CHAPTER 10

URBAN STREET CONCEPTS

CONTENTS

EXHIBITS

I. INTRODUCTION

In this chapter, capacity and quality of service concepts for urban streets are introduced. The term “urban streets,” as used in this manual, refers to urban arterials and collectors, including those in downtown areas. Methodologies found in Chapter 15 (Urban Streets), Chapter 16 (Signalized Intersections), and Chapter 17 (Unsignalized Intersections) can be used in conjunction with this chapter.

II. URBAN STREETS

In the hierarchy of street transportation facilities, urban streets (including arterials and collectors) are ranked between local streets and multilane suburban and rural highways. The difference is determined principally by street function, control conditions, and the character and intensity of roadside development.

Arterial streets are roads that primarily serve longer through trips. However, providing access to abutting commercial and residential land uses is also an important function of arterials. Collector streets provide both land access and traffic circulation within residential, commercial, and industrial areas. Their access function is more important than that of arterials, and unlike arterials their operation is not always dominated by traffic signals.

Downtown streets are signalized facilities that often resemble arterials. They not only move through traffic but also provide access to local businesses for passenger cars, transit buses, and trucks. Turning movements at downtown intersections are often greater than 20 percent of total traffic volume because downtown flow typically involves a substantial amount of circulatory traffic.

Pedestrian conflicts and lane obstructions created by stopping or standing taxicabs, buses, trucks, and parking vehicles that cause turbulence in the traffic flow are typical of downtown streets. Downtown street function may change with the time of day; some downtown streets are converted to arterial-type operation during peak traffic hours.

Multilane suburban and rural highways differ from urban streets in the following ways: roadside development is not as intense, density of traffic access points is not as high, and signalized intersections are more than 3.0 km apart. These conditions result in a smaller number of traffic conflicts, smoother flow, and dissipation of the platoon structure associated with traffic flow on an arterial or collector with traffic signals.

The urban streets methodology described in this chapter and in Chapter 15 can be used to assess the mobility function of the urban street. The degree of mobility provided is assessed in terms of travel speed for the through-traffic stream. A street’s access function is not assessed by this methodology. The level of access provided by a street should also be considered in evaluating its performance, especially if the street is intended to provide such access. Oftentimes, factors that favor mobility reflect minimal levels of access and vice versa.

Functional class defined

The functional classification of an urban street is the type of traffic service the street provides. Within the functional classification, the arterial is further classified by its design category. Illustrations 10-1 through 10-4 show typical examples of four design categories that are described in the following sections.

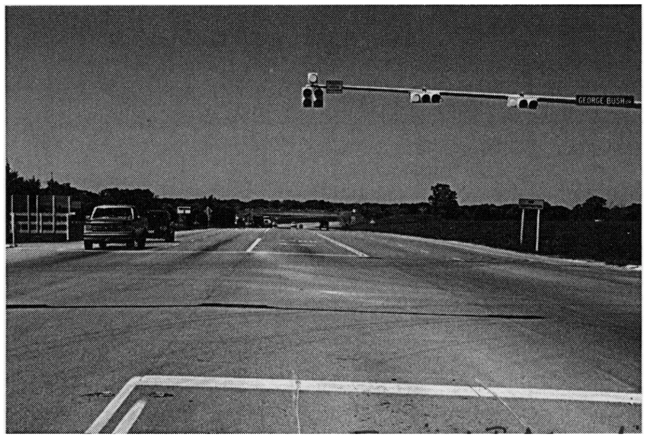


ILLUSTRATION 10-1. Typical high-speed design.

ILLUSTRATION 10-2. Typical suburban design.

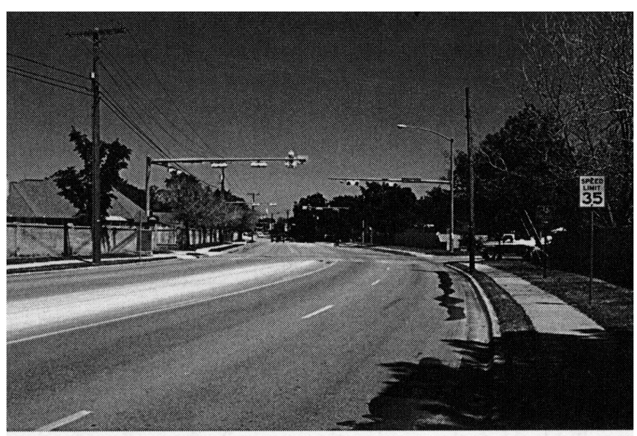


ILLUSTRATION 10-3. Typical intermediate design.

ILLUSTRATION 10-4. Typical urban design.

FLOW CHARACTERISTICS

The speed of vehicles on urban streets is influenced by three main factors: street environment, interaction among vehicles, and traffic control. As a result, these factors also affect quality of service.

The street environment includes the geometric characteristics of the facility, the character of roadside activity, and adjacent land uses. Thus, the environment reflects the number and width of lanes, type of median, driveway/access-point density, spacing between signalized intersections, existence of parking, level of pedestrian activity, and speed limit.

The interaction among vehicles is determined by traffic density, the proportion of trucks and buses, and turning movements. This interaction affects the operation of vehicles at intersections and, to a lesser extent, between signals.

Traffic control (including signals and signs) forces a portion of all vehicles to slow or stop. The delays and speed changes caused by traffic control devices reduce vehicle speeds; however, such controls are needed to establish right-of-way.

Free-Flow Speed

The street environment affects the driver's speed choice. When vehicle interaction and traffic control are not factors, the speed chosen by the average driver is referred to as the free-flow speed (FFS). FFS is the average speed of the traffic stream when traffic volumes are sufficiently low that drivers are not influenced by the presence of other vehicles and when intersection traffic control (i.e., signal or sign) is not present or is sufficiently distant as to have no effect on speed choice. As a consequence, FFS is typically observed along midblock portions of the urban street segment.

Running Speed

A driver can seldom travel at the FFS. Most of the time, the presence of other vehicles restricts the speed of a vehicle in motion because of differences in speeds among drivers or because downstream vehicles are accelerating from a stop and have not yet reached FFS. As a result, vehicle speeds tend to be lower than the FFS during moderate to high-volume conditions.

One speed characteristic that captures the effect of interaction among vehicles is the average running speed. This speed is computed as the length of the segment divided by the average running time. The running time is the time taken to traverse the street segment, less any stop-time delay.

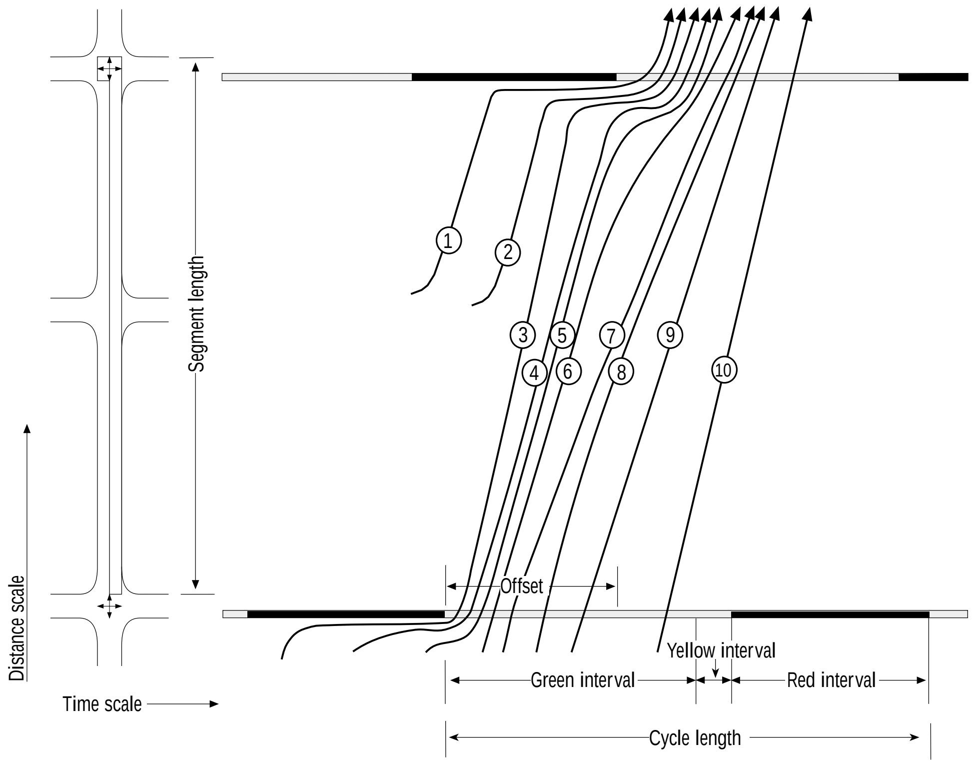
Travel Speed

The presence of traffic control on a street segment tends to reduce vehicle speeds below the average running speed. A speed characteristic that captures the effect of traffic control is average travel speed. This speed is computed as the length of segment divided by the average travel time. The travel time is the time taken to traverse the street segment, inclusive of any stop-time delay.

Time-Space Trajectory

Exhibit 10-1 shows simplified time-space trajectories of representative vehicles along one lane of an urban street. The slope of each line reflects the corresponding vehicle speed at a given time. Steeper slopes represent higher speeds; horizontal slopes represent stopped vehicles.

EXHIBIT 10-1. TYPICAL SPEED PROFILES OF VEHICLES ON URBAN STREETS



Vehicles 1 and 2 turned onto the street from side streets, while the other vehicles were discharged from the upstream signal. Vehicles 1, 2, and 3 arrived at the downstream signal during the red interval and had to stop. Vehicle 4 could have arrived at the stop line on the green but had to stop because it was blocked by Vehicle 3, which was not yet in motion.

Vehicles 5, 6, and 7 did not stop but had to reduce their speeds because they were affected by the stoppages caused by the signal. Vehicle 8 was slowed by Vehicle 7. The speeds of Vehicles 9 and 10 were not affected by the presence of other vehicles or the downstream traffic control.

LEVELS OF SERVICE

The average travel speed for through vehicles along an urban street is the determinant of the operating level of service (LOS). The travel speed along a segment, section, or entire length of an urban street is dependent on the running speed between signalized intersections and the amount of control delay incurred at signalized intersections.

Urban street LOS is based on average through-vehicle travel speed for the segment, section, or entire urban street under consideration. The following general statements characterize LOS along urban streets. Refer to Exhibit 15-2 for speed ranges for each LOS.

LOS is based on average through-vehicle travel speed for the urban street segment

LOS A describes primarily free-flow operations at average travel speeds, usually about 90 percent of the FFS for the given street class. Vehicles are completely unimpeded in their ability to maneuver within the traffic stream. Control delay at signalized intersections is minimal.

LOS B describes reasonably unimpeded operations at average travel speeds, usually about 70 percent of the FFS for the street class. The ability to maneuver within the traffic stream is only slightly restricted, and control delays at signalized intersections are not significant.

LOS C describes stable operations; however, ability to maneuver and change lanes in midblock locations may be more restricted than at LOS B, and longer queues, adverse signal coordination, or both may contribute to lower average travel speeds of about 50 percent of the FFS for the street class.

LOS D borders on a range in which small increases in flow may cause substantial increases in delay and decreases in travel speed. LOS D may be due to adverse signal progression, inappropriate signal timing, high volumes, or a combination of these factors. Average travel speeds are about 40 percent of FFS.

LOS E is characterized by significant delays and average travel speeds of 33 percent or less of the FFS. Such operations are caused by a combination of adverse progression, high signal density, high volumes, extensive delays at critical intersections, and inappropriate signal timing.

LOS F is characterized by urban street flow at extremely low speeds, typically one-third to one-fourth of the FFS. Intersection congestion is likely at critical signalized locations, with high delays, high volumes, and extensive queuing.

REQUIRED INPUT DATA AND ESTIMATED VALUES

Estimating speed, delay, and LOS for an urban street or an intersection requires geometric data and demand data. Signal control data will be discussed in the signalized intersections section. Exhibit 10-2 gives default values for input parameters in the absence of local data.

EXHIBIT 10-2. REQUIRED INPUT DATA FOR URBAN STREETS

Item	Default
Geometric Data	
Urban street class	Exhibits 10-3, 10-4
Length	-
Free-flow speed	Exhibit 10-5
Intersection Control Data	
Signal density	Exhibit 10-6
Intersection delay	See Section III of this chapter

The analyst should note that taking field measurements for use as inputs to an analysis is the most reliable way to generate parameter values. Default values should be considered only when this is not feasible.

Urban Street Class

The urban street classification system is somewhat different from that used by the American Association of State Highway and Transportation Officials (AASHTO) (*1*).

AASHTO's functional classes are based on travel volume, mileage, and the characteristic of service the urban street is intended to provide. The analysis method in this manual makes use of the AASHTO distinction between principal arterial and minor arterial. But a second classification step is used herein to determine the appropriate design category for the arterial. The design category depends on the posted speed limit, signal density, driveway/access-point density, and other design features. The third step is to determine the appropriate urban street class on the basis of a combination of functional category and design category. Exhibits 10-3 and 10-4 are useful for establishing urban street class.

Four urban street classes are defined in this manual. The classes are designated by number (i.e., I, II, III, and IV) and reflect unique combinations of street function and design, as shown in Exhibit 10-3. The functional component is separated into two categories: principal arterial and minor arterial. The design component is separated into four categories: high-speed, suburban, intermediate, and urban. The characteristics associated with each category are described in the remainder of this section. Exhibit 10-4 summarizes these characteristics.

EXHIBIT 10-3. URBAN STREET CLASS BASED ON FUNCTIONAL AND DESIGN CATEGORIES

	Functional Category	
Design Category	Principal Arterial	Minor Arterial
High-Speed	I	N/A
Suburban	II	II
Intermediate	II	III or IV
Urban	III or IV	IV

EXHIBIT 10-4. FUNCTIONAL AND DESIGN CATEGORIES

	Functional Category			
Criterion	Principal Arterial		Minor Arterial	
Mobility function	Very important		Important	
Access function	Very minor		Substantial	
Points connected	Freeways, important activity centers, major traffic generators		Principal arterials	
Predominant trips served	Relatively long trips between major points and through-trips entering, leaving, and passing through the city		Trips of moderate length within relatively small geographical areas	
	Design Category			
Criterion	High-Speed	Suburban	Intermediate	Urban
Driveway/access density	Very low density	Low density	Moderate density	High density
Arterial type	Multilane divided; undivided or two-lane with shoulders	Multilane divided; undivided or two-lane with shoulders	Multilane divided or undivided; one-way, two-lane	Undivided one-way, two-way, two or more lanes
Parking	No	No	Some	Significant
Separate left-turn lanes	Yes	Yes	Usually	Some
Signals/km	0.3–1.2	0.6–3.0	2–6	4–8
Speed limit	75–90 km/h	65–75 km/h	50–65 km/h	40–55 km/h
Pedestrian activity	Very little	Little	Some	Usually
Roadside development	Low density	Low to medium density	Medium to moderate density	High density

A principal arterial serves major through movements between important centers of activity in a metropolitan area and a substantial portion of trips entering and leaving the area. It also connects freeways with major traffic generators. In smaller cities (population under 50,000), its importance is derived from the service provided to traffic passing through the urban area. Service to abutting land is subordinate to the function of moving through traffic.

A minor arterial connects and augments the principal arterial system. Although its main function is traffic mobility, it performs this function at a lower level and places more emphasis on land access than does the principal arterial. A system of minor arterials serves trips of moderate length and distributes travel to geographical areas smaller than those served by the principal arterial.

The urban street is further classified by its design category. Exhibit 10-3 shows urban street classes based on functional and design categories.

High-speed design defined

High-speed design represents an urban street with a very low driveway/access-point density, separate left-turn lanes, and no parking. It may be multilane divided or undivided or a two-lane facility with shoulders. Signals are infrequent and spaced at long distances. Roadside development is low density, and the speed limit is typically 75 to 90 km/h. This design category includes many urban streets in suburban settings.

Suburban design defined

Suburban design represents a street with a low driveway/access-point density, separate left-turn lanes, and no parking. It may be multilane divided or undivided or a two-lane facility with shoulders. Signals are spaced for good progressive movement (up to three signals per kilometer). Roadside development is low to medium density, and speed limits are usually 65 to 75 km/h.

Intermediate design defined

Intermediate design represents an urban street with a moderate driveway/access-point density. It may be a multilane divided, an undivided one-way, or a two-lane facility. It may have some separate or continuous left-turn lanes and some portions where parking is permitted. It has a higher density of roadside development than the typical suburban design and usually has two to six signals per kilometer. Speed limits are typically 50 to 65 km/h.

Urban design defined

Urban design represents an urban street with a high driveway/access-point density. It frequently is an undivided one-way or two-way facility with two or more lanes. Parking is usually permitted. Generally, there are few separate left-turn lanes, and some pedestrian interference is present. It commonly has four to eight signals per kilometer. Roadside development is dense with commercial uses. Speed limits range from 40 to 55 km/h.

In addition to the above definitions, Exhibit 10-4 can be used as an aid in the determination of functional and design categories. Once the functional and design categories have been determined, the urban street classification may be established by referring to Exhibit 10-3.

In practice, there are sometimes ambiguities in determining the proper categories. The measurement or estimation of the free-flow speed is a great aid in this determination, because each urban street class has a characteristic range of free-flow speeds, as shown in Chapter 15.

Length

The portion of the urban street being analyzed should be at least 1.5 km long in a downtown area and 3.0 km long elsewhere for the LOS speed criteria to be meaningful. Study lengths shorter than 1.5 km should be analyzed as individual intersections and the LOS assessed according to individual intersection criteria.

Free-Flow Speed

Measure free-flow speed as far as possible from nearest signal or stop-controlled intersection and at flows < 200 veh/h/ln

The free-flow speed is used to determine the urban street class and to estimate the segment running time. If FFS cannot be measured in the field, the analyst should attempt to take measurements on a similar facility in the same area or should resort to established local policies. Lacking any of these options, the analyst might rely on the posted speed limit (or some value around that limit) or on default values in this manual.

Free-flow speed on an urban street is the speed that a vehicle travels under low-volume conditions when all the signals on the urban street are green for the entire trip. Thus, all delay at signalized intersections, even under low flow conditions, is excluded from the computation of urban street FFS. The best location to measure urban street FFS is midblock and as far as possible from the nearest signalized or stop-controlled intersection. This measurement should be made under low flow conditions (less than 200 veh/h/ln). Exhibit 10-5 gives default FFS by urban street class for use in the absence of local data.

EXHIBIT 10-5. FREE-FLOW SPEED BY URBAN STREET CLASS

Urban Street Class	Default (km/h)
I	80
II	65
III	55
IV	45

Signal Density

Signal density defined

Signal density is the number of signalized intersections in the study portion of the urban street divided by its length. If signalized intersections are used to define both the beginning and the ending points of the study portion of the urban street, then the number of signals in the study portion should be reduced by one in computing the signal density. Exhibit 10-6 gives defaults by urban street class that may be used in the absence of local data.

EXHIBIT 10-6. SIGNAL DENSITY BY URBAN STREET CLASS

Urban Street Class	Default (signals/km)
I	0.5
II	2
III	4
IV	6

Peak-Hour Factor

In the absence of field measurements of peak-hour factor (PHF), approximations can be used. For congested conditions, 0.92 is a reasonable approximation for PHF. For conditions in which there is fairly uniform flow throughout the peak hour but a recognizable peak does occur, 0.88 is a reasonable estimate for PHF.

Length of Analysis Period

The analytical procedures for estimating speed for an urban street depend on the estimation of delay for the signalized and unsignalized intersections on the street. The delay equations for signalized and unsignalized intersections are most accurate when the demand is less than capacity for the selected analysis period. If the demand exceeds capacity, the intersection delay equations will estimate the delay for all vehicles arriving during the analysis period but will not determine the effect of the excess demand (the residual queue for the next period) on the vehicles arriving during the next analysis period.

The typical analysis period is 15 min. However, if demand creates a residual queue for the 15-min analysis period (i.e., v/c greater than 1.00), the analyst should consider the use of multiple analysis periods or a single longer analysis period to improve the delay estimate.

If a multiple-period analysis is selected, the analyst must carry over the residual queue from one period to the next as discussed in Chapter 16, Appendix F (Extension of Signal Delay Models to Incorporate the Effect of an Initial Queue). The analyst will have to modify or adapt these procedures in the case of unsignalized intersections. Speed, delay, and LOS can then be computed for each analysis period. The analyst must determine how to report these results, since averaging LOS across multiple analysis periods may obscure some of the results.

If a single longer analysis period is selected (such as 1 h), the analyst should use caution in performing the analysis and interpreting the results. The peak-hour factor (which normally is used to compute the peak 15-min flow rate from a 1-h volume) may have to be modified to provide the appropriate flow rate for the longer analysis period. The analyst must also recognize that LOS criteria for urban streets, signalized intersections, and unsignalized intersections were developed for a 15-min analysis period. Conditions that persist for longer periods (presumably with worse peak conditions within those periods) may no longer meet the 15-min LOS criteria provided in this manual.

SERVICE VOLUME TABLE

Exhibit 10-7 is an example service volume table for the four urban street classes. This table is useful for estimates of how many vehicles an urban street can carry at a given level of service, for a particular class and number of lanes (per direction). It is most accurate when the defaults shown in Exhibit 10-7 are applicable. If conditions on a given street vary considerably from those used to create this table, the tabular values are not appropriate.

III. SIGNALIZED INTERSECTIONS

The capacity of an urban street is related primarily to the signal timing and the geometric characteristics of the facility as well as to the composition of traffic on the facility. Geometrics are a fixed characteristic of a facility. Thus, while traffic composition may vary somewhat over time, the capacity of a facility is generally a stable value that can be significantly improved only by initiating geometric improvements.

At signalized intersections, the additional element of time allocation is introduced into the concept of capacity. A traffic signal essentially allocates time among conflicting traffic movements that seek to use the same space. The way in which time is allocated significantly affects the operation and the capacity of the intersection and its approaches.

Lane group defined

In analyzing a signalized intersection, the physical unit of analysis is the lane group. A lane group consists of one or more lanes on an intersection approach. The outputs from application of the method in this manual are reported on the basis of each lane group.

SIGNALIZED INTERSECTION FLOW CHARACTERISTICS

For a given lane group at a signalized intersection, three signal indications are displayed: green, yellow, and red. The red indication may include a short period during which all indications are red, referred to as an all-red interval, which with the yellow indication forms the change and clearance interval between two green phases.

Exhibit 10-8 provides a reference for much of the discussion in this section. It presents some fundamental attributes of flow at signalized intersections. The diagram represents a simple situation of a one-way approach to a signalized intersection having two phases in the cycle.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions used. It should not be used for operational analyses or final design. This table was derived using assumed values listed in the footnote.

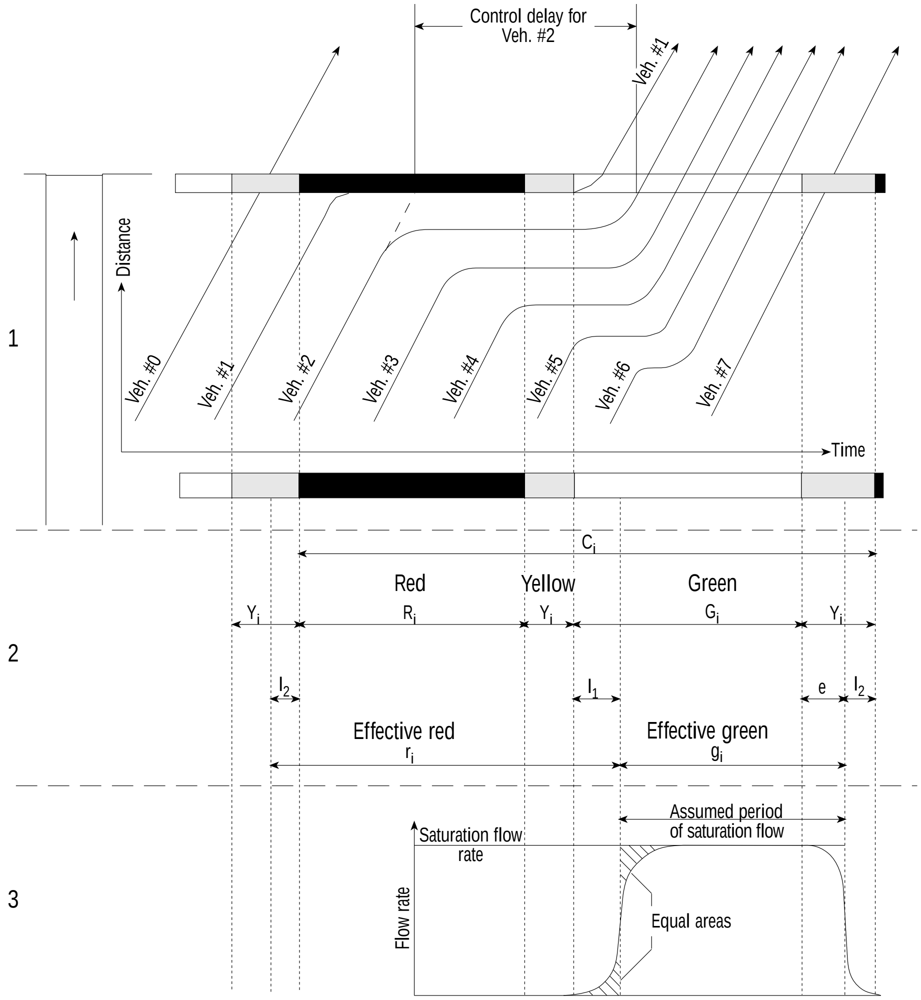
EXHIBIT 10-7. EXAMPLE SERVICE VOLUMES FOR URBAN STREETS
(SEE FOOTNOTES FOR ASSUMED VALUES)

Lanes	Service Volumes (veh/h)				
	A	B	C	D	E
Class I					
1	N/A	740	920	1010	1110
2	N/A	1490	1780	1940	2120
3	N/A	2210	2580	2790	3040
4	N/A	2970	3440	3750	4060
Class II					
1	N/A	N/A	620	820	860
2	N/A	N/A	1290	1590	1650
3	N/A	N/A	1920	2280	2370
4	N/A	N/A	2620	3070	3190
Class III					
1	N/A	N/A	600	790	840
2	N/A	N/A	1250	1530	1610
3	N/A	N/A	1870	2220	2310
4	N/A	N/A	2580	2960	3080
Class IV					
1	N/A	N/A	270	690	790
2	N/A	N/A	650	1440	1520
3	N/A	N/A	1070	2110	2180
4	N/A	N/A	1510	2820	2900

Notes
N/A - not achievable given assumptions below.
This table was derived from the conditions listed in the following table.

	Class			
	I	II	III	IV
Signal density (sig/km)	0.5	2	3	6
Free-flow speed (km/h)	80	65	55	45
Cycle length (s)	110	90	80	70
Effective green ratio	0.45	0.45	0.45	0.45
Adj. sat. flow rate	1850	1800	1750	1700
Arrival type	3	4	4	5
Unit extension (s)	3	3	3	3
Initial queue	0	0	0	0
Other delay	0	0	0	0
Peak-hour factor	0.92	0.92	0.92	0.92
% lefts, % rights	10	10	10	10
Left-turn bay	Yes	Yes	Yes	Yes
Lane utilization factor	According to Exhibit 10-23, Default Lane Utilization Factors			

EXHIBIT 10-8. FUNDAMENTAL ATTRIBUTES OF FLOW AT SIGNALIZED INTERSECTIONS



The exhibit is divided into three parts. The first part shows a time-space plot of vehicles on the northbound approach to the intersection. The intervals for the signal cycle are indicated in the diagram. The second part repeats the timing intervals and labels the various time intervals of interest with the symbols used throughout this chapter. The third part is an idealized plot of flow rate passing the stop line, indicating how saturation flow is defined. Further definitions of these variables and other basic terms are provided in Exhibit 10-9.

Effective green defined

Effective red defined

The signal cycle for a given lane group has two simplified components: effective green and effective red. Effective green time is the time that may be used by vehicles on the subject lane group at the saturation flow rate. Effective red time is defined as the cycle length minus the effective green time.

It is important that the relationship between the actual green, yellow, and red times shown on signal faces and the effective green and red times be understood. Each time a movement is started and stopped, two lost times are experienced. At the beginning of movement, the first several vehicles in the queue experience start-up losses that result in movement at less than the saturation flow rate (Exhibit 10-8). At the end of a movement, a portion of the change and clearance interval (yellow and all-red) is not used for vehicular movement.

EXHIBIT 10-9. SYMBOLS, DEFINITIONS, AND UNITS FOR FUNDAMENTAL VARIABLES OF TRAFFIC FLOW AT SIGNALIZED INTERSECTIONS

Name	Symbol	Definition	Unit
Change and clearance interval	Y_i	The yellow plus all-red interval that occurs between phases of a traffic signal to provide for clearance of the intersection before conflicting movements are released	s
Clearance lost time	l_2	The time between signal phases during which an intersection is not used by any traffic	s
Control delay	d_i	The component of delay that results when a control signal causes a lane group to reduce speed or to stop; it is measured by comparison with the uncontrolled condition	s
Cycle		A complete sequence of signal indications	
Cycle length	C_i	The total time for a signal to complete one cycle	s
Effective green time	g_i	The time during which a given traffic movement or set of movements may proceed; it is equal to the cycle length minus the effective red time	s
Effective red time	r_i	The time during which a given traffic movement or set of movements is directed to stop; it is equal to the cycle length minus the effective green time	s
Extension of effective green time	e	The amount of the change and clearance interval, at the end of the phase for a lane group, that is usable for movement of its vehicles	s
Green time	G_i	The duration of the green indication for a given movement at a signalized intersection	s
Interval		A period of time in which all traffic signal indications remain constant	
Lost time	t_L	The time during which an intersection is not used effectively by any movement; it is the sum of clearance lost time plus start-up lost time	s
Phase		The part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals	
Red time	R_i	The period in the signal cycle during which, for a given phase or lane group, the signal is red	s
Saturation flow rate	s_i	The equivalent hourly rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal is available at all times and no lost times are experienced	veh/h
Start-up lost time	l_1	The additional time consumed by the first few vehicles in a queue at a signalized intersection above and beyond the saturation headway, because of the need to react to the initiation of the green phase and to accelerate	s
Total lost time	L	The total lost time per cycle during which the intersection is effectively not used by any movement, which occurs during the change and clearance intervals and at the beginning of most phases	s

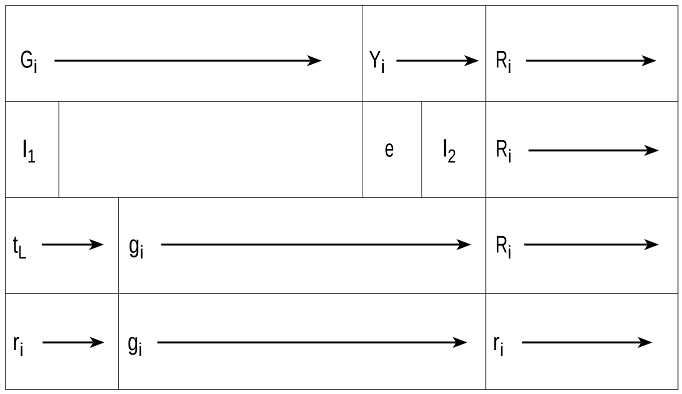
Start-up and clearance lost times are combined and considered to occur at the start of a lane group movement

At the beginning of green, the start-up losses are called start-up lost time (l_1). At the beginning of the yellow, when vehicles tend to continue to enter the intersection for a short period of time, an extension of effective green (e) is experienced. When this extension of green has been exhausted, the remainder of the change and clearance interval is considered to be clearance lost time (l_2). The lost time for a lane group, t_L, is the sum of the start-up and clearance lost times.

Research (*2*) has shown that start-up lost time (l_1) is about 2 s and that the extension of effective green (e) is about 2 s (sometimes longer under congested conditions). Thus, the relationship shown in Equation 10-1 exists for typical conditions, and the relationship among actual green, lost time, extension of effective green, and effective green is shown in Exhibit 10-10. When $l_1 = 2$ and $e = 2$ (typical), then $t_L = Y_i$.

$$t_L = l_1 + l_2 = l_1 + Y_i - e \tag{10-1}$$

EXHIBIT 10-10. RELATIONSHIP AMONG ACTUAL GREEN, LOST-TIME ELEMENTS, EXTENSION OF EFFECTIVE GREEN, AND EFFECTIVE GREEN



As shown in Exhibit 10-10, the lost time for the movement is deducted from the beginning of the actual green phase. Thus, a small portion of G_i becomes part of the effective red, r_i. This portion is equal to the lost time for the movement, t_L. Because all of the lost time for the movement is deducted at the beginning of the green, effective green can be assumed to run through the end of the yellow-plus-all-red change and clearance interval, Y_i. Thus, for any given movement, effective green time is computed by Equation 10-2 and effective red time by Equation 10-3.

$$g_i = G_i + Y_i - t_L \tag{10-2}$$

$$r_i = R_i + t_L \tag{10-3}$$

A lost time is applied each time a movement is started

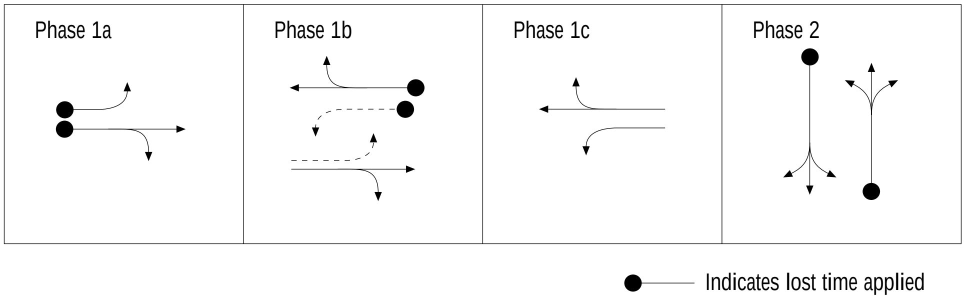
The simplified concept of applying all of the lost time at the beginning of a movement makes it easier to analyze more complex signalization involving protected-plus-permitted left-turn phasing. As a general rule, a lost time, t_L, is applied each time a movement is started. Thus, where a given movement starts in a protected phase and continues through a permitted phase (or vice versa), only one lost time is deducted. No lost time is assumed to occur at the boundary between the permitted and protected phases for continuing movements.

Exhibit 10-11 diagrams a more complex situation involving a protected-plus-permitted and permitted-plus-protected compound left-turn phasing, a classic lead-lag phasing scheme in which left turns are protected in Phase 1a [eastbound (EB)] and Phase 1c [westbound (WB)] and permitted during the common Phase 1b. The question of how many lost times are included in such a phase sequence is important. Using the general rule that the entire lost time for a movement is applied at the time the movement begins, the following may be determined:

- In Phase 1a, the EB through and left-turn movements begin. Thus, a lost time is applied to both movements.
- In Phase 1b, the EB through and left-turn movements continue. No lost times are assigned to the continuing movements in this phase. The WB through and left-turn movements begin in this phase, and a lost time is applied to these movements.
- In Phase 1c, only the WB through and left turns continue. Because these movements did not start in this phase, no lost time is applied here. Further, because no movements begin in Phase 1c, no lost time is applied to any movement in Phase 1c.

• In Phase 2, northbound (NB) and southbound (SB) movements begin, and a lost time is applied.

EXHIBIT 10-11. LOST TIME APPLICATION FOR COMPOUND LEFT-TURN PHASING



Total lost time is the sum of lost time for the path through the critical movements

The total lost time in the signal cycle, L, is also important. This is the total lost time involved in the critical path through the signal cycle. Determining the critical path and finding L are discussed in Chapter 16.

TRAFFIC SIGNAL CHARACTERISTICS

Modern traffic signals allocate time in a variety of ways, from the simplest two-phase pretimed mode to the most complex multiphase actuated mode.

There are three types of traffic signal controllers:

• Pretimed, in which a sequence of phases is displayed in repetitive order. Each phase has a fixed green time and change and clearance interval that are repeated in each cycle to produce a constant cycle length.

• Fully actuated, in which the timing on all of the approaches to an intersection is 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 varies from cycle to cycle.

• Semiactuated, in which some approaches (typically on the minor street) have detectors and some of the approaches (typically on the major street) have no detectors.

While 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 level-of-service analysis, it is no longer sufficient to use 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.

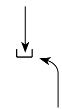
Each traffic movement may be served by a phase that is either actuated or nonactuated. Signal phases may be coordinated with neighboring signals on the same route, or they may function in an isolated mode without influence from other signals. Nonactuated phases generally operate with fixed minimum green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

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.

Not only the allocation of green time but also the manner in which turning movements are accommodated within the phase sequence significantly affects capacity and operations at a signalized intersection. Signal phasing can provide for protected, permitted, or not opposed turning movements.

Permitted turning movement

Protected turning movement



A permitted turning movement is made through a conflicting pedestrian or bicycle flow or opposing vehicle flow. Thus, a left-turn movement concurrent with the opposing through movement is considered to be permitted, as is a right-turn movement concurrent with pedestrian crossings in a conflicting crosswalk. Protected turns are those made without these conflicts, such as turns made during an exclusive left-turn phase or a right-turn phase during which conflicting pedestrian movements are prohibited. Permitted turns experience the friction of selecting and passing through gaps in a conflicting vehicle or pedestrian flow. Thus, a single permitted turn often consumes more of the available green time than a single protected turn. Either permitted or protected turning phases may be more efficient in a given situation, depending on the turning and opposing volumes, intersection geometry, and other factors.

Turning movements that are not opposed do not receive a dedicated left-turn phase (i.e., a green arrow), but because of the nature of the intersection, they are never in conflict with through traffic. This condition occurs on one-way streets, at T-intersections, and with signal phasing plans that provide complete separation between all movements in opposite directions (i.e., split-phase operation). Such movements must be treated differently in some cases because they can be accommodated in shared lanes without impeding the through traffic. Left turns that are not opposed at any time should be distinguished from those that may be unopposed during part of the signal cycle and opposed during another part. Left turns that are opposed during any part of the sequence will impede through traffic in shared lanes.

SATURATION FLOW RATE

Saturation flow rate is a basic parameter used to derive capacity. It is defined in Exhibits 10-8 and 10-9. It is essentially determined on the basis of the minimum headway that the lane group can sustain across the stop line as the vehicles depart the intersection. Saturation flow rate is computed for each of the lane groups established for the analysis. A saturation flow rate for prevailing conditions can be determined directly from field measurement and can be used as the rate for the site without adjustment. If a default value is selected for base saturation flow rate, it must be adjusted for a variety of factors that reflect geometric, traffic, and environmental conditions specific to the site under study.

SIGNALIZED INTERSECTION CAPACITY

Lane group capacity defined

Capacity at intersections is defined for each lane group. The lane group capacity is the maximum hourly rate at which vehicles can reasonably be expected to pass through the intersection under prevailing traffic, roadway, and signalization conditions. The flow rate is generally measured or projected for a 15-min period, and capacity is stated in vehicles per hour (veh/h).

Traffic conditions include volumes on each approach, the distribution of vehicles by movement (left, through, and right), the vehicle type distribution within each movement, the location and use of bus stops within the intersection area, pedestrian crossing flows, and parking movements on approaches to the intersection. Roadway conditions include the basic geometrics of the intersection, including the number and width of lanes, grades, and lane use allocations (including parking lanes). Signalization conditions include a full definition of the signal phasing, timing, and type of control, and an evaluation of signal progression for each lane group. The analysis of capacity at signalized intersections (Chapter 16) focuses on the computation of saturation flow rates, capacities, v/c ratios, and level of service for lane groups.

LEVEL OF SERVICE

Control delay is the service measure that defines LOS

Level of service for signalized intersections is defined in terms of control delay, which is a measure of driver discomfort, frustration, fuel consumption, and increased travel time. The delay experienced by a motorist is made up of a number of factors that relate to control, geometrics, traffic, and incidents. Total delay is the difference between the travel time actually experienced and the reference travel time that would result during base conditions: in the absence of traffic control, geometric delay, any incidents, and any other vehicles. Specifically, LOS criteria for traffic signals are stated in terms of the average control delay per vehicle, typically for a 15-min analysis period. Delay is a complex measure and depends on a number of variables, including the quality of progression, the cycle length, the green ratio, and the v/c ratio for the lane group.

The critical v/c ratio is an approximate indicator of the overall sufficiency of an intersection. The critical v/c ratio depends on the conflicting critical lane flow rates and the signal phasing. The computation of the critical v/c ratio is described in detail in Appendix A and in Chapter 16.

Back of queue defined

The average back of queue is another performance measure that is used to analyze a signalized intersection. 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. The computation of average back of queue is explained in Appendix G of Chapter 16.

Levels of service are defined to represent reasonable ranges in control delay.

LOS A describes operations with low control delay, up to 10 s/veh. This LOS occurs when progression is extremely favorable and most vehicles arrive during the green phase. Many vehicles do not stop at all. Short cycle lengths may tend to contribute to low delay values.

LOS B describes operations with control delay greater than 10 and up to 20 s/veh. This level generally occurs with good progression, short cycle lengths, or both. More vehicles stop than with LOS A, causing higher levels of delay.

Cycle failure occurs when a given green phase does not serve queued vehicles, and overflows occur

LOS C describes operations with control delay greater than 20 and up to 35 s/veh. These higher delays may result from only fair progression, longer cycle lengths, or both. Individual cycle failures may begin to appear at this level. Cycle failure occurs when a given green phase does not serve queued vehicles, and overflows occur. The number of vehicles stopping is significant at this level, though many still pass through the intersection without stopping.

LOS D describes operations with control delay greater than 35 and up to 55 s/veh. At LOS D, the influence of congestion becomes more noticeable. Longer delays may result from some combination of unfavorable progression, long cycle lengths, and high v/c ratios. Many vehicles stop, and the proportion of vehicles not stopping declines. Individual cycle failures are noticeable.

LOS E describes operations with control delay greater than 55 and up to 80 s/veh. These high delay values generally indicate poor progression, long cycle lengths, and high v/c ratios. Individual cycle failures are frequent.

LOS F describes operations with control delay in excess of 80 s/veh. This level, considered unacceptable to most drivers, often occurs with oversaturation, that is, when arrival flow rates exceed the capacity of lane groups. It may also occur at high v/c ratios with many individual cycle failures. Poor progression and long cycle lengths may also contribute significantly to high delay levels.

Delays in the range of LOS F (unacceptable) can occur while the v/c ratio is below 1.0. Very high delays can occur at such v/c ratios when some combination of the following conditions exists: the cycle length is long, the lane group in question is disadvantaged by the signal timing (has a long red time), and the signal progression for the subject movements is poor. The reverse is also possible (for a limited duration): a saturated lane group (i.e., v/c ratio greater than 1.0) may have low delays if the cycle length is short or the signal progression is favorable, or both.

Thus, the designation LOS F does not automatically imply that the intersection, approach, or lane group is over capacity, nor does an LOS better than E automatically imply that unused capacity is available.

The method in this chapter and Chapter 16 requires the analysis of both capacity and LOS conditions to fully evaluate the operation of a signalized intersection.

REQUIRED INPUT DATA AND ESTIMATED VALUES

Exhibit 10-12 gives default values for input parameters in the absence of local data. If intersection saturation flow is to be estimated as well, then additional saturation flow adjustment data are required. The analyst should note that taking field measurements for use as inputs to an analysis is the most reliable means of generating parameter values. Default values should be considered only when this is not feasible.

EXHIBIT 10-12. REQUIRED DATA FOR SIGNALIZED INTERSECTIONS

Item	Default
Geometric Data	
Exclusive turn lanes	Exhibit 10-13
Demand Data	
Intersection turning movements	-
PHF	0.92
Length of analysis period	0.25 h
Intersection Data	
Control type	-
Cycle	Exhibit 10-16
Lost time	Exhibit 10-17
g/C	-
Arrival type (AT)	3 uncoordinated, 4 coordinated
Unit extension time (UE)	3.0 s
Actuated control adjustment factor (k)	0.40 (planning)
Upstream filtering adjustment factor (I)	1.00
Adjusted saturation flow rate	Exhibit 10-19
Saturation Flow Data	
Base saturation flow rate	1900 pc/h/ln
Lane widths	3.6 m
Heavy vehicles	2 %
Grades	0 %
Parking maneuvers	Exhibit 10-20
Local bus	Exhibit 10-21
Pedestrians	Exhibit 10-22
Area type	-
Lane utilization	Exhibit 10-23

Lane Additions and Drops at Intersections

Short through-lane additions on the approaches to an intersection and short through-lane drops exiting the intersection may not function as full through lanes. The analyst should take this into consideration in determining the equivalent number of through lanes for the approach and in selecting the lane utilization factor for the approach.

Exclusive Turn Lanes

This section summarizes suggestions for establishing the geometric design of an intersection when it has not been defined by existing conditions or by state or local practice (*3*). These suggestions may also be applied when analysis indicates intersection deficiencies that are to be corrected by changes in geometric design. However, nothing in this section should be construed as constituting a strict guideline or standard. This material should not be used in place of applicable state and local standards, guidelines, policies, or practice. Rather it is presented here to indicate general possibilities for improvement of signalized intersections.

Exclusive Left-Turn Lanes

The presence of exclusive left-turn lanes is determined by the volume of left-turn traffic, opposing volumes, and safety considerations (*4*). For analyses of future conditions requiring assumptions about lane configurations, Exhibit 10-13 shows relationships between left-turn volumes and the probable need for left-turn lanes in the absence of local data (*5*).

EXHIBIT 10-13. TURN VOLUMES PROBABLY REQUIRING EXCLUSIVE LEFT-TURN LANES AT SIGNALIZED INTERSECTIONS

Turn Lane	Minimum Turn Volume (veh/h)
Single exclusive left-turn lane	100
Double exclusive left-turn lanes	300

Exclusive left-turn lanes are also required when an exclusive left-turn phase is warranted at a signalized intersection. In the absence of forecast turn volumes, the analyst should assume that exclusive left-turn lanes will be the standard design for all future intersections, except possibly in the central business district (CBD) (if severe right-of-way constraints exist), on a one-way street, or where the operating jurisdiction does not typically construct such lanes.

Exclusive Right-Turn Lanes

Although right turns are generally made more efficiently than left turns, exclusive right-turn lanes are often provided for many of the same reasons that left-turn lanes are used. Right turns may face a conflicting pedestrian or bicycle flow, but they do not face a conflicting vehicular flow. In general, an exclusive right-turn lane should be considered if the right-turn volume exceeds 300 veh/h and the adjacent mainline volume exceeds 300 veh/h/ln.

Number of Lanes

The number of lanes required on an approach depends on a variety of factors, including the signal design. In general, enough main roadway lanes should be provided to prevent the total of the through plus right-turn volume (plus left-turn volume, if present) from exceeding 450 veh/h/ln. This is a very general suggestion. Higher volumes can be accommodated on major approaches if a substantial portion of available green time can be allocated to the subject approach. If the number of lanes is unknown, the foregoing value is a reasonable starting point for analysis.

Other Features

If lane widths are unknown, the 3.6-m standard lane width should be assumed unless known restrictions prevent such width. Parking conditions consistent with local practice should be assumed. If no information exists, no curb parking and no local buses should be assumed for analysis purposes.

The storage bay length of exclusive turn lanes should be sufficient to handle the turning traffic without reducing the safety or capacity of the approach. A method for estimating the required length of the storage bay is presented in Appendix G of Chapter 16.

Intersection Turning Movements

Intersection turning movements are used in the analysis of signalized intersections on urban streets. If signal timing is not known, then the turning movements may also be used to estimate the effective green ratios (g/C) and cycle length for each intersection.

Default Values in Absence of Turning Movement Data

See Chapter 9 for means of estimating peak demands from ADT

An urban street analysis can be performed in the absence of intersection turning movement data if the analyst can obtain directional volume counts for the urban street and estimate the average percentage of turning vehicles on the street approaches at the intersections. Peak-hour volumes by direction can be estimated from average daily traffic (ADT). The estimated percentage of turns is used to reduce the total urban street approach volume at intersections where exclusive turn lanes are provided. The percentage of turns made from exclusive lanes at an intersection is subject to local conditions. In the complete absence of local information, default values of 10 percent to represent right turns and 10 percent for left turns as a percentage of the total approach traffic are suggested.

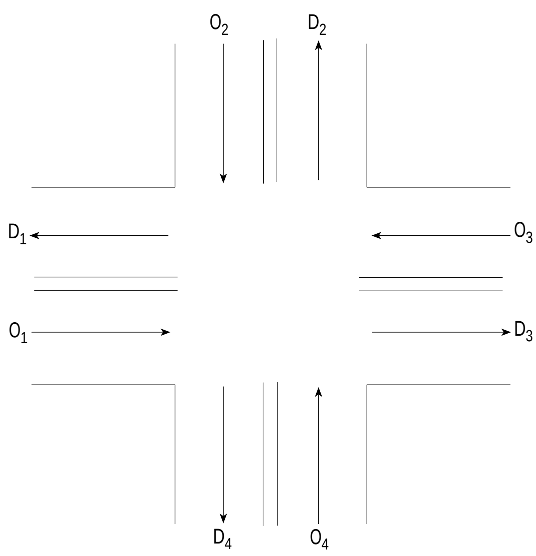
If some of the intersections have exclusive lanes and some do not, the delay at each intersection along the urban street should be computed and summed to obtain the intersection delay for the section. If this is not feasible, the analyst can divide the intersections into two categories: intersections with exclusive lanes and intersections without exclusive lanes. The delay can be computed for each category of intersection, multiplied by the number of intersections in that category, and summed to obtain intersection delay on the urban street.

Turning Movement Estimation

Peak-hour turning movement counts or forecasts are the best source of information on turning movements. In the absence of such information, turning movements can be estimated from approach and departure volumes for each leg of the intersection.

Each approach to the intersection is considered an origin. Each departure leg is a destination as shown in Exhibit 10-14. The problem then becomes one of estimating the origin-destination (O-D) table given the entering and exiting volume on each leg of the intersection.

EXHIBIT 10-14. ORIGIN-DESTINATION LABELS FOR INTERSECTION TURNING MOVEMENTS



This estimation procedure is derived from research (*6*). The procedure assumes that the number of vehicles going from one leg to another is directly proportional to the total volume entering the one leg and the total volume exiting on the other leg. This assumption may not be valid when other factors or geometric situations are present, such as a nearby freeway on-ramp, which may attract a much higher than normal trip volume. Equation 10-4 is used to estimate the turning movement O-D matrix:

$$T_{ij} = \frac{T_i * T_j}{\sum_i T_{ij}} \tag{10-4}$$

where

T_{ij} = number of trips going from origin leg i to destination leg j,
T_i = number of trips originating at origin i, and
T_j = number of trips leaving at destination j.

U-turns ($T_{i=j}$) trips are assigned a value of zero unless the analyst is aware of a reason for U-turns to be a significant number. Note that Equation 10-4 does not ensure that the final estimates of total trips exiting each leg of the intersection will match the initial value. An iterative procedure can be used to increase or reduce the T_{ij} as necessary to ensure that the sum of the T_{ij} is close to the initial demand estimates for each entering and departing leg of the intersection. This procedure is known as a matrix balancing process (*7*).

The steps of the iterative procedure use Equations 10-5, 10-6, 10-7, and 10-8.

Step 1. Compute the ratio of desired to actual exiting volume for each departure leg.

$$R_j = \frac{T_j}{\sum_i T_{ij}} \tag{10-5}$$

where

R_j = ratio of desired to actual exiting volume for exit leg j,
T_j = desired exiting volume for exit leg j, and
T_{ij} = current estimate of volume going from origin i to destination j.

Step 2. Multiply all T_{ij} for that exit leg by ratio R_j. Repeat for each exit leg.

Step 3. Compute ratio of desired to actual entering volumes for each entering leg i.

$$R_i = \frac{T_i}{\sum_j T_{ij}} \tag{10-6}$$

where

R_i = ratio of desired to actual entering volume for entry leg i,
T_i = desired entering volume for entry leg i, and
T_{ij} = current estimate of volume going from origin i to destination j.

Step 4. Multiply all T_{ij} for that entry leg by ratio R_i. Repeat for each entry leg.

Step 5. Determine whether the user-specified number of iterations has been exhausted or the user-specified closure criterion is met for all entry and exit legs.

$$\text{diff}_i = T_i - \sum_j T_{ij} \quad \text{for entry legs} \tag{10-7}$$

$$\text{diff}_j = T_j - \sum_i T_{ij} \quad \text{for exit legs} \tag{10-8}$$

Step 6. If any of the computed differences is greater than the closure criterion (a closure criterion of 10 veh/h is suggested) and the iteration limit has not been exceeded, then go back to Step 1.

Peak-Hour Factor

Refer to the peak-hour factor discussion in this chapter under Section II, Urban Streets, Required Input Data and Estimated Values.

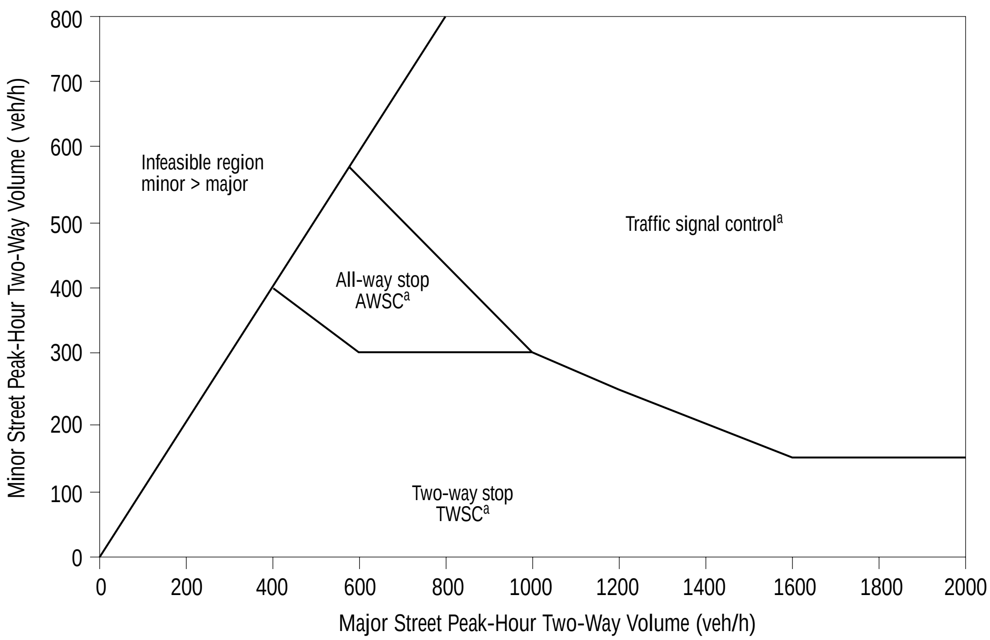
Length of Analysis Period

Refer to the length of analysis period discussion in this chapter under Section II, Urban Streets, Required Input Data and Estimated Values.

Intersection Control Type

The intersection control type for an existing facility is known, by definition. In the case of future facilities, the likely intersection control types can be forecast using Exhibit 10-15 and the forecast two-way peak-hour volumes on the major and minor streets. Note that this exhibit is based on a set of specific assumptions, which are identified in a footnote.

EXHIBIT 10-15. INTERSECTION CONTROL TYPE AND PEAK-HOUR VOLUMES (SEE FOOTNOTE FOR ASSUMED VALUES)



Notes

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

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

Cycle Length

Greater accuracy can be achieved when using the computational methodology if the cycle length for each intersection along the urban street is known or can be calculated on the basis of intersection-specific data. In the absence of a known cycle length or intersection-specific data, the cycle lengths for signalized intersections along an urban street can be estimated using the default values in Exhibit 10-16.

EXHIBIT 10-16. DEFAULT CYCLE LENGTHS BY AREA TYPE

Area Type	Default (s)
CBD	70
Other	100

If the results of the urban street or intersection analysis indicate that the critical volume/capacity ratios for one or more intersections will be greater than 1.00, then the analyst should perform an overall review of geometrics, signal timing, and signal phasing and should consider increasing the cycle length until the v/c ≤ 1.00. Special analysis procedures for actuated signals are given in Appendix B of Chapter 16. A simpler approach is also presented in Appendix A of this chapter.

Lost Time and Estimation of Signal Phasing

The total lost time in the signal cycle can be obtained from Exhibit 10-17 on the basis of whether left turns are protected or permitted for the major street and the minor street.

EXHIBIT 10-17. DEFAULT LOST TIME PER CYCLE BY LEFT PHASE TYPE

Major Street	Minor Street	Number of Phases	L (s)
Protected	Protected	4	16
Protected	Permitted	3	12
Permitted	Protected	3	12
Permitted	Permitted	2	8

Note:
Protected and permitted refer to left turns.

Unopposed left turns (left turns made from a one-way street or from the unopposed leg of a T-intersection) are treated as permitted, while protected plus permitted left turns can be treated as protected when using Exhibit 10-17. Exhibit 10-17 shows that 4 s of lost time occurs between phases of the signal. Note that the term "phase" is used here as it is defined in this manual and should not be confused with the term "NEMA phase," which is used in traffic-actuated control to refer to the green time for a single movement. Thus, an eight-phase NEMA controller (which has protected left turns for all four approaches) has four phases.

The actual left-turn treatment should be used, if known, in determining the lost time. If this is unknown, the choice should be made using local policies or practices. Many agencies use the product of the left-turn and opposing through traffic volumes as an indicator of the need for protected phasing. The following criteria and thresholds may be used to determine whether a left turn is likely to need a protected left-turn phase.

Criteria for consideration of providing a protected left-turn phase

Left turns should be considered for protected phases if any of the following criteria is met:

- More than one turning lane is provided,
- The left turn has a demand in excess of 240 veh/h over 1 h, or
- The cross product of left-turn demand and the opposing through demand for 1 h exceeds 50,000 for one opposing through lane, 90,000 for two opposing through lanes, or 110,000 for three or more opposing through lanes.

Note that these thresholds should only be used for planning applications. For design and operational purposes many other factors should be considered, including accident experience, field observations, and conditions that may exist outside of the analysis period. The *Traffic Control Devices Handbook* (*8*, pp. 4–18) has more information on left-turn phase warrants.

Unprotected left turns from exclusive lanes receive no explicit assignment of green time because they are assumed to be accommodated by green time for the concurrent through movement.

Split phase operation provides complete separation between movements in opposing directions by allowing all movements from only one approach to proceed at the same time. This alternative can be assumed for planning purposes only if

- A pair of opposing approaches is physically offset by more than 20 m,

- A protected left-turn phase must be provided to two opposing single-lane approaches, or
- Both opposing left turns are protected and one of the left turns is accommodated with an exclusive lane plus an optional lane for through and left-turning traffic.

Effective Green Ratio

It is best to use the actual effective green time ratio (g/C) for each movement. In the case of semiactuated and fully actuated signals, the g/C ratios are measured in the field and averaged over several signal cycles during the analysis period. The next best method is to estimate the g/C ratio from intersection turning movements, as described in Appendix A and Appendix B of Chapter 16.

Weighted g/C for urban street

A through g/C for each intersection is desirable; however, for planning purposes, a weighted g/C may be appropriate. The weighted g/C of an urban street is the average of the critical intersection through g/C and the average of all the other g/C's for the urban street. For example, if there are four signals with a through g/C of 0.50 and one signal with a through g/C of 0.40, the weighted average g/C for the urban street is 0.45. The weighted g/C takes into account the adverse effect of the critical intersection and the overall quality of flow for the urban street. An overall default of 0.45 may be used for the major street through movement, but it should be recognized that appropriate ratios can vary depending on characteristics of the urban street.

Arrival Type

The quality of progression is used to determine the arrival type as shown in Exhibit 10-18. The arrival type is used to adjust the computed delay for traffic at each signal. If the arrival type is unknown, a default value of 3 is used for uncoordinated movements and a default value of 4 is used for coordinated movements. Exhibit 15-4 provides a more precise means of determining the arrival type when the proportion of vehicles arriving on green is known.

EXHIBIT 10-18. PROGRESSION QUALITY AND ARRIVAL TYPE

Progression Quality	Arrival Type	Conditions Under Which Arrival Type Is Likely To Occur
Very poor	1	Occurs for coordinated operation on two-way street where one direction of travel does not receive good progression. Signals are spaced less than 500 m apart.
Unfavorable	2	A less extreme version of Arrival Type 1. Signals spaced at or more than 500 m but less than 1000 m apart.
Random arrivals	3	Isolated signals spaced at or more than 1000 m apart (whether or not coordinated).
Favorable	4	Occurs for coordinated operation, often only in one direction on a two-way street. Signals are typically between 500 m and 1000 m apart.
Highly favorable	5	Occurs for coordinated operation. More likely to occur with signals less than 500 m apart.
Exceptional	6	Typical of one-way streets in dense networks and central business districts. Signal spacing is typically under 250 m.

Progression Adjustment Factor

Exhibit 16-12 in Chapter 16 provides guidance on selecting the progression adjustment factor (PF) on the basis of arrival type. The PF = 1.00 for Arrival Type 3. The PF for Arrival Type 4 is 0.84, when g/C is equal to 0.45.

Incremental Delay Adjustment

The incremental delay adjustment (or actuated control adjustment factor), k, is set to 0.50 for pretimed signal control. The same value is used for the nonactuated movements at an intersection with actuated control or semiactuated control.

For the actuated movements, the factor can vary from 0.04 to 0.50, depending on the unit extension for each vehicle actuation and the volume/capacity ratio (X) for the movement. The value of the adjustment factor increases as the volume/capacity ratio increases. Exhibit 15-8 provides guidance on the selection of k given the unit extension and the degree of saturation. In the absence of this information, the analyst may use a value of 0.40 for k, which corresponds to a unit extension of 3 s and a volume/capacity ratio between 0.85 and 0.90. For operational analyses in Chapter 16, the value of k should be based on the computed v/c ratio.

Upstream Filtering/Metering Adjustment Factor

The incremental delay adjustment term (I) accounts for the effects of upstream signals metering arrivals at the study intersection. If the nearest upstream signal is 1.0 km or more away from the subject movement at the study intersection, then I is set to 1.0. Otherwise, I varies from 0.92 to 0.09, decreasing with increasing degree of saturation for the upstream signal. In the absence of this information, the analyst may use a value of 1.0, which applies to isolated intersections, for I.

Base Saturation Flow Rate

The default base saturation flow rate is 1,900 pc/h/ln

This manual uses a default base saturation flow rate of 1,900 pc/h/ln. This value may be increased or decreased on the basis of local field measurements. Approaches with lower approach speeds (less than 50 km/h) often have lower base saturation flow rates, on the order of 1,800 pc/h/ln. Approaches with higher approach speeds (greater than 80 km/h) may have base saturation flow rates higher than 1,900 pc/h/ln.

Adjusted Saturation Flow Rates

The adjusted saturation flow rate per lane is used to compute the delay to through movements at each intersection due to signal control. Initial estimates of the adjusted saturation flow rate for through lanes can be obtained from Exhibit 10-19.

EXHIBIT 10-19. ADJUSTED SATURATION FLOW RATE BY AREA TYPE

Area Type	Default Value (veh/h/ln)	Range (veh/h/ln)
CBD	1700	1600–1800
Other	1800	1700–1950

These adjusted saturation flow rates can also be used for intersection analyses. However, if local data are available on the specific geometry and demand conditions on each approach of an intersection, then the adjusted saturation flow rates can be estimated more accurately. Chapter 15 provides a more accurate method for computing adjusted saturation flow rates when additional data are available.

Lane Widths

The typical lane width is 3.6 m. Urban street lane widths can be as narrow as 3.0 m. The lane closest to a raised median may be extra wide to allow for some shy distance between vehicles and the median. The rightmost lane may be several meters wider than the standard. Lanes wider than 6.0 m should be evaluated to determine whether drivers use the lane as two lanes or as a single wide lane. Drivers making right turns from a wide through lane next to the curb may often form a second queue next to the through vehicles. In this case, the wide through lanes might be analyzed as two lanes: a standard-width through lane plus a narrow exclusive right-turn lane.

The lane width used for the analysis excludes parking lanes (occupied by parked vehicles). Bicycle lanes that are striped to allow right-turning vehicles to enter them for the last several meters before the intersection may be included in the overall lane width of the curb lane.

Heavy Vehicles

The local highway performance management system can be used to obtain local information on the percentage of heavy vehicles by facility and area type. The breakdown between recreational vehicles, trucks, and buses is not used in the computation of adjusted saturation flow rates at signalized intersections. In the absence of local data, a default value of 2 percent heavy vehicles (including all types) may be used for urban streets.

Grades

The approach grade becomes important only when it is significantly steeper than 4 percent. The maximum grades encountered on urban streets typically range from 6 to 11 percent but can reach 31 percent in extreme situations, such as in San Francisco, California. The analyst, in the absence of specific local data, may use 0 percent for essentially flat approaches, 3 percent for moderate grades, and 6 percent for relatively steep grades.

Parking Maneuvers

The number of parking maneuvers per hour is best measured in the field. In the absence of such data, it can be estimated from the number of parking spaces within 80 m of the stop line and the average turnover rate for each space. The number of spaces within 80 m of the stop line is estimated assuming 8 m per space. Each turnover (one car leaving and one car arriving) generates two parking maneuvers. Exhibit 10-20 gives default values for maneuvers per hour, based on 80 percent occupancy of the spaces. These default values may be used in the absence of local data.

EXHIBIT 10-20. PARKING MANEUVER DEFAULTS

Street Type	Number of Spaces in 80 m	Parking Time Limit (h)	Turnover Rate per Hour	Maneuvers per Hour
Two-way	10	1	1	16
		2	0.5	8
One-way	20	1	1	32
		2	0.5	16

Note:
Assumed parking space occupancy of 80 percent.

Local Buses

The number of buses per hour stopping at bus stops within 80 m of the stop line can be estimated from bus schedules for existing conditions. In the absence of such data, the default values in Exhibit 10-21 may be used for intersections where buses are expected to stop.

EXHIBIT 10-21. BUS FREQUENCY DEFAULTS

Area Type	Average Bus Headway (min)	Buses Stopping/h
CBD	5	12
Other	30	2

Pedestrians

Field counts are the best source of information on pedestrian flows. In the absence of counts the defaults listed in Exhibit 10-22 may be used. The analyst should recognize the tendency of field observers to underestimate pedestrian flows when casually observing intersection operations. Relatively infrequent appearances of pedestrians at the intersection, such as one pedestrian per cycle or one pedestrian per minute, can add up to fairly significant hourly pedestrian flows of 60 p/h.

EXHIBIT 10-22. DEFAULTS FOR PEDESTRIAN FLOWS

Area Type	Pedestrian Volume (p/h)
CBD	400
Other	50

Area Type

Only two area types are recognized for signalized intersection analysis: CBD and other. The base saturation flow rate for an intersection is reduced 10 percent for CBD conditions compared with other areas. This adjustment is in addition to the saturation flow reductions for the higher number of parking maneuvers, pedestrians, and bus stops typical of CBDs.

Lane Utilization

The assumed lane utilization can be based on default values unless field data are available or the analyst is aware of special conditions (such as short lane drops or a downstream freeway on-ramp) that might cause drivers to distribute themselves unevenly across the available lanes on the approach. As demand approaches capacity, the analyst may use lane utilization factors closer to 1.0, which would indicate a more uniform use of the available lanes and less opportunity for drivers to freely select their lane. Exhibit 10-23 summarizes lane utilization adjustment factors for different lane group movements and numbers of lanes.

EXHIBIT 10-23. DEFAULT LANE UTILIZATION ADJUSTMENT FACTORS

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

Note:
a. If lane group has more lanes than shown in this exhibit, it is recommended that surveys be made or the smallest f_{LU} shown for that type of lane group be used.

SERVICE VOLUME TABLE

Exhibit 10-24 shows the example service volumes (veh/h) that can be accommodated by a given LOS and number of through lanes to provide the desired LOS on an approach (assuming that all other approaches have the same hourly volume). The values are based on assumptions given in a footnote to the exhibit.

EXHIBIT 10-24. EXAMPLE SERVICE VOLUMES FOR SIGNALIZED INTERSECTION
(SEE FOOTNOTE FOR ASSUMED VALUES)

Through Lanes	LOS				
	A	B	C	D	E
	Service Volumes (veh/h)				
1	N/A	130	350	530	590
2	N/A	200	860	1090	1220
3	N/A	N/A	1230	1510	1680

Notes
N/A - not achievable given assumptions listed below.
This table is derived from the following assumptions.
a. Entries are total hourly volumes for subject approach including turns.
b. All approaches of intersection have the same demand as the subject approach.
c. Left turns equal to 10% of approach demand. Right turns equal to 10% of approach demand.
d. Phasing is protected lefts with exclusive left-turn lane in addition to through lanes.
e. All approaches are two-way streets.
f. Peak-hour factor = 0.92.
g. Saturation flow for each approach is computed assuming the following defaults: lane width = 3.6 m, percent heavy vehicles = 2%, grades = 0%, parking = 8/h, bus = 2/h, pedestrians = 50/h, area type = CBD, lane utilization = 1.05 for 2 lane, 1.10 for 3 lanes, base saturation flow rate = 1900 pc/h/ln. These assumptions result in adjusted saturation flow rates of 1770 for the left-turn lanes and 1560 veh/h/ln for the through lanes.
h. Lost time is 16 s.
i. No upstream signal (I = 1.0).
j. Pretimed signal (k = 0.50).
k. Arrival type = 3.
l. Analysis period (T) = 0.25 h.
m. Initial queue = 0.
n. Signal timing set to maximize service volumes subject to pedestrian clearance time requirements, minimum phase lengths, and maximum signal length of 150 s.
o. Cycle lengths are as follows.

Through Lanes	LOS A	LOS B	LOS C	LOS D	LOS E
1	N/A	60 s	60 s	110 s	150 s
2	N/A	70 s	80 s	110 s	150 s
3	N/A	N/A	80 s	100 s	150 s

Note that minimum cycle lengths to serve pedestrians are 60 s for single-lane approach intersection, 70 s for two-lane approach intersection, and 80 s for a three-lane approach intersection. Minimum pedestrian clearance times are 3 s per through lane plus 3 s for left-turn lane plus 6 s walk display. Minimum left-turn phase was set at 6 s plus phase change and clearance interval of 4 s.

IV. UNSIGNALIZED INTERSECTIONS

Three types of unsignalized intersections are addressed in this manual: two-way stop-controlled (TWSC), all-way stop-controlled (AWSC), and roundabouts.

CHARACTERISTICS OF TWSC INTERSECTIONS

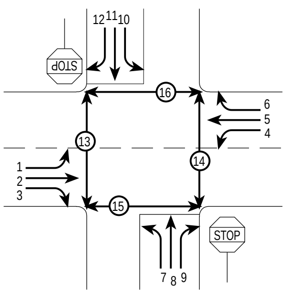
TWSC intersections are common in the United States and abroad. Stop signs are used to control vehicle movements at such intersections. At TWSC intersections, the stop-controlled approaches are referred to as the minor street approaches; they can be either public streets or private driveways. The intersection approaches that are not controlled by stop signs are referred to as the major street approaches.

A three-leg intersection is considered to be a standard type of TWSC intersection if the single minor street approach (i.e., the stem of the T configuration) is controlled by a stop sign. Three-leg intersections where two of the three approaches are controlled by stop signs are a special form of unsignalized intersection control.

FLOW AT TWSC INTERSECTIONS

TWSC intersections assign the right-of-way among conflicting traffic streams according to the following hierarchy:

Four-leg intersection



Rank	Traffic stream
1	2, 3, 5, 6, 15, 16
2	1, 4, 13, 14, 9, 12
3	8, 11
4	7, 10

Rank 1 All conflicting movements yield the right-of-way to any through or right-turning vehicle on the major street approaches. The major street through and right-turning movements are the highest-priority movements at a TWSC intersection.

Rank 2 Vehicles turning left from the major street onto the minor street yield only to conflicting major street through and right-turning vehicles. All other conflicting movements at a TWSC intersection yield to these major street left-turning movements. Vehicles turning right from the minor street onto the major street yield only to conflicting major street through movements.

Rank 3 Minor street through vehicles yield to all conflicting major street through, right-turning, and left-turning movements.

Rank 4 Minor street left-turning vehicles yield to all conflicting major street through, right-turning, and left-turning vehicles and to all conflicting minor street through and right-turning vehicles.

Even though the hierarchy described above suggests that the highest-priority movements experience no delay as they travel through a TWSC intersection, experience shows that their right-of-way is sometimes preempted by other conflicting movements. Such preemptions most often occur during periods of congestion when vehicles in the conflicting movements are experiencing long delays and queues (or when separate left-turn bays are not provided on the major street).

GAP ACCEPTANCE MODELS

Gap acceptance models begin with the recognition that TWSC intersections give no positive indication or control to the driver on the minor street as to when it is safe to leave the stop line and enter the major traffic stream. The driver must determine both when a gap in the major stream is large enough to permit safe entry and when it is the driver's turn to enter on the basis of the relative priority of the competing traffic streams. This decision-making process has been formalized into what is commonly known as gap acceptance theory. Gap acceptance theory includes three basic elements: the size and distribution (availability) of gaps in the major traffic stream, the usefulness of these gaps to the minor stream drivers, and the relative priority of the various traffic streams at the intersection.

Availability of Gaps

The first element to consider is the proportion of gaps of a particular size in the major traffic stream offered to the driver entering from the minor stream, as well as the pattern of arrival times of vehicles. The distribution of gaps between the vehicles in the different streams has a major effect on the performance of the intersection.

Usefulness of Gaps

Critical gap is the minimum time between successive major street vehicles where minor street vehicles make a maneuver

Follow-up time is the time span between the departure of one vehicle from the minor street and the departure of the next vehicle using the same gap

The second element to consider is the extent to which drivers find gaps of a particular size useful when attempting to enter the intersection. It is generally assumed in gap acceptance theory that drivers are both consistent and homogeneous. In reality, this assumption is not entirely correct. Studies have demonstrated that different drivers have different gap acceptance thresholds and even that the gap acceptance threshold of an individual driver often changes over time (*9*). In this manual the critical gap and follow-up times are considered representative of a statistical average of the driver population in the United States.

Relative Priority of Various Streams at the Intersection

Different streams have different ranking in a priority hierarchy. The gap acceptance process evaluates them with impedance terms through the order of departures. Typically, gap acceptance processes assume that drivers on the major road or stream are unaffected by the minor stream drivers. If this is not the case, the gap acceptance process has to be modified.

CAPACITY OF TWSC INTERSECTIONS

At TWSC intersections, drivers on the controlled approaches are required to select gaps in the major street flow through which to execute crossing or turning maneuvers on the basis of judgment. In the presence of a queue, each driver on the controlled approach must also use some time to move into the front-of-queue position and prepare to evaluate gaps in the major street flow. Thus, the capacity of the controlled legs is based on three factors: the distribution of gaps in the major street traffic stream, driver judgment in selecting gaps through which to execute the desired maneuvers, and the follow-up time required by each driver in a queue.

The basic capacity model assumes that gaps in the conflicting stream are randomly distributed. When traffic signals on the major street are within 0.4 km of the subject intersection, flows may not be random but will likely have some platoon structure.

Pedestrians crossing an intersection impede lower-ranked minor street vehicles, but only one lane at a time. This is because vehicles performing a given through or turning movement tend to pass in front of or behind pedestrians once a driver’s target lane is clear. Pedestrian flows are counted somewhat differently than are vehicle flows. If the typical pattern is for pedestrians to walk individually, each pedestrian is counted individually in the pedestrian flow. However, if pedestrians tend to cross in groups, the number of groups is counted. The important factor is to determine the number of blockages. In most cases, this will be a combination of individual pedestrians and groups of pedestrians. Thus, as defined for the purpose of determining the pedestrian impedance, the pedestrian volume is the sum of individual pedestrians crossing individually and groups of pedestrians crossing together during the analysis time period.

The existence of a raised or striped median or a two-way left-turn lane (TWLTL) on the major street often causes some degree of a gap acceptance phenomenon known as “two-stage gap acceptance.” For example, the existence of a raised or striped median causes a significant proportion of the minor street drivers to first cross part of the major street approach and then pause in the middle of the road to wait for another gap in the other approach. If a TWLTL exists on the major street, the minor street left-turn vehicle usually merges into the TWLTL first, then seeks a usable gap on the other approach while slowly moving some distance along the TWLTL. Both of these behaviors can increase capacity.

The geometric elements near the stop line on the stop-controlled approaches of many intersections may result in a higher capacity than the shared-lane capacity equation may predict. This is because, at such approaches, two vehicles may occupy or depart from the stop line simultaneously as a result of a large curb radius, a tapered curb, or a parking prohibition. The magnitude of this effect will depend in part on the turning movement volumes and the resultant probability of two vehicles being simultaneously at the stop line and on the storage length available to feed the second position at the stop line.

Often, two or three movements share a single lane on the minor approach. With this lane sharing, vehicles from different movements do not have simultaneous access to gaps, nor can more than one vehicle from the sharing movements use the same gap, which influences capacity.

The existence of nearby signalized intersections (i.e., traffic signals on the major street within 0.4 km of the subject intersection) typically causes vehicles to arrive at the intersection in platoons. This influences the size and distribution of available gaps and may cause an increase in the minor street capacity. The greater the number of vehicles traveling in platoons, the higher the minor street capacity for a given opposing volume. This is due to the greater proportion of large gaps that more than one minor street vehicle can use. If signalized intersections exist upstream of the subject intersection in both directions, the effect is much more complex.

CHARACTERISTICS OF AWSC INTERSECTIONS

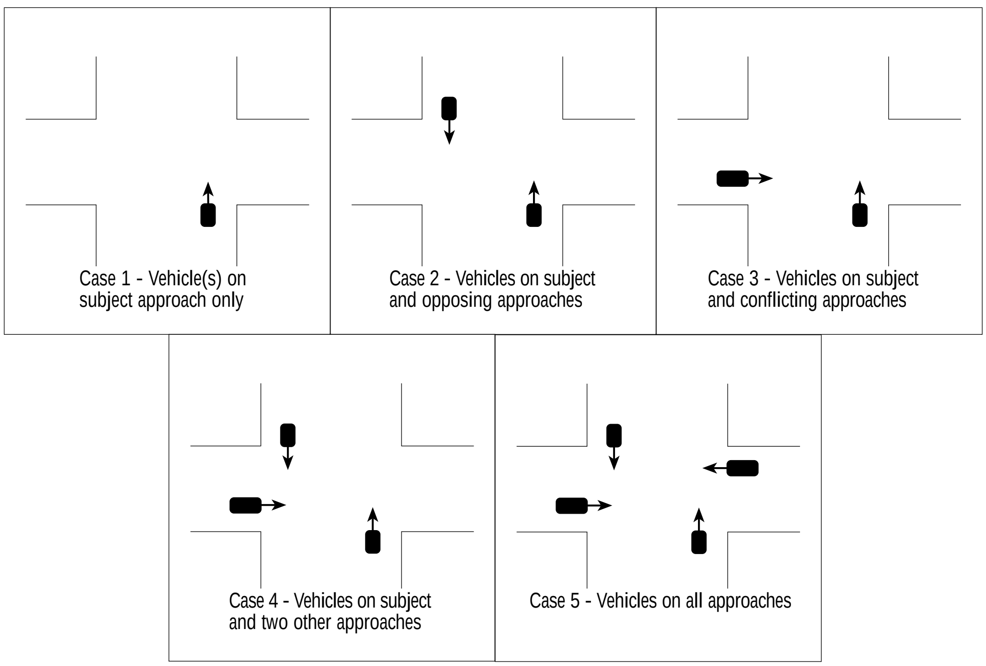
AWSC intersections require every vehicle to stop at the intersection before proceeding. Since each driver must stop, the judgment as to whether to proceed into the intersection is a function of traffic conditions on the other approaches. If no traffic is present on the other approaches, a driver can proceed immediately after the stop is made. If there is traffic on one or more of the other approaches, a driver proceeds only after determining that there are no vehicles currently in the intersection and that it is the driver’s turn to proceed.

FLOW AT AWSC INTERSECTIONS

Field observations indicate that AWSC intersections operate in either a two-phase or a four-phase pattern, based primarily on the complexity of the intersection geometry. Flows are determined by a consensus of right-of-way that alternates between the north-south and east-west streams (for a single-lane approach) or proceeds in turn to each intersection approach (for a multilane approach intersection).

If traffic is present on the subject approach only, vehicles depart as rapidly as individual drivers can safely accelerate into and clear the intersection. This is illustrated as Case 1 in Exhibit 10-25.

EXHIBIT 10-25. ANALYSIS CASES FOR AWSC INTERSECTIONS



If traffic is present on the other approaches, as well as on the subject approach, the saturation headway on the subject approach will increase somewhat, depending on the degree of conflict that results between the subject approach vehicles and the vehicles on the other approaches. In Case 2 some uncertainty is introduced with a vehicle on the opposing approach, and thus the saturation headway will be greater than for Case 1. In Case 3, vehicles on one of the conflicting approaches further restrict the departure rate of vehicles on the subject approach, and the saturation headway will be longer than for Cases 1 or 2. In Case 4, two vehicles are waiting on opposing or conflicting approaches. When all approaches have vehicles as in Case 5, saturation headways are even longer than in the other cases, since the potential for conflict between vehicles is greatest. The increasing degree of potential conflict translates directly into both longer driver decision times and saturation headways. Since no traffic signal controls the stream movement or allocates the right-of-way to each conflicting traffic stream, the rate of departure is controlled by the interactions between the traffic streams themselves.

CHARACTERISTICS OF ROUNDABOUTS

Three main features of roundabouts are illustrated in Exhibit 10-26: the central island, the circulating roadway, and the splitter island (*10*). A roundabout is distinguished from a traffic circle in general by the following characteristics:

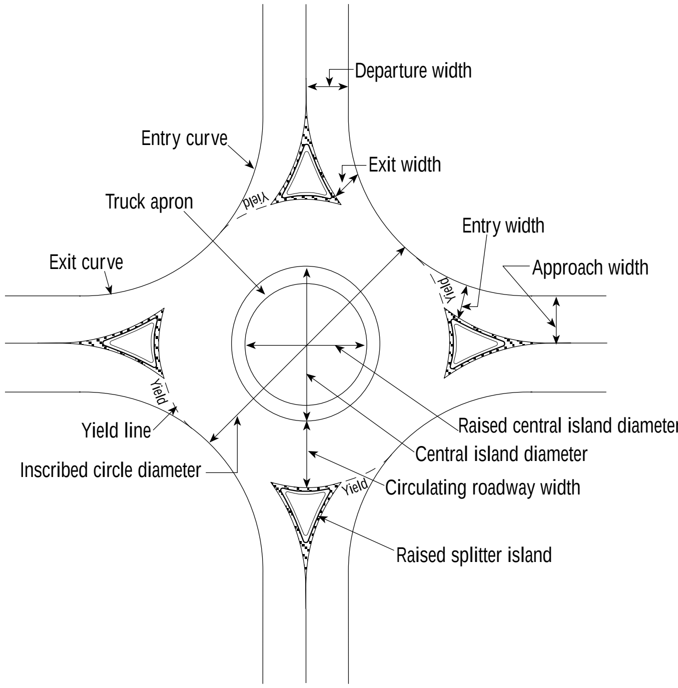
- Vehicles entering a roundabout are required to yield to vehicles within the circulating roadway. Because of right-of-way constraints, some small traffic circles are unable to deflect vehicle paths properly to achieve the desired speed reduction.
- The circulating vehicles are not subjected to any other right-of-way conflicts, and weaving is kept to a minimum. This provides the means by which the priority is distributed and alternated among vehicles. A vehicle entering as a subordinate vehicle immediately becomes a priority vehicle until it exits the roundabout. Some traffic circles impose control measures within the circulating roadway or are designed with weaving areas to resolve conflicts between movements.
- Some small circles do not control speed because of the small central island diameter and because the radius of the vehicle path is large.
- No parking is allowed on the circulating roadway. Parking maneuvers, if allowed, would prevent the roundabout from operating in a manner consistent with its design. Some larger traffic circles permit parking within the circulating roadway.
- No pedestrian activities take place on the central island. Pedestrians are not expected to cross the circulating roadway. Some larger traffic circles provide for pedestrians crossing to, and activities on, the central island.
- All vehicles circulate counterclockwise (in countries with a drive right policy), passing to the right of the central island. In some small traffic circles (sometimes called mini–traffic circles) left-turning vehicles are expected to pass to the left of the central island.
- Roundabouts are designed to properly accommodate specified design vehicles. Some smaller traffic circles are unable to accommodate large vehicles, usually because of right-of-way constraints.
- Roundabouts have raised splitter islands on all approaches. Splitter islands are an essential safety feature, required to separate traffic moving in opposite directions and to provide refuge for pedestrians. Some smaller traffic circles do not provide raised splitter islands.
- When pedestrian crossings are provided for on the approach roads, they are placed approximately one car length back from the entry point. Some traffic circles accommodate pedestrians in other places, such as the yield point.
- Vehicle speed into and through roundabouts is controlled by the physical features of a roundabout and not by signs or pavement markings.

PERFORMANCE MEASURES

Four measures are used to describe the performance of TWSC intersections: control delay, delay to major street through vehicles, queue length, and v/c ratio. The primary measure that is used to provide an estimate of LOS is control delay. This measure can be estimated for any movement on the minor (i.e., the stop-controlled) street. By summing delay estimates for individual movements, a delay estimate for each minor street movement and minor street approach can be achieved.

For AWSC intersections, the average control delay (in seconds per vehicle) is used as the primary measure of performance. Control delay is the increased time of travel for a vehicle approaching and passing through an AWSC intersection, compared with a free-flow vehicle if it were not required to slow or stop at the intersection.

EXHIBIT 10-26. BASIC ROUNDABOUT GEOMETRICS



REQUIRED INPUT DATA AND ESTIMATED VALUES

Unsignalized intersections require basic input data. The data requirements are summarized below for TWSC intersections, AWSC intersections, and roundabouts.

Two-Way Stop-Controlled Intersections

Intersection Geometry

Major street through lanes—Through lanes include those shared by through and turning traffic. Separate left-turn or right-turn lanes are not included in this lane count.

Major street left-turn lanes—The major street left-turn lanes affect the estimated delay for major street through traffic. Exhibit 10-27 can be used to decide whether left-turn lanes are likely to be in place in the future at unsignalized TWSC intersections on two-lane highways.

Minor street lanes—The analyst must provide the number and use of the lanes on the minor street approaches. Any shared lanes should be noted. These data affect the capacity of the minor street movements.

Channelization—Raised or painted islands that separate conflicting flows from each other should be noted. They affect the impedance adjustments and calculation of conflicting traffic flows.

Approach grade—Grades are needed for all approaches and are expressed as a percentage, with positive values for upgrades and negative values for downgrades. The grade affects the calculation of critical gaps.

Control

Movement controls—The analyst should note which movements are stop-controlled and which are yield-controlled (if any). These data affect the calculation of conflicting traffic flows.

EXHIBIT 10-27. MINIMUM APPROACH VOLUMES (veh/h) FOR LEFT-TURN LANES ON TWO-LANE HIGHWAYS AT UNSIGNALIZED INTERSECTIONS

Opposing Volume	5% Left Turns	10% Left Turns	20% Left Turns	30% Left Turns
Free-Flow Speed = 60 km/h				
800	330	240	180	160
600	410	305	225	200
400	510	305	275	245
200	640	470	350	305
100	720	515	390	340
Free-Flow Speed = 80 km/h				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
Free-Flow Speed = 100 km/h				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Source: AASHTO *Policy on Geometric Design of Highways and Streets* (*1*, p. 791).

Volumes

Turning movement volumes—The peak-hour turning volumes are required for all intersection approaches. Vehicle classification is used to calculate the percentage of heavy vehicles.

Peak-hour factor (PHF)—The peak-hour volumes must be divided by the PHF before beginning the computations. If the analyst has peak 15-min flow rates, these flow rates can be entered directly with the PHF set to 1.00.

Length of study period (T)—Refer to urban street description of length of study period in this chapter under required input data and estimated values.

All-Way Stop-Controlled Intersections

Intersection Geometry

The number and configuration of lanes on each approach are used to determine the complexity of the analysis and to select applicable geometry groups.

Volumes

Turning movement volumes—Peak-hour turning volumes by movement for all intersection approaches are required, including vehicle classification (to calculate the percentage of heavy vehicles).

Peak-hour factor (PHF)—The peak-hour volumes must be divided by the PHF before beginning the computations. If the analyst has peak 15-min flow rates, these flow rates can be entered directly with the PHF set to 1.00.

Length of study period (T)—Refer to urban street description of length of study period in this chapter under required data and estimated values.

Roundabouts

Intersection Geometry

The geometric layout of the intersection must be consistent with the characteristics of a single-lane roundabout (see Chapter 17, "Unsignalized Intersections," for details).

Volumes

Turning movement volumes—Peak-hour turning volumes by movement for all intersection approaches are required.

Peak-hour factor (PHF)—For the analysis to reflect conditions during the peak 15 min, the peak-hour volumes must be divided by the PHF before beginning the computations. If the analyst has peak 15-min flow rates, these flow rates can be entered directly with the PHF set to 1.00.

SERVICE VOLUME TABLES

Exhibits 10-28 and 10-29 show the number of lanes required to achieve a desired LOS for a TWSC intersection. The LOS shown in these example tables reflects conditions for the worst movement at the intersections, which is usually the left turn from the stop-controlled minor street. These example service volumes reflect the specific assumptions listed in the exhibit footnotes. Exhibit 10-28 is for a T-intersection and gives example service volumes for a single-lane approach on the minor street. Exhibit 10-29 gives example service volumes for four-leg intersections with varying lane configurations and two-way hourly volumes on the major street approaches ranging between 500 and 1,500 veh/h.

Exhibit 10-30 can be used to estimate the number of through lanes for each approach required to achieve a desired LOS for an AWSC intersection. Since the intersection is all-way stop controlled, each vehicle is required to stop before proceeding. Adding left-turn or right-turn pockets will not significantly reduce delay. The entries in the exhibit are maximum hourly approach volumes for any one of the four approaches to the intersection. These example service volumes reflect the specific assumptions found in the exhibit footnotes.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions used. It should not be used for operational analyses or final design. This table was derived using assumed values listed in the footnote.

EXHIBIT 10-28. EXAMPLE OF MINOR STREET SERVICE VOLUMES FOR T-INTERSECTION TWO-WAY STOP INTERSECTION (SEE FOOTNOTE FOR ASSUMED VALUES)

Major Street 2-Way Volume (veh/h)	LOS				
	A	B	C	D	E
	Minor Street Approach Service Volumes for Single-Lane Approach (veh/h)				
200	110	450	630	700	760
400	N/A	280	460	530	590
600	N/A	150	320	390	440
800	N/A	40	210	270	320
1000	N/A	N/A	120	180	230

Note:
Assumptions: minor street left turns and right turns are equal; major street left turns and right turns are each 10% of the approach volume; PHF = 0.92; heavy vehicles = 2%; grade = 0%; pedestrian flow = 0; no flared minor approach; no channelization; 50/50 split of major street traffic; two-lane major street; major street left-turn pocket.
N/A = not achievable under given conditions.

EXHIBIT 10-29. EXAMPLE OF MINOR STREET SERVICE VOLUMES FOR FOUR-LEG INTERSECTION, TWO-WAY STOP
(SEE FOOTNOTE FOR ASSUMED VALUES)

Major Street 2-Way Volume (veh/h)	LOS A	LOS B	LOS C	LOS D	LOS E
Minor Street Approach Service Volumes (veh/h), Major Street = 1 Lane Plus Turn Pockets, Minor Street = 1 Lane, No Turn Pockets					
500	N/A	90	220	260	300
1000	N/A	N/A	30	70	100
1500	N/A	N/A	N/A	N/A	N/A
Minor Street Approach Service Volumes (veh/h), Major Street = 1 Lane Plus Turn Pockets, Minor Street = 1 Lane, Plus Turn Pockets					
500	N/A	170	370	420	470
1000	N/A	N/A	60	130	180
1500	N/A	N/A	N/A	N/A	10
Minor Street Approach Service Volumes (veh/h), Major Street = 2 Lane Plus Turn Pockets, Minor Street = 1 Lane, No Turn Pockets					
500	N/A	120	240	300	340
1000	N/A	N/A	40	100	130
1500	N/A	N/A	N/A	N/A	20
Minor Street Approach Service Volumes (veh/h), Major Street = 2 Lane Plus Turn Pockets, Minor Street = 1 Lane, Plus Turn Pockets					
500	N/A	240	440	500	550
1000	N/A	N/A	110	180	230
1500	N/A	N/A	N/A	N/A	40

Note:
Assumptions: both approach legs of minor streets have same volume. Minor street left turns and right turns are equal to 33% of total minor street approach volume. Major street left turns and right turns are each 10% of the approach volume; PHF = 0.92; a default PCE of 1.10 was used; no flared minor approach; no channelization; no heavy vehicles.
N/A = not achievable under given conditions.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions used. It should not be used for operational analyses or final design. This table was derived using assumed values listed in the footnote.

EXHIBIT 10-30. EXAMPLE OF APPROACH SERVICE VOLUMES FOR ALL-WAY STOP INTERSECTIONS FOR SINGLE APPROACH
(SEE FOOTNOTE FOR ASSUMED VALUES)

	LOS A	LOS B	LOS C	LOS D	LOS E
Through Lanes	Approach Service Volumes (veh/h)				
1	170	260	310	340	350
2	180	320	430	480	520

Note:
Assumptions: equal demand on all approaches, identical lanes on all four approaches, PHF = 0.92, 10% left turns, 10% right turns, and no heavy vehicles.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions used. It should not be used for operational analyses or final design. This table was derived using assumed values listed in the footnote.

V. REFERENCES

1. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets.* Washington, D.C., 1990.
2. Messer, C. J., and J. A. Bonneson. *Capacity of Interchange Ramp Terminals*. Final Report, NCHRP Project 3-47. Texas A&M Research Foundations, College Station, April 1997.

3. Messer, C. J. *Guidelines for Signalized Left-Turn Treatments, Implementation Package*. FHWA-IP-81-4. Federal Highway Administration, U.S. Department of Transportation, 1981.
4. Koepke, F. J., and H. S. Levinson. *NCHRP Report 348: Access Management Guidelines for Activity Centers*. TRB, National Research Council, Washington, D.C., 1992.
5. Harwood, D. W., M. T. Pietrucha, M. D. Wooldridge, R. E. Brydia, and K. Fitzpatrick. *NCHRP Report 375: Median Intersection Design*. TRB, National Research Council, Washington, D.C., 1995.
6. Pedersen, N. J., and D. R. Samdahl. *NCHRP Report 255: Highway Traffic Data for Urbanized Area Project Planning and Design*. TRB, National Research Council, Washington, D.C., 1982.
7. Ortuzar, J. D., and L. G. Willumsen. *Modeling Transport*, 2nd ed. John Wiley and Sons, New York, 1994.
8. *Traffic Control Devices Handbook*. Federal Highway Administration, U.S. Department of Transportation, 1983.
9. Kittelson, W. K., and M. A. Vandehey. Delay Effects on Driver Gap Acceptance Characteristics at Two-Way Stop-Controlled Intersections. In *Transportation Research Record 1320,* TRB, National Research Council, Washington, D.C., 1991, pp. 154–159.
10. Kyte, M., Z. Tian, Z. Mir, Z. Hameedmansoor, W. Kittelson, M. Vandehey, B. Robinson, W. Brilon, L. Bondzio, N. Wu, and R. Troutbeck. *Capacity and Level of Service at Unsignalized Intersections*. Final Report, NCHRP Project 3-46. April 1996.

APPENDIX A. QUICK ESTIMATION METHOD FOR SIGNALIZED INTERSECTIONS

X_{cm} is the measure of effectiveness for quick estimation methods

A quick estimation method for determining the critical v/c ratio, X_{cm}, signal timing, and delay for a signalized intersection is described in this appendix. This procedure can be used when only minimal data are available for the analysis and only approximate results are desired.

The quick estimation method consists of six steps: assembly of the input data, determination of left-turn treatment, lane volume computations, estimation of signal timing plan, calculation of the critical v/c ratio, and calculation of average vehicle delay.

INPUT REQUIREMENTS

The overall data requirements are summarized in Exhibit A10-1. The input worksheet is shown in Exhibit A10-2. Some of the input requirements may be met by assumed values or default values that represent reasonable values for operating parameters. Other data items are site specific and must be obtained in the field. The objective of using the quick estimation method is to minimize the need for collection of detailed field data.

The calculations integrated into the quick estimation method and specifically indicated in the worksheets include several default values for variables. The values have been selected to be generally representative and to simplify the analysis. The computations for the quick estimation method must be based on the traffic volumes and lane configuration of each approach to the intersection.

The worksheets for computation are shown in Exhibits A10-2, A10-3, A10-4, and A10-5. The first step is to input geometric data and all volume-related parameters on the input worksheet. Critical lane volume (V_{CL}) and left-turn volume (V_{LT}) are obtained according to the method discussed below.

EXHIBIT A10-1. INPUT DATA REQUIREMENTS FOR QUICK ESTIMATION METHOD

Data Item	Comments
Volumes	By movement as projected
Lanes	Left, through, or right; exclusive or shared
Adjusted saturation flow rate	Includes all adjustments for PHF, CBD, grades, etc.
Left-turn treatment (phasing plan)	Use actual treatment, if known. See discussion of phasing plan development.
Cycle length (min and max)	Use actual value, if known. May be estimated using signal operations worksheet.
Lost time	May be estimated using signal operations worksheet
Green times	Use actual values, if known. May be estimated using signal operations worksheet.
Coordination	Isolated intersection versus intersection influenced by upstream signals
Peak-hour factor	Use default value of 0.90 if not known
Parking	On-street parking is or is not present
Area type	Signal is or is not in CBD

DETERMINATION OF LEFT-TURN TREATMENT

Do not use this quick estimation method as the sole basis for determining need for protected left-turn phasing

The signal timing needs of permitted left turns are not considered in the syntheses of the traffic signal timing plan in the quick estimation technique. Therefore, failure to assume protected left-turn phases for heavy left-turn flow rates will generally produce an overly optimistic assessment of the critical v/c ratio and intersection operations.

Exhibit A10-3 provides a procedure for determining whether a permitted left turn should be protected instead in the quick estimation of intersection operations. This left-turn treatment check is not necessary if the left turn is unopposed or if the analyst wishes to analyze only protected left turns. Above all, the left-turn treatment checks should not be used as the sole determinant of the need for protected left-turn phasing.

Even if the analyst already knows that permitted left-turn treatment will be implemented, this left-turn treatment check must still be used to check that the left-turn treatment does not conflict with the assumptions on which this quick estimation method is based. The more robust methodology presented in Chapter 16, “Signalized Intersections,” should be used whenever the analyst wishes to analyze an intersection with permitted left turns that fails the left-turn treatment checks in Exhibit A10-3.

The determination of the recommended left-turn treatment is accomplished in four steps. The first step recommends left-turn protection if there is more than one left-turn lane on the approach. The second step recommends left-turn protection if there are more than 240 veh/h (unadjusted) turning left. The third step recommends left-turn protection if the cross-product of the unadjusted left turn and opposing mainline volumes exceeds the minimum values shown in Exhibit A10-3. The opposing mainline volume is usually the summation of the opposing through and right-turning vehicles. If the analyst recognizes that the opposing approach geometry is such that left-turn vehicles can safely ignore the opposing right-turn vehicles, the opposing right-turn vehicles can be excluded. Such situations occur if there is an exclusive right-turn lane on the opposing approach and the right-turning vehicles have their own lane to turn into on the cross street without interfering with the left-turning vehicles.

The final step compares the left-turn demand with the average number that can sneak through on the yellow phase and recommends left-turn protection if the opposing flows are high enough to result in a left-turn equivalence factor (computed later) exceeding 3.5.

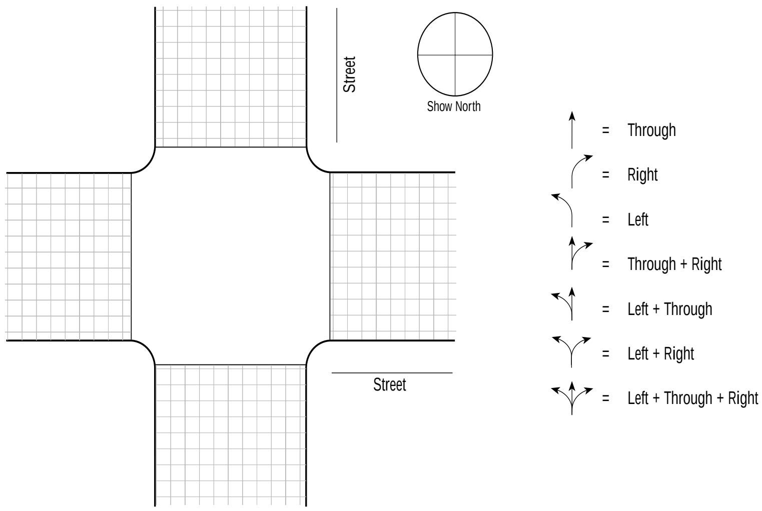
Once it is determined that left-turn protection is recommended for a given approach, the subsequent checks for that approach are unnecessary.

EXHIBIT A10-2. QUICK ESTIMATION INPUT WORKSHEET

QUICK ESTIMATION 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 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)												
Proportion of LT or RT (P_{LT} or P_{RT})[2]		-			-			-			-	
Parking (Yes/No)												
Left-turn treatment (permitted, protected, not opposed) (if known)												

Peak-hour factor, PHF ________

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

Lost time/phase ________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.

EXHIBIT A10-3. LEFT-TURN TREATMENT WORKSHEET

LEFT-TURN TREATMENT WORKSHEET

General Information

Description ____

Check #1. Left-Turn Lane Check

Approach	EB	WB	NB	SB
Number of left-turn lanes				
Protect left turn (Y or 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				
Protect left turn (Y or 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)				
Opposing mainline volume, V_o (veh/h)				
Cross-product (V_L * V_o)				
Opposing through lanes				
Protected left turn (Y or N)?				

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				
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.

EXHIBIT A10-4. QUICK ESTIMATION LANE VOLUME WORKSHEET

QUICK ESTIMATION LANE VOLUME WORKSHEET

General Information

Description/Approach ____

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

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) $V_{TH} = \frac{V_{tot}}{(N_{TH} \times f_{DL})}$			
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.

EXHIBIT A10-5. QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET

General Information

Description____

East-West Phasing Plan

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

North-South Phasing Plan

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

Intersection Status Computation

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

Green Time Calculation

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

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).

LANE VOLUME COMPUTATIONS

The purpose of the lane volume worksheet (Exhibit A10-4) is to establish the individual lane flow rates (in veh/h/ln) on all approaches. This information is used on the control delay and LOS worksheet to synthesize the signal timing plan. The lane volume worksheet contains additional items such as left-turn treatment alternatives, parking adjustments, left-turn equivalence, and adjustment factors for shared lanes with permitted left turns. The directional designations refer to the movements as they approach the intersection.

Each of the three left-turn treatment alternatives identified must be processed differently in computing the lane volumes. Therefore, the lane volume worksheet contains three columns, each representing one of the alternatives. Only one of the three columns should be used for each approach. The following instructions cover the procedure for completing all of the items on the lane volume worksheet.

Right-turn volumes (veh/h) (with the estimated right-turn-on-red volume having been removed from the total) from either a shared through and right-turn lane or from an exclusive turn lane should be entered. The right-turn adjustment factor is 0.85 for a single lane or a shared lane and 0.75 for two lanes. The total right-turn volume should be divided by the product of the number of exclusive right-turn lanes and the right-turn adjustment factor.

Left-turn flow rate is adjusted for left-turn sneakers

The next worksheet computation involves the left-turn volume. In the case of protected-plus-permitted phasing with an exclusive left-turn lane, two vehicles per cycle should be removed from the left-turn volume to account for the effect of sneakers. If the cycle length has not been established, the maximum allowable cycle length should be used. To prevent unreasonably short protected left-turn phases, this volume adjustment step should not reduce the left-turn volume to a value below four vehicles per cycle. The opposing mainline volume is the total approach volume minus the left-turn volume from exclusive lanes or from a single lane (veh/h). The number of exclusive left-turn lanes is the number of lanes exclusively designated to accommodate the left-turn volumes. The left-turn adjustment factor applies only to protected left turns from exclusive left-turn lanes or to left turns that are not opposed. This factor is given as 0.95 for single lanes and 0.92 for double lanes. If the left-turn movement is not opposed because of a one-way street or T-intersection, pedestrian interference must be considered. The corresponding value of 0.85 for one lane and 0.75 for two lanes is used. The total left-turn volume is divided by the product of the number of exclusive left-turn lanes and the left-turn adjustment factor. The left-turn volume is entered directly if there is no exclusive left-turn lane. The result is expressed in units of veh/h/ln. Zero should always be entered if the left turns are permitted.

Total through volume for the approach, excluding left and right turns, is placed in the appropriate row to correspond to the applicable treatment for left turns (permitted, protected, or not opposed). The parking adjustment factor and the number of through lanes are noted. Exclusive turn lanes should be excluded. For an unopposed shared lane, the total approach volume is the sum of the shared-lane right-turn volume, the through volume, and the left-turn volume. Through-lane volume is then computed by dividing total approach volume by the number of through lanes. Critical lane volume is normally the same as through-lane volume, unless the right turn has an exclusive lane or the left turn is not opposed and either of these movements is more critical than the through movement. If both conditions apply, the critical lane volume will be the largest of the left-lane volume, exclusive right-lane volume, and through-lane volume.

The computation of critical lane volume in the case of shared left-turn lanes is more complicated and requires a more detailed computational procedure. The total approach volume is computed in nearly the same manner as for exclusive left-turn lanes. The proportion of left turns in a lane group is self-explanatory. Left-turn equivalence is one of the factors needed to compute the applicable formulas in Appendix C of Chapter 16 for shared-lane permitted left turns. It is not used at all when the left turn is protected. The left-turn adjustment factor for through traffic, f_{DL}, is computed according to Exhibit A10-6. This reduction factor is applied to the through volumes to account for the effect of left-turning vehicles waiting for a gap in the opposing traffic to make the turn. Note that for lanes that are not opposed, the factor is 1.0 because these vehicles will have gaps through which to turn.

EXHIBIT A10-6. SHARED-LANE, LEFT-TURN ADJUSTMENT COMPUTATION FOR QUICK ESTIMATION

Permitted Left Turn	
Lane groups with two or more lanes: $f_{DL} = \frac{(N_{TH} - 1) + e^{\left(\frac{-N_{TH} V_L E_{L1}}{600}\right)}}{N_{TH}}$	Subject to a minimum value that applies at very low left-turning volumes when some cycles will have no left-turn arrivals: $f_{DL(min)} = \frac{(N_{TH} - 1) + e^{\left(\frac{-V_L C_{max}}{3600}\right)}}{N_{TH}}$
Lane groups with only one lane for all movements: $f_{DL} = e^{\left[-0.02(E_{L1} + 10 P_{LT})\frac{V_L C_{max}}{3600}\right]}$	
Protected-Plus-Permitted Left Turn (one direction only)	
If $V_o < 1220$: $f_{DL} = \frac{1}{1 + \left(\frac{P_{LT}(235 + 0.435 V_o)}{1400 - V_o}\right)}$	If $V_o \geq 1220$: $f_{DL} = \frac{1}{1 + 4.525 P_{LT}}$

The through-lane volume is then computed. Note that the number of lanes is reduced by the shared left-turn adjustment factor to account for the effect of the left-turning vehicles.

In the exclusive left-turn lane case, the critical lane volume is the maximum of either the through-lane volume or the right-turn volume from an exclusive right-turn lane. If one or more left turns have been designated as permitted (i.e., no protected phase has been assigned), the need for a protected phase should be reexamined at this point.

As indicated in Chapter 16, Appendix C, values for the left-turn equivalence above 3.5 imply that left-turn capacity is derived substantially from sneakers. Therefore, if the left-turn equivalence is greater than 3.5 and the left-turn volume is greater than two vehicles per cycle, it is likely that the subject left turn will not have adequate capacity without a protected phase.

SIGNAL TIMING ESTIMATION

The purpose of this step is to estimate a feasible signal timing plan for the intersection. The signal timing plan is required to estimate delay. Note that the signal timing plan estimated using the method described below is not necessarily the optimal timing plan. The timing plan is estimated in five substeps: phasing plan development, computation of critical sum, estimation of total lost time, cycle length estimation, and effective green time estimation.

Phasing Plan Development

The phase plan is selected from six alternatives presented in Exhibit A10-7. If the phasing plan is not known, the selection is made on the basis of the user-specified left-turn protection and the dominant left-turn movements identified from the left-turn treatment worksheet (Exhibit A10-3).

EXHIBIT A10-7. PHASE PLANS FOR QUICK ESTIMATION METHOD

Phase Plan	Eastbound	Westbound	Northbound	Southbound
1	Permitted	Permitted	Permitted	Permitted
	Permitted	Not Opposed	Permitted	Not Opposed
	Not Opposed	Permitted	Not Opposed	Permitted
2a	Permitted	Protected	Permitted	Protected
2b	Protected	Permitted	Protected	Permitted
3a	Protected[a]	Protected	Protected[a]	Protected
3b	Protected	Protected[a]	Protected	Protected[a]
4	Not Opposed	Not Opposed	Not Opposed	Not Opposed

Note:
a. Dominant left turn for each opposing movement.

Computation of Critical Sum

When the phase plan has been selected, the movement codes, critical phase volume (CV), and lost time per phase are entered on the quick estimation control delay and LOS worksheet. The critical phase volume is the volume for the movement that requires the most green time during the phase. If two opposing lefts are moving during the same phase, the critical phase volume is the higher-volume left turn. The appropriate choice for critical lane volume is given in the phase plan summary shown in Exhibit A10-8, along with a code that identifies the movements that are allowed to proceed on each phase. For example, NBSBTH indicates that the northbound and southbound through movements have the right-of-way on the specified phase. Exhibit A10-8 also indicates the lost time to be assigned to each phase.

The movement codes and CVs must be determined for each phase from Exhibit A10-8 and entered on the quick estimation control delay and LOS worksheet. When all phases have been completed, the critical sum (CS) of the CVs is entered on the next line.

EXHIBIT A10-8 PHASE PLAN SUMMARY FOR QUICK ESTIMATION METHOD

			East-West		North-South	
Phase Plan	Phase No.	Lost Time (s)	Movement Code	Critical Volume	Movement Code	Critical Volume
1	1	4	EBWBTH	Max(EBTH, EBLT, WBTH, WBLT)	NBSBTH	Max(NBTH, NBLT, SBTH, SBLT)
2a	1	4	WBTHLT	WBLT	SBTHLT	SBLT
	2	4	EBWBTH	Max(WBTH-WBLT,EBTH)	NBSBTH	Max(SBTH-SBLT, NBTH)
2b	1	4	EBTHLT	EBLT	NBTHLT	NBLT
	2	4	EBWBTH	Max(EBTH-EBLT, WBTH)	NBSBTH	Max(NBTH-NBLT, SBTH)
3a	1	4	EBWBLT	WBLT	NBSBLT	SBLT
	2	0	EBTHLT	EBLT-WBLT	NBTHLT	NBLT-SBLT
	3	4	EBWBTH	Max(WBTH, EBTH-(EBLT-WBLT))	NBSBTH	Max(SBTH, NBTH-(NBLT-SBLT))
3b	1	4	EBWBLT	EBLT	NBSBLT	NBLT
	2	0	WBTHLT	WBLT-EBLT	SBTHLT	SBLT-NBLT
	3	4	EBWBTH	Max(EBTH, WBTH-(WBLT-EBLT))	NBSBTH	Max(NBTH, SBTH-(SBLT-NBLT))
4	1	4	EBTHLT	Max(EBTH, EBLT)	NBTHLT	Max(NBTH, NBLT)
	2	4	WBTHLT	Max(WBTH, WBLT)	SBTHLT	Max(SBTH, SBLT)

Estimation of Total Lost Time

The total lost time per cycle is computed on the quick estimation control delay and LOS worksheet. For planning purposes, a lost time value of 4 s per phase is assumed, in which any movement is both started and stopped. For example, if Phase Plan 1 were selected for both streets, then there would be a total of 8 s of lost time per cycle (4 s for each street). When the lost times have been determined for each phase, the total lost time per cycle (L) may be computed and entered on the quick estimation control delay and LOS worksheet.

Cycle Length Estimation

A cycle length that will accommodate the observed flow rates with a specified degree of saturation is computed by Equation A10-1. If the cycle length is known, that value should be used.

$$C = \frac{L}{1 - \left[\frac{\min(CS, RS)}{RS}\right]} \tag{A10-1}$$

where

C = cycle length (s),
L = total lost time (s),
CS = critical sum (veh/h),
RS = reference sum flow rate (1,710 ∗ PHF ∗ f_a) (veh/h),
PHF = peak-hour factor, and
f_a = area type adjustment factor (0.90 if CBD, 1.00 otherwise).

RS is the reference sum of phase flow rates representing the theoretical maximum value that the intersection could accommodate at an infinite cycle length. The recommended value for the reference sum, RS, is computed as an adjusted saturation flow rate. The value of 1,710 is about 90 percent of the base saturation flow rate of 1,900 pc/h/ln. The objective is to produce a 90 percent v/c ratio for all critical movements.

The CS volume is the sum of the critical phase volume for each street. The critical phase volumes are identified in the quick estimation control delay and LOS worksheet on the basis of the phasing plan selected from Exhibit A10-8.

The cycle length determined from this equation should be checked against reasonable minimum and maximum values. The cycle length must not exceed a maximum allowable value set by the local jurisdiction (such as 150 s), and it must be long enough to serve pedestrians (use 60 s if local data are not available).

Green Time Estimation

The total cycle time is divided among the conflicting phases in the phase plan on the basis of the principle of equalizing the degree of saturation for the critical movements. The lost time per cycle must be subtracted from the total cycle time to determine the effective green time per cycle, which must then be apportioned among all phases. This is based on the proportion of the critical phase flow rate sum for each phase determined in a previous step.

The effective green time (including change and clearance time) for each phase can be computed using Equation A10-2.

$$g = \left[(C - L)\left(\frac{CV}{CS}\right)\right] \tag{A10-2}$$

where

g = effective green time (s),
CV = critical phase flow rate (veh/h),
CS = critical sum (veh/h),

C = cycle length (s), and
L = total lost time per cycle (s).

The analyst should note that this method of estimating green time will not necessarily minimize the overall delay at the intersection.

COMPUTATION OF CRITICAL v/c RATIO

The critical v/c ratio, X_{cm}, is an approximate indicator of the overall sufficiency of the intersection geometrics. The computational method involves the summation of conflicting critical lane flow rates for the intersection. The computations depend on traffic signal phasing, which in turn depends on the type of protection assigned to each left-turn movement. The quick estimation critical v/c ratio, X_{cm}, is the ratio of the critical sum, CS, to the sum of the critical lane flow rates that can accommodate demand at the given cycle length and is computed by Equation A10-3.

$$X_{cm} = \frac{CS}{RS\left(1 - \frac{L}{C}\right)} \quad \text{(A10-3)}$$

where

X_{cm} = critical v/c ratio,
CS = critical sum (veh/h),
L = total lost time (s),
C = cycle length (s), and
RS = reference sum (veh/h/ln).

The critical sum is the sum of the critical phase volumes per lane at the intersection. A phasing plan identifying the protected left turns at the intersection must be known or developed for the intersection to compute the critical sum. The computation of the critical phase volumes per lane and the critical sum and the development of an estimated phasing plan are described in the section on the estimation of a signal timing plan.

The reference sum was derived on the basis of minimum acceptable operational conditions and typical traffic flow conditions. More refined saturation flow rates are computed by using the methodology of Chapter 16.

Although it is not appropriate to assign an LOS to the intersection on the basis of X_{cm}, it is appropriate to evaluate the operational status of the intersection for quick estimation purposes. Exhibit A10-9 expresses the intersection status as over, at, near, or under capacity.

EXHIBIT A10-9. INTERSECTION STATUS CRITERIA FOR SIGNALIZED INTERSECTION PLANNING ANALYSIS

Critical v/c Ratio (X_{cm})	Relationship to Capacity
$X_{cm} \le 0.85$	Under capacity
> 0.85 - 0.95	Near capacity
> 0.95 - 1.00	At capacity
$X_{cm} > 1.00$	Over capacity

COMPUTATION OF DELAY

First, the lane groups are established for all approaches. Lane grouping is explained in Chapter 16. Exhibit 16-5 shows different types of lane groups for analysis. Lane group volumes are computed by summing the adjusted volumes obtained from the quick estimation lane volume worksheet (Exhibit A10-4) for each lane group on each approach. The green ratio (g/C) is computed using green time and cycle length values from the top portion of the quick estimation control delay and LOS worksheet. The lane group saturation flow rate is equal to the reference sum times the number of lanes in the lane group. The lane group v/c ratio is calculated using adjusted lane group volumes and lane group saturation flow rates computed previously and g/C ratios. Lane group capacity is calculated using lane group adjusted volumes and lane group v/c ratios. The progression adjustment factor for uniform delay calculation, PF, is selected from Exhibit 16-12. For quick estimation purposes, the analyst may assume Arrival Type 3 for an uncoordinated signal and Arrival Type 4 for coordinated operation (for coordinated lane groups only).

The final step is to compute the various components of control delay using Equations 16-10 through 16-15 in Chapter 16, "Signalized Intersections." This chapter provides guidance on selecting default values for parameters used in the delay equations. Delay by approach, approach volumes, and intersection delay require the delay to be averaged across actual volumes on the quick estimation method input worksheet, not adjusted volumes used to compute capacity. The values V_A and V used in these equations should be computed using unadjusted volumes that are inputs on the quick estimation method input worksheet.

Note that this procedure does not provide sufficient information for the computation of delay for permitted left turns from an exclusive turn lane. The analyst may ignore this delay in the computations or may use the more detailed capacity and LOS procedure provided in Chapter 16, "Signalized Intersections." In addition, the quick estimation method does not include the delay associated with the two sneakers per cycle, which has been subtracted from the left-turn volumes for permitted and protected/permitted left turns.

APPENDIX B. WORKSHEETS

QUICK ESTIMATION INPUT WORKSHEET

LEFT-TURN TREATMENT WORKSHEET

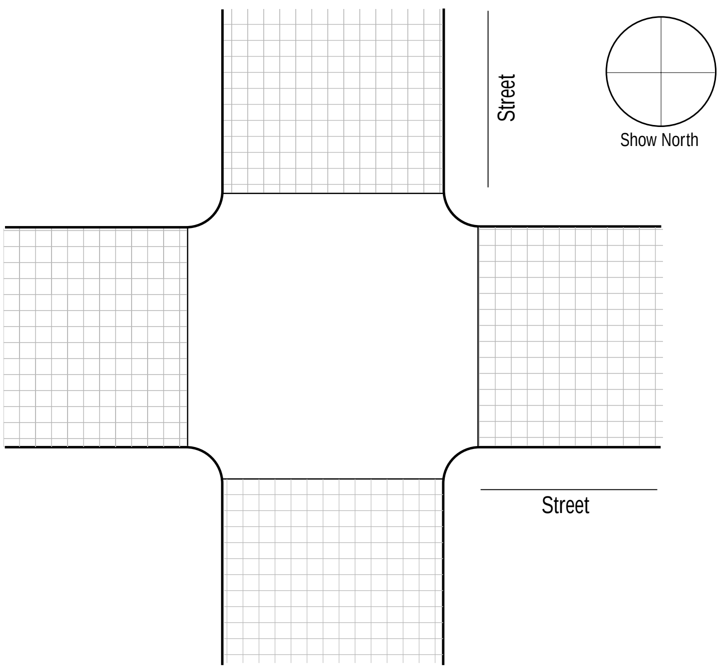
QUICK ESTIMATION LANE VOLUME WORKSHEET

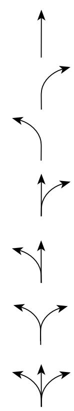
QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET

QUICK ESTIMATION 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





= Through
= Right
= Left
= Through + Right
= Left + Through
= Left + Right
= Left + Through + Right

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)												
Proportion of LT or RT (P_{LT} or P_{RT})[2]		-			-			-			-	
Parking (Yes/No)												
Left-turn treatment (permitted, protected, not opposed) (if known)												

Peak-hour factor, PHF _________

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

Lost time/phase _________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.

LEFT-TURN TREATMENT WORKSHEET

General Information

Description ____________________

Check #1. Left-Turn Lane Check

Approach	EB	WB	NB	SB
Number of left-turn lanes				
Protect left turn (Y or 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				
Protect left turn (Y or 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)				
Opposing mainline volume, V_o (veh/h)				
Cross-product (V_L * V_o)				
Opposing through lanes				
Protected left turn (Y or N)?				

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.

QUICK ESTIMATION LANE VOLUME WORKSHEET

General Information

Description/Approach ____

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

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
Planning 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.

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET

General Information

Description____

East-West Phasing Plan

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

North-South Phasing Plan

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

Intersection Status Computation

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

Green Time Calculation

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

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) $\dfrac{\Sigma(d)(V)}{\Sigma V}$				
Approach flow rate, V_A (veh/h)				
Intersection delay, d_I (s/veh) $d_I = \dfrac{\Sigma(d_A)(V_A)}{\Sigma V_A}$		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).