single-lane approach opposed by another single-lane approach has a number of unique features that must be reflected in the model. The most critical of these is the effect of opposing left turns. An opposing left-turning vehicle in effect creates a gap in the opposing flow through which a subject left turn may be made. This gap can occur during the clearance of the opposing queue as well as during the unsaturated portion of the green phase.

Single-lane approaches are legs of intersections with only one approach lane

Thus, the assumption in the multilane model that there is no flow during the period $g_q - g_f$ (where $g_q > g_f$) is not applicable to opposing single-lane approaches, on which there is flow during this period at a reduced rate reflecting the blocking effect of left-turning vehicles as they await an opposing left turn. Left-turning vehicles during the period $g_q - g_f$ are assigned a through-car equivalent value E_{L2} based on simple queuing analysis, which can be converted to an adjustment factor for application during this period of the green.

Since vehicles do not have the flexibility to choose lanes on a single-lane approach, regression relationships for predicting g_f and g_q are also different from those for the multilane case. Further, for a single-lane approach, $f_{LT} = f_m$, and $P_L = P_{LT}$. As in the multilane case, the opposing single-lane model has no term to account for sneakers but has a practical minimum value of $f_{LT} = 2(1 + P_{LT})/g$.

The basic model of left-turn lanes with opposing single-lane approaches is defined by Equations C16-8 and C16-9:

$$f_{LT} = f_m = \left(\frac{g_f}{g}\right)(1.0) + \left(\frac{g_{diff}}{g}\right)\left[\frac{1}{1 + P_{LT}(E_{L2} - 1)}\right] + \left(\frac{g_u}{g}\right)\left[\frac{1}{1 + P_{LT}(E_{L1} - 1)}\right] \quad \text{(C16-8)}$$

$$f_{LT} = \left(\frac{g_f}{g}\right) + \left(\frac{g_{diff}}{g}\right)\left[\frac{1}{1 + P_{LT}(E_{L2} - 1)}\right] + \left(\frac{g_u}{g}\right)\left[\frac{1}{1 + P_{LT}(E_{L1} - 1)}\right] \quad \text{(C16-9)}$$

where

g_{diff} = max $(g_q - g_f, 0)$. Note that when no opposing left turns are present, the value of g_{diff} is set to zero.

To implement this model, it is again necessary to estimate the subportions of the effective green phase, g_f, g_q, and g_u, using Equations C16-10 through C16-12.

1. Compute g_f:

$$g_f = Ge^{(-0.860 LTC^{0.629})} - t_L \qquad \text{when } 0 \le g_f \le g \quad \text{(C16-10)}$$

where

G = actual green time for permitted phase (s);
LTC = left turns per cycle, computed as $v_{LT}C/3600$;
v_{LT} = adjusted left-turn flow rate (veh/h);
C = cycle length (s); and

t_L = lost time for subject left-turn lane group (s).

2. Compute g_q:

$$g_q = 4.943 v_{olc}^{0.762}\, qr_o^{1.061} - t_L \quad \text{when } 0 \le g_q \le g \qquad \text{(C16-11)}$$

where

v_{olc} = adjusted opposing flow rate per lane per cycle, computed as $v_o C/(3600) f_{LUo}$;
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, computed as $1 - R_{po}(g_o/C)$, $qr_o \ge 0$;
R_{po} = platoon ratio for opposing flow, obtained from Exhibit 16-11 on basis of opposing arrival type;
g_o = effective green for opposing flow (s); and
t_L = lost time for opposing lane group (s).

3. Compute g_u:

$$g_u = g - g_q \quad \text{when } g_q \ge g_f$$

$$g_u = g - g_f \quad \text{when } g_q < g_f$$

where

g = effective green time for subject permitted left turn (s).

Note that when $g_q < g_f$, that is, when the first left-turning vehicle does not arrive until after the opposing queue clears, an effective adjustment factor of 1.0 is applied throughout g_f and a factor based on E_{L1} thereafter.

4. Select the appropriate value of E_{L1} from Exhibit C16-3 on the basis of the opposing flow rate, v_o, and the lane utilization adjustment factor of the opposing flow, f_{LUo}.

5. Compute E_{L2}:

$$E_{L2} = \frac{(1 - P_{THo}^{n})}{P_{LTo}} \qquad \text{when } E_{L2} \ge 1.0 \qquad \text{(C16-12)}$$

where

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 $1 - P_{LTo}$; and
n = maximum number of opposing vehicles that could arrive during $g_q - g_f$, computed as $(g_q - g_f)/2$. Note that n is subject to a minimum value of zero.

6. Compute f_{LT} using Equation C16-9.

SPECIAL CASES

Single-lane approach in combination with multilane approach

Two special cases for permitted left turns must be addressed: a single-lane approach opposed by a multilane approach and vice versa. When the subject lane in these cases is the single-lane approach, it is opposed by a multilane opposing flow. Even if the opposing approach is a single through lane and an exclusive left-turn lane, opposing left turns will not open gaps in the opposing flow. Thus, the special structure of the single-lane model does not apply. The multilane model is applied, except that $f_{LT} = f_m$. The value of g_f, however, should be computed using the single-lane equation, $g_f = Ge^{(-0.860 LTC^{0.629})} - t_L$.

When the multilane approach is considered, the reverse is true. The opposing flow is in a single lane, and opposing left turns could open gaps for subject left turners. The single-lane model may be applied, with several notable revisions. The term g_f should be computed using Equation C16-5. P_L must be estimated and substituted for P_{LT} in the single-lane model. P_L may be estimated from P_{LT} using Equation C16-7. Also, f_{LT} does not equal f_m. Thus, the conversion must be made using the multilane equations, except when the subject approach is a double left-turn lane as shown in Equation C16-4.

Worksheets that may be used to assist in implementing the special models for permitted left-turn movements are presented later in this appendix. These worksheets do not account for the modifications that must be made to analyze single-lane approaches opposed by multilane approaches and vice versa.

MORE COMPLEX PHASING WITH PERMITTED LEFT TURNS

The models and worksheets presented in previous sections of Appendix C apply directly to situations in which left turns are made only on permitted phases and in which no protected phases or opposing leading green phases exist. The models may, however, be applied to these more complex cases with some modifications.

In general, protected-plus-permitted phases are analyzed by separating the portions of the phase into two lane groups for the sake of analysis. Each portion of the phase is then handled as if the other were not present. The protected portion of the phase is treated as a protected phase, and a left-turn adjustment factor appropriate to a protected phase is selected. The permitted portion of the phase is treated as a permitted phase, and the special procedures outlined here are used to estimate a left-turn adjustment factor (with modifications as defined in this section).

With the foregoing procedure, separate saturation flow rates may be computed for each portion of the phase. This method does not require that the demand volume for the protected-plus-permitted movement be divided between the two portions of the phase. However, the computation of the critical v/c ratio, X_c, does require this apportionment. The following is a reasonable and conservative approach to apportioning the volumes for purposes of computing X_c.

- The first portion of the phase, whether protected or permitted, is assumed to be fully utilized, that is, to have a v/c of 1.0, unless total demand is insufficient to use the capacity of that portion of the phase.
- Any remaining demand not handled by the first portion of the phase is assigned to the second portion of the phase, whether protected or permitted.

This approach assumes that when the movement is initiated, a queue exists that uses available capacity in the initial portion of the phase. In cases of a failed cycle, the unserved queue will exist after the end of the second portion of the phase, with those vehicles queued and ready to use the initial portion of the phase on the next cycle. In this sense, the initial portion of the movement can never operate at a v/c of more than 1.0.

Modification to account for leading and lagging phases or protected-plus-permitted phasing

In the analysis of the permitted portion of such phases, as well as those with opposing leading protected left-turn phases, the basic models described previously may be applied. The difficulty is in selecting values of G, g, g_f, g_q, and g_u for use in these models. The equation for g_f is indexed to the beginning of effective green in the subject direction, and g_q is indexed to the beginning of the effective green for the opposing flow. When leading or lagging phasing or protected-plus-permitted phasing exists, these equations must be modified to account for shifts in the initiation and overlap of various green times.

Some common examples are shown in Exhibits C16-4 through C16-8. G, g, g_f, and g_q are computed as shown in the models and worksheets. These values are modified as shown and replaced on the worksheets with G*, g*, g_f*, and g_q* for the permitted portion of protected-plus-permitted phasing. This extended notation is required to cover the general case of complex left-turn phasing. In most practical cases, it will not be necessary to use all the superscripted terms.

Exhibit C16-4 shows the standard case. Exhibit C16-5 shows Case 2 with a leading green phase. The equations shown are valid for either exclusive-lane or shared-lane operation, except that g_f is zero by definition for the exclusive-lane case. For exclusive-lane operation, the leading green, G_1, is followed by G/Y_1, a period during which the left-turn change-and-clearance interval is displayed, and the through movement continues with a green indication. G_2 has a green indication for both the through and left-turn movements, followed by a full change-and-clearance interval for all north-south movements, Y_2.

EXHIBIT C16-4. CASE 1 - PERMITTED TURNS: STANDARD CASE

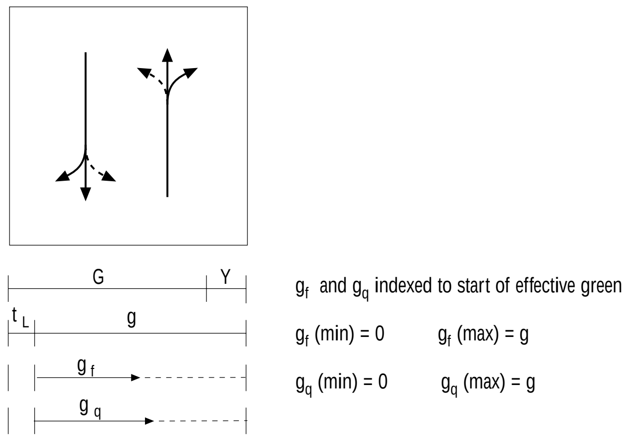


EXHIBIT C16-5. CASE 2 - GREEN-TIME ADJUSTMENTS FOR LEADING GREEN

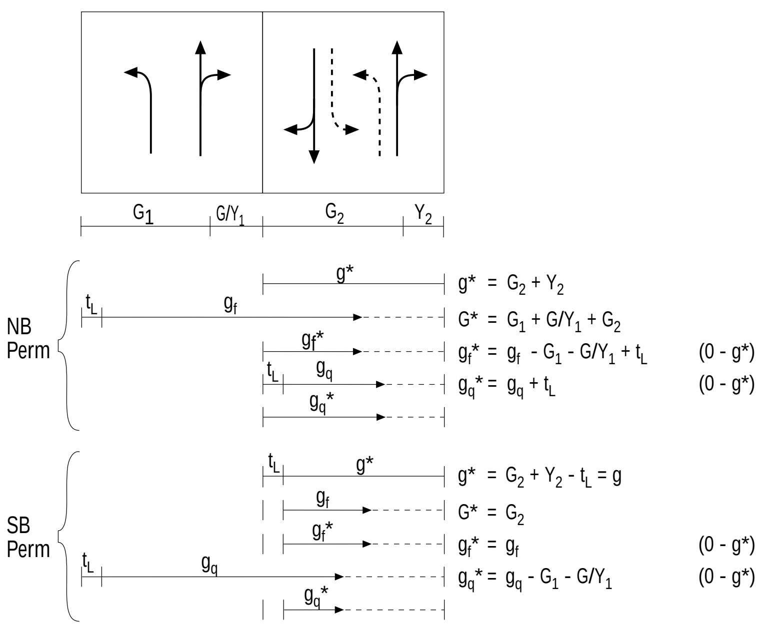


EXHIBIT C16-6. CASE 3 - GREEN-TIME ADJUSTMENTS FOR LAGGING GREEN

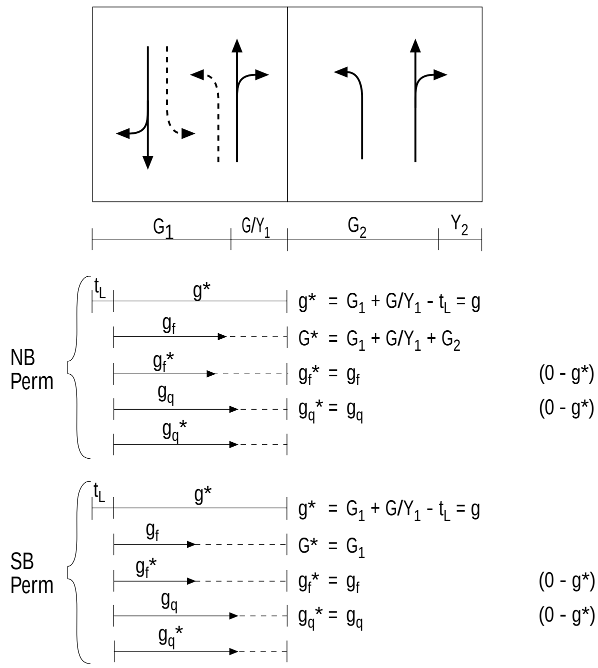


EXHIBIT C16-7. CASE 4 - GREEN-TIME ADJUSTMENTS FOR LEADING AND LAGGING GREEN

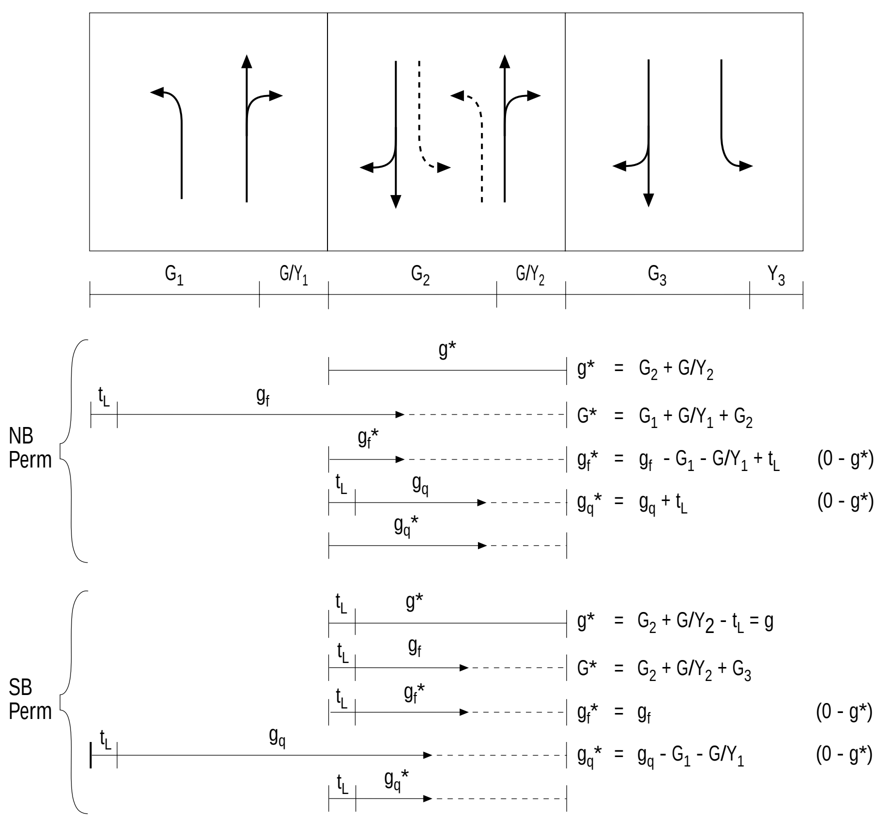
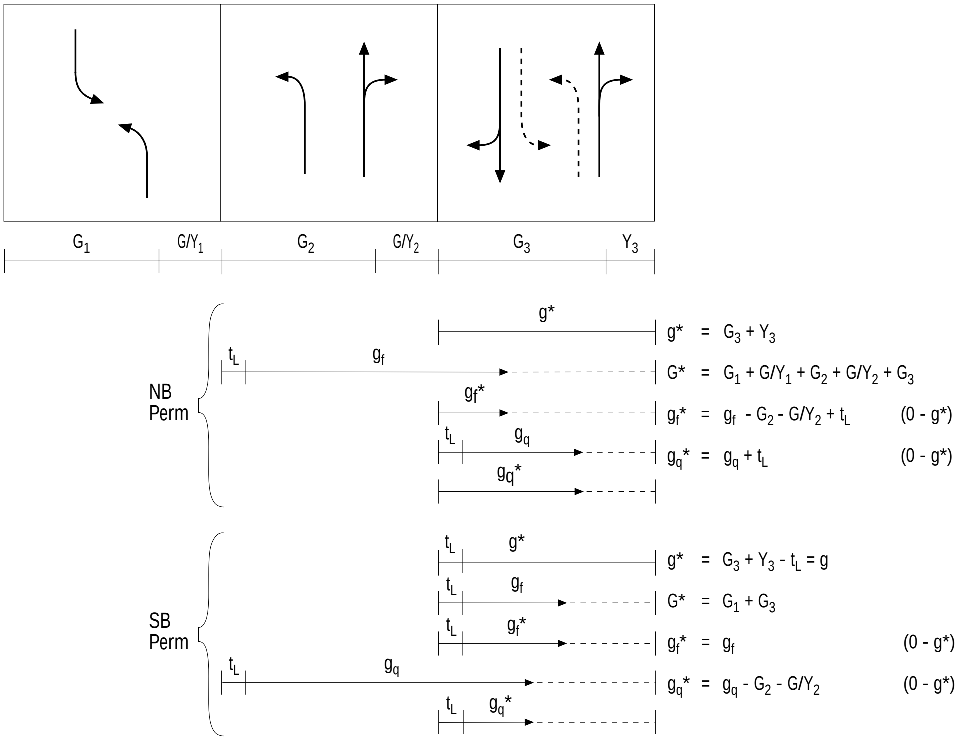


EXHIBIT C16-8. CASE 5 - GREEN-TIME ADJUSTMENTS FOR LT PHASE WITH LEADING GREEN



The effective green time for the permitted phase, g*, is equal to $G_2 + Y_2$ for the NB direction and $G_2 + Y_2 - t_L$ for the SB direction. Note that there is no lost time for the NB movements, since both were initiated in the leading phase, and the lost time is assessed there. Thus, the NB and SB effective green times that must be used are not equal.

For the NB phase, g_f is computed using the total green time for the NB left-turn movement, $G_1 + G/Y_1 + G_2$. The computed value, however, begins with the leading-phase effective green, as shown. The value that needs to be applied to the permitted phase, however, is that portion of g_f that overlaps g*, which results in $g_f^* = g_f - G_1 - G/Y_1 + t_L$. This computation would be done for a shared lane, and the result, g_f^*, would have to be a value between 0 and g*. For an exclusive-lane case, g_f and g_f^* are by definition zero. For the SB phase, g_f as normally computed is the same as g_f^*, and no adjustment is necessary.

For the NB phase, g_q is referenced to the beginning of the opposing (SB) effective green. Again, the value needed is the portion of the NB g* blocked by the clearance of the opposing queue. Because the NB effective green (g*) does not account for lost time, $g_q^* = g_q + t_L$. For the SB phase, the usual computation of g_q is indexed to the start of the opposing (NB) flow, which begins in the leading phase. For analysis of the permitted phase, however, only the portion that blocks the SB permitted effective green is of interest. Thus, $g_q^* = g_q - G_1 - G/Y_1$.

The foregoing discussion is illustrative. The relationship between the normal calculations of g, G, g_f, and g_q and their adjusted counterparts, g*, G*, g_f^*, and g_q^*, is best described by Exhibits C16-5 through C16-8, which may be used in conjunction with the standard worksheets to arrive at the appropriate left-turn adjustment factor for the permitted portion of a protected-plus-permitted phase plan. Obviously, north and south can be reversed or replaced by east and west without any change in the equations shown.

PROCEDURES FOR APPLICATION

The procedure described in this appendix is used at the point in the analysis where the base saturation flow is being adjusted to site conditions. Exhibits C16-9 and C16-10 show worksheets that are used in the computation of the left-turn adjustment factor when permitted left-turn phasing exists. These worksheets are applied to the permitted portion of left turns, including permitted-only and protected-plus-permitted phasing, whether made from an exclusive or shared lane for Cases 2, 3, 5, and 6.

The basic methodology for each worksheet assumes that the subject approach is a multilane approach if the opposing approach is a multilane approach (Exhibit C16-9) and that the subject approach is a single-lane approach if the opposing approach is a single-lane approach (Exhibit C16-10). For cases in which the two approaches are not of the same type as well as for cases of protected-plus-permitted phasing and a phasing in which the opposing through movement has a lead phase, the worksheets may still be used, but the special instructions cited earlier in this appendix must be followed carefully.

There is a column for each approach on the worksheets, although only those approaches with permitted left-turn conditions would be included. Since the worksheets are quite similar, they are discussed together here, with exceptions and differences noted where appropriate.

The first set of entries consists of input variables that should be entered directly from values appearing on previous worksheets.

1. The cycle length is entered from the Input Worksheet.

2. The actual green time for the permitted phase is entered from the Input Worksheet. If the permitted phase is part of a protected-plus-permitted phasing or the opposing approach has a lead phase, see the special instructions earlier in this appendix.

3. The effective green time for the permitted phase is entered. This entry is generally the actual green time from the Input Worksheet plus the yellow-plus-all-red change-and-clearance interval minus the movement's lost time. If the permitted phase is part of a protected-plus-permitted phasing or the opposing approach has a lead phase, see the special instructions in this appendix.

4. The effective green time for the opposing approach is entered for the permitted phase. This entry is generally the actual green time from the Input Worksheet plus the yellow-plus-all-red change-and-clearance interval minus the movement's lost time. If the permitted phase is part of a protected-plus-permitted phasing or the opposing approach has a lead phase, see the special instructions in this appendix.

5. The number of lanes in the subject lane group is entered from the Input Worksheet. If the left turn is opposed by a multilane approach (Exhibit C16-9), the number of lanes in the opposing lane group is entered from the Input Worksheet as well. If left or right turns are made from exclusive turn lanes on the opposing approach, these lanes are not included in the number of opposing lanes.

6. The adjusted left-turn flow rate is entered from the Volume Adjustment and Saturation Flow Rate Worksheet.

7. The proportion of left turns in the lane group is entered from the Volume Adjustment and Saturation Flow Rate Worksheet. When an exclusive left-turn lane group is involved, P_{LT} = 1.0. If the left turn is opposed by a single-lane approach (Exhibit C16-10), the proportion of left turns in the opposing flow is entered from the Volume Adjustment and Saturation Flow Rate Worksheet.

8. The adjusted opposing flow rate is entered from the Volume Adjustment and Saturation Flow Rate Worksheet. If left or right turns are made from exclusive turn lanes on the opposing approach, these adjusted volumes are not included in the opposing flow rate.

9. The lost time for the left-turn lane group is entered as determined from the Input Worksheet.

EXHIBIT C16-9. SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS WHERE APPROACH IS OPPOSED BY MULTILANE APPROACH

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} (refer to Exhibit 16-11)				
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} (refer to Exhibit C16-3)				
$P_L = P_{LT}\left[1 + \frac{(N-1)g}{(g_f + g_u/E_{L1} + 4.24)}\right]$ (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 Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 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 $f_{LT} = f_m$.

EXHIBIT C16-10. SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS WHERE APPROACH IS OPPOSED BY SINGLE-LANE 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} (refer to Exhibit 16-11)				
$g_f = G[e^{-0.860(LTC^{0.629})}] - t_L$ $g_f \le g$ (except exclusive left-turn lanes)[3]				
Opposing queue ratio, $qr_o = \max[1 - R_{po}(g_o/C), 0]$				
$g_q = 4.943 v_{olc}^{0.762} qr_o^{1.061} - t_L$ $g_q \le g$				
$g_u = g - g_q$ 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 Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 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 $g_{diff} = 0$.

The equations used in subsequent computations are shown in the remaining rows of the worksheet. These equations are based on the input variables that have been entered. Some of these computations deserve further discussion, as follows.

- The opposing platoon ratio, R_{po}, may be determined in two ways. If the arrival type of the opposing traffic appears on the Input Worksheet, the default platoon ratio from Exhibit 16-11 is used. If the proportion of arrivals on green appears on the Input Worksheet, Equation 16-1 based on the g/C ratio is used.
- The equation shown for g_f in Exhibit C16-9 assumes that the subject approach is a multilane approach like the opposing approach. If the subject approach is a single-lane approach, the equation for g_f from Exhibit C16-10 should be used. Conversely, the

equation shown for g_f in Exhibit C16-10 assumes that the subject approach is a single-lane approach like the opposing approach. If the subject approach is a multilane approach, the equation for g_f from Exhibit C16-9 should be used. In either case, if the subject lane group is an exclusive left-turn lane, then $g_f = 0$.

• For multilane lane groups (Exhibit C16-9), P_L is computed as the proportion of left turns in the left-hand lane of the lane group. If this value is determined to be 1.0 or higher, the lane groups for the approach should be reassigned showing this left-hand lane as an exclusive left-turn lane (a de facto left-turn lane), since it is occupied entirely by left-turning vehicles. This change requires redoing all of the computations for this approach. If a multilane lane group is opposed by a single-lane approach, Exhibit C16-10 should be used, but a value of P_L should be estimated and substituted for P_{LT}, as described in this appendix. In this case, the same de facto left-turn check should be applied.

• Exhibit C16-3 is used to determine the value of E_{L1} based on the opposing flow rate and the lane utilization factor of the opposing flow. For the single-lane approach (Exhibit C16-4), E_{L2} is computed by formula, not by Exhibit C16-3.

• The value of f_m is computed. The maximum value is 1.0 and the minimum value is $2(1 + P_L)/g$. These limits are used if the computed value falls outside this range.

• The left-turn adjustment factor, f_{LT}, is computed. For a single-lane group, $f_{LT} = f_m$. If a multilane lane group is opposed by a single-lane approach, Exhibit C16-10 is used, but f_{LT} is calculated on the basis of f_m and the number of lanes as shown in Exhibit C16-9 except when the subject lane group contains multiple exclusive left-turn lanes.

APPENDIX D. PEDESTRIAN AND BICYCLE ADJUSTMENT FACTORS

Appendix D provides the procedure used to calculate pedestrian and bicycle adjustment factors for calculating saturation flow of turning vehicles. Exhibit D16-1 shows sample conflict zones where intersection users compete for space. The variables required for input are shown in Exhibit D16-2. These variables are divided into two groups, qualitative and quantitative, and they are displayed in computational order. A flowchart in Exhibit D16-3 illustrates a step-by-step procedure for computation, and a supplemental worksheet is also provided to facilitate the computation.

There are four steps for computing pedestrian-bicycle saturation flow rate adjustment factors.

Step 1: Determine average pedestrian occupancy, OCC_{pedg}

Average pedestrian occupancy is derived from pedestrian volume, v_{ped}. If pedestrian flow rate, v_{pedg}, rather than pedestrian volume, v_{ped}, is collected in the field, average pedestrian occupancy can be calculated directly from pedestrian flow rate. Otherwise, pedestrian flow rate first has to be converted from pedestrian volume using Equation D16-1.

$$v_{pedg} = v_{ped} * (C/g_p) \qquad (v_{pedg} \le 5000) \qquad \text{(D16-1)}$$

Then average pedestrian occupancy can be calculated using Equation D16-2.

$$OCC_{pedg} = v_{pedg}/2000 \qquad (v_{pedg} \le 1000 \text{ and } OCC_{pedg} \le 0.5) \qquad \text{(D16-2)}$$

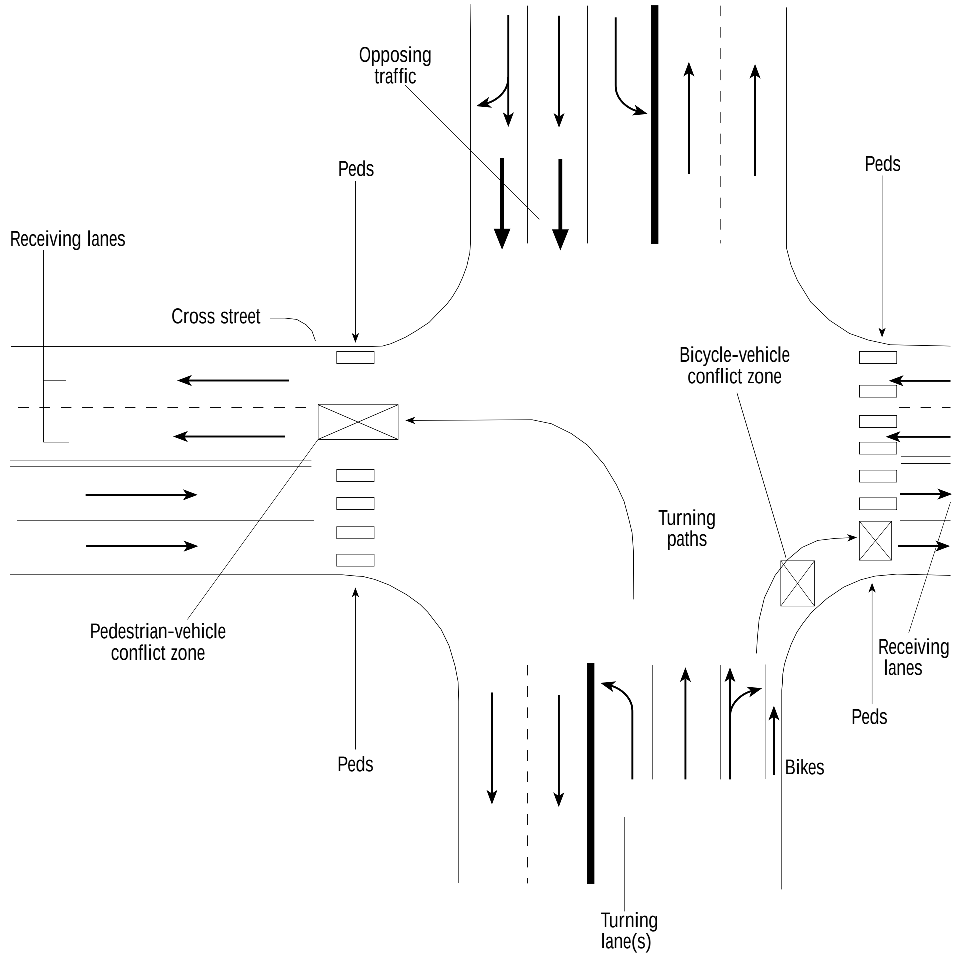
or

$$OCC_{pedg} = 0.4 + v_{pedg}/10{,}000 \qquad (1000 < v_{pedg} \le 5{,}000 \text{ and } 0.5 < OCC_{pedg} \le 0.9)$$

Step 2: Determine relevant conflict zone occupancy, OCC_r

If bicycle traffic weaves with right-turning vehicles in advance of the stop line, the bicycle volume should be ignored in the analysis because this interaction does not take place within the intersection. Only pedestrian interference should be considered.

EXHIBIT D16-1. CONFLICT ZONE LOCATIONS



• For right-turn movements with no bicycle interference or for left-turn movements from a one-way street, Equation D16-3 is used.

$$OCC_r = OCC_{pedg} \tag{D16-3}$$

• For right-turn movements with bicycle interference, bicycle flow rate, v_{bicg}, first has to be converted from bicycle volume, v_{bic}. If bicycle flow rate data are collected in the field, no conversion is needed. The relationship between bicycle flow rate and bicycle volume is given by Equation D16-4.

$$v_{bicg} = v_{bic}(C/g) \qquad (v_{bicg} \leq 1900) \tag{D16-4}$$

The bicycle conflict zone occupancy, OCC_{bicg}, is determined by Equation D16-5.

$$OCC_{bicg} = 0.02 + v_{bicg}/2700 \qquad (v_{bicg} \leq 1900 \text{ and } OCC_{bicg} \leq 0.72) \tag{D16-5}$$

Then the relevant occupancy is determined from combined pedestrian occupancy and bicycle conflict zone occupancy using Equation D16-6.

$$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg}) \tag{D16-6}$$

• For left-turn movements from a two-way street, opposing queue clearing time, g_q, is first compared with pedestrian green, g_p. If $g_q \geq g_p$, then $f_{Lpb} = 1.0$, and the procedure ends here because the opposing queue consumes the entire pedestrian green time. Hence, the adjustment factor is 1.

EXHIBIT D16-2. INPUT VARIABLES

Qualitative Variables	
Turning movements (left-turn or right-turn)	
Street types (one-way or two-way)	
Turn lane types (exclusive or shared)	
Signal phasing types (protected, permitted, or protected-permitted)	
Quantitative Parameters	**Symbols**
Cycle length, s	C
Opposing queue clearing time,[a] s	g_q
Opposing flow rate after queue clears,[a] veh/h	V_o
Number of effective turning lanes	N_{turn}
Number of effective receiving lanes	N_{rec}
Proportion of left-turn volumes[b]	P_{LT}
Proportion of right-turn volumes[b]	P_{RT}
Proportion of left turns using protected phase [c]	P_{LTA}
Proportion of right turns using protected phase[c]	P_{RTA}
Pedestrian volume,[d] p/h	v_{ped}
Pedestrian flow rate,[d] p/h	v_{pedg}
Bicycle volume,[e] bicycles/h	v_{bic}
Bicycle flow rate,[e] bicycles/h	v_{bicg}
Effective green,[e] s	g
Pedestrian Green WALK + flashing DON'T WALK,[f] s	g_p

Notes:
a. Only for left-turn movements from a two-way street.
b. Only for right-turn movements from a single-lane approach or for shared turning lanes.
c. Only for cases with both protected and permitted green phases.
d. Without consideration of noncompliant pedestrians.
e. Only for right-turn movements impeded by bicycles.
f. If pedestrian signal timing is unknown, g_p may be assumed to be equal to g.

Otherwise, pedestrian occupancy after the opposing queue clears, OCC_{pedu}, is determined by Equation D16-7.

$$OCC_{pedu} = OCC_{pedg}[1 - 0.5(g_q/g_p)] \qquad \text{(D16-7)}$$

After the opposing queue clears, left-turning vehicles complete their maneuvers on the basis of accepted gap availability in opposing traffic, V_o. Relevant occupancy is a function of the probability of accepted gap availability and pedestrian occupancy and is computed by Equation D16-8.

$$OCC_r = OCC_{pedu}\left[e^{-(5/3600)v_o}\right] \qquad \text{(D16-8)}$$

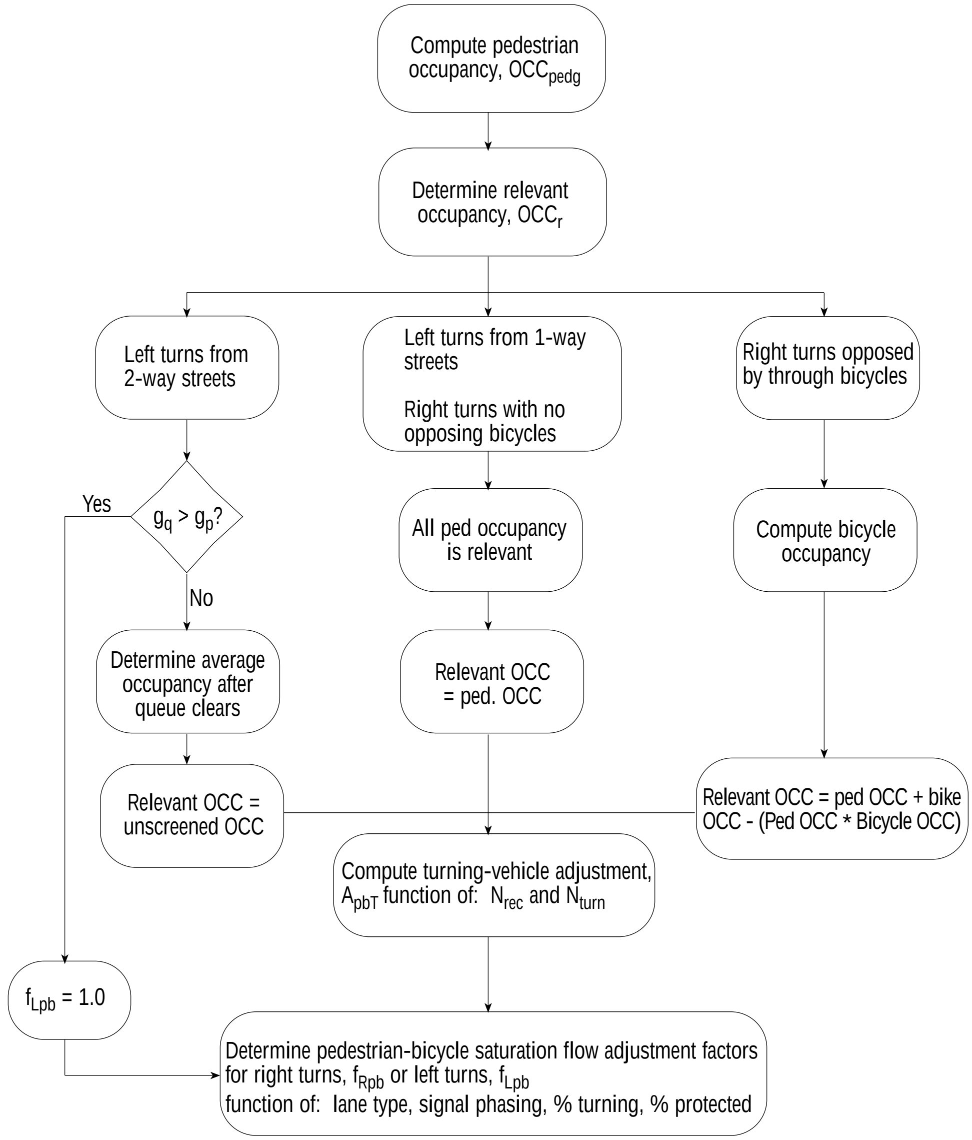
Step 3: Determine permitted phase pedestrian-bicycle adjustment factors for turning movements, A_{pbT}.

The number of turning lanes, N_{turn}, and receiving lanes, N_{rec}, should be determined from field observation rather than on the basis of intersection striping because some vehicles may consistently and deliberately make illegal turns from an outer lane, or sometimes proper turning cannot be executed because the receiving lane is blocked by double-parked vehicles. Two conditions are being considered in this step.

• If the number of cross-street receiving lanes is equal to the number of turning lanes, turning vehicles will not be able to maneuver around pedestrians or bicycles; the adjustment factor is the proportion of time the conflict zone is unoccupied as shown in Equation D16-9.

$$A_{pbT} = 1 - OCC_r \quad (N_{rec} = N_{turn}) \qquad \text{(D16-9)}$$

EXHIBIT D16-3. OUTLINE OF COMPUTATIONAL PROCEDURE FOR f_{Rpb} AND f_{Lpb}



• If the number of cross-street receiving lanes exceeds the number of turning lanes, turning vehicles will more likely maneuver around pedestrians or bicycles and pedestrian-bicycle effects on saturation flow are reduced. The adjustment factor can be calculated using Equation D16-10.

$$A_{pbT} = 1 - 0.6(OCC_r) \quad (N_{rec} > N_{turn}) \tag{D16-10}$$

Step 4: Determine saturation flow adjustment factors for turning movements, f_{Lpb}, for left-turn movements and f_{Rpb} for right-turn movements.

Saturation flow adjustment factors account for pedestrian-bicycle effects on saturation flow for turning vehicles, and the factors are dependent on the proportion of turning traffic using protected phases. The proportion of right-turn movements is roughly equal to the proportion of the protected green phase. The proportion of left-turn movements is approximately equal to [1 – (permitted phase f_{LT})]/0.95.

• For right-turn movements, the pedestrian-bicycle adjustment factor, f_{Rpb}, can be calculated using Equation D16-11.

$$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA}) \quad \text{(D16-11)}$$

• For left-turn movements, the pedestrian adjustment factor, f_{Lpb}, can be calculated using Equation D16-12.

$$f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA}) \quad \text{(D16-12)}$$

The supplemental worksheet for pedestrian-bicycle effects on permitted left turns and right turns is shown in Exhibit D16-4.

EXHIBIT D16-4. SUPPLEMENTAL WORKSHEET FOR PEDESTRIAN-BICYCLE EFFECTS

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 $P_{RTA} = 0$.

APPENDIX E. ESTIMATING UNIFORM CONTROL DELAY (d_1) FOR PROTECTED-PLUS-PERMITTED OPERATION

Left turns from exclusive lanes that are allowed to proceed on both protected and permitted phases in the signal sequence must be treated as a special case for purposes of computing the uniform delay. Such movements are analyzed for both phases on the Capacity and LOS Worksheet, on which the protected phase and the permitted phase are identified.

Inputs needed

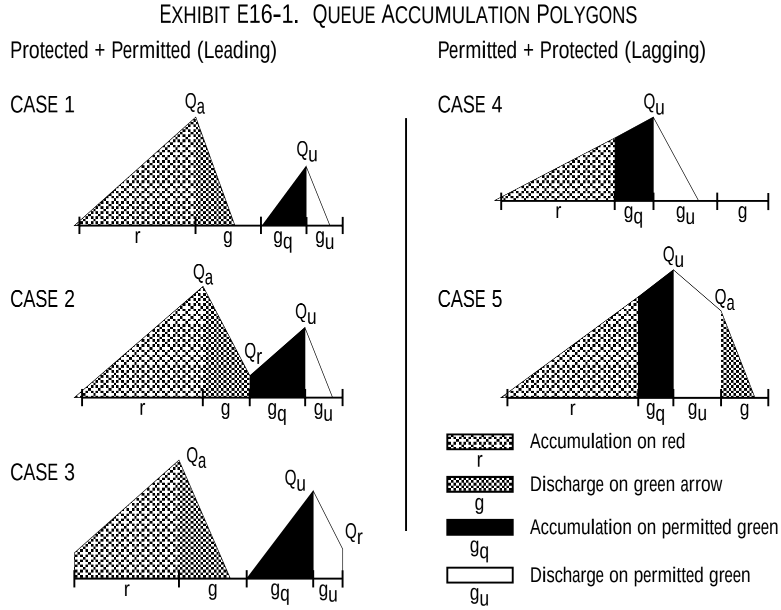
Specifically, the following parameters must be known for evaluating the uniform delay:

- Arrival rate, q_a (veh/s), presumed to be uniform over entire cycle;
- Saturation flow rate for protected phase, s_p (veh/s);
- Saturation flow rate for unsaturated portion of permitted phase, s_s (veh/s) (unsaturated portion begins when queue of opposing vehicles has been served);
- Effective green time for protected phase in which a green arrow is displayed to left turns, g (s);
- Green time during permitted phase when opposing through movement blocks permitted left turns, g_q (s) (this interval begins at start of permitted green and continues until queue of opposing through vehicles has been fully discharged);
- Green time available for left-turning vehicles to filter through gaps in oncoming traffic, g_u (s) (this interval begins when queue of opposing through vehicles has been satisfied, i.e., at end of g_q, and continues until end of permitted green phase); and
- Red time during which signal is effectively red for left turn, r (s).

This terminology will be used in the following description of the supplemental uniform delay procedures and on the worksheet.

The input-output relationships that determine the shape and area of the polygon are shown in Exhibit E16-1. Note that the queuing polygon may assume any one of five shapes depending on the relationship of arrivals and departures. Slightly mathematical formulas must be applied to determine the area for each of the shapes. In all cases, the arrival rate must be adjusted to ensure that, for purposes of uniform delay computation, the v/c ratio is not greater than 1.0. This adjustment is also necessary for the analysis of simple protected operation as described previously. If the v/c ratio is greater than 1.0, the area contained by the polygon will not be defined. The effect of v/c ratios greater than 1.0 is expressed by the second term of the delay equation.

EXHIBIT E16-1. QUEUE ACCUMULATION POLYGONS



It is first necessary to distinguish between protected-plus-permitted (leading left-turn) phasing and permitted-plus-protected (lagging left-turn) phasing. Three of the five cases shown in Exhibit E16-1 are associated with leading left-turn phases and the other two with lagging left-turn phases. The five cases are identified as follows:

- Case 1—Leading left-turn phase: no queue remains at the end of the protected or permitted phase.
- Case 2—Leading left-turn phase: a queue remains at the end of the protected phase but not at the end of the permitted phase.
- Case 3—Leading left-turn phase: a queue remains at the end of the permitted phase but not at the end of the protected phase. Note that it is not possible to have a queue at the end of both the protected and permitted phases if the v/c ratio is not allowed to exceed 1.0 when the uniform delay term is calculated.
- Case 4—Lagging left-turn phase: no queue remains at the end of the permitted phase. In this case there will be no queue at the end of the protected phase either, because the protected phase follows immediately after the permitted phase and will therefore accommodate all of its arrivals without further delay.
- Case 5—Lagging left-turn phase: a queue remains at the end of the permitted phase. If the v/c ratio is kept below 1.0 as just discussed, this queue will be fully served during the protected phase.

Some intermediate computations are required to provide a consistent framework for dealing with all of these cases. Three queue lengths may be determined at various transition points within the cycle. These values are defined as follows:

- Queue size at the beginning of the green arrow, Q_a (veh);
- Queue size at the beginning of the unsaturated interval of the permitted green phase, Q_u (veh); and
- Residual queue size at the end of either the permitted or protected phase, Q_r (veh).

These queue sizes dictate the shape of the polygon whose area determines the value of uniform delay. Separate equations are given for computing each of the queue sizes for the five cases. Equations are provided on the supplemental worksheet (Exhibit 16-23) for computing the uniform delay as a function of queue sizes.

SUPPLEMENTAL UNIFORM DELAY WORKSHEET

The worksheet is presented in Exhibit 16-23. Input data must first be obtained from other worksheets and entered here, namely, the adjusted left-turn volume from the Volume Adjustment and Saturation Flow Rate Worksheet (Exhibit 16-21) and the v/c ratio, X, for the lane group, obtained from the Capacity and LOS Worksheet (Exhibit 16-22).

The following signal timing intervals are obtained from previous computations:

- Protected-phase effective green, g, determined from the Capacity and LOS Worksheet (Exhibit 16-22);
- Permitted-phase effective green intervals, g_q and g_u, from the supplemental worksheets for permitted left turns (see Appendix C); and
- Red time, r, computed as $C – (g + g_q + g_u)$, where C is the cycle length .

These values are entered in the appropriate rows on the worksheet. Note that extremely heavy opposing traffic may reduce g_u to zero, which means that all of the left turns on the permitted phase will be accommodated as sneakers. The effect of sneakers was approximated on the Volume Adjustment and Saturation Flow Rate Worksheet (Exhibit 16-22) by imposing a lower limit on f_{LT}. Because of the lower limit on f_{LT}, a lower limit must also be imposed on the value of g_u to be entered on the supplemental uniform delay worksheet. The necessary time should be transferred from g_q to g_u to ensure that the value of g_u does not fall below 4 s.

The delay computations begin with determination of the arrival and departure rates in units of vehicles per second for compatibility with the remaining worksheet computations. The arrival rate is determined by dividing the left-turn flow rate, v, by

3600. This value is adjusted to ensure that for purposes of uniform delay computation, the arrivals do not exceed the capacity of the intersection. If the v/c ratio, X, exceeds 1.0, the arrival rate is divided by X, as indicated on the worksheet.

Two departure rates are determined:

- Protected-phase departure rate, s_p = s/3600, where s is obtained from the Capacity and LOS Worksheet (Exhibit 16-22); and
- Permitted-phase departure rate, s_s, is computed using Equation E16-1.

$$s_s = \frac{s(g_q + g_u)}{g_u * 3600} \qquad \text{(E16-1)}$$

where s is the adjusted saturation flow rate for the permitted phase from the Capacity and LOS Worksheet (Exhibit 16-22) and the other values have already been determined as described.

Constraint on permitted-phase departure rate, s_s

When g_u is very short, the permitted-phase departures will be mostly sneakers. Since sneakers move with very low headway, it is possible to have extremely high values of s_s. As a practical matter, the per-lane value of s_s should not exceed the base saturation flow rate for the lane group divided by 3600.

Next, the v/c ratios for the protected and permitted phases, X_{prot} and X_{perm}, are determined from the equations on the worksheet. Note that different equations are used for leading and lagging left-turn phases. Because of the adjustment of the arrival rate performed in the previous step, it will not be possible for both X_{prot} and X_{perm} to exceed 1.0. It will, however, be possible for one or the other to exceed 1.0. It is possible to define five separate cases for delay computation, depending on which of the X values exceed 1.0 and on the left-turn phasing (leading or lagging). The case number is now determined and entered at the bottom of the worksheet.

When the case number is known, the size of the queue at three transition points (Q_a, Q_u, and Q_r) may be determined from the formulas at the bottom of the worksheet. When these values have been computed and entered on the worksheet, uniform delay, d_1, has been determined.

APPENDIX F. EXTENSION OF SIGNAL DELAY MODELS TO INCORPORATE EFFECT OF AN INITIAL QUEUE

INTRODUCTION

d_3 = initial queue delay

The delay model represented by Equations 16-9 to 16-12 is based on the assumption that there is no initial queue at the start of the analysis period of duration T. In cases where X > 1.0 for a 15-min period, the following period begins with an initial queue. This initial queue is referred to as Q_b, in vehicles. Q_b is observed at the start of the red period and excludes any vehicles in queue due to random, cycle-by-cycle fluctuations in demand (overflow queue due to cycle failures). When $Q_b \neq 0$, vehicles arriving during the analysis period will experience an additional delay because of the presence of an initial queue. The magnitude of this additional delay depends on several factors, including the size of the initial queue, the length of the analysis period, and the volume to capacity ratio during the analysis period. The initial queue delay term is designated d_3.

Five scenarios emerge in the estimation of control delay, labeled Cases I to V. Cases I and II occur when there is no initial queue and the period is either undersaturated (Case I) or oversaturated (Case II). In both Cases d_3 = 0, and the delay model in Equation 16-11 applies. Cases III, IV, and V are shown in Exhibits F16-1, F16-2, and F16-3, respectively. Case III occurs when the initial queue Q_b can be fully served in time period T. For this to happen, the sum of Q_b and the total demand in T, qT, must be less than the available capacity, cT. Case IV in Exhibit F16-2 occurs when there is still unmet demand at the end of T but the size of the unmet demand is decreasing. For this to happen, the demand in T (i.e., qT), should be less than the capacity, cT. Finally, Case V in Exhibit F16-3 occurs when demand in T exceeds the capacity. Here the unmet demand will increase at the end of the period T.

Case	Initial Queue	Queue at End of Analysis Period
I	No	No
II	No	Yes
III	Yes	No
IV	Yes	Yes, but smaller
V	Yes	Yes, but larger

EXHIBIT F16-1. CASE III: INITIAL QUEUE DELAY WITH INITIAL QUEUE CLEARING DURING T

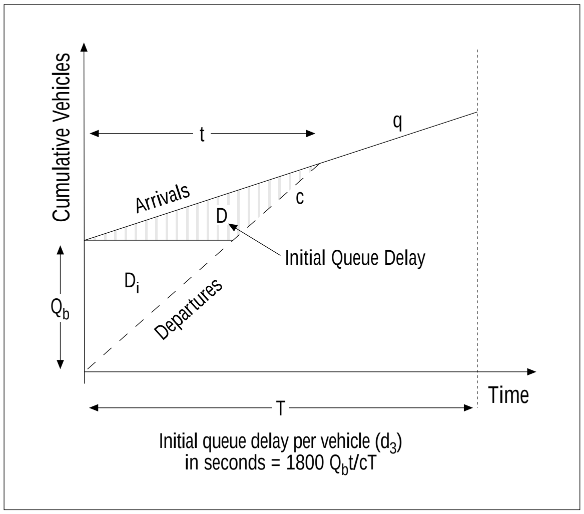
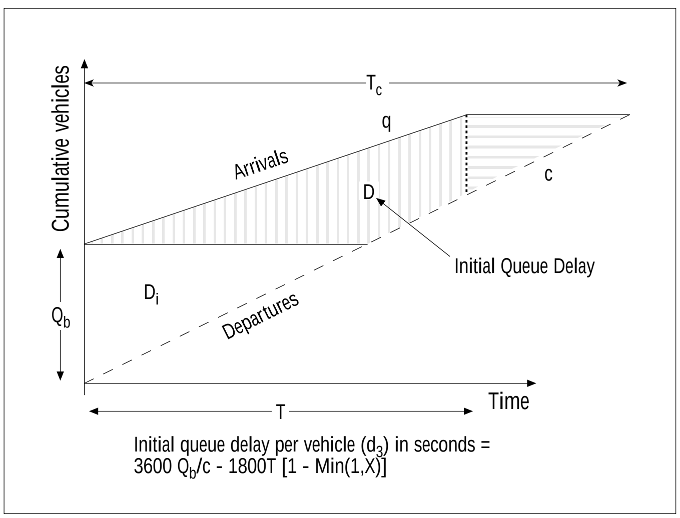


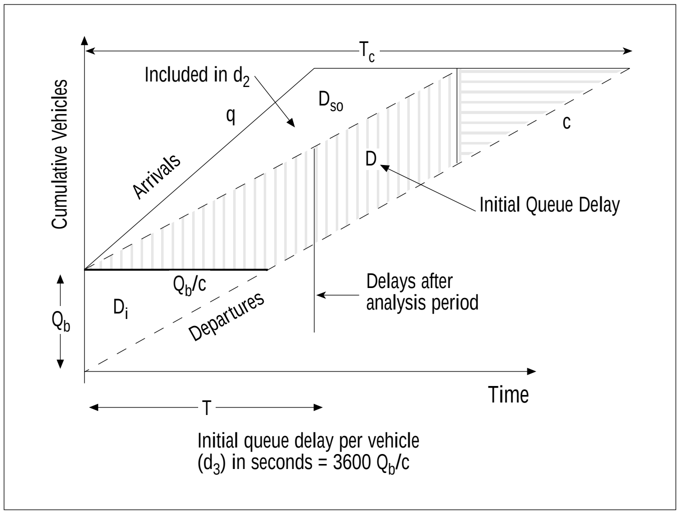
EXHIBIT F16-2. CASE IV: INITIAL QUEUE DELAY WITH INITIAL QUEUE DECREASING DURING T



The total initial queue delay due to an initial queue that is incurred in the average cycle is depicted as the shaded area in Exhibits F16-1 to F16-3, labeled D. It represents the delay experienced by all vehicles arriving during the analysis period, including delay that is experienced in subsequent time periods (Exhibits F16-2 and F16-3). Excluded from this delay are two components: the delay incurred by vehicles in the initial queue (labeled D_i in the exhibits) and the oversaturation delay corresponding to a zero initial queue (labeled D_{so} in Exhibit F16-3). This last term is already accounted for in the d_2 term component of the delay model in Equation 16-12.

Delay estimates for the analysis period include delay experienced by vehicles arriving during the period but leaving after it

EXHIBIT F16-3. CASE V: INITIAL QUEUE DELAY WITH INITIAL QUEUE INCREASING DURING T



ESTIMATION OF d_3

A generalized form of d_3 appears as Equation F16-1, which provides estimation of the initial queue delay per vehicle (in seconds) when an initial queue of size Q_b is present at the start of the analysis period T. d_3 is a term in the delay model given in Equation 16-9.

$$d_3 = \frac{1800 Q_b (1 + u) t}{cT} \quad \text{(F16-1)}$$

where

Q_b = initial queue at the start of period T (veh),
c = adjusted lane group capacity (veh/h),
T = duration of analysis period (h),
t = duration of unmet demand in T (h), and
u = delay parameter.

The parameters t and u are determined according to the prevailing case. Equations F16-2 and F16-3 may be used to estimate the values for Cases III, IV, and V:

$$t = 0 \text{ if } Q_b = 0, \text{ else } t = \min\left\{T, \frac{Q_b}{c[1 - \min(1, X)]}\right\} \quad \text{(F16-2)}$$

where

X = lane group degree of saturation, v/c

$$u = 0 \text{ if } t < T, \text{ else } u = 1 - \frac{cT}{Q_b[1 - \min(1, X)]} \quad \text{(F16-3)}$$

Initial queue clearing time defined

In addition to computation of the initial queue delay term, the analyst may be interested in computing the time at which the last vehicle that arrives during the analysis period clears the intersection (measured from the start of the time period T) because of the presence of an initial queue of length Q_b. This time is referred to as the initial queue clearing time, T_c. In Cases I, II, and III, all vehicles will clear at the end of the period T (in addition to the normal delays $d_1 + d_2$). For Cases IV and V, the last vehicle arriving in T will clear the intersection at time $T_c > T$ (again, in addition to $d_1 + d_2$). Therefore, a general formula for the initial queue clearing time in the case of an initial queue, measured from the start of the analysis period, T, is given as Equation F16-4:

$$T_c = \max\left(T, \frac{Q_b}{c} + TX\right) \qquad \text{(F16-4)}$$

To summarize the procedure for estimating control delay, Exhibit F16-4 gives a comparison of the model parameters for Cases I through V. Note that in order to decide whether Case III (t < T) or IV (t = T) applies, the value of t must first be computed from Equation F16-2.

EXHIBIT F16-4. SELECTION OF DELAY MODEL VARIABLES BY CASE

Case No.	X	Q_b	d_1	d_2	t	u	d_3	T_c
I	≤ 1.0	0	Eq. 16-14	Eq. 16-15	0	0	0	T
II	> 1.0	0	Eq. 16-14	Eq. 16-15	0	0	0	TX
III	≤ 1.0	> 0	Eq. F16-5	Eq. 16-15	Eq. F16-2	0	Eq. F16-1	T
IV	≤ 1.0	> 0	Eq. F16-5	Eq. 16-15	T	Eq. F16-3	Eq. F16-1	Eq. F16-4
V	> 1.0	> 0	Eq. F16-5	Eq. 16-15	T	1	Eq. F16-1	Eq. F16-4

d_1 must also be evaluated differently if initial queue is present

For Cases III, IV, and V, the uniform control delay component (d_1) must be evaluated using X = 1.0 for the period when an oversaturation queue exists (t) and using the actual X value for the remainder of the analysis period (T – t). Therefore, in these cases, a time-weighted value of d_1 is to be used as shown in Equation F16-5.

$$d_1 = d_s * \frac{t}{T} + d_u * PF * \frac{(T - t)}{T} \qquad \text{(F16-5)}$$

where

d_s = saturated delay (d_1 evaluated for X = 1.0), and
d_u = undersaturated delay (d_1 evaluated for actual X value).

In Equation F16-5 for Cases IV and V, the d_u term drops out because t = T. Equation 16-11 is used to evaluate the d_s and d_u components in all cases except for left turns made from exclusive lanes with compound left-turn protection (protected-permitted and permitted-protected), using X = 1.0 in the equation to compute d_s and using the actual X value to compute d_u. For compound left-turn protection, the supplemental uniform delay worksheet in Exhibit 16-23 is used as a means to approximate the d_s and d_u components, again using X = 1.0 for d_s and the actual X value for d_u. When X = 1.0 is used for the d_s component in Exhibit 16-23, the left-turn volume (v) must also be adjusted by the actual X value (use $v' = v/X$) to meet the basic assumptions of the supplemental uniform delay worksheet.

Note that the only place where PF is used in the initial queue analysis of this appendix is for the undersaturated d_u portion of a Case III condition because the existence of the initial queue defeats the value of the progression under all other conditions. Analysts are advised to be aware of a similar concern in the use of PF in a Case II analysis (oversaturated) using Equation 16-9 because all but the first cycle will be blocked by initial queues due to the oversaturated condition.

NUMERICAL EXAMPLE OF DELAYS WITH INITIAL QUEUE

To demonstrate the application of the delay model extension, an analysis of the EB lane group in Example Problem 1 is carried out with and without an initial queue. The following input values are considered:

Lane group capacity (c) = 780 veh/h,
Lane group degree of saturation (X) = 1.026,
Analysis period length (T) = 0.25 h,
Initial queue = 20 vehicles (across the two-lane lane group).

Scenario I: No initial queue

In this case, $d_3 = 0$ as per Exhibit F16-4, Case II. The average control delay per vehicle is $d_1PF + d_2 + d_3 = 22.0 * 0.923 + 39.0 + 0.0 = 59.3$ s. The corresponding LOS for a control delay of 59.3 s is E. Finally, the supplemental clearing time $T_c = 15.4$ min for Case II.

Scenario II: Initial queue of 20 vehicles

Since X > 1.0 and $Q_b = 20$, Case V in Exhibit F16-4 applies. Here, t = 0.25 and u = 1. Substituting in Equation F16-1 gives

$$d_3 = \frac{(1800)(20)(1+1)(0.25)}{(780)(0.25)} = \textit{92.3 s/veh}$$

Therefore, the average control delay per cycle is

$$d = 22.0 + 39.0 + 92.3 = 153.3 \text{ s/veh (LOS F)}$$

which is more than twice the delay calculated when no initial queue is assumed. Note that PF is not applied for Case V. Thus, the impact of an initial queue can be substantial and must be accounted for in delay and LOS estimation.

Finally, the initial queue clearing time, T_c, is estimated from Equation F16-4:

$$T_c = \max\left(0.25, \frac{20}{780} + 0.25 * 1.026\right) = 0.282 \text{ h}$$

or 16.9 min from the start of the peak period. The last vehicle entering in the peak 15-min period will experience an additional 1.9 min of delay because of the presence of the initial queue of 20 vehicles.

EXTENSION TO MULTIPLE TIME PERIODS

The procedure described above can be extended to analyze multiple time periods, each of duration T and each having a fixed demand during T. The analysis is performed sequentially, carrying over the final initial queue Q_b (if any) from one time period to the beginning of the next. In general, for time period i the final initial queue $Q_{b,i+1}$ at the start of the next time period T can be estimated from Equation F16-6:

$$Q_{b,i+1} = \max\left[0,\ Q_{b,i} + cT(X_i - 1)\right], \text{ for } i = 1,2, \ldots, n \qquad \text{(F16-6)}$$

where

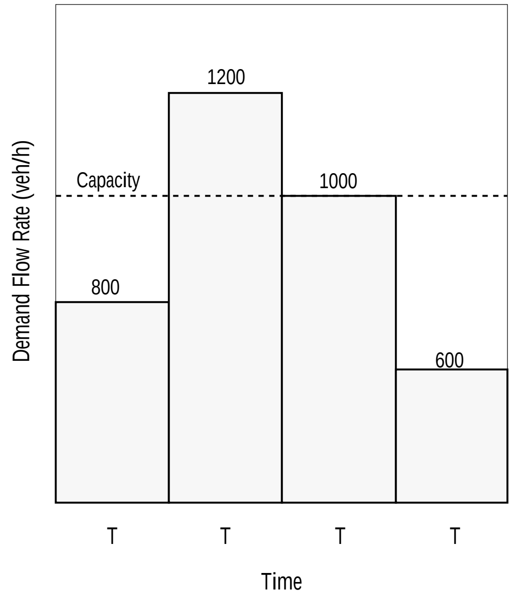
$Q_{b,i}$, X_i = initial queue and degree of saturation for period i.

Typically, a multiple-time-period analysis would start with an undersaturated time period, particularly for $Q_{b,1} = 0$. Once the initial queue is calculated, delays are estimated according to the method described in the previous section. An important feature of multiple-period analysis is that the actual counts taken during each time period should be used in the procedure, that is, the PHF is unity. Counts are then converted into hourly flow rates by dividing each count by T (in hours). The procedure is best described using a numerical example.

NUMERICAL EXAMPLE FOR MULTIPLE-PERIOD ANALYSIS

In this example, consider a signalized lane group with no initial queue that has a fixed capacity of 1,000 veh/h. The demand profile based on 15-min counts (factored to hourly rates) is depicted in Exhibit F16-5. The lane group receives 40 s of effective green time in a 100-s cycle. Arrivals are considered to be random (Arrival Type 3). Calculate the delay and LOS for vehicles arriving in each 15-min time period and for the overall analysis period of 1 h.

EXHIBIT F16-5. DEMAND PROFILE FOR MULTIPLE-PERIOD ANALYSIS WITH 15-MIN PERIODS



Period 1

Period 1 is undersaturated, with a degree of saturation X = 800/1000 = 0.80. Therefore, from Equation F16-6, there is no initial queue ($Q_{b,2}$) at the start of Period 2, assuming no initial queue at the start of Period 1 ($Q_{b,1}$ = 0). The average control delay to vehicles arriving in Period 1 will be labeled $d_{c,1}$ and is estimated as follows:

$$d_{c,1} = \frac{0.50 * 100 * \left(1 - \frac{40}{100}\right)^2}{1 - \frac{40}{100}\min(1, 0.80)} * 1.0 + 900 * 0.25\left[(0.80 - 1) + \sqrt{(0.80 - 1)^2 + \frac{8 * 0.50 * 0.80}{1000 * 0.25}}\right] = 33.2 \text{ s}$$

Period 2

Period 2 is oversaturated, with a degree of saturation X = 1200/1000 = 1.20. There is no initial queue at the start of the period, so again the two-component delay formula can be used:

$$d_{c,2} = \frac{0.50 * 100 * \left(1 - \frac{40}{100}\right)^2}{1 - \frac{40}{100}\min(1, 1.20)} * 1.0 + 900 * 0.25\left[(1.20 - 1) + \sqrt{(1.20 - 1)^2 + \frac{8 * 0.50 * 1.20}{1000 * 0.25}}\right] = 129.7 \text{ s}$$

Period 3

Period 3 is fully saturated, with a degree of saturation X = 1000/1000 = 1.00. The residual queue from the previous period, which is equivalent to the initial queue of Period 3, is calculated from Equation F16-6 as follows:

$$Q_{b,3} = \max\left[0, 0 + 1000 * 0.25 * (1.2 - 1)\right] = 50 \text{ veh}$$

Here, the initial queue delay term (d_3) must be added. First, the values of t and u are determined from Equations F16-2 and F16-3, respectively:

$$t = \min\left[0.25, \frac{50}{1000(1 - 1)}\right] = 0.25$$

$$u = 1 - \frac{1000 * 0.25[1 - \min(1.0, 1.0)]}{50} = 1.0$$

Substituting in Equation F16-1 gives

$$d_3 = \frac{1800 * 50 * (1 + 1) * 0.25}{1000 * 0.25} = 180 \text{ s}$$

So the average control delay in Period 3 is

$$d_{c,3} \frac{0.50 * 100 * \left(1 - \frac{40}{100}\right)^2}{1 - \frac{40}{100}\min(1, 1.0)} * \frac{0.25}{0.25} + 900 * 0.25\left[(1.0 - 1) + \sqrt{(1.0 - 1)^2 + \frac{8 * 0.50 * 1.0}{1000 * 0.25}}\right] + 180 = 238.5 \text{ s}$$

Period 4

Period 4 is undersaturated, with a degree of saturation X = 600/1000 = 0.60. The residual queue from the previous period, which is equivalent to the initial queue of Period 4, is calculated from Equation F16-6 as follows:

$$Q_{b,4} = \max[0, 50 + 1000 * 0.25 * (1.0 - 1)] = 50 \text{ veh}$$

In essence, since the previous period was at capacity, the residual queue at the end of the period is equivalent to that at the start of the period. Again, the computation of d_3 requires the values of t and u, which are calculated as follows:

$$t = \min\left[0.25, \frac{50}{1000(1 - 0.60)}\right] = 0.125$$

Since t < 0.25, then u = 0 from Equation F16-3. Substituting in Equation F16-1 gives

$$d_3 = \frac{1800 * 50 * (1 + 0.0) * 0.125}{1000 * 0.25} = 45.0 \text{ s}$$

Since t < T, the uniform delay component is calculated as per Equation F16-5:

$$d_1 = \frac{0.50 * 100\left(1 - \frac{40}{100}\right)^2}{\left(1 - \frac{40}{100} * 1.0\right)} * \frac{0.125}{0.25} + \frac{0.50 * 100\left(1 - \frac{40}{100}\right)^2}{1 - \frac{40}{100} * \min(1, 0.60)} * 1.0 * \frac{0.25 - 0.125}{0.25} = 26.8 \text{ s/veh}$$

Thus, the average control delay in Period 4 is

$$d_{c,4} = 26.8 + 900 * 0.25 * \left[(0.60 - 1.0) + \sqrt{(0.60 - 1.0)^2 + \frac{8 * 0.50 * 0.60}{1000 * 0.25}}\right] + 45 = 74.5 \text{ s/veh}$$

The contribution of each delay term in each period is shown in Exhibit F16-6. The impact of the initial queue delay term is evident, particularly for Periods 3 and 4.

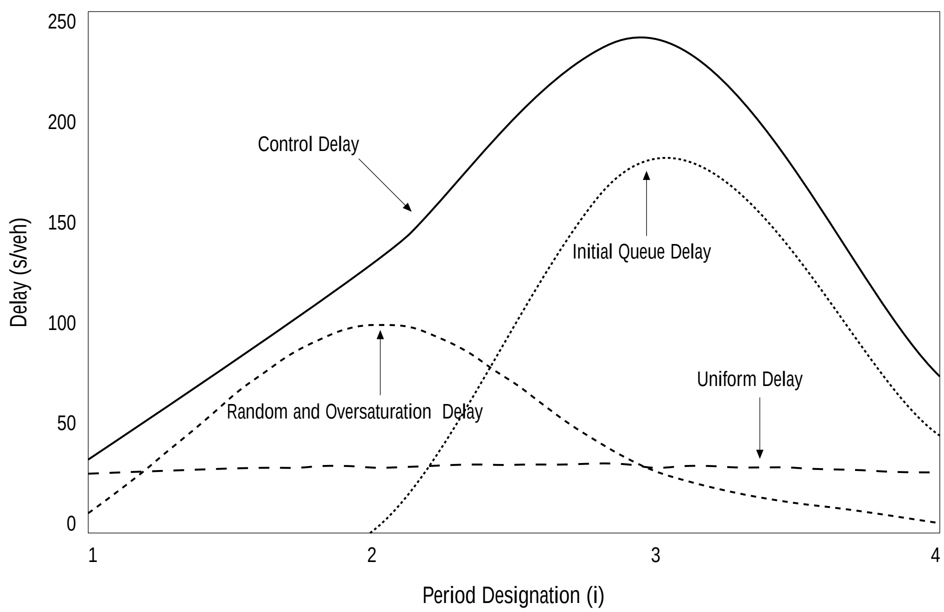
Finally, the average overall control delay to all vehicles arriving in the hour is calculated as a volume-weighted delay of the individual period delays:

$$d_{c,t} = \frac{[800 * 33.2 + 1200 * 129.7 + 1000 * 238.5 + 600 * 74.5]}{[800 + 1200 + 1000 + 600]} = 129.3 \text{ s/veh}$$

The example demonstrates how misleading information can result from analyzing only one portion of an oversaturated time period

The average control delay in the entire period is virtually identical to the delay to vehicles arriving in the peak Period 2, but it is much smaller than the worst delay (and LOS) that is experienced in Period 3, which immediately follows the peak. Thus, a single-period analysis may not be sufficient to determine the worst LOS in an oversaturated time period. When residual queues occur at the end of a peak period, it is recommended that a delay analysis be carried out over subsequent time intervals to ensure that the most severe LOS period is identified.

EXHIBIT F16-6. ILLUSTRATION OF DELAY MODEL COMPONENTS FOR MULTIPLE-PERIOD ANALYSIS



PROCEDURES FOR MAKING CALCULATIONS

Time periods should be of equal lengths in multiperiod analyses

Exhibit F16-7 provides a worksheet that can be used in the calculation of delay with initial queue. The following steps should be followed for this worksheet. Each time period will utilize a separate worksheet to calculate the delay when any of the movements at the intersection start with an initial queue during that time period. Each time period should be numbered and the period number and time entered. The duration of the time period, in hours, should also be entered. Note that in a multiperiod analysis the length of each time period should be the same.

Undersaturated lane groups should be entered for purposes of averaging

Lane groups to be analyzed are those used in the Capacity and LOS Worksheet. Lane groups that do not have an initial queue may appear on this worksheet so that their delay values can be averaged with oversaturated lane groups. In this case, their d_1 and d_2 values will be unchanged and d_3 will be 0. The v/c, lane group capacity, unadjusted uniform delay, and incremental delay values are taken from the Capacity and LOS Worksheet. Unadjusted uniform delay is the product of uniform delay and the progression adjustment factor.

The initial queue for each lane group may either be physically observed in the field (excluding any vehicles in queue due to random cycle-by-cycle fluctuation in demand) or be carried over as the residual queue from the previous analysis time period.

The duration of unmet demand, t, is calculated using Equation F16-2. If there is no initial queue ($Q_b = 0$), then $t = 0$, and the value of t is limited to be no larger than the length of the time period, T.

The adjusted uniform delay term, d_1, is calculated using Equation F16-5. When $t = 0$, the result is the same as for the unadjusted value of d_1. Progression effects are included, as appropriate, in this adjusted uniform delay result. The values of g and C for the lane group from the Capacity and LOS Worksheet must be used to make this calculation. Note that the unadjusted value of d_2 is used (from the Capacity and LOS Worksheet) in the final delay calculations and that this delay value includes the oversaturation delay when $v/c > 1$.

EXHIBIT F16-7. INITIAL QUEUE DELAY WORKSHEET

INITIAL QUEUE DELAY WORKSHEET				
General Information				
Project Description ______________________				
Input Parameters				
Period (i) ______________ Duration, T ______________ h Cycle length, C ______________ s				
	EB	WB	NB	SB
Lane group				
Initial queue, Q_b (veh)				
Green ratio, g/C				
v/c ratio, X X = v/c				
Adjusted lane group capacity, c (veh/h)				
Duration of unmet demand in T (h) $t = \min\left\{T, \frac{Q_b}{c[1 - \min(1,X)]}\right\}$				
Case				
Cases I and II (Q_b = 0)				
Initial queue delay, d_3 = 0, and uniform delay , d_1, is as shown on Capacity and LOS Worksheet				
Case III (Q_b > 0) (X ≤ 1.0) (t < T)				
Initial queue delay, d_3 (s) $d_3 = \frac{1800\,Q_b t}{cT}$				
Uniform delay, d_1 (s) $d_1 = 0.50C(1 - g/C)\frac{t}{T} + \frac{0.5C(1-g/C)^2}{1 - g/C\min(1,X)}\left[\frac{(T-t)}{T}\right] * PF$				
Case IV (Q_b > 0) (X ≤ 1.0) (t = T)				
Delay parameter, u $u = 1 - \frac{cT}{Q_b}\left[1 - \min(1,X)\right]$				
Initial queue delay, d_3 (s) $d_3 = \frac{1800\,Q_b\,(1 + u)}{c}$				
Uniform delay, d_1 (s) $d_1 = 0.50C(1 - g/C)$				
Case V (Q_b > 0) (X > 1.0) (t = T)				
Initial queue delay, d_3 (s) $d_3 = \frac{3600\,Q_b}{c}$				
Uniform delay, d_1 (s) $d_1 = 0.50C\,(1 - g/C)$				

The initial queue parameter, u, is calculated using Equation F16-3. When t < T, u = 0 (Cases I, II, and III); otherwise the equation is used (Case IV), or u = 1 (Case V).

The final residual queue is calculated using Equation F16-6. This is the estimate of the number of vehicles in queue at the end of the analysis period. If its value is nonzero, this value indicates that the subsequent analysis period should be analyzed to determine the average delay per vehicle that results because of this initial queue for that time period.

The initial queue delay, d_3, is calculated using Equation F16-1. This value is the additional delay that results from the existence of the initial queue. Note that this value does not include any of the oversaturation delay, which is accounted for in d_2. The d_3 value is obtained from the Initial Queue Delay Worksheet (Exhibit F16-7) and is entered in the Capacity and LOS Worksheet (Exhibit 16-22).

Delay and LOS are found by adding the three delay terms d_1, d_2, and d_3 for each lane group. Note that the d_1 value includes any appropriate effects of PF on the d_1 term. The LOS corresponding to this delay, taken from Exhibit 16-2, is the result.

The control delay per vehicle is found for each approach by adding the product of the lane group flow rate and the delay for each lane group on the approach and dividing the sum by the total approach flow rate on the Capacity and LOS Worksheet. The LOS is determined from Exhibit 16-2.

The control delay per vehicle for the intersection as a whole is found by adding the product of the approach flow rate and the approach delay for all approaches and dividing by the total intersection flow rate. The intersection LOS is then found from Exhibit 16-2.

APPENDIX G. DETERMINATION OF BACK OF QUEUE

Appendix G provides the procedure to calculate the queue at signalized intersections. The queue length definition used in this model is the back of queue. A relationship for the back of queue is developed as described in the next sections.

Back of queue defined

The back of queue is the number of vehicles that are queued depending on arrival patterns of vehicles and vehicles that do not clear the intersection during a given green phase (overflow). The model predicts the average back of queue, and 70th-, 85th-, 90th-, 95th-, and 98th-percentile backs of queue.

The methodology is applicable to single-lane situations

The model described in this appendix is for use on an individual lane. To apply the method to a lane group, the flow rate, saturation flow rate, capacity, and initial queue demand values for the lane group are converted to individual lane values. If initial queue (Q_b) is present in a lane group, the lane group flow rate is adjusted to include the initial queue present according to Equation G16-1.

$$v_l = v + \frac{Q_b}{T} \qquad \text{(G16-1)}$$

where

v_l = lane group flow rate including initial queue present (veh/h),
v = arrival flow rate (veh/h),
Q_b = lane group initial queue at start of analysis period (veh), and
T = length of analysis period (h).

Parameters are averaged across the number of lanes

Other parameters for individual lanes are calculated for each lane group by dividing the total lane group values by the number of lanes in the lane group as shown in Equations G16-2 through G16-5. The queue calculated by this method is assumed to be the queue found in any lane of the lane group (all lane queues are assumed to be nominally equal) and reflects the impact of unequal lane usage only to the degree that unequal lane utilization affects saturation flow for the entire lane group. Specifically, it does not reflect the queue in the lane with the longest queue due to unequal lane utilization. If the lane with the longest queue due to unequal lane utilization is desired, this can be calculated by applying the entire HCM methodology on a lane-by-lane basis. As an alternative, the lane with the longest queue can be approximated by determining that lane's unequal lane volume (v_L in Equation G16-2) on the basis of the lane utilization factors instead of dividing by N_{LG}.

$$v_L = \frac{v_l}{N_{LG}} \qquad \text{(G16-2)}$$

$$s_L = \frac{s}{N_{LG}} \qquad \text{(G16-3)}$$

$$c_L = \frac{c}{N_{LG}} \qquad \text{(G16-4)}$$

$$Q_{bL} = \frac{Q_b}{N_{LG}} \qquad \text{(G16-5)}$$

where

v_L = lane group flow rate per lane (veh/h),
s = lane group saturation flow rate (veh/h),
s_L = lane group saturation flow rate per lane (veh/h),
c = lane group capacity (veh/h),
c_L = lane group capacity per lane (veh/h),
Q_{bL} = lane group initial queue at start of analysis period per lane (veh), and
N_{LG} = number of lanes in lane group.

AVERAGE BACK OF QUEUE

The average back-of-queue measure is the basis to calculate percentile back of queue. Equation G16-6 shows average back-of-queue characteristics at signalized intersections.

$$Q = Q_1 + Q_2 \qquad \text{(G16-6)}$$

where

Q = maximum distance in vehicles over which queue extends from stop line on average signal cycle (veh),
Q_1 = first-term queued vehicles (veh), and
Q_2 = second-term queued vehicles (veh).

Q_1 is the number of vehicles arriving assuming uniform pattern, adjusted for progression

The first term, Q_1, is the average back of queue, determined first by assuming a uniform arrival pattern and then adjusting for the effects of progression for a given lane group. The first term is calculated using Equation G16-7.

$$Q_1 = PF_2 \frac{\frac{v_L C}{3600}\left(1 - \frac{g}{C}\right)}{1 - \left[\min(1.0, X_L)\frac{g}{C}\right]} \qquad \text{(G16-7)}$$

where

Q_1 = first-term queued vehicles (veh),
PF_2 = adjustment factor for effects of progression,
v_L = lane group flow rate per lane (veh/h),
C = cycle length (s),
g = effective green time (s), and
X_L = ratio of flow rate to capacity (v_L/c_L ratio).

Q_1 represents the number of vehicles that arrive during the red phases and during the green phase until the queue has dissipated.

The adjustment factor for the effects of progression is calculated by Equation G16-8.

$$PF_2 = \frac{\left(1 - R_p \frac{g}{C}\right)\left(1 - \frac{v_L}{s_L}\right)}{\left(1 - \frac{g}{C}\right)\left[1 - R_p\left(\frac{v_L}{s_L}\right)\right]} \qquad \text{(G16-8)}$$

where

PF_2 = adjustment factor for effects of progression,
v_L = lane group flow rate per lane (veh/h),
s_L = lane group saturation flow rate per lane (veh/h),
g = effective green time (s),
C = cycle length (s), and
R_p = platoon ratio [P(C/g)].

Q_2 is the overflow queue

The second term, Q_2, is an incremental term associated with randomness of flow and overflow queues that may result because of temporary failures, which can occur even when demand is below capacity. This value can be an approximate cycle overflow queue when there is no initial queue at the start of the analysis period. Initial queue at the start of the analysis period is also accounted for in the second term, Q_2. Equation G16-9 is used to compute the second term of the average back of queue.

$$Q_2 = 0.25 c_L T \left[(X_L - 1) + \sqrt{(X_L - 1)^2 + \frac{8 k_B X_L}{c_L T} + \frac{16 k_B Q_{bL}}{(c_L T)^2}} \right] \qquad \text{(G16-9)}$$

where

Q_2	=	second term of queued vehicles, estimate for average overflow queue (veh);
c_L	=	lane group capacity per lane (veh/h);
T	=	length of analysis period (h);
X_L	=	v_L/c_L ratio;
k_B	=	second-term adjustment factor related to early arrivals;
Q_{bL}	=	initial queue at start of analysis period (veh); and
C	=	cycle length (s).

The second-term adjustment factor related to early arrivals is calculated using Equation G16-10.

$$k_B = 0.12\, I \left(\frac{s_L g}{3600} \right)^{0.7} \text{(pretimed signals)}$$

$$k_B = 0.10\, I \left(\frac{s_L g}{3600} \right)^{0.6} \text{(actuated signals)} \qquad \text{(G16-10)}$$

where

k_B	=	second-term adjustment factor related to early arrivals,
s_L	=	lane group saturation flow rate per lane (veh/h),
g	=	effective green time (s), and
I	=	upstream filtering factor for platoon arrivals (Chapter 15).

Exhibit G16-1 depicts the development of the back of queue over a typical undersaturated cycle in an analysis period. The exhibit shows the queue developing over the red interval and into the green interval until the front of the queue reaches the back of queue, thus dissipating the entire queue. The diagram shows the randomness of arrival rate. The variation in demand may cause individual cycle failures even though the demand over the analysis period is less than the capacity available. During some cycles queue overflow may be experienced as shown in Exhibit G16-2. Thus, the model for average back of queue accounts both for the queuing that occurs with a basic regular flow rate and for that which occurs because of randomness. Thus, there are two basic terms in the equation for determining average back of queue.

Exhibits G16-3 and G16-4 depict the relative contribution of each of the terms for Q across a range of v/c ratios. Exhibit G16-3 is calculated for poor progression, and Exhibit G16-4 shows results for good progression. Unlike the associated delay term, the portion of the queue arising from uniform flow will grow as v/c increases. However, the contribution of the second term (Q_2), the portion of the queue that results from random arrivals and overflow queues, grows proportionally as v/c increases.

EXHIBIT G16-1. UNDERSATURATED CYCLE BACK OF QUEUE

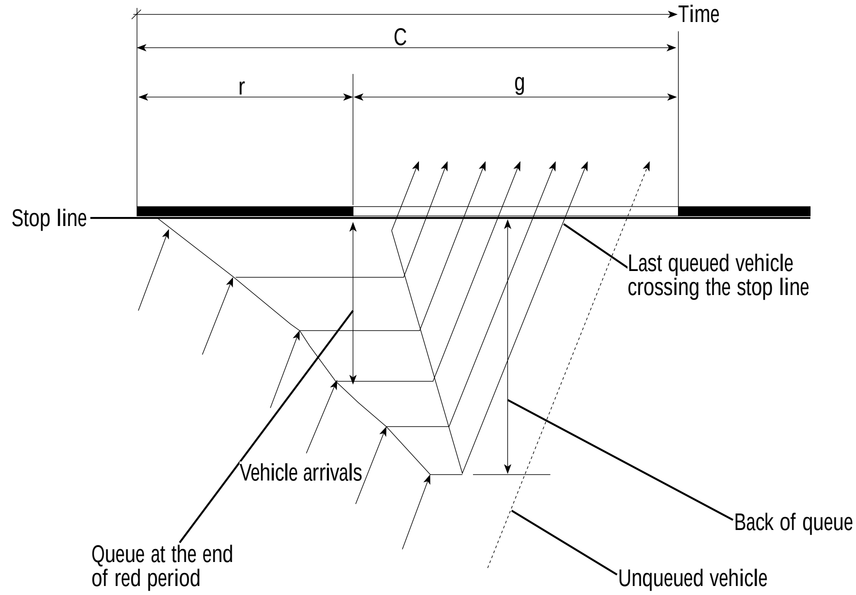


EXHIBIT G16-2. OVERSATURATED CYCLE BACK OF QUEUE

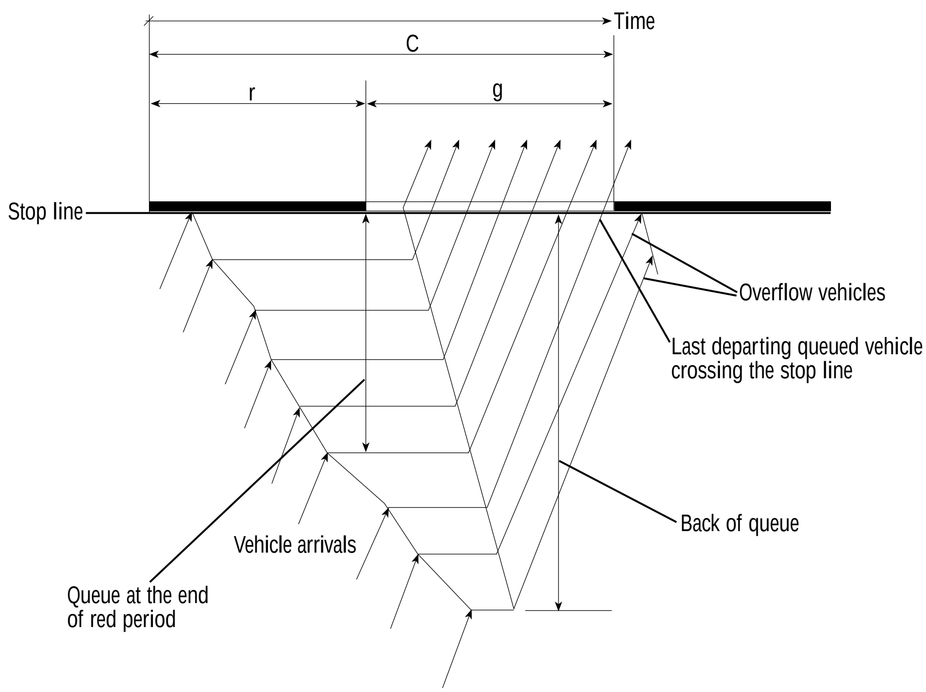
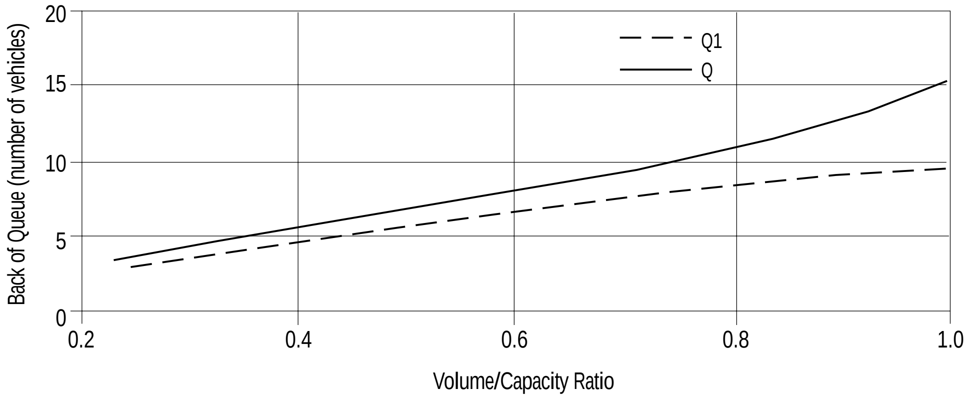
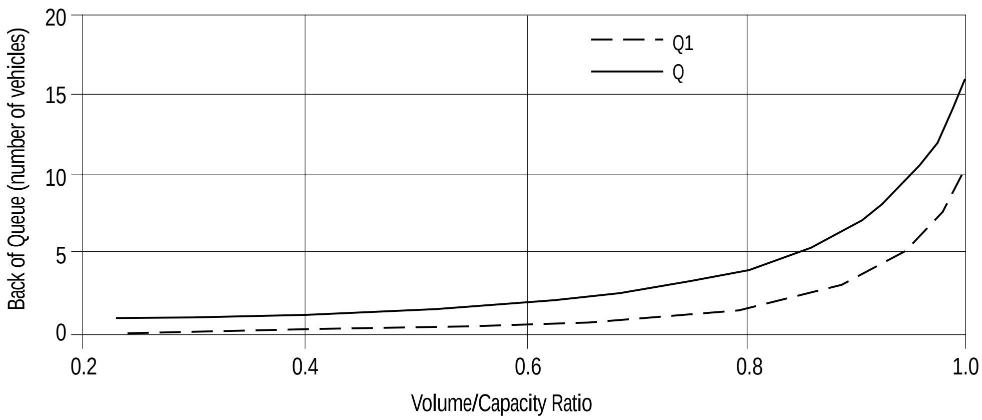


EXHIBIT G16-3. CONTRIBUTION OF THE FIRST AND SECOND TERMS OF BACK OF QUEUE WITH POOR PROGRESSION
(SEE FOOTNOTE FOR ASSUMED VALUES)



Note:
Assumes Arrival Type 1, g/C = 0.44, T = 0.25 h, Q_{bL} = 0, and s_L = 1,000 veh/h.

EXHIBIT G16-4. CONTRIBUTION OF THE FIRST AND SECOND TERMS OF BACK OF QUEUE WITH GOOD PROGRESSION
(SEE FOOTNOTE FOR ASSUMED VALUES)



Note:
Assumes Arrival Type 6, g/C = 0.44, T = 0.25 h, Q_{bL} = 0, s_L = 1,000 veh/h.

PERCENTILE BACK OF QUEUE

The percentile back of queue is computed by applying the percentile back-of-queue factor to the average back of queue. Equation G16-11 shows this relationship.

$$Q_{\%} = Q\, f_{B\%} \qquad \text{(G16-11)}$$

where

$Q_{\%}$ = percentile back of queue (veh),
Q = average number of vehicles in queue (veh), and
$f_{B\%}$ = percentile back-of-queue factor.

The percentile back-of-queue factor is calculated using Equation G16-12.

$$f_{B\%} = p_1 + p_2 e^{\frac{-Q}{p_3}} \qquad \text{(G16-12)}$$

where

$f_{B\%}$ = percentile back-of-queue factor,

p_1 = first parameter for percentile back-of-queue factor (Exhibit G16-5),
p_2 = second parameter for percentile back-of-queue factor (Exhibit G16-5),
p_3 = third parameter for percentile back-of-queue factor (Exhibit G16-5), and
Q = average number of vehicles in queue (veh).

Exhibit G16-5 gives the first, second, and third parameters of the percentile back-of-queue factor for pretimed and actuated signals.

EXHIBIT G16-5. PARAMETERS FOR 70TH-, 85TH-, 90TH-, 95TH-, AND 98TH-PERCENTILE BACK OF QUEUE

	Pretimed Signals			Actuated Signals		
	p_1	p_2	p_3	p_1	p_2	p_3
$f_{B70\%}$	1.2	0.1	5	1.1	0.1	40
$f_{B85\%}$	1.4	0.3	5	1.3	0.3	30
$f_{B90\%}$	1.5	0.5	5	1.4	0.4	20
$f_{B95\%}$	1.6	1.0	5	1.5	0.6	18
$f_{B98\%}$	1.7	1.5	5	1.7	1.0	13

QUEUE STORAGE RATIO

Queue storage ratio is a test for possible blockage

The back-of-queue measure is useful for dealing with the blockage of available queue storage distance, determined by the queue storage ratio. If the queue storage ratio is less than 1, blockage will not occur. If the queue storage ratio is equal to or greater than 1, blockage will occur. Equations G16-13 and G16-14 are used to calculate average queue storage ratio and percentile queue storage ratio, respectively.

$$R_Q = \frac{L_h Q}{L_a} \qquad \text{(G16-13)}$$

where

R_Q = average queue storage ratio,
L_h = average queue spacing in a stationary queue (m),
L_a = available queue storage distance (m), and
Q = average number of vehicles in queue (veh).

$$R_{Q\%} = \frac{L_h Q_{\%}}{L_a} \qquad \text{(G16-14)}$$

where

$R_{Q\%}$ = percentile queue storage ratio, and
$Q_{\%}$ = percentile back of queue (veh).

One must assume an average distance between front bumpers of successive vehicles standing in queue

Average queue spacing is the average length between the front bumpers of two successive vehicles in a stationary queue.

APPLICATION

Exhibit G16-6 provides a worksheet to perform back-of-queue computations. Queue lengths are calculated for each lane in the lane group. Lane group information, flow rates, capacities, and saturation flow rates for each lane group are taken from the Input Worksheet, Volume Adjustment and Saturation Flow Rate Worksheet, and Capacity and LOS Worksheet after they are adjusted for initial queue present and computed on a per-lane basis. The proportion of vehicles arriving on the green should be observed in the field. If field information is available, average arrival rate on green can be calculated. Initial queue at the start of the analysis period also should be observed in the field.

However, if field information is not available, successive period analyses, beginning with a period in which there is no initial queue, can be performed.

EXHIBIT G16-6. BACK-OF-QUEUE WORKSHEET

BACK-OF-QUEUE WORKSHEET

General Information

Project Description ____

Average Back of Queue

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Lane group												
Initial queue per lane at the start of analysis period, Q_{bL}												
Flow rate per lane, v_L (veh/h)												
Saturation flow rate per lane, s_L (veh/h)												
Capacity per lane, c_L (veh/h)												
Flow ratio, v_L/s_L												
v/c ratio, $X_L = v_L/c_L$												
Effective green time, g (s)												
Green ratio, g/C												
Upstream filtering factor, I												
Proportion of vehicles arriving on green, P												
Platoon ratio, R_p $\quad R_p = \left(\frac{P}{g/C}\right)$												
Effects of progression adjustment factor, PF_2 $\quad PF_2 = \frac{\left(1 - R_p\frac{g}{C}\right)\left(1 - \frac{v_L}{s_L}\right)}{\left(1 - \frac{g}{C}\right)\left[1 - R_p\left(\frac{v_L}{s_L}\right)\right]}$												
First-term queued vehicles, Q_1 (veh) $\quad Q_1 = PF_2 \frac{\frac{v_L C}{3600}\left(1 - \frac{g}{C}\right)}{\left[1 - \min(1.0, X_L)\left(\frac{g}{C}\right)\right]}$												
Second-term adjustment factor, k_B $k_B = 0.12\, I \left(\frac{s_L g}{3600}\right)^{0.7}$ (pretimed signals) $k_B = 0.10\, I \left(\frac{s_L g}{3600}\right)^{0.6}$ (actuated signals)												
Second-term queued vehicles, Q_2 $\quad Q_2 = 0.25 c_L T \left[(X_L - 1) + \sqrt{(X_L-1)^2 + \frac{8 k_B X_L}{c_L T} + \frac{16 k_B Q_{bL}}{(c_L T)^2}}\right]$												
Average number of queued vehicles, Q $Q = Q_1 + Q_2$												
Percentile Back of Queue	❑ **70%**			❑ **85%**			❑ **90%**			❑ **95%**		❑ **98%**
Percentile back-of-queue factor,[1] $f_{B\%}$												
Percentile back-of-queue, $Q_{\%}$ (veh), $Q_{\%} = Qf_{B\%}$												
Queue Storage Ratio												
Average queue spacing, L_h (m)												
Available queue storage, L_a (m)												
Average queue storage ratio, $R_Q = \frac{L_h Q}{L_a}$												
Percentile queue storage ratio, $R_{Q\%} = \frac{L_h Q_{\%}}{L_a}$												

Notes

1. $f_{B\%} = p_1 + p_2 e^{\left(\frac{-Q}{p_3}\right)}$, where p_1, p_2, and p_3 are obtained from Exhibit G16-5.

After these parameters have been gathered, the average back of queue, Q, is calculated. Then percentile back of queue is calculated for any desired percentile.

Queue storage ratios for average back of queue and desired percentile back of queue are calculated. The available queue storage distance should be determined by field observation. If the facility has not yet been built (planning application), right-of-way limitations or agency requirements should be used to determine available queue storage distance. The average queue spacing in a stationary queue is the average length between the front bumper of the queued vehicle to the front bumper of the queued vehicle in front. Average queue spacing can be determined according to the traffic composition.

The queue values produced by the estimation procedure in this appendix may be higher than those from other procedures found in the literature, especially at high degrees of saturation or high percentiles, for two reasons. First, many procedures report only average values, applying no queue expansion factor to estimate a higher-percentile approximation. Second, overflow queues that occur occasionally at the end of a cycle are forgiven in many procedures that compute the average back of queue. It must be recognized that the common wisdom that has evolved from popular estimation techniques may be unduly optimistic and that low probabilities of queue overflow may be difficult to achieve when demand is near capacity.

APPENDIX H. DIRECT MEASUREMENT OF PREVAILING SATURATION FLOW RATES

GENERAL NOTES

The default base saturation flow rate used in the methodology of this chapter is 1,900 pc/h/ln. This value must be adjusted for prevailing traffic conditions such as lane width, left turns, right turns, heavy vehicles, grade, parking, parking blockage, area type, bus blockage, and left-turn blockage. These computations are made in the Volume Adjustment and Saturation Flow Rate Worksheet. As an alternative to these computations, the actual saturation flow rate can be measured directly in the field.

Stability of saturation flow rates

Saturation flow rate is the maximum discharge rate during the green time. It is usually achieved after about 10 to 14 s of green, which corresponds to the front axle of the fourth to sixth passenger car crossing the stop line after the beginning of green.

The base saturation flow rate is defined as the discharge rate from a standing queue in a 3.6-m-wide lane that carries only through passenger cars and is otherwise unaffected by conditions such as grade, parking, and turning vehicles. Vehicles are recorded when their front axles cross the stop line. The measurement starts at the beginning of the green time or when the front axle of the first vehicle in the queue passes the stop line. Saturation flow, however, is calculated only from the headways after the fourth vehicle in queue passes the stop line. Other reference points on the vehicle, on the road, or in time may yield different saturation flow rates. In order to maintain consistency with the method described in this chapter and to allow for information exchange, maintaining the roadway and vehicle reference points identified here is essential.

The base saturation flow rate is usually stable over a period of time for similar traffic conditions in a given community. Values measured in the same lane during repetitive weekday traffic conditions (e.g., a.m. or p.m. peaks) normally exhibit relatively narrow distributions. On the other hand, saturation flow rates for different communities or different traffic conditions and compositions, even at the same location, may vary significantly.

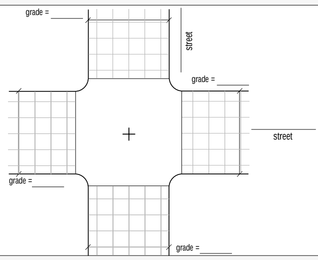
For practical purposes, prevailing saturation flow rates are usually expressed in vehicles per hour per lane. As a result, their values also depend on traffic flow composition. The default value is expressed in passenger cars per hour per lane (i.e., passenger cars only). Preferably, local prevailing saturation flow rates should be observed directly. Alternatively, the computation module can be used, with the measured regional base saturation flow rates as the starting values. The default value should be used only as an approximate substitute. Severe weather conditions, unusual traffic mixes, or other special local conditions can yield saturation flow rates that differ markedly from those estimated using the computation procedures. The procedure for measuring prevailing saturation flow rates is summarized below. A sample field worksheet for recording observations is included as Exhibit H16-1.

EXHIBIT H16-1. FIELD SATURATION FLOW RATE STUDY WORKSHEET

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	Cycle 1			Cycle 2			Cycle 3			Cycle 4			Cycle 5			Cycle 6		
queue	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

MEASUREMENT TECHNIQUE

The following example describes a single-lane saturation flow survey. A two-person field crew is recommended. However, one person with a tape recorder, push-button event recorder, or a notebook computer with appropriate software will suffice. The field notes and tasks identified in the following section must be adjusted according to the type of equipment used.

1. General tasks: Measure and record the area type and width and grade of the lane being studied. Fill out the survey identification data shown in Exhibit H16-1. Select an observation point where the stop line for the surveyed lane and the corresponding signal heads are clearly visible. The reference point is normally the stop line. Vehicles should consistently stop behind this line. When a vehicle crosses it unimpeded, it has entered the intersection conflict space for the purpose of saturation flow measurement. Left- or right-turning vehicles yielding to opposing through traffic or yielding to pedestrians are not recorded until they proceed through the opposing traffic.

2. Recorder tasks: Note the last vehicle in the stopped queue when the signal turns green. Describe the last vehicle to the timer. Note on the worksheet which vehicles are heavy vehicles and which vehicles turn left or right. Record the time called out by the timer.

3. Timer tasks: Start stopwatch at beginning of green and notify the recorder. Count aloud each vehicle in the queue as its front axle crosses the stop line and note the time of crossing. Call out the time of the fourth, tenth, and last vehicle in the stopped queue as its front axle is crossing the stop line.

If queued vehicles are still entering the intersection at the end of the green, call out (saturation through the end of green - last vehicle was number XX). Note any unusual events that may have influenced the saturation flow rate, such as buses, stalled vehicles, and unloading trucks.

The period of saturation flow begins when the front axle of the fourth vehicle in the queue crosses the stop line or reference point and ends when the front axle of the last queued vehicle crosses the stop line. The last queued vehicle may be a vehicle that joined the queue during the green time.

Measurements are taken cycle by cycle. To reduce the data for each cycle, the time recorded for the fourth vehicle is subtracted from the time recorded for the last vehicle in the queue. This value is the sum of all headways for (n – 4) vehicles, where n is the number of the last vehicle surveyed (this may not be the last vehicle in the queue). This sum is divided by the number of headways after the fourth vehicle [i.e., divided by (n – 4)] to obtain the average headway per vehicle under saturation flow. The saturation flow rate is 3,600 divided by this value.

For example, if the time for the fourth vehicle was observed as 10.2 s and the time for the 14th and last vehicle surveyed is 36.5 s, the average saturation headway per vehicle is

$$\frac{(36.5-10.2)}{(14-4)} = \frac{26.3}{10} = 2.63 \text{ s/veh}$$

and the prevailing saturation flow rate in that cycle is

$$\frac{3600}{2.63} = 1369 \text{ veh/h/ln}$$

Guidelines for attaining statistically satisfying results

In order to obtain a statistically significant value, a minimum of 15 signal cycles with more than eight vehicles in the initial queue is typically required. An average of the saturation flow rate values in individual cycles represents the prevailing local saturation flow rate for the surveyed lane. The percentage of heavy vehicles and turning vehicles in the sample used in the computations should be determined and noted for reference.

APPENDIX I. WORKSHEETS

INPUT WORKSHEET

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET

CAPACITY AND LOS WORKSHEET

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

TRAFFIC-ACTUATED CONTROL INPUT DATA WORKSHEET

SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS OPPOSED BY MULTILANE APPROACH

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

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

INITIAL QUEUE DELAY WORKSHEET

BACK-OF-QUEUE WORKSHEET

INTERSECTION CONTROL DELAY WORKSHEET

FIELD SATURATION FLOW RATE STUDY WORKSHEET

INPUT WORKSHEET

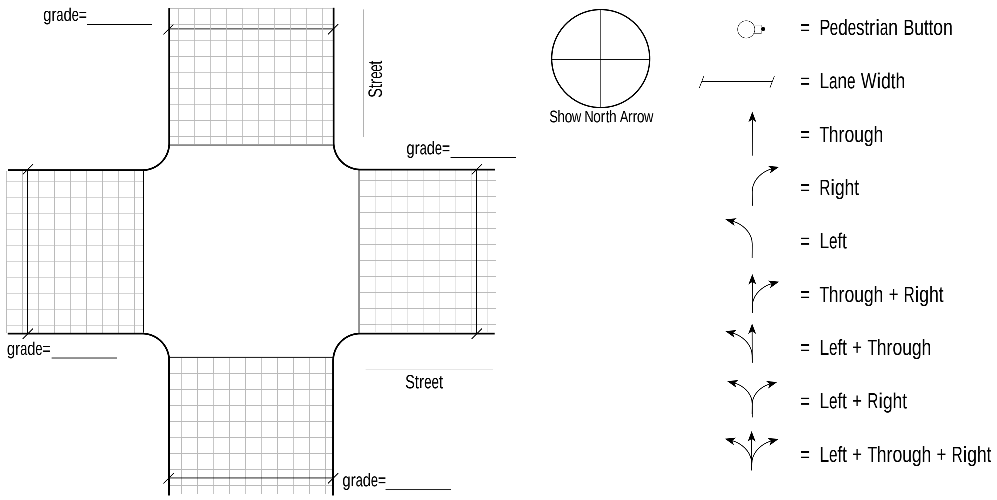
General Information

Analyst	________
Agency or Company	________
Date Performed	________
Analysis Time Period	________

Site Information

Intersection	________	
Area Type	❑ CBD	❑ Other
Jurisdiction	________	
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
D I A G R A M								
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 16-2.

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 (see Exhibit 16-7 to determine adjustment factors)

	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.

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 $d_2 = 900T[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}]$ (s/veh)												
Initial queue delay, d_3 (s/veh) (Appendix F)												
Uniform delay, d_1 (s/veh) (Appendix F)												
Progression adjustment factor, PF												
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)												
LOS by lane group (Exhibit 16-2)												

Delay by approach, $d_A = \frac{\sum (d)(v)}{\sum v}$ (s/veh)				
LOS by approach (Exhibit 16-2)				
Approach flow rate, v_A (veh/h)				
Intersection delay, $d_I = \frac{\sum (d_A)(v_A)}{\sum v_A}$ (s/veh)		Intersection LOS (Exhibit 16-2)		

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 Exhibit 16-13.
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.

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	$[0.50/(q_a C)][(r + g_q)Q_u + g_u(Q_u + Q_a) + Q_a^2/(s_p - q_a)]$

TRAFFIC-ACTUATED CONTROL INPUT DATA WORKSHEET

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

Approach-Specific Data

	EB	WB	NB	SB
Left-turn treatment code[1]				
Left-turn position (leading, lagging, or N/A)				
Left-turn max sneakers, v_{LS} (veh)				
Left-turn free queue, Q_f (veh)				
Approach speed, S_A (km/h)				
Ring 1 and 2 termination (simultaneous, independent, or N/A)				

Phase-Specific Data

	Ring 1				Ring 2			
	1. WBLT	2. EBTH	3. NBLT	4. SBTH	5. EBLT	6. WBTH	7. SBLT	8. NBTH
Phase type[2] (L, T, G, N, X)								
Phase reversal (Yes or No)								
Detector length, DL (m)								
Detector setback, DS (m)								
Max initial interval, MxI (s)								
Added initial per actuation, AI (s)								
Min allowable gap, MnA (s)								
Gap reduction rate, GR								
Ped Walk + Don't Walk, WDW (s)								
Maximum green, MxG (s)								
Intergreen time, Y (s)								
Recall mode (min, max, ped, none)								
Min veh phase time, MnV (s)								

Notes

1. (0) Does not exist (1) Permitted (2) Protected (3) Protected + Permitted (4) Not opposed
2. (L) Protected left turn on a green arrow
   (T) Through and right-turning traffic only
   (G) Permitted left turns and compound left-turn protection
   (N) In addition to other movements, left turns are not opposed
   (X) Inactive phases

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} (refer to Exhibit 16-11)				
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} (refer to Exhibit C16-3)				
$P_L = P_{LT}\left[1 + \frac{(N-1)g}{(g_f + g_u/E_{L1} + 4.24)}\right]$ (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 Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 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 $f_{LT} = f_m$.

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

	EB	WB	NB	SB
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} (refer to Exhibit 16-11)				
$g_f = G[e^{-0.860(LTC^{0.629})}] - t_L$ $\quad g_f \le g$ (except exclusive left-turn lanes)[3]				
Opposing queue ratio, $qr_o = \max[1 - R_{po}(g_o/C), 0]$				
$g_q = 4.943v_{olc}^{0.762}qr_o^{1.061} - t_L$ $\quad g_q \le g$				
$g_u = g - g_q$ 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 Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 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 $g_{diff} = 0$.

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				
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 – 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 $P_{RTA} = 0$.

INITIAL QUEUE DELAY WORKSHEET

General Information

Project Description ______________________________

Input Parameters

Period (i) ________________

Duration, T ________________ h

Cycle length, C ________________ s

	EB			WB			NB			SB		
Lane group												
Initial queue, Q_b (veh)												
Green ratio, g/C												
v/c ratio, X X = v/c												
Adjusted lane group capacity, c (veh/h)												
Duration of unmet demand in T (h) $t = \min\left\{T, \frac{Q_b}{c[1 - \min(1,X)]}\right\}$												
Case												

Cases I and II (Q_b = 0)

Initial queue delay, d_3 = 0, and uniform delay , d_1, is as shown on Capacity and LOS Worksheet

Case III (Q_b > 0) (X ≤ 1.0) (t < T)

Initial queue delay, d_3 (s) $d_3 = \frac{1800\ Q_b t}{cT}$												
Uniform delay, d_1 (s) $d_1 = 0.50C(1 - g/C)\frac{t}{T} + \frac{0.5C(1 - g/C)^2}{1 - g/C \min(1, X)}\left[\frac{(T - t)}{T}\right] * PF$												

Case IV (Q_b > 0) (X ≤ 1.0) (t = T)

Delay parameter, u $u = 1 - \frac{cT}{Q_b}\left[1 - \min(1,X)\right]$												
Initial queue delay, d_3 (s) $d_3 = \frac{1800\ Q_b\ (1 + u)}{c}$												
Uniform delay, d_1 (s) $d_1 = 0.50C(1 - g/C)$												

Case V (Q_b > 0) (X > 1.0) (t = T)

Initial queue delay, d_3 (s) $d_3 = \frac{3600\ Q_b}{c}$												
Uniform delay, d_1 (s) $d_1 = 0.50C\ (1 - g/C)$												

BACK-OF-QUEUE WORKSHEET

General Information

Project Description ____

Average Back of Queue

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Lane group												
Initial queue per lane at the start of analysis period, Q_{bL}												
Flow rate per lane, v_L (veh/h)												
Saturation flow rate per lane, s_L (veh/h)												
Capacity per lane, c_L (veh/h)												
Flow ratio, v_L/s_L												
v/c ratio, $X_L = v_L/c_L$												
Effective green time, g (s)												
Green ratio, g/C												
Upstream filtering factor, I												
Proportion of vehicles arriving on green, P												
Platoon ratio, R_p $\quad R_p = \left(\frac{P}{g/C}\right)$												
Effects of progression adjustment factor, PF_2 $\quad PF_2 = \frac{\left(1 - R_p\frac{g}{C}\right)\left(1 - \frac{v_L}{s_L}\right)}{\left(1 - \frac{g}{C}\right)\left[1 - R_p\left(\frac{v_L}{s_L}\right)\right]}$												
First-term queued vehicles, Q_1 (veh) $\quad Q_1 = PF_2 \frac{\frac{v_L C}{3600}\left(1 - \frac{g}{C}\right)}{\left[1 - \min(1.0, X_L)\left(\frac{g}{C}\right)\right]}$												
Second-term adjustment factor, k_B $k_B = 0.12\, I\left(\frac{s_L g}{3600}\right)^{0.7}$ (pretimed signals) $k_B = 0.10\, I\left(\frac{s_L g}{3600}\right)^{0.6}$ (actuated signals)												
Second-term queued vehicles, Q_2 $\quad Q_2 = 0.25 c_L T\left[(X_L - 1) + \sqrt{(X_L - 1)^2 + \frac{8k_B X_L}{c_L T} + \frac{16 k_B Q_{bL}}{(c_L T)^2}}\right]$												
Average number of queued vehicles, Q $Q = Q_1 + Q_2$												

Percentile Back of Queue

❑ 70% ❑ 85% ❑ 90% ❑ 95% ❑ 98%

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Percentile back-of-queue factor,[1] $f_{B\%}$												
Percentile back-of-queue, $Q_{\%}$ (veh), $Q_{\%} = Qf_{B\%}$												

Queue Storage Ratio

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Average queue spacing, L_h (m)												
Available queue storage, L_a (m)												
Average queue storage ratio, $R_Q = \frac{L_h Q}{L_a}$												
Percentile queue storage ratio, $R_{Q\%} = \frac{L_h Q_{\%}}{L_a}$												

Notes

1. $f_{B\%} = p_1 + p_2 e^{\left(\frac{-Q}{p_3}\right)}$, where p_1, p_2, and p_3 are obtained from Exhibit G16-5.

INTERSECTION CONTROL DELAY WORKSHEET

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

Input Initial Parameters

Number of lanes, N	________	Total vehicles arriving, V_{tot}	________
Free-flow speed, FFS (km/h)	________	Stopped-vehicle count, V_{stop}	________
Survey count interval, I_s (s)	________	Cycle length, C (s)	________

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue									
		Count Interval									
		1	2	3	4	5	6	7	8	9	10
Total											

Computations

Total vehicles in queue, ΣV_{iq} =	________	Number of cycles surveyed, N_c =	________
Time-in-queue per vehicle, $d_{vq} = (I_s * \frac{\Sigma V_{iq}}{V_{tot}}) * 0.9$	________ s	Fraction of vehicles stopping, $FVS = \frac{V_{stop}}{V_{tot}}$	________
No. of vehicles stopping per lane each cycle = $\frac{V_{stop}}{(N_c \times N)}$	________	Accel/Decel correction delay, d_{ad} = FVS * CF	________ s
Accel/Decel correction factor, CF (Ex. A16-2)	________	Control delay/vehicle, $d = d_{vq} + d_{ad}$	________ s

FIELD SATURATION FLOW RATE STUDY WORKSHEET

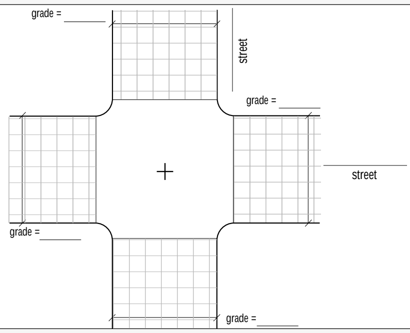
General Information

Analyst ______________________
Agency or Company ______________________
Date Performed ______________________
Analysis Time Period ______________________

Site Information

Intersection ______________________
Area Type ❑ CBD ❑ Other
Jurisdiction ______________________
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