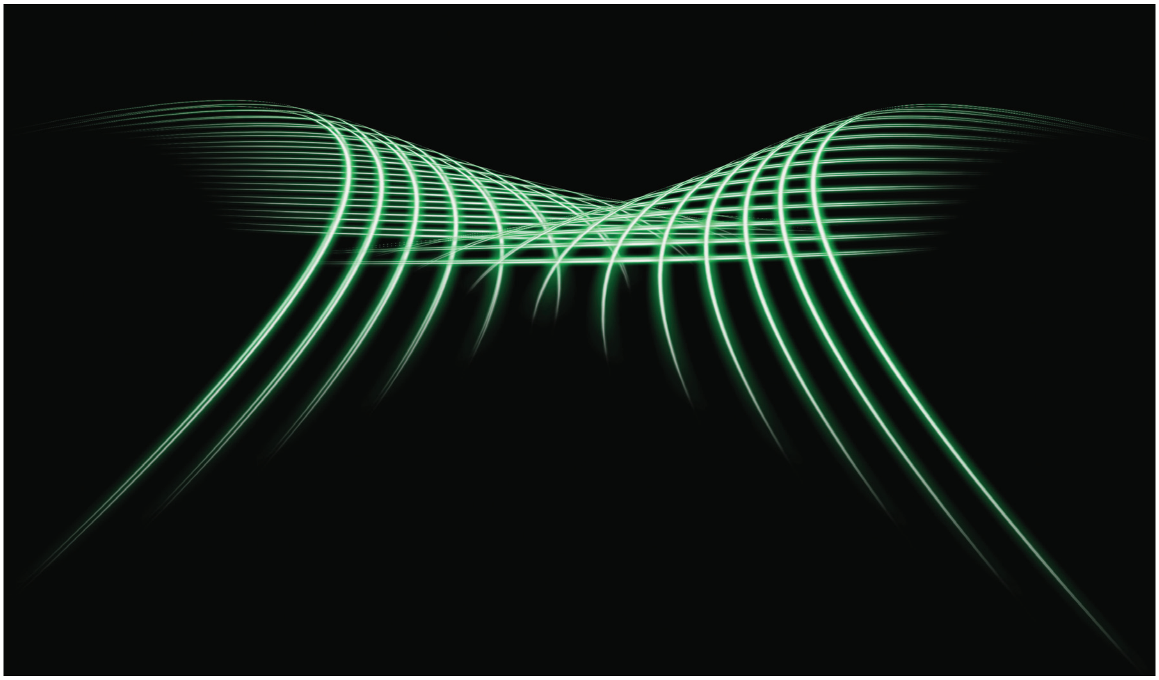
HCM2010

HIGHWAY CAPACITY MANUAL



CHAPTER 22

INTERCHANGE RAMP TERMINALS

TRB TRANSPORTATION RESEARCH BOARD
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HCM2010

HIGHWAY CAPACITY MANUAL

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CHAPTER 22
INTERCHANGE RAMP TERMINALS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Interchange ramp terminals are critical components of the highway network. They provide the connection between various highway facilities (i.e., freeway–arterial, arterial–arterial, etc.), and thus their efficient operation is essential. Interchanges have to be designed to work in harmony with the freeway, the ramps, and the arterials. In addition, they need to provide adequate capacity to avoid affecting the connecting facilities.

SCOPE OF THE CHAPTER

Chapter 22, Interchange Ramp Terminals, presents the methodology for the analysis of interchanges involving freeways and surface streets (i.e., service interchanges), and it was developed primarily on the basis of research conducted through the National Cooperative Highway Research Program (*1*–*3*) and elsewhere (*4*).

The methodology addresses interchanges with signalized intersections, interchanges with roundabouts, and the impact and operations of adjacent closely spaced intersections. Interchanges with two-way STOP-controlled intersections or interchanges consisting of a signalized intersection and a roundabout cannot be evaluated with the procedures of this chapter. Traffic circles (e.g., intersections with a circular island in the middle and signals at the approaches) are not considered in this chapter. The scope of this chapter includes the operational analysis for a full range of service interchange types, including diamond, partial cloverleaf (parclo), and single-point urban interchanges (SPUIs). It also includes a methodology for assessing the operational performance of various types of interchanges for purposes of interchange type selection. The chapter can be used to obtain guidance for assessing various interchange types with respect to their operational performance; it does not provide guidance for selecting an appropriate interchange type with respect to economic, environmental, land use, and other such concerns. The methodology addresses at-grade intersections, not including the freeway proper, and focuses on surface streets; it does not analyze freeway-to-freeway interchanges.

LIMITATIONS OF THE METHODOLOGY

The methodology does not address oversaturated conditions, particularly cases when the downstream queue spills back into the upstream intersection (i.e., when the internal queue exceeds the available storage of the link). It does not address spillback from inadequate turning pocket length. It does not explicitly evaluate the impact of spillback on freeway operations; however, it does estimate the expected queue on the ramps. It does not consider the impacts of ramp metering and spillback from the freeway into the interchange. The method does not estimate lane utilizations for cases when one or both intersections contain an approach that is not part of the prescribed interchange configuration given in the Types of Interchanges section; however, guidance is provided for addressing those cases. The methodology does not specifically address diverging interchanges or continuous flow interchanges (there is limited information with regard to the operation of these emerging interchange types, but it is likely that future research will focus on developing methods for evaluating their operational performance). The methodology provides delay estimates but does not provide link travel times and speeds.

In cases where the user is interested in the analysis of conditions that fall under the above methodological limitations or in the investigation of dynamic traffic operations (i.e., those that evolve in time and space), the use of another analysis tool, such as simulation modeling, is advised. Section 3 includes information on the use of alternative tools for the analysis of interchange ramp terminals.

The operational analysis is only one factor to be considered in the design or redesign of an interchange ramp terminal. Other important factors include right-of-way availability and economic and environmental constraints. The scope of this chapter does not include such considerations; the chapter focuses only on the traffic operational performance of signalized interchanges.

TYPES OF INTERCHANGES

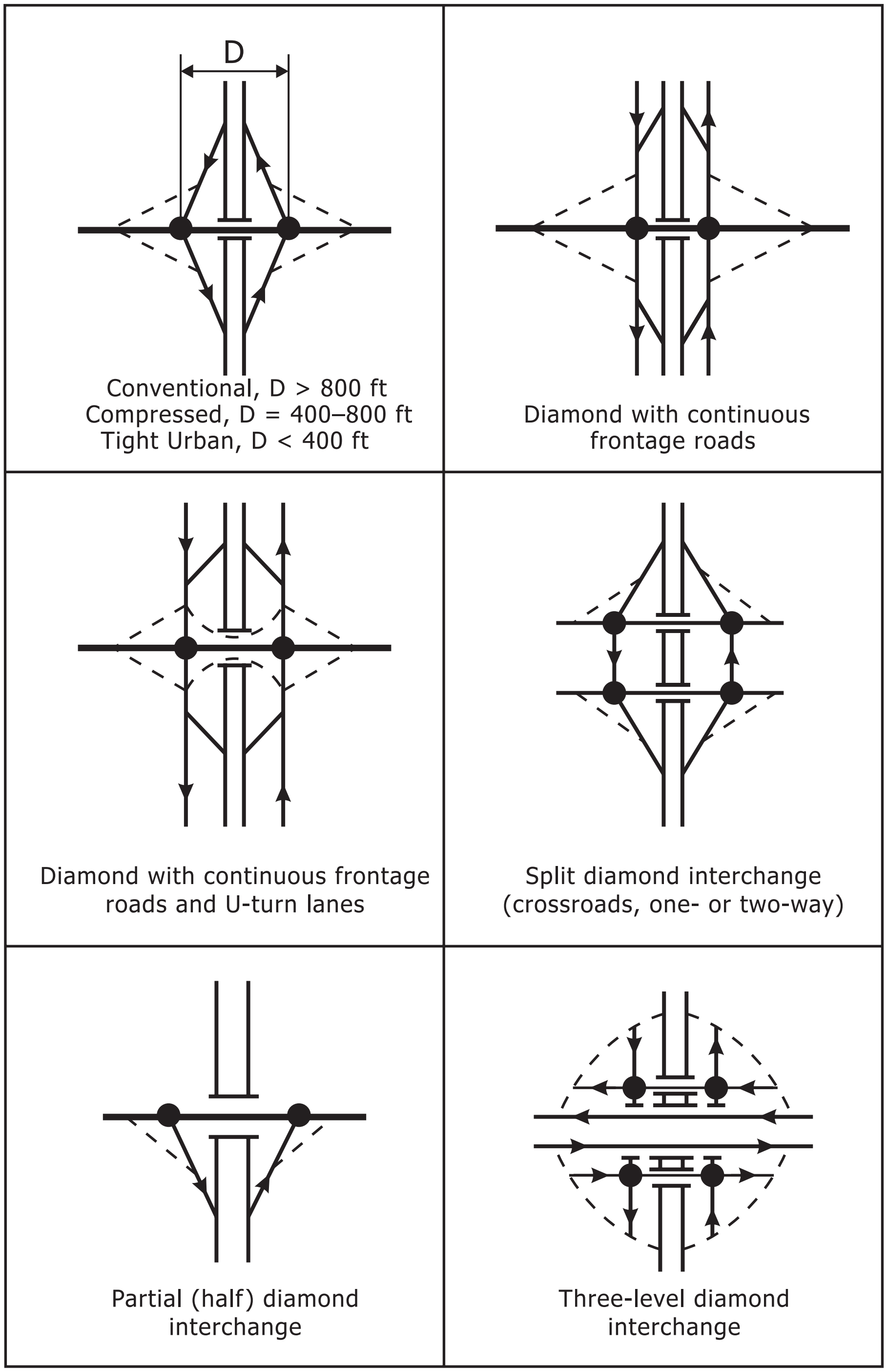
A number of different types of interchanges are recognized in the literature. *A Policy on Geometric Design of Highways and Streets* (*5*) provides extensive information on interchange designs and their characteristics. This section illustrates and discusses the interchange designs considered in this chapter, namely diamond interchanges, parclos, SPUIs, and interchanges with roundabouts.

Diamond Interchanges

Most forms of diamond interchanges result in two or more closely spaced surface intersections, as illustrated in Exhibit 22-1. On a diamond interchange, only one connection is made for each freeway entry and exit, with one connection per quadrant. Left- and right-turning movements are used for entry to or exit from the two directions of the surface facility, which necessitates left-turning movements. When demands are low (generally in rural areas), the junction of diamond interchange ramps with the surface facility is typically controlled by stop or yield signs. If traffic demands are sufficiently high, signalization becomes necessary.

There are many variations of the diamond interchange. The typical diamond configuration has three subcategories defined by the spacing of the intersections formed by the ramp–street connections. Conventional diamond interchanges provide a separation of 800 ft or more between the two intersections. Compressed diamond interchanges have intersections spaced between 400 and 800 ft, and tight urban diamond interchanges feature spacing of less than 400 ft. Because of right-of-way constraints, compressed diamonds are more likely to be used in urban areas, while conventional diamond interchanges are more likely to be used in rural or suburban settings.

Exhibit 22-1
Types of Diamond Interchanges



Note: ———— Possible alternative configuration of signal bypasses operating as unsignalized movements; these are movements that are not using the ramp terminals.

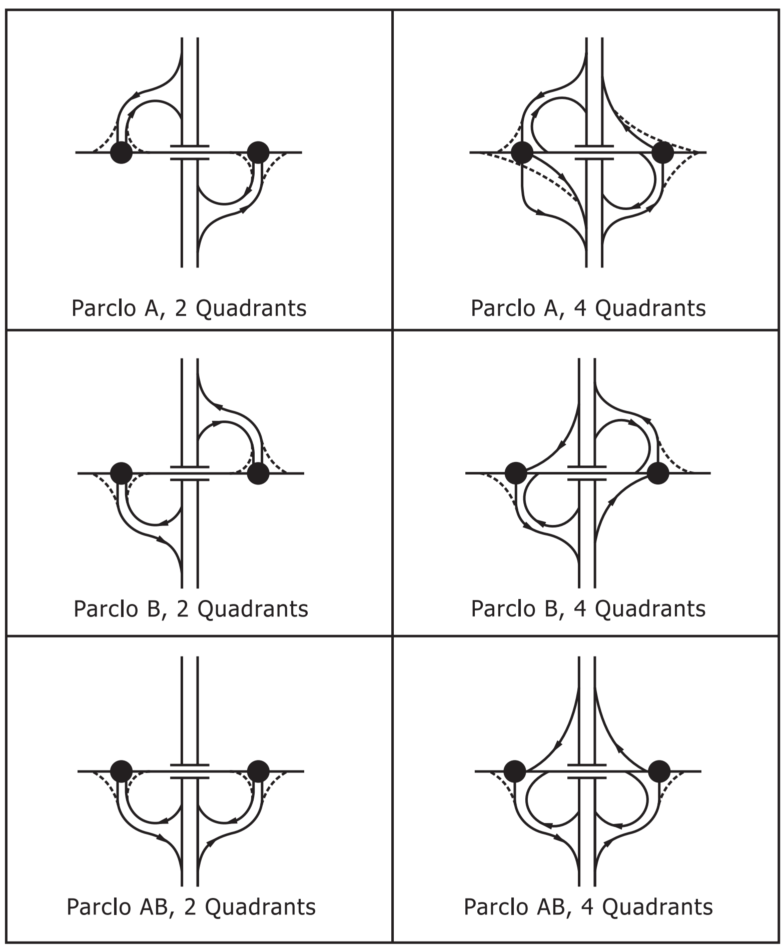
Split diamond interchanges have freeway entry and exit ramps separated at the street level, creating four intersections. Diamond configurations also can be combined with continuous one-way frontage roads. The frontage roads become one-way arterials, and turning movements at the intersections created by the diamond interchange become even more complex. Separated U-turn roadways may be added, removing U-turns from the signal scheme, if there is a signal. A partial diamond interchange has fewer than four ramps, and not all freeway–street or street–freeway movements are served. A three-level diamond interchange features two divided levels, so that ramps are necessary on both facilities to allow continuous through movements.

All these forms of diamond interchanges are depicted in Exhibit 22-1. The methodology in this chapter is applicable to all diamond interchange forms except the split diamond and the three-level diamond. The methodology addresses interchanges where both terminals are signalized or both terminals are roundabouts.

Parclo Interchanges

Parclo interchanges are depicted in Exhibit 22-2. A variety of parclo interchanges can be created with one or two loop ramps. In such cases, one or two of the outer ramps take the form of a diamond ramp, allowing a movement to take place by means of a right turn. In some parclo configurations, left turns also may be made onto or off of a loop ramp. The methodology in this chapter is applicable to parclo interchanges where both terminals are signalized or both terminals are roundabouts.

Exhibit 22-2
Types of Parclo Interchanges

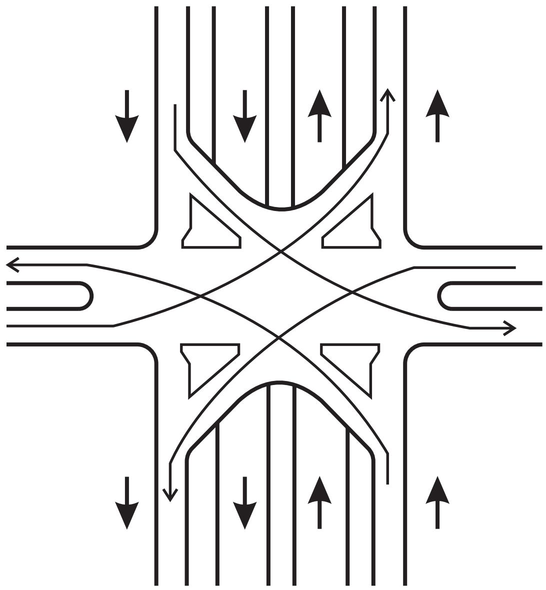


Note: — — — — Possible alternative configuration of signal bypasses operating as unsignalized movements; these are movements that are not using the ramp terminals.

Single-Point Urban Interchanges

A SPUI combines all the ramp movements into a single signalized intersection and has the advantage of operating as such. The design eliminates the critical issue of coordinating the operation of two closely spaced intersections. The SPUI is depicted in Exhibit 22-3.

Exhibit 22-3
Single-Point Urban Interchange

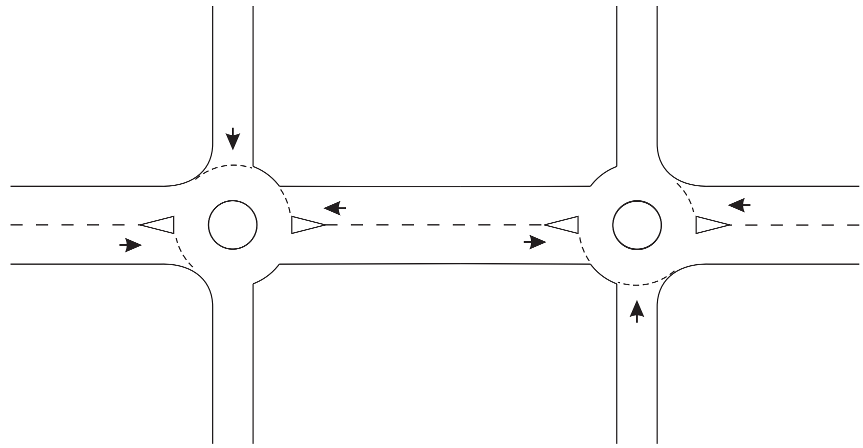


Interchanges with Roundabouts

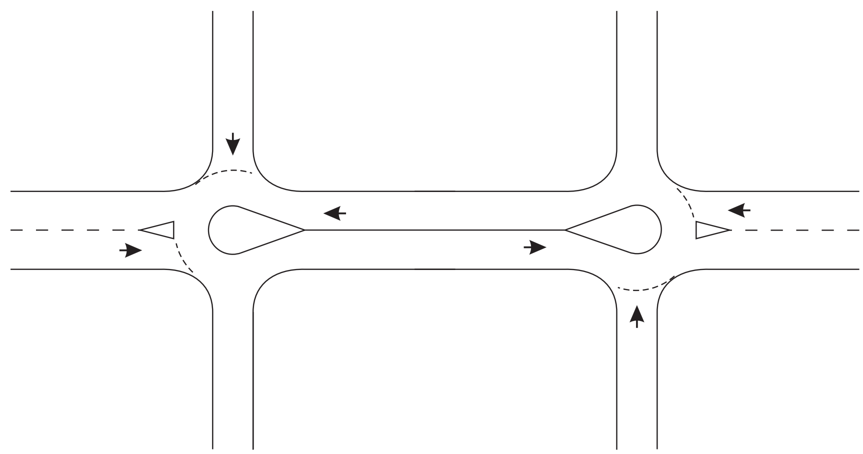
Roundabout intersections can replace signalized or stop-controlled intersections as interchange ramp terminals. Three types of roundabout intersection designs are typically used in the United States and are illustrated in Exhibit 22-4. The first design consists of two traditional roundabouts at the two nodes of the interchange. The second design is called the raindrop roundabout interchange, and it restricts certain movements within each roundabout by creating raindrop-shaped central islands. The two designs are essentially the same, except that the first should be provided when U-turns are allowed or when there is an additional approach to the roundabout. The last design consists of a single roundabout spanning both sides of the freeway via over- or underpasses.

These three designs are applicable to both diamond and parclo interchanges. Their major advantage is that they can reduce the number of lanes needed between terminals, thereby significantly reducing structure-related costs. They also eliminate the need for coordinating signal operations at the two closely spaced intersections. A potential disadvantage of using roundabouts is that spillback from a downstream facility into the roundabout may result in gridlock for all movements at the roundabout, since all movements must use the circulating roadway.

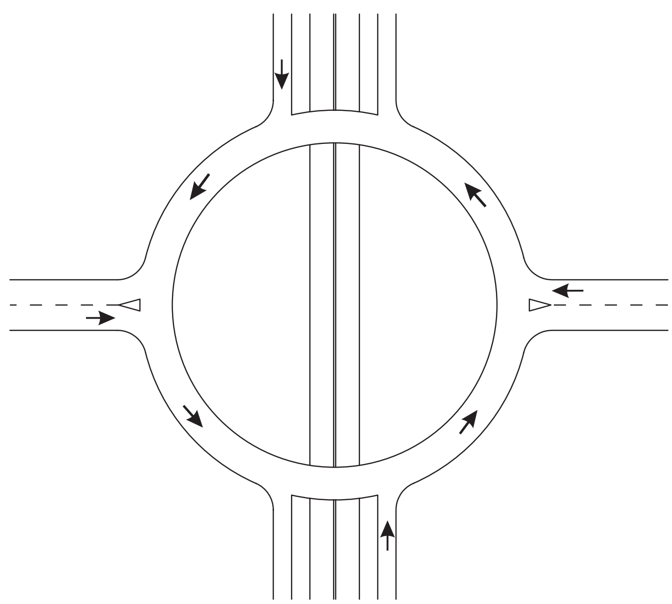
Exhibit 22-4
Interchanges with Roundabouts



Diamond Interchange with Circular Ramp Terminals



Diamond Interchange with Raindrop-Shaped Ramp Terminals



Single-Point Roundabout Interchange

UNIQUE OPERATIONAL CHARACTERISTICS OF INTERCHANGES

Influence of Interchange Type on Turning Movements

The type of interchange has a major influence on turning movements. Movements that involve a right-side merge in one configuration become left turns in another. Movements approaching the interchange on the surface facility are also affected by the interchange type, depending on whether the ramp movements involve left or right turns. Thus, the lane utilization of the external approaches to the interchange varies as a function of the type of interchange and the relative proportion of the turning movements at the downstream intersection.

In selecting an appropriate type of interchange, the impacts on the turning movements should be considered. Left-turning movements are always the most difficult in terms of efficiency of operation, and high-volume left-turning movements should be avoided, if possible. By selecting a type of interchange that requires left turns only for low-demand movements, the overall operation can be enhanced significantly. However, it is not always possible to accomplish this. Right-of-way limitations or agency policies may preclude the use of loop ramps, and economic and environmental constraints may make multilevel structures undesirable.

Because of the influence of interchange type on turning movements, and to be able to compare various interchange types, their level of service (LOS) is based on origin–destination (O-D) demands through the interchange, which are identical regardless of the interchange type. The methodology in this chapter uses both O-D demands and turning movement demands; one set of demands can be derived from the other. Exhibit 22-5 illustrates the O-D demands at an interchange and gives their respective notation. To simplify the mapping process, it is assumed that the freeway is oriented north-south (N-S) and the surface arterial east-west (E-W). Note that Demands K and L indicate the through movement that travels across the surface street, not the freeway through traffic. These demands are expected to be minimal; however, they are included in the methodology because they constitute allowable movements through the interchange. Exhibit 22-6 illustrates the routing of all demands through a diamond interchange, while Exhibit 22-7 illustrates their routing through a Parclo AB-2Q. As shown, the O-D Demand E, eastbound from the arterial and northbound into the freeway, is a left turn in the diamond configuration, while it becomes a right turn in Parclo AB-2Q.

Exhibit 22-5
Illustration and Notation of O-D Demands at an Interchange

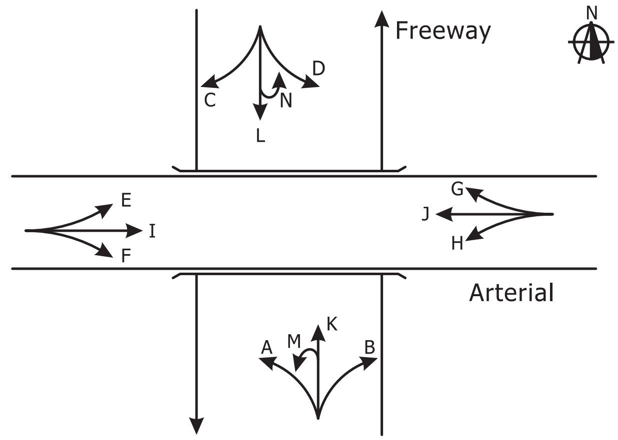


Exhibit 22-6
Illustration and Notation of O-D Demands at a Diamond Interchange

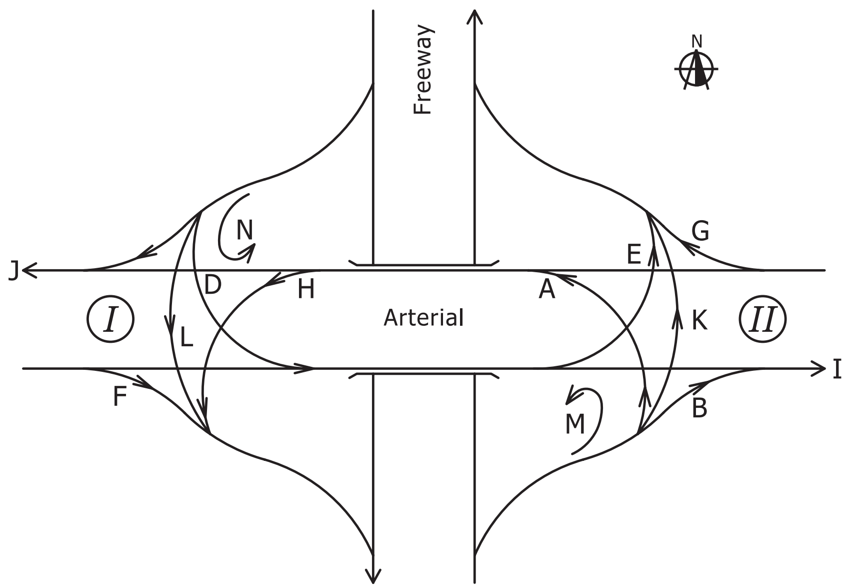
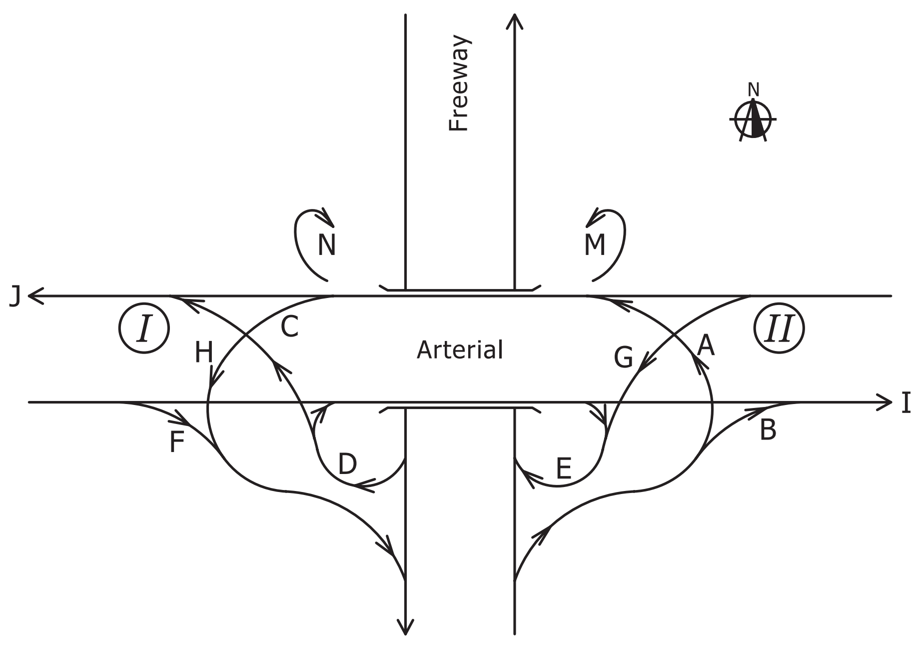


Exhibit 22-7
Illustration of O-D Demands Through a Parclo AB-2Q Interchange



Operational Effects of the Intersection Spacing

Those configurations with two closely spaced signalized intersections present unique challenges because the two intersections do not operate in isolation. Examples of such configurations include the diamond interchanges and two-quadrant parclos. The distance separating the two intersections limits the amount of storage available for queued vehicles. Also, the presence of a downstream queue may reduce or completely block the discharge from the upstream intersection.

Queuing at the downstream intersection can have one of the following two impacts on the discharge from the upstream intersection:

1. Queuing conditions at the downstream intersection are not severe enough to affect the upstream intersection; or
2. Queues at the downstream intersection reduce the rate of discharge upstream because of proximity of the back of the queue. When that occurs, portions of the upstream green cannot be used because of downstream blockage.

Queued vehicles within a short segment (or link) limit the effective length of the link, and vehicles can travel freely only from the upstream stop line to the back of the downstream queue. Because this distance may be small, the impact on the upstream discharge rate is significant. In this methodology, the effects of the presence of a queue at the downstream link are considered by estimating the amount of additional lost time experienced at the upstream intersection. The additional lost time is calculated as a function of the distance to the downstream queue at the beginning of the green for each of the upstream phases.

The extent of queuing at the downstream intersection depends on several factors, including the signal control at the upstream and downstream signals, the number and use of lanes at both intersections, and the upstream flow rates that feed the downstream intersection.

Some of these effects may also exist at locations where signalized intersections are closely spaced, particularly where heavy left-turn movements exist. This chapter addresses the interactions of interchange operations with those of adjacent closely spaced signalized intersections. Furthermore, the principles described in this chapter may be applied to similar situations in which closely spaced signalized intersections (other than those at interchanges) interact.

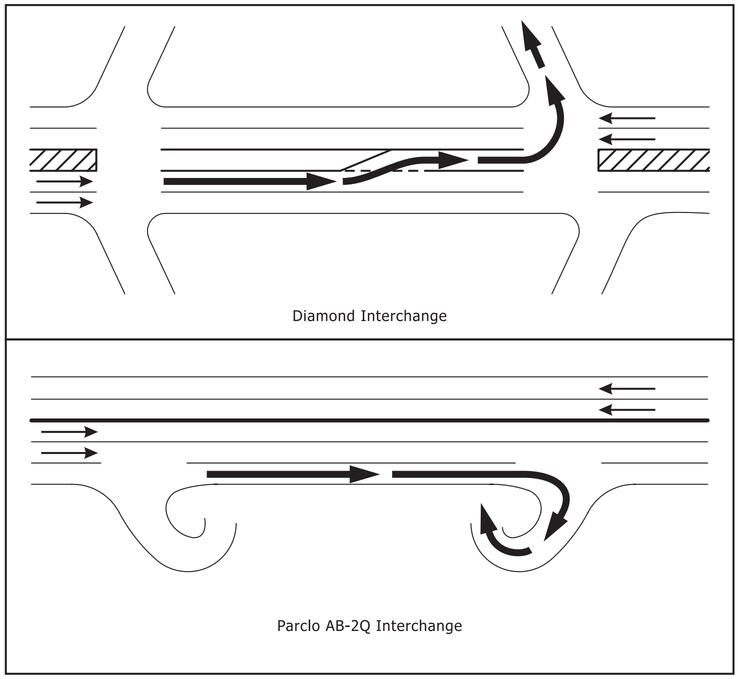
Similar issues may exist at interchanges with roundabouts that are near signalized intersections. When the queue from the signalized intersection reaches the roundabout, it might cause complete gridlock, since all movements through the roundabout must use the circulating roadway.

Lane Utilization for the External Through Movements

For two-intersection signalized interchanges, the lane utilization for the external through movements approaching the interchange on the surface facility is significantly affected by the direction and demand of the turning movements at the downstream intersection. As shown in Exhibit 22-8, high-volume downstream left turns will gravitate toward the left-side lanes at the upstream intersection, while the remaining through and right-turning vehicles will tend toward the right. Conversely, heavy-volume downstream right turns will gravitate toward the right side at the upstream intersection. This can create lane-use imbalances that exceed those at single intersections.

The methodology in this chapter identifies the highest-utilization lane at each of the upstream external through movements as a function of the interchange type, the number of through lanes, the distance between the two intersections, and the O-D demands.

Exhibit 22-8
Impact of Interchange Type on Lane Utilization



This chapter also considers the lane utilization of the arterial approaches at intersections adjacent to the interchange. Lane utilization at those intersections may be affected by turning movement demands at the interchange.

Demand Starvation

Demand starvation occurs when a signalized approach has adequate capacity but a significant portion of the traffic demand is held upstream because of the signalization pattern. For two-intersection signalized interchanges, demand starvation occurs when a portion of the green at the downstream intersection is not used because the upstream intersection signalization prevents vehicles from reaching the stop line. Thus, portions of the downstream green are unused while demand is stuck at the upstream intersection. Exhibit 22-9 illustrates the concept of demand starvation for an interchange. As shown, the internal left turn in the eastbound direction is green, blocking all westbound vehicles from reaching the westbound internal link green. Thus, demand starvation is experienced by the internal westbound through movement, where the signal is green, while the demand for it is blocked upstream.

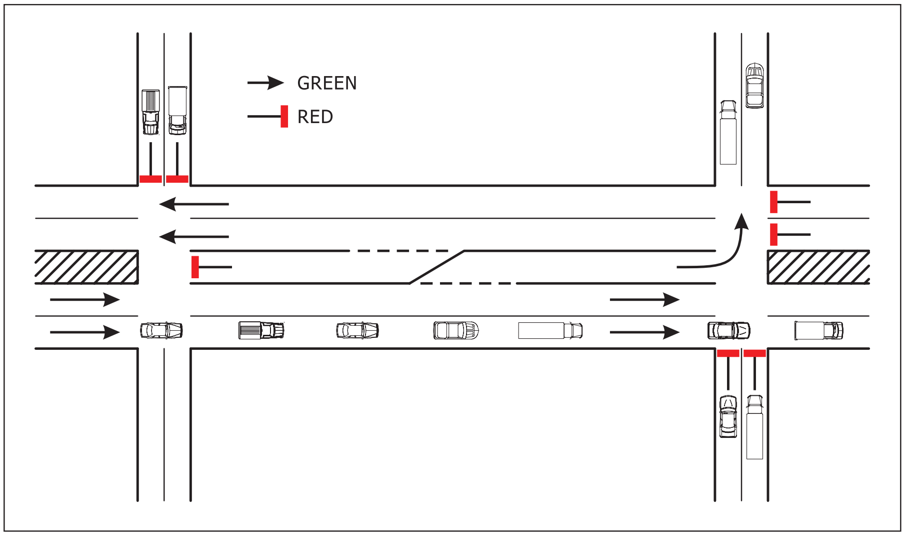


Exhibit 22-9
Demand Starvation at the Internal Link of a Diamond Interchange

LOS FRAMEWORK

Signalized Interchanges

The LOS designation is based on the operational performance of O-D demands (shown in Exhibit 22-5) through the interchange. The LOS for each O-D is based on the total average control delay experienced by that demand as it travels through the interchange. For example, for the diamond interchange shown in Exhibit 22-10, the delay for the O-D Movement H is equal to the sum of the average control delays (d_{WBTH}, d_{WBL}) at each of the lane group flows v_{WBTH}, v_{WBL} along its path. Thus, the delay d_W for O-D H is as follows:

Equation 22-1

$$d_w = d_{WBTH} + d_{WBL}$$

where d_{WBTH} is average control delay of the external westbound through movement (s/veh) and d_{WBL} is average control delay of the internal westbound left movement (s/veh).

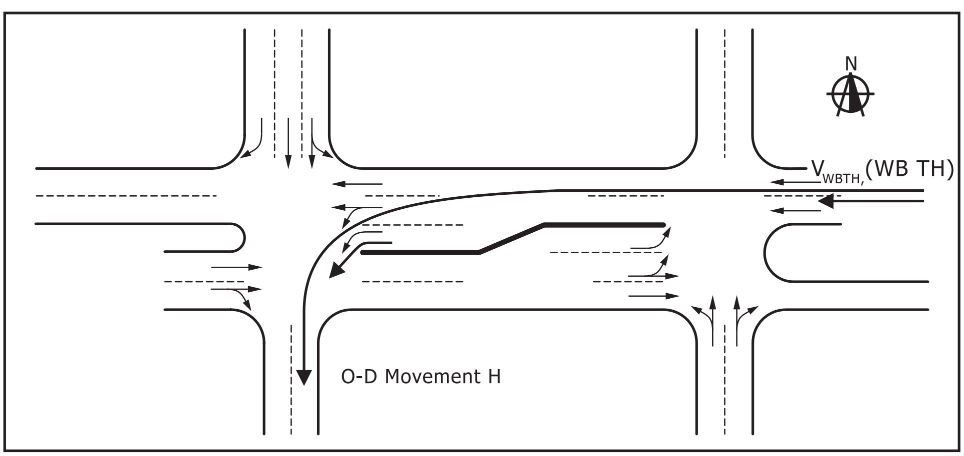


Exhibit 22-10
Illustration of the LOS Concept at a Diamond Interchange

Furthermore, LOS F is defined to occur when either the volume-to-capacity ratio (v/c) or the average queue-to-storage ratio (R_Q) for any of the lane groups that contain this O-D exceed 1, where R_Q refers to the average per lane queue storage ratio within the lane group. Storage is defined as the distance available for queued vehicles on a particular movement, and it is provided on a per lane basis. For example, if the left-turning lane group shown in Exhibit 22-10 has $v/c > 1$, then the LOS for the entire O-D Movement H will be LOS F. If a particular lane group has $v/c > 1$, then all O-Ds that travel through this lane group will operate in LOS F, regardless of their delay. Similarly, if the average per lane queue in a particular lane group exceeds its available storage, then all O-Ds traveling through this lane group will operate in LOS F, regardless of their delay.

Exhibit 22-11 summarizes the LOS criteria for each O-D of an interchange. The values presented in Exhibit 22-11 are greater than those for signalized intersections by a factor of 1.5 to reflect the fact that some of the O-D movements would travel through two intersections, while others would travel through only one.

Exhibit 22-11
LOS Criteria for Each O-D of a Signalized Interchange

Control Delay (s/veh)	O-D LOS $v/c < 1$ and $R_Q < 1$ for Every Lane Group	$v/c > 1$ for Any Lane Group	$R_Q > 1$ for Any Lane Group
≤15	A	F	F
>15–30	B	F	F
>30–55	C	F	F
>55–85	D	F	F
>85–120	E	F	F
>120	F	F	F

Interchanges with Roundabouts

Similar to signalized interchanges, the LOS designation for interchanges with roundabouts is based on the operational performance of O-D demands through the interchange. The LOS for each O-D is based on the total average control delay experienced by that demand as it travels through the interchange. For example, for the interchange shown in Exhibit 22-12, the delay for the O-D Movement H is equal to the sum of the average control delays (d_{15}, d_7) at each of the approach flows v_{15}, v_7 along its path. Furthermore, LOS F is defined to occur when either the v/c ratio or the average R_Q for any of the lane groups that contain this O-D exceeds 1, where R_Q refers to the average per lane queue storage ratio within the lane group.

Exhibit 22-12
Illustration of the LOS Concept at an Interchange with Roundabouts

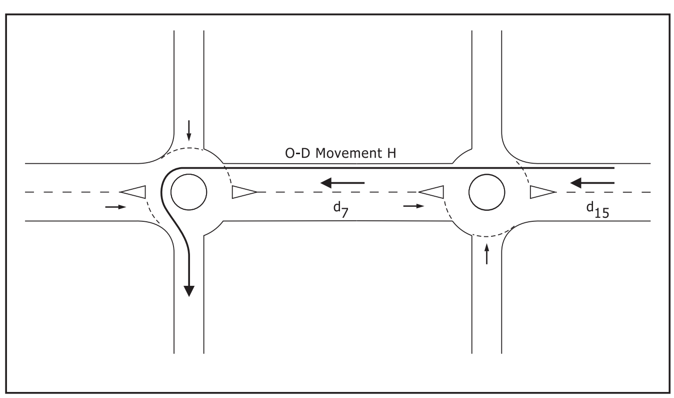


Exhibit 22-13 summarizes the LOS criteria for each O-D of an interchange with one or two roundabouts. The values presented in Exhibit 22-13 are greater than those for noninterchange roundabouts to reflect the fact that some of the O-D movements might travel through two roundabouts while others might travel through only one. The values are also generally lower than the respective ones for signalized interchanges, since drivers would likely expect lower delays at roundabouts.

Exhibit 22-13
LOS Criteria for Interchanges with Roundabouts

Control Delay (s/veh)	O-D LOS: $v/c < 1$ and $R_Q < 1$ for All Approaches	O-D LOS: $v/c > 1$ for Any Approach	O-D LOS: $R_Q > 1$ for Any Approach
≤15	A	F	F
>15–25	B	F	F
>25–35	C	F	F
>35–50	D	F	F
>50–75	E	F	F
>75	F	F	F

Other Interchange Types

Interchange types and control not explicitly included in this chapter (e.g., two-way STOP-controlled diamond interchanges) do not have LOS criteria defined on an O-D basis. In the absence of such LOS criteria, analyses of these interchange types and comparisons with other interchange types can be made by using control delay for each O-D and other applicable performance measures. These performance measures can be determined with procedures in this and other *Highway Capacity Manual* (HCM) chapters, alternative tools, or both, aggregated as appropriate into O-D performance measures by using the techniques in this chapter.

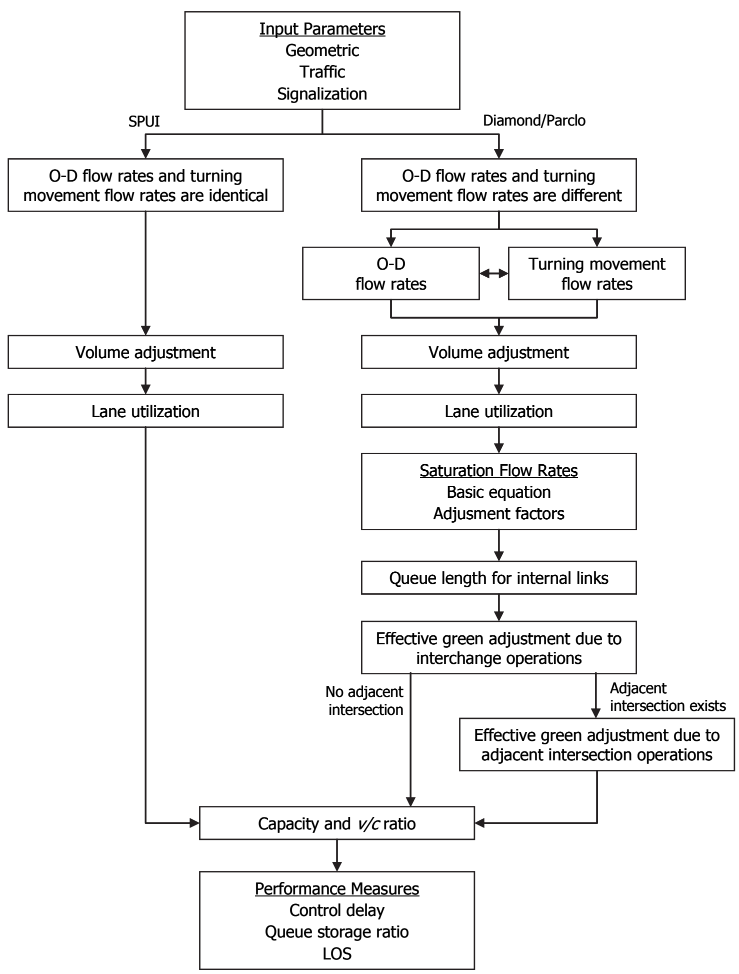
2. METHODOLOGIES

There are two general types of analysis for signalized interchange ramp terminals: (*a*) final design and traffic operational analysis and (*b*) operational analysis for interchange type selection. The methodology for final design and traffic operational analysis for signalized interchanges is presented first. The next section presents the methodology for analyzing interchanges with roundabouts and is followed by a brief discussion of operations at interchanges with unsignalized intersections. The last section presents the methodology used in interchange type selection.

FINAL DESIGN AND OPERATIONAL ANALYSIS FOR SIGNALIZED INTERCHANGES

Exhibit 22-14 summarizes the basic methodology for the final design and operational analysis of signalized interchange ramp terminals. The methodology is similar to that of Chapter 18, Signalized Intersections, with additional consideration for imbalanced lane utilizations, additional lost times due to downstream queues, demand starvation, and additional lost times due to interactions with closely spaced intersections.

Exhibit 22-14
Interchange Ramp Terminals Methodology: Final Design and Operational Analysis for Signalized Interchanges



The analysis of SPUIs is outlined on the left part of the graph. The graph highlights only the components added to the signalized intersection methodology for analyzing SPUIs. The right part of the graph highlights the components added to the signalized intersection methodology for analyzing diamond and parclo interchanges. Each of the steps outlined in Exhibit 22-14 is explained and discussed below.

The analysis begins with the assembly of all pertinent input data, such as geometric characteristics, traffic demands, and signalization information. Exhibit 22-15 provides a summary of all input data required to conduct an operational analysis for interchange ramp terminals.

Exhibit 22-15
Summary of Required Input Data for Final Design and Operational Analysis of Signalized Interchanges

Type of Condition	Parameter
Geometric conditions	Area type Number of lanes (N) Average lane width (W, ft) Grade (G, %) Existence of exclusive left- or right-turn lanes Length of storage for each lane group (L_a, ft) Distance corresponding to the internal storage between the two intersections in the interchange (D, ft) Distances corresponding to the internal storage between interchange intersections and adjacent closely spaced intersections (ft) Turning radii for all turning movements (ft)
Traffic conditions	Demand volume by O-D or turning movement (V, veh/h) Right-turn-on-red flow rates Base saturation flow rate (s_0, pc/hg/ln) Peak hour factor (PHF) Percent heavy vehicles (HV, %) Approach pedestrian flow rates (v_{ped}, ped/h) Approach bicycle flow rates (v_b, bicycles/h) Local bus stopping rate (N_B, buses/h) Parking activity (N_m, maneuvers/h) Arrival type (AT) Upstream filtering adjustment factor Approach speed (S_A, mi/h)
Signalization conditions	Type of signal control Phase sequence Cycle length (if appropriate) (C, s) Green times (if appropriate) (G, s) Yellow-plus-all-red change-and-clearance interval (intergreen) (Y, s) Offset (if appropriate) Maximum, minimum green, passage times, phase recall (for actuated control) Pedestrian push button Minimum pedestrian green (G_p, s) Phase plan

O-D Demands and Movement Demands

The analyst may have either O-D demands or intersection turning movements available for the study interchange. Since both are needed in the analysis, the first step in the methodology consists of calculating either the turning movements by using the O-D demands or the O-D demands by using the turning movements. If the interchange is a SPUI (i.e., only has one intersection), the O-D demands and the turning movement demands are the same, and the analysis proceeds similarly to the methodology of Chapter 18, Signalized Intersections, to estimate capacity, v/c, delay, and queue storage ratios. The

Applications section of this chapter provides guidance on converting O-D movements to turning movements and vice versa for each type of interchange configuration addressed in this methodology.

Lane Group Determination

As in the case of signalized interchanges, the methodology for interchange ramp terminals is disaggregate; that is, it is designed to consider individual intersection approaches and individual lane groups within approaches. The segmentation of the interchange into lane groups generally follows the same guidelines that apply for the analysis of signalized intersections.

Lane Utilization

Vehicles at interchanges do not distribute evenly among lanes in a lane group, and their lane selection is highly affected by their ultimate destination. For example, for two-intersection interchanges, when there is a high-volume left turn at the downstream intersection, traffic at the upstream intersection will gravitate toward the left lanes, while through and right-turning vehicles will tend toward the right. While this may occur at any intersection, because the internal link is generally short and because turning movements are typically high, there is generally greater variation in lane distribution and the adjustment factors that result from it. Segregation at the upstream intersection may occur by driver selection or by designated signing and striping.

To account for these phenomena, lane utilization models have been developed specifically for the external through approaches (surface streets) of two-intersection interchanges. The lane utilization factors for all other interchange approaches (freeway ramps, internal approaches, and SPUI approaches) are estimated by using the procedures of Chapter 18. These lane utilization factors are then used to adjust the saturation flow rates for each lane group.

Adjustment for Lane Utilization

The lane utilization factor accounts for the unequal distribution of traffic among the lanes in a lane group with more than one lane. The factor provides an adjustment to the base saturation flow rate. The adjustment factor is based on the flow in the lane with the highest volume and is calculated by Equation 22-2:

Equation 22-2

$$f_{LU} = \frac{1}{\%V_{Lmax} \times N}$$

where

f_{LU} = adjustment factor for lane utilization;

$\%V_{Lmax}$ = percent of the total approach flow in the lane with the highest volume, expressed as a decimal; and

N = number of lanes in lane group.

A series of models have been developed to predict $\%V_{Lmax}$ for the external arterial approaches of two-intersection interchanges as a function of the downstream turning movements. The remaining approaches should use lane utilization factors based on either field data or on values obtained from Exhibit 18-30.

Exhibit 22-16 through Exhibit 22-20 provide the models developed for each type of interchange configuration and for two-, three-, and four-lane arterials. In these exhibits, L1 represents the leftmost lane, L2 represents the second lane from the left, and so forth. The notation used in these exhibits is provided immediately after Exhibit 22-20.

The models estimate the percent of traffic expected to use each through lane as a function of the O-D demands in the subject approach. These models predict, for the external arterial approaches, the percent of traffic that is expected to use a particular lane as a function of the downstream turning movements. These turning movements are expressed in terms of their respective O-D flows. O-D flows (A through N) are shown in Exhibit 22-21 for each configuration type. When the patterns of the eastbound and westbound approaches are symmetrical, the calibration parameters for the eastbound and the westbound directions are identical, and only the O-D flows differ. Interchange approaches with identical turning movement patterns in the subject direction (eastbound or westbound) are grouped together, and the models developed apply to all configurations in the group. For example, the Parclo B-2Q, B-4Q, and AB-4Q–westbound approach are grouped together, and their lane utilization models are presented in Exhibit 22-16.

In Exhibit 22-16, Exhibit 22-18, and Exhibit 22-19, when an external approach has an exclusive right-turning lane, the O-D for that movement (v_F or v_G) should be assumed to be zero in the respective equation. In Exhibit 22-17, Exhibit 22-18, Exhibit 22-19, and Exhibit 22-20, when there is an additional leg in the upstream intersection, the analyst should use the lane utilization factors of Chapter 18. The models shown in these exhibits are valid for values of D less than 800 ft. The empirical models shown in Exhibit 22-16 through Exhibit 22-20 did not consider configurations with longer distances; for these longer distances between the two intersections vehicles tend not to preposition themselves in anticipation of a downstream turn. In those cases, and in the absence of field data, it is recommended that the default values of Exhibit 18-30 be used. In cases when the internal link contains dual left turns extending to the upstream approach, the volume in the most heavily traveled left-turning lane can be approximated as follows:

1. Use the model with number of lanes $N - 1$, where N is the number of lanes of the subject external approach;
2. Estimate the leftmost lane volume; and
3. Multiply by 0.515.

Research has shown that as operations approach congested conditions, the lane utilization factor tends to approach 1 (i.e., traffic becomes more uniformly distributed).

Actual lane volume distributions observed in the field should be used, if available, because these distributions are highly dependent on the existing land uses and access points in the vicinity of the interchange. A lane utilization factor of 1.0 can be used when uniform traffic distribution can be assumed across all lanes in the lane group or when a lane group comprises a single lane. The lane utilization factors are used in the next step of the methodology to adjust the saturation flow rates for each lane group of the interchange.

Exhibit 22-16
Lane Utilization Models for the External Arterial Approaches of Diamond Interchanges

TWO LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} - 0.154 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) + 0.187 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) - 0.0181 \times \left(\frac{D \times v_E}{10^6}\right)$
Eastbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} - 0.154 \times \left(\frac{v_G}{v_G + v_H + v_J}\right) + 0.187 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.0181 \times \left(\frac{D \times v_H}{10^6}\right)$
Westbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
THREE LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} - 0.245 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) + 0.465 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Eastbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Eastbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} + 0.609 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) - 0.326 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} - 0.245 \times \left(\frac{v_G}{v_G + v_H + v_J}\right) + 0.465 \times \left(\frac{v_H}{v_G + v_H + v_J}\right)$
Westbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Westbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} + 0.609 \times \left(\frac{v_G}{v_G + v_H + v_J}\right) - 0.326 \times \left(\frac{v_H}{v_G + v_H + v_J}\right)$
FOUR LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} - 0.328 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) + 0.684 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Eastbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Eastbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} + 0.64 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) - 0.233 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} - 0.328 \times \left(\frac{v_G}{v_G + v_H + v_J}\right) + 0.684 \times \left(\frac{v_H}{v_G + v_H + v_J}\right)$
Westbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Westbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} + 0.64 \times \left(\frac{v_G}{v_G + v_H + v_J}\right) - 0.233 \times \left(\frac{v_H}{v_G + v_H + v_J}\right)$

Notes: Definitions of variables are provided after Exhibit 22-20.
If there is an exclusive right-turn lane on the external approach, then the respective O-D demand (v_F or v_G) should be zero in the respective equation.

Exhibit 22-17
Lane Utilization Models for the External Arterial Approaches of Parclo A-2Q

TWO LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} - 0.527 \times \left(\frac{v_E}{v_E + v_I}\right)$
Eastbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} - 0.527 \times \left(\frac{v_H}{v_H + v_J}\right)$
Westbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
THREE LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} - 0.363 \times \left(\frac{v_E}{v_E + v_I}\right)$
Eastbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Eastbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} + 0.655 \times \left(\frac{v_E}{v_E + v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} - 0.363 \times \left(\frac{v_H}{v_H + v_J}\right)$
Westbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Westbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} + 0.655 \times \left(\frac{v_H}{v_H + v_J}\right)$
FOUR LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} - 0.257 \times \left(\frac{v_E}{v_E + v_I}\right)$
Eastbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Eastbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} + 0.747 \times \left(\frac{v_E}{v_E + v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} - 0.257 \times \left(\frac{v_H}{v_H + v_J}\right)$
Westbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Westbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} + 0.747 \times \left(\frac{v_H}{v_H + v_J}\right)$

Notes: Definitions of variables are provided after Exhibit 22-20.

If the intersection for which lane utilizations are being estimated has an additional leg, the analyst should not use the equations of this exhibit. The procedures of Chapter 18 should be used instead.

Exhibit 22-18
Lane Utilization Models for the External Arterial Approaches of Parclo B-2Q, B-4Q, and AB-4Q (Westbound Only)

TWO LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} + 0.387 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) - 0.344 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Eastbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} + 0.387 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.344 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
THREE LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} + 0.559 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) - 0.218 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Eastbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Eastbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} - 0.429 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) + 0.695 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} + 0.559 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.218 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Westbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} - 0.429 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) + 0.695 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
FOUR LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} + 0.643 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) - 0.103 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Eastbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Eastbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} - 0.359 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) + 0.794 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} + 0.643 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.103 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Westbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} - 0.359 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) + 0.794 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$

Notes: Definitions of variables are provided after Exhibit 22-20.

If there is an exclusive right-turn lane on the external approach, then the respective O-D demand (v_F or v_G) should be zero in the respective equation.

If the intersection for which lane utilizations are being estimated has an additional leg, the analyst should not use the equations of this exhibit. The procedures of Chapter 18 should be used instead.

Exhibit 22-19
Lane Utilization Models for the External Arterial Approaches of Parclo A-4Q, AB-2Q (Eastbound Only), and AB-4Q (Eastbound Only)

TWO LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{2} - 0.306 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) - 0.484 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Eastbound Right lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1}$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{2} - 0.306 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.484 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Right lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1}$
THREE LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{3} - 0.333 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) - 0.289 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Eastbound Middle lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Eastbound Right lane ($\%V_{L3}$)	$\%V_{L3} = \frac{1}{3} + 0.579 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) + 0.428 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{3} - 0.333 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.289 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Middle lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Westbound Right lane ($\%V_{L3}$)	$\%V_{L3} = \frac{1}{3} + 0.579 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) + 0.428 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
FOUR LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{4} - 0.233 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) - 0.237 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Eastbound Middle lanes ($\%V_{L2}$, $\%V_{L3}$)	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Eastbound Right lane ($\%V_{L4}$)	$\%V_{L4} = \frac{1}{4} + 0.703 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) + 0.641 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{4} - 0.233 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.237 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Middle lanes ($\%V_{L2}$, $\%V_{L3}$)	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Westbound Right lane ($\%V_{L4}$)	$\%V_{L4} = \frac{1}{4} + 0.703 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) + 0.641 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$

Notes: Definitions of variables are provided after Exhibit 22-20.
If there is an exclusive right-turn lane on the external approach, then the respective O-D demand (v_F or v_G) should be zero in the respective equation.
If the intersection for which lane utilizations are being estimated has an additional leg, the analyst should not use the equations of this exhibit. The procedures of Chapter 18 should be used instead.

Exhibit 22-20
Lane Utilization Models for the External Arterial Approaches of Parclo AB-2Q (Westbound Only) Interchanges

TWO LANES IN THE LANE GROUP	
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} + 0.468 \times \left(\frac{v_H}{v_H + v_J}\right)$
Westbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
THREE LANES IN THE LANE GROUP	
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} + 0.735 \times \left(\frac{v_H}{v_H + v_J}\right)$
Westbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Westbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} - 0.308 \times \left(\frac{v_H}{v_H + v_J}\right)$
FOUR LANES IN THE LANE GROUP	
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} + 0.768 \times \left(\frac{v_H}{v_H + v_J}\right)$
Westbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Westbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} - 0.202 \times \left(\frac{v_H}{v_H + v_J}\right)$

Note: Definitions of variables are provided after Exhibit 22-20.
If the intersection where lane utilizations are estimated has an additional leg, the analyst should not use the equations of this exhibit. The procedures of Chapter 18 should be used instead.

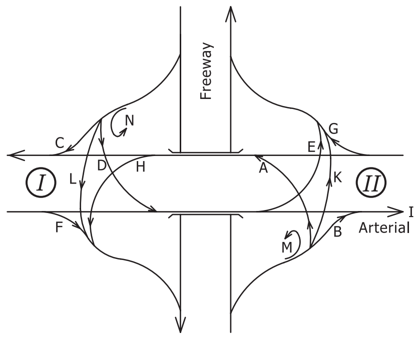
Definitions of variables used in Exhibit 22-16 through Exhibit 22-20 are as follows:

$\%V_{Li}$ = percent of traffic present in lane L*i*, where L1 represents the leftmost lane, L2 represents the second lane from the left, and so forth;

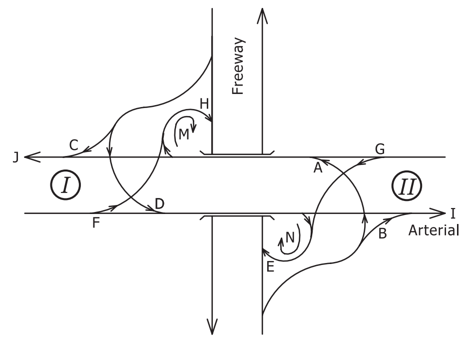
D = distance between the two intersections of the interchange (ft); and

v_i = O-D demand flow rates for movement *i* (veh/h), as illustrated in Exhibit 22-21 for each interchange type.

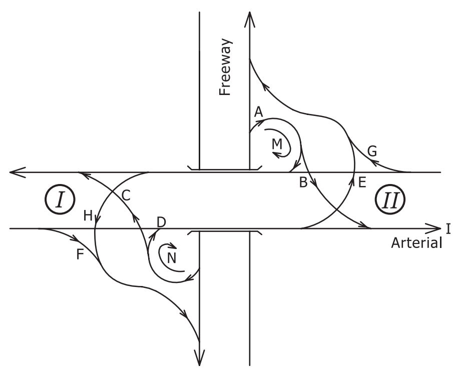
Exhibit 22-21
O-D Flows for Each Interchange Configuration



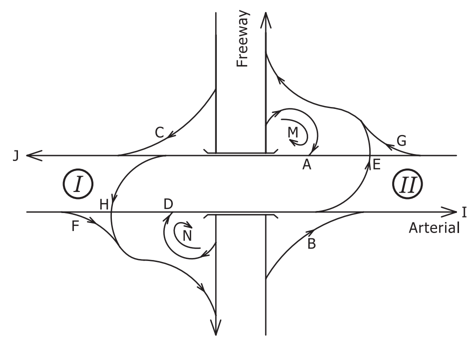
(a) Diamond Interchange



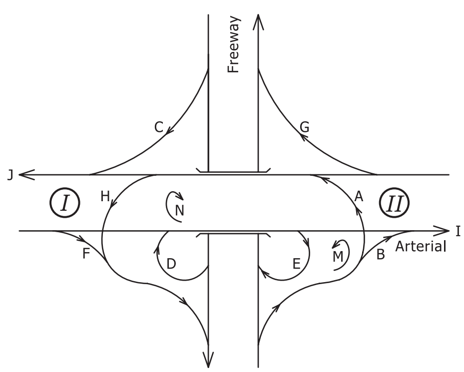
(b) Parclo A-2Q



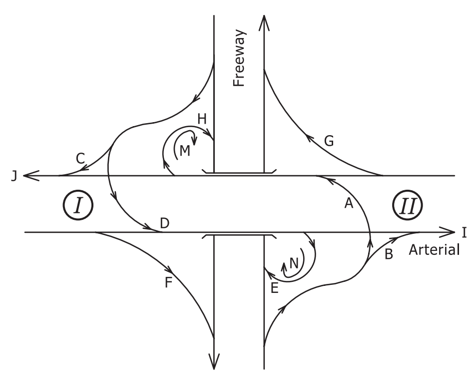
(c) Parclo B-2Q



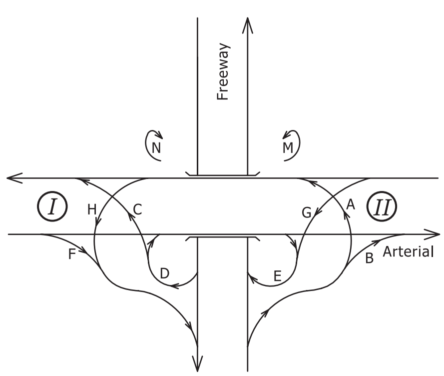
(d) Parclo B-4Q



(e) Parclo AB-4Q



(f) Parclo A-4Q



(g) Parclo AB-2Q

Saturation Flow Rates

The saturation flow rate for each lane group can either be measured in the field or estimated with the following equation:

Equation 22-3

$$s = s_0 N f_w f_{HV} f_g f_p f_{bb} f_a f_{RT} f_{LT} f_{Lpb} f_{Rpb} f_{LU} f_v$$

where

s = adjusted saturation flow rate (veh/h),

s_0 = base saturation flow rate per lane (1,900 pc/hg/ln),

N = number of lanes in the lane group,

f_w = adjustment factor for lane width (from Chapter 18),

f_{HV} = adjustment factor for heavy-vehicle presence (from Chapter 18),

f_g = adjustment factor for approach grade (from Chapter 18),

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to the lane group (from Chapter 18),

f_{bb} = adjustment factor for local bus blockage (from Chapter 18),

f_a = adjustment factor for the area type (from Chapter 18),

f_{RT} = adjustment for right-turning vehicle presence in the lane group (from Chapter 18),

f_{LT} = adjustment for left-turning vehicle presence in the lane group (from Chapter 18),

f_{Lpb} = pedestrian adjustment factor for left turns (from Chapter 18),

f_{Rpb} = pedestrian–bicycle adjustment factor for right turns (from Chapter 18),

f_{LU} = adjustment factor for lane utilization (from Equation 22-2 and Exhibit 22-16 to Exhibit 22-20 as discussed in the previous section), and

f_v = adjustment for traffic pressure (from Exhibit 22-22).

All factors in Equation 22-3 are obtained from Chapter 18, Signalized Intersections, except the last two. A third adjustment factor, f_R, which quantifies the effect of turn radius on the saturation flow rate of a left- or right-turn movement, is used to modify the adjustment factors for protected turn movements provided in Chapter 18. The lane utilization factor was discussed in the previous section, while the other two are discussed below.

Adjustment for Traffic Pressure

Saturation flow rates have generally been found to be higher during peak traffic demand periods than during off-peak periods (*3*). Traffic pressure reflects the display of aggressive driving behavior for a large number of drivers during high-demand traffic conditions. Under such conditions, a large number of drivers accept shorter headways during queue discharge than they would under different circumstances.

The effect of traffic pressure has been found to vary by traffic movement. The left-turn movements tend to be more affected by traffic pressure than through or right movements. To account for this phenomenon, the saturation flow rates at interchange approaches are adjusted by using the traffic pressure factor. This factor is computed with the following equation:

$$f_v = \begin{cases} \dfrac{1}{1.07 - 0.00672 v_i'} & \text{(left turn)} \\ \dfrac{1}{1.07 - 0.00486 v_i'} & \text{(through or right turn)} \end{cases}$$

Equation 22-4

where f_v is the adjustment factor for traffic pressure and v_i' is demand flow rate per cycle per lane (veh/cycle/ln).

For values of v_i' higher than 30 veh/cycle/ln, 30 veh/cycle/ln should be used, since the effects of demands higher than that value are not known.

Exhibit 22-22 tabulates the results of Equation 22-4 for various demands and for each turning movement type.

Exhibit 22-22
Adjustment Factor for Traffic Pressure (f_v)

Demand Flow Rate v_i' (veh/cycle/ln)	Movement Type: Left Turn	Movement Type: Through and Right Turn
3.0	0.953	0.947
6.0	0.971	0.961
9.0	0.991	0.974
12.0	1.011	0.988
15.0	1.032	1.003
18.0	1.054	1.018
21.0	1.077	1.033
24.0	1.100	1.049

When the lane group is shared by several movements, the adjustment factor for traffic pressure is estimated as the weighted (on the basis of the respective flows) average of the respective movements.

Adjustment for Turn Radius Effects on Left- or Right-Turning Movements

Traffic movements that discharge along a curved travel path do so at rates lower than those of through movements (*3*). The turning radius has been found to have a unique effect on saturation flows for turning movements at interchanges (*3*). The adjustment factor to account for the effects of travel path radius, f_R, is calculated with the following equation:

$$f_R = \frac{1}{1 + \dfrac{5.61}{R}}$$

Equation 22-5

where R is the radius of curvature of the left- or right-turning path (at the center of the path), in feet.

The turning radius adjustment factor f_R is used to revise the adjustment factors for protected turn movements provided in Chapter 18. The revised left- and right-turn adjustment factors are as follows:

For protected, exclusive left-turn lanes:

Equation 22-6

$$f_{LT} = f_R$$

For protected, shared left-turn lanes:

Equation 22-7

$$f_{LT} = \frac{1}{1 + P_{LT}\left(\frac{1}{f_R} - 1\right)}$$

For protected, exclusive right-turn lanes:

Equation 22-8

$$f_{RT} = f_R$$

For protected, shared right-turn lanes:

Equation 22-9

$$f_{RT} = \frac{1}{1 + P_{RT}\left(\frac{1}{f_R} - 1\right)}$$

Exhibit 22-23 tabulates the adjustment factor for turn radius for several radii.

Exhibit 22-23
Adjustment Factor for Turn Radius (f_R)

Radius of the Travel Path (ft)	Movement Type: Left and Right Turn	Movement Type: Through
25	0.817	1.00
50	0.899	1.00
100	0.947	1.00
150	0.964	1.00
200	0.973	1.00
250	0.978	1.00
300	0.982	1.00
350	0.984	1.00

When the lane group is shared by several movements, the adjustment factor for turn radii is estimated as the weighted (on the basis of the respective flows) average of the respective movements. The adjustment factors for permissive phasing are estimated by using the procedures of Chapter 18.

Additional Lost Time for the Upstream Approaches due to the Presence of a Downstream (Internal) Link Queue

The presence of a downstream queue may reduce or block the discharge of the upstream movements, increasing the amount of lost time for the upstream phases. In the analysis of interchange ramp terminals, the effects of the presence of a queue at the downstream link (through movement) are considered by estimating the amount of additional lost time experienced at the upstream intersection. The methodology takes into consideration the duration of common green times between various phases at the two intersections. In this chapter common green time between Phase A in Intersection I and Phase B in Intersection II is defined as the amount of time (in seconds) during which both phases have a green indication. Exhibit 22-24 provides an illustrative example of common green times between the upstream and downstream through phases (CG_{UD}) and between the upstream ramp and the downstream through phases (CG_{RD}).

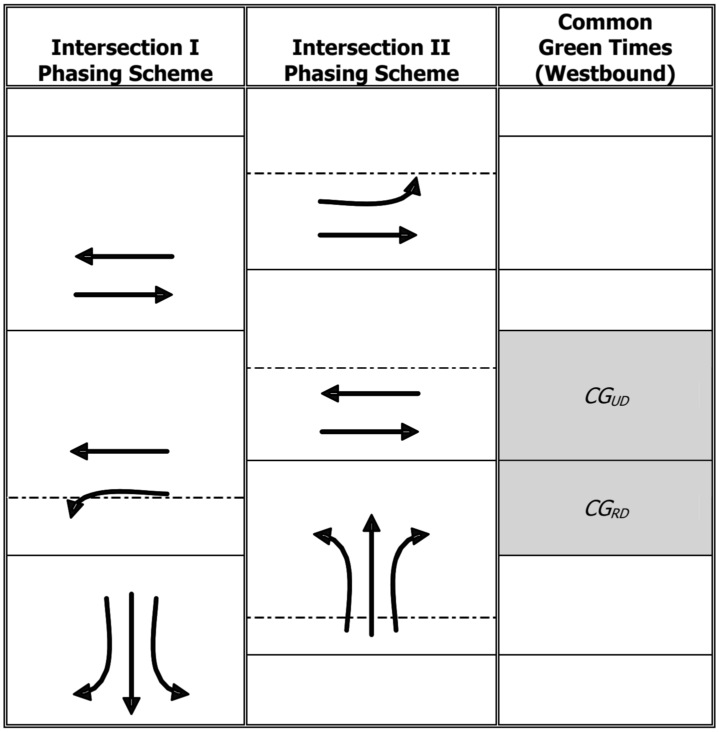


Exhibit 22-24
Illustration of Common Green Times

The additional lost time due to the presence of a downstream queue in the internal through movement is calculated for each of the upstream approaches with the following equations:

Additional lost time on the external arterial approach:

$$L_{D\text{-}A} = G_A - 0.106 \times DQ_A - 5.39 \times \frac{CG_{UD}}{C}$$

Equation 22-10

Additional lost time on the ramp approach:

$$L_{D\text{-}R} = G_R - 0.106 \times DQ_R - 5.39 \times \frac{CG_{RD}}{C}$$

Equation 22-11

where

$L_{D\text{-}A}$ = lost time on external arterial approach due to presence of downstream queue (s),

$L_{D\text{-}R}$ = lost time on external ramp approach due to presence of downstream queue (s),

G_A = green interval for external arterial approach (s),

G_R = green interval for left-turning ramp movement (s),

DQ_A = distance to downstream queue at beginning of upstream arterial green (ft),

DQ_R = distance to downstream queue at beginning of upstream ramp green (ft),

CG_{UD} = common green time between upstream and downstream arterial through green (s),

CG_{RD} = common green time between upstream ramp green and downstream arterial through green (s), and

C = cycle length (s).

If Equation 22-10 or Equation 22-11 results in negative values, then the respective lost times L_{D-A} or L_{D-R} are zero. Furthermore, if DQ_A or DQ_R exceeds 200 ft, then the lost time will be zero.

DQ_A and DQ_R are calculated as follows:

Equation 22-12

$$DQ_A = D - Q_A$$

Equation 22-13

$$DQ_R = D - Q_R$$

where

D = distance corresponding to storage space between the two intersections of the interchange (ft),

Q_A = estimated average per lane queue length for through movement in downstream (internal) link at beginning of upstream arterial Phase A (ft), and

Q_R = estimated average per lane queue length for through movement in downstream (internal) link at beginning of upstream ramp Phase R (ft).

The downstream queue length (averaged across all through lanes) at the beginning of each upstream phase is calculated with the following equations:

Queue at the beginning of the upstream arterial Phase A:

Equation 22-14

$$Q_A = \left[0.0107 \frac{v_R}{N_R} - 7.96 \times \frac{G_D}{C} - 0.082 CG_{UD} + 7.96 \frac{G_R}{C} \right] \times L_h$$

Queue at the beginning of the upstream ramp Phase R:

Equation 22-15

$$Q_R = \left[0.0107 \frac{v_A}{N_A} - 7.96 \times \frac{G_D}{C} - 0.082 CG_{RD} + 7.96 \frac{G_A}{C} \right] \times L_h$$

where

Q_A = queue at the beginning of upstream arterial Phase A (ft);

Q_R = queue at the beginning of upstream ramp Phase R (ft);

v_R = ramp flow feeding subject queue (veh/h);

v_A = arterial flow feeding subject queue (veh/h);

N_R = number of ramp lanes feeding the subject queue;

N_A = number of arterial lanes feeding the subject queue;

G_R = green interval for upstream left ramp movement (s);

G_A = green interval for upstream arterial through movement (s);

G_D = green interval for downstream arterial through movement (s);

CG_{UD} = common green time between upstream arterial green and downstream through green (s);

CG_{RD} = common green time between upstream ramp green and downstream through green (s);

L_h = average queue spacing in a stationary queue, measured from front bumper to front bumper between successive vehicles (ft/veh); and

C = cycle length (s).

The variables v_R, N_R, v_A, and N_A refer to the movement flows that feed the subject queue. For example, for the diamond interchange, v_R is the left-turning flow from the ramp, and the variable becomes $v_{\text{Ramp-}L}$. For actuated signals, the analyst should first determine the equivalent pretimed signal timing plan on the basis of the average duration of each phase during the study hour and estimate the parameters described above on the basis of that plan.

If Q_A or Q_R is calculated to be less than zero, then the expected queue is zero, and no additional lost time due to the presence of a downstream queue will be experienced. Similarly, if the lost times $L_{D\text{-}A}$ or $L_{D\text{-}R}$ are estimated to be negative, then the expected lost time will be zero for the respective approach. Conversely, if Q_A or Q_R exceeds the available storage, its value should be set equal to that storage, and the respective distance to the downstream queue, DQ_A or DQ_R, should be set to zero.

Additional Lost Time for the Downstream (Internal) Approaches due to Demand Starvation

This methodology accounts for the effects of demand starvation in interchange operations by computing the lost time experienced at the downstream intersection that results from demand starvation. Lost time due to demand starvation (L_{DS}) is defined as the amount of green time during which there is no queue present to be discharged from the internal link *and* there are no arrivals from either of the upstream approaches due to signalization. The common green time between two phases that may lead to demand starvation is called *common green time with demand starvation potential* (CG_{DS}). Exhibit 22-25 provides an illustrative example of an interval with demand starvation potential. In that example, there is potential for demand starvation for the westbound internal through movement of the interchange.

The following equation is used to estimate lost time due to demand starvation:

$$L_{DS} = CG_{DS} - Q_{\text{INITIAL}} \times h_I$$

Equation 22-16

where

L_{DS} = additional lost time due to demand starvation (s);

CG_{DS} = common green time with demand starvation potential (s);

h_I = saturation headway for internal through approach (= 3,600/saturation flow per lane) (s); and

Q_{INITIAL} = length of queue stored at internal approach at beginning of interval during which this approach has demand starvation potential, calculated from Equation 22-17.

Equation 22-17

$$Q_{\text{INITIAL}} = \left(\frac{v_{\text{Ramp}-L} \times C}{N_{\text{Ramp}-L} \times 3{,}600} - \frac{(CG_{RD} - t_L)}{h_I} \right) + \left(\frac{v_{\text{Arterial}} \times C}{N_{\text{Arterial}} \times 3{,}600} - \frac{(CG_{UD} - t_L)}{h_I} \right)$$

where

$v_{\text{Ramp-}L}$ = upstream ramp left-turning flow (v/h),

v_{Arterial} = upstream arterial through flow (v/h),

C = cycle length (s),

$N_{\text{Ramp-}L}$ = number of lanes for upstream ramp left-turning movement,

N_{Arterial} = number of lanes for upstream arterial through movement,

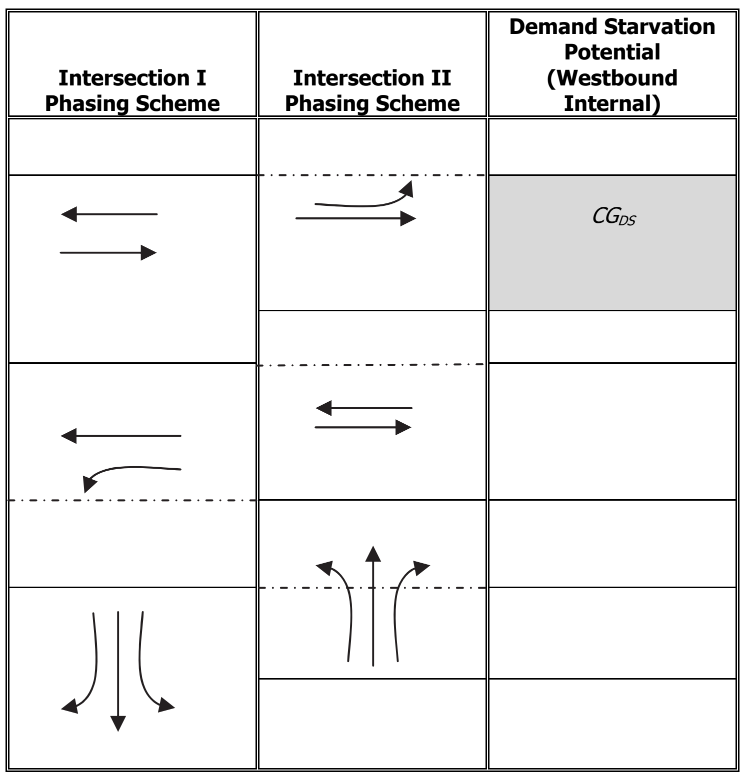
CG_{RD} = common green time between upstream ramp and downstream through green phase (s),

CG_{UD} = common green time between upstream through and downstream through green phase (s),

h_I = saturation headway for internal through approach (= 3,600/saturation flow per lane) (s), and

t_L = lost time per phase (s).

Exhibit 22-25
Illustration of Interval with Demand Starvation Potential



Equation 22-16 calculates the amount of time that would not be used because the internal link queue has completely discharged and the upstream demand is blocked and cannot arrive to the internal link stop line. The initial queue at the beginning of the demand starvation interval is estimated as a function of the demands of the upstream approaches and of the respective common intervals between the upstream and downstream green.

Equation 22-17 is valid for values of CG_{RD} and $CG_{UD} \geq t_L$. If CG_{RD} or $CG_{UD} < t_L$, the analyst should assume that CG_{RD} or $CG_{UD} = t_L$. Also, in applying Equation 22-17 it is assumed that no vehicles will have to wait for more than one cycle (i.e., none of the approaches is oversaturated). If the time required to discharge the queue is equal to or larger than the CG_{DS}, the lost time due to demand starvation will be zero. The model for estimating lost time due to demand starvation assumes uniform arrivals and departures and assumes that operations at the interchange are not oversaturated.

Effective Green Adjustments

The effective green adjustment involves two components: (*a*) adjustment in the effective green of the upstream approaches due to the presence of a downstream queue and (*b*) adjustment in the effective green of the downstream (internal) approaches due to demand starvation.

The adjusted lost time t_L' for the arterial approaches and for the ramp approaches is estimated as follows:

Arterial approaches:

$$t_L' = l_1 + L_{D\text{-}A} + Y - e$$

Equation 22-18

Ramp approaches:

$$t_L' = l_1 + L_{D\text{-}R} + Y - e$$

Equation 22-19

where

- t_L' = adjusted lost time (i.e., time when the signalized intersection is not used effectively by any movement) (s),
- l_1 = start-up lost time (s),
- $L_{D\text{-}A}$ = lost time on external arterial approach due to presence of a downstream queue (s),
- $L_{D\text{-}R}$ = lost time on external ramp approach due to presence of a downstream queue (s),
- Y = yellow-plus-all-red change-and-clearance interval (s), and
- e = extension of effective green time into the clearance interval (s).

The adjusted lost time t_L'' for the internal approaches is estimated as follows:

$$t_L'' = l_1 + L_{DS} + Y - e$$

Equation 22-20

where L_{DS} is the additional lost time due to demand starvation (s).

The effective green time adjusted due to the presence of a downstream queue is then calculated for the external approaches by using the following equation:

$$g' = G + Y - t_L'$$

Equation 22-21

where g' is the effective green time adjusted due to presence of a downstream queue (s), G is the green time (s), and t_L' is the adjusted lost time for external approaches (s).

Similarly, the effective green time adjusted due to demand starvation is calculated for the internal approaches as follows:

Equation 22-22

$$g' = G + Y - t''_L$$

where g' is the effective green time adjusted due to demand starvation (s), G is the green time (s), and t''_L is the adjusted lost time for the internal approaches (s).

Closely Spaced Adjacent Intersections

The presence of closely spaced signalized intersections in the vicinity of an interchange may affect the operations of the entire interchange system and can present unique operational challenges. First, the lane utilizations of the arterial approaches would be affected by the presence of the interchange as vehicles position themselves to make a turn downstream or as they enter the arterial from the interchange. Queuing from the adjacent intersections could affect the discharge rate of the upstream (internal) link of the interchange. Furthermore, demand starvation in the internal link can coexist with queues upstream, in the external approaches of the interchange. If this external link is short, queue spillback may affect the adjacent intersections and have a significant and long-lasting impact throughout the interchange area. Generally, closely spaced signalized intersections whose traffic signals are poorly timed can cause flow blockages on the next upstream link due to queue spillback, even during nominally undersaturated conditions.

In the analysis of interchange ramp terminals, the effects of the presence of closely spaced intersections are considered by adjusting the lane utilizations of the intersections' arterial approaches, by estimating the amount of additional lost time experienced at the upstream intersection due to the presence of the downstream queue, and by estimating the additional lost time due to demand starvation.

The lane utilization factors for the through approaches of closely spaced intersections should be estimated by subtracting 0.05 from the lane utilization factors obtained from Exhibit 18-30. Research has shown that those lane utilization factors are generally lower than those at a typical intersection approach.

The additional lost times experienced at the approaches to closely spaced intersections are estimated as discussed in the previous section. A brief overview is provided here for convenience.

Additional lost time may be experienced at any of the upstream approaches to the closely spaced intersections. The additional lost time due to the presence of the downstream queue is calculated for each of the upstream approaches i by using the following equation:

Equation 22-23

$$L_{D\text{-}U_i} = G_{U_i} - 0.106 \times DQ_i - 5.39 \times \frac{CG_{U_iD}}{C}$$

where

$L_{D\text{-}U_i}$ = lost time on upstream approach i due to presence of a downstream queue (s),

G_{U_i} = green interval for upstream approach i (s),

DQ_i = distance to downstream queue at beginning of upstream green for approach i (ft), and

CG_{U_iD} = common green time between upstream approach i and downstream through green (s).

The distance to the downstream queue at the beginning of the upstream green is calculated on the basis of the estimated average per lane queue length (in feet) for the through movement in the downstream link at the beginning of the respective upstream phase with Equation 22-12 through Equation 22-15.

When a significant portion of the traffic demand is held at the upstream adjacent intersection, demand starvation can occur on the external approaches to the interchange. The lost time caused by demand starvation on the external approaches to the interchange is estimated with the following equation:

$$L_{DS} = CG_{DS} - Q_{\mathrm{INITIAL}} \times h_I$$

Equation 22-24

where

L_{DS} = additional lost time due to demand starvation (s),

CG_{DS} = common green time with demand starvation potential (s),

h_I = saturation headway for through approach (= 3,600/saturation flow per lane) (s), and

Q_{INITIAL} = initial queue length at beginning of interval with demand starvation potential.

When the operations of adjacent closely spaced intersections affect and are affected by operations at the interchange, the external and internal approaches of the interchange could experience both lost time due to a downstream queue and demand starvation. For example, the internal approach of a diamond interchange may experience lost time due to a downstream queue created at the downstream intersection, and at the same time it may experience demand starvation. In those cases, the procedures of this chapter should not be applied; simulation or other alternative tools should be used instead.

LOS Determination

The determination of LOS for each O-D in the interchange involves the calculation of three performance measures: the queue storage ratios (R_Q), the v/c ratios, and the average control delays. First, the queue storage ratios and v/c ratios for each lane group are estimated. If for any given lane group one or both of these variables exceed 1.0, then the LOS for every O-D that travels through this particular lane group will be F. Next, the average control delay for each lane group is estimated. Finally, the average control delay for each O-D is estimated as the sum of the control delays for each lane group through which the O-D travels.

Queue Storage Ratio Estimation

The procedure to estimate the queue storage ratio (R_Q) is described in detail in Chapter 31, Signalized Intersections: Supplemental.

v/c Ratio Estimation

For a given lane group i, X_i is computed with the following equation:

Equation 22-25

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

where

X_i = $(v/c)_i$ ratio for lane group i,

v_i = actual or projected demand flow rate for lane group i (veh/h),

s_i = saturation flow rate for lane group i (veh/h),

g_i = effective green time for lane group i (s), and

C = cycle length (s).

Note that the effective green time g should be replaced by the adjusted green time g' if there is additional lost time due to a downstream queue and by the adjusted green time g'' if there is lost time due to demand starvation.

Average Control Delay Estimation for Each O-D

The average control delay for each lane group and movement is estimated by using the procedures provided in Chapter 18, Signalized Intersections. The average control delay for each O-D is estimated as the total delay experienced by that O-D. If the O-D travels only through one intersection, then its average control delay is equal to the average control delay of the respective lane group. If the O-D travels through both intersections, then its average control delay is the sum of the delays experienced at each of the lane groups along its path.

Operations at the closely spaced intersections are generally assessed by using the procedures of Chapter 18. The additional lost time estimation, which is computed with the procedures of this chapter, is used to determine the adjusted effective green time for all affected approaches.

FINAL DESIGN AND OPERATIONAL ANALYSIS FOR INTERCHANGES WITH ROUNDABOUTS

Roundabouts are generally analyzed with the procedures of Chapter 21 of the HCM. This chapter provides guidance for translating O-D demands into movement demands at a roundabout to apply the procedures of Chapter 21.

Exhibit 22-26 defines the movements traveling through an interchange with two roundabouts, while Exhibit 22-27 lists the O-D demands contributing to each of these movements. For example, for diamond interchanges, O-D Movements G, H, and J constitute Movement 15 in Exhibit 22-26.

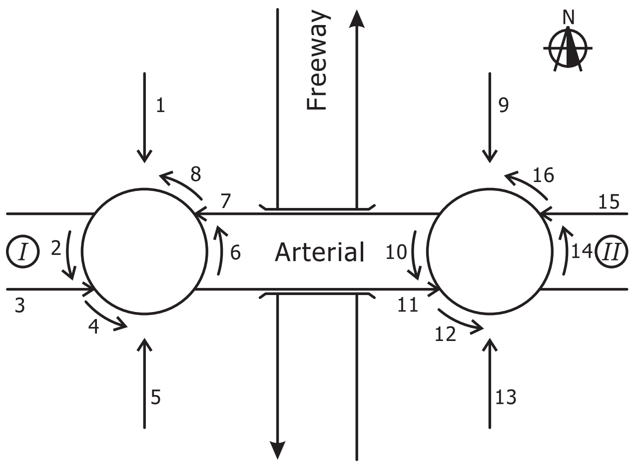


Exhibit 22-26
Illustration and Notation of O-D Demands at an Interchange with Roundabouts

Exhibit 22-27
Notation of O-D Demands at Interchanges with Roundabouts

Movement	Diamond	Parclo A-2Q	Parclo B-2Q	Parclo B-4Q
1	C, D, L, N	C, D, N	--	C
2	D, H, L, M, N	D, N	H, M, N	H, M
3	E, F, I	E, F	E, F, I	E, F, I
4	D, E, F, H, I, L, M, N	D, E, F, I, N	E, F, H, I, M	E, F, H, I, M
5	--	--	C	D, N
6	--	F	C	--
7	A, H, J, M	A, H, J, M	A, H, J, M	A, H, J, M
8	J, M	A, F, H, J, M	A, C, H, J, M	A, H, J, M
9	--	--	A, B, M	A, M
10	--	G	B	-
11	D, E, I, N	D, E, I, N	D, E, I, N	D, E, I, N
12	D, E, I, N	D, E, G, I, N	B, D, E	D, E, I, N
13	A, B, K, M	A, B, M	--	B
14	A, E, K, M, N	A, M	E, N	E, N
15	G, H, J	G, H, J	G, H, J	G, H, J
16	A, E, G, H, J, K, M, N	A, G, H, J, M	E, G, H, J, N	E, G, H, J, N
Movement	**SPUI**	**Parclo AB-4Q**	**Parclo A-4Q**	**Parclo AB-2Q**
1	C, D, L, N	C	C, D, N	--
2	D, H, L, M, N	H, M	D, N	H, M
3	E, F, I	E, F, I	E, F, I	E, F, I
4	D, E, I, N	E, F, H, I, M	D, E, F, I, N	E, F, H, I, M
5	A, B, K, M	D, N	--	C, D, N
6	A, E, K, M, N	--	--	C
7	G, H, J	A, H, J, M	A, H, J, M	A, H, J, M
8	A, H, J, M	A, H, J, M	A, H, J, M	A, C, H, J, M
9	--	--	--	--
10	--	--	--	G
11	--	D, E, I, N	D, E, I, N	D, E, I, N
12	--	D, E, I, N	D, E, I, N	D, E, G, I, N
13	--	A, B, M	A, B, M	A, B, M
14	--	A, M	A, M	A, M
15	--	G, H, J	G, H, J	G, H, J
16	--	A, G, H, J, M	A, G, H, J, M	A, G, H, J, M

Note: -- indicates movements that do not exist for a given interchange form.

In analyzing interchanges with roundabouts, Exhibit 22-26 and Exhibit 22-27 should be used to establish the roundabout movements. The procedures of Chapter 21 should then be applied to estimate the capacity and delay for each roundabout approach. Finally, Exhibit 22-13 should be used to determine the LOS for each O-D demand through the interchange.

INTERCHANGES WITH UNSIGNALIZED INTERSECTIONS

Interchanges with unsignalized intersections cannot be evaluated with the procedures of this chapter, since research has not yet been done on the operation of two closely spaced unsignalized intersections. In the absence of research, the intersections of such interchanges can be analyzed individually with the procedures of Chapter 19, Two-Way STOP-Controlled Intersections, or Chapter 20, All-Way STOP-Controlled Intersections.

OPERATIONAL ANALYSIS FOR INTERCHANGE TYPE SELECTION

The operational analysis for interchange type selection can be used to evaluate the operational performance of various interchange types. It allows the user to compare eight fundamental types of interchanges for a given set of demand flows. The eight signalized interchange types covered by the interchange type selection analysis methodology are as follows:

1. SPUI,
2. Tight urban diamond interchange (TUDI),
3. Compressed urban diamond interchange (CUDI),
4. Conventional diamond interchange (CDI),
5. Parclo A—four quadrants (Parclo A-4Q),
6. Parclo A—two quadrants (Parclo A-2Q),
7. Parclo B—four quadrants (Parclo B-4Q), and
8. Parclo B—two quadrants (Parclo B-2Q).

Other types of signalized interchanges cannot be investigated with this interchange type selection analysis methodology. Also, the operational analysis methodology does not distinguish between the TUDI, CUDI, and CDI types. In general, the interchange type selection analysis methodology categorizes diamond interchanges by the distance between the centerlines of the ramp roadways that form the signalized intersections. This distance is generally between 200 and 400 ft for the TUDI, between 600 and 800 ft for the CUDI, and between 1,000 and 1,200 ft for the CDI.

The method is based on research (*4*). The research also provides a methodology for selecting unsignalized interchanges. Since unsignalized interchanges are not covered by this chapter, users should consult the original source for this information.

The methodology is based on the estimation of the sums of critical flow ratios through the interchange and their use to estimate interchange delay. A combination of simulation and field data was used to develop critical relationships for the methodology.

The sum of critical flow ratios is based on an identification of all flows served during a particular signal phase and the determination of maximum flow ratios among the movements served by that phase. The models are similar to those used in Chapter 18 for signalized intersections; they are modified to take into account the fact that each signal phase involves two signalized intersections. Interchange delay is defined as the total of all control delays experienced by all interchange movements involved in signalized ramp terminal movements divided by the sum of all external movement flows. Additional information is available in the source report (*4*).

Because signalization is not specified for an interchange type selection analysis, it is assumed that the following interchange types are operated by a single signal controller: SPUI, TUDI, and CUDI. All other types are assumed to be operated by separate controllers at each signalized ramp terminal. In all cases, optimal signal timing and phasing are assumed.

Inputs and Applications

This interchange type selection analysis methodology can be used in several ways:

1. For a given set of O-D interchange movements, eight basic types of signalized interchanges may be compared on the basis of interchange delay;
2. For a given type of interchange, the impact of intersection spacing on interchange delay can be examined (within the range of applicability for each interchange type); and
3. For a given type of interchange, the impact of the number of lanes on ramp and surface arterial approaches and the movements assigned to these lanes can be examined, again by using interchange delay as the measure of effectiveness.

For any of these applications, all interchange O-D movements must be specified, generally by using full peak-hour volumes. The interchange type selection methodology is not detailed enough to use flow rates or to consider such factors as the presence of heavy vehicles.

In addition, for any given computation, it is necessary to specify the number of lanes assigned to each phase movement and to specify the distance between the centerlines of the two ramps, measured along the surface arterial.

Saturation Flow Rates

Implementation of the interchange type selection methodology requires the adoption of default values for saturation flow rate. Research (*4*) suggests the use of 1,900 veh/hg/ln for some basic cases. This is, however, based on a suggested base saturation flow rate of 2,000 pc/hg/ln, which is higher than the default values suggested in Chapter 18, Signalized Intersections. For consistency with the base saturation flow rate of 1,900 pc/hg/ln specified in Chapter 18, and to recognize the impact of various movements on saturation flow rate, it is recommended that the default values shown in Exhibit 22-28 be used in conjunction with the interchange type selection methodology. Alternatively, and if relevant information is available, the default values provided in Chapter 18, Signalized Intersections (Exhibit 18-28), may be used.

Exhibit 22-28
Default Values of Saturation Flow Rate for Use with the Operational Analysis for Interchange Type Selection

Interchange Type	Default Saturation Flow Rate (veh/hg/ln)		
	Left Turns	Through	Right Turns
SPUI	1,800	1,800	1,800
TUDI	1,700	1,800	1,800
CUDI	1,700	1,800	1,800
CDI	1,700	1,800	1,800
Parclo A-4Q	1,700	1,800	1,800
Parclo A-2Q	1,700	1,800	1,800
Parclo B-4Q	1,700	1,800	1,800
Parclo B-2Q	1,700	1,800	1,800

Where turning movements are in shared lanes, the "through" saturation flow rates should be used for analysis.

Step 1: Mapping O-D Flows into Interchange Movements

Since the primary objective of an interchange type selection analysis is to compare up to eight interchange types against a given set of design volumes, it is necessary first to convert a given set of design origin and destination volumes to movement flows through the signalized interchange. The methodology identifies volumes by signal phase by using the standard National Electrical Manufacturers Association (NEMA) numbering sequence for interchange phasing. Thus, movements are numbered 1 through 8 on the basis of the signal phase that accommodates the movement. Not all configurations and signalizations include all eight NEMA phases, and for some interchange forms some movements are not signalized and do not, therefore, contribute to interchange delay.

As for the operational analysis methodology, to simplify the mapping process, it is assumed that the freeway is oriented north-south and the surface arterial east-west. If the freeway is oriented in the east-west direction, rotate the interchange drawing or diagram clockwise until the freeway is in the north-south direction. In rotating clockwise, the westbound freeway direction becomes northbound and the eastbound freeway direction becomes southbound; the northbound arterial direction becomes eastbound and the southbound arterial direction becomes westbound. The methodology allows for separate consideration of freeway U-turn movements through the interchange. Thus, there are 14 basic movements that must be mapped for each interchange type.

For interchange types using two controllers, phase movements through the left (Intersection I) and right (Intersection II) intersections of the interchange are separately mapped and used in the procedure.

Exhibit 22-29 indicates the appropriate mapping of O-D demand volumes into phase movement volumes for the eight covered interchange types. The designation of the O-D demands is shown in Exhibit 22-21. The mapped phase movement volumes are then used in Step 2 to compute critical flow ratios.

Step 2: Computation of Critical Flow Ratios

The subsections that follow detail the computation of the critical flow ratio Y_c for the interchange for the eight basic configurations covered by this methodology.

Exhibit 22-29
Mapping of Interchange Origins and Destinations into Phase Movements for Operational Interchange Type Selection Analysis

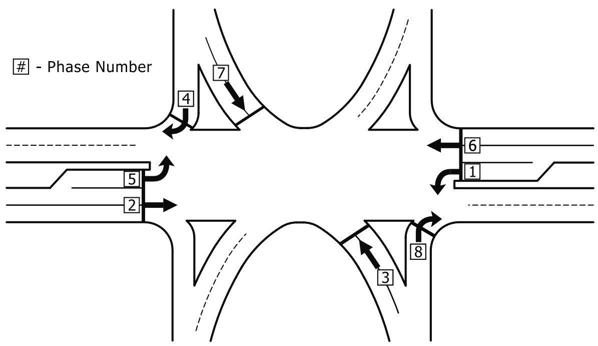
Interchange Type	NEMA Phase Movement Number 1	2	3	4	5	6	7	8
SPUI	H	I+**F**	A+M	**C**	E	J+**G**	D+N	**B**
TUDI /CUDI	H+M	E+I+**F**	--	D+**C**+N	E+N	H+J+**G**	--	A+M+**B**
CDI (I)	H+M	E+I+**F**	--	D+**C**+N	--	J+A	--	--
CDI (II)	--	I+D	--	--	E+N	H+J+**G**	--	A+M+**B**
Parclo A-4Q (I)	--	E+I	--	D+N+**C**	--	J+A+**M**+**H**	--	--
Parclo A-4Q (II)	--	I+D+**N**+**E**	--	--	--	J+H	--	A+M+**B**
Parclo A-2Q (I)	--	E+I	--	D+N+**C**	F	J+A+**H**+**M**	--	--
Parclo A-2Q (II)	G	I+D+**E**+**N**	--	--	--	H+J	--	A+M+**B**
Parclo B-4Q (I)	H+M	I+E+**F**	--	--	--	J+A	--	--
Parclo B-4Q (II)	--	I+D	--	--	E+N	H+J+**G**	--	--
Parclo B-2Q (I)	H+M	E+I+**F**	--	--	--	J+A	--	C
Parclo B-2Q (II)	--	I+D	--	B	E+N	H+J+**G**	--	--

Notes: -- indicates that phase movement does not exist for this interchange configuration.
Bold indicates movements not included when they operate from a separate lane with YIELD or STOP control.

Single-Point Urban Interchange

The phase movements in a SPUI are illustrated in Exhibit 22-30.

Exhibit 22-30
Phase Movements in a SPUI



Source: Bonneson et al. (*4*).

The sum of critical flow ratios is estimated as follows:

Equation 22-26

$$Y_c = A + R$$

$$A = \max\left[\left(\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2}\right); \left(\frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6}\right)\right]$$

$$R = \max\left[\left(\frac{v_3}{s_3 n_3} + \frac{v_4}{s_4 n_4}\right); \left(\frac{v_7}{s_7 n_7} + \frac{v_8}{s_8 n_8}\right)\right]$$

where

Y_c = sum of the critical flow ratios,

v_i = phase movement volume for phase *i* (veh/h),

n_i = number of lanes serving phase movement *i*,

s_i = saturation flow rate for phase movement *i* (veh/hg/ln),

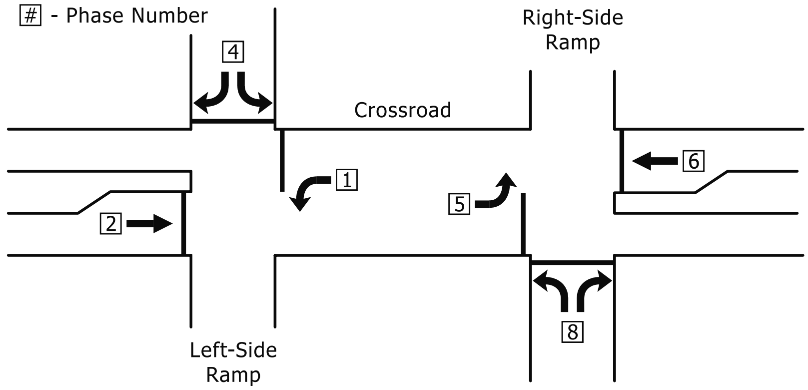
A = critical flow ratio for the arterial movements, and

R = critical flow ratio for the exit-ramp movements.

Tight Urban Diamond Interchange

Phase movements in a TUDI are illustrated in Exhibit 22-31.

Exhibit 22-31
Phase Movements in a Tight Urban or Compressed Urban Diamond Interchange



Source: Bonneson et al. (*4*).

The sum of critical flow ratios is computed as follows:

Equation 22-27

$$Y_c = A + R$$

$$A = \max\left[\left(\frac{v_2}{s_2 n_2} + \frac{v_4}{s_4 n_4} - y_3\right); \left(\frac{v_5}{s_5 n_5} + y_7\right)\right]$$

$$R = \max\left[\left(\frac{v_1}{s_1 n_1} + y_3\right); \left(\frac{v_6}{s_6 n_6} + \frac{v_8}{s_8 n_8} - y_7\right)\right]$$

$$y_3 = \min\left[\frac{v_4}{s_4 n_4}; y_t\right]$$

$$y_7 = \min\left[\frac{v_8}{s_8 n_8}; y_t\right]$$

where y_3 and y_7 are the effective flow ratios for concurrent (or transition) Phases 3 and 7, respectively; y_t is the effective flow ratio for the concurrent phase when dictated by travel time; and other variables are as previously defined.

For preliminary design applications, the default values of Exhibit 22-32 are recommended for y_t. The distance between the two intersections is measured from the centerline of the left ramp roadway to the centerline of the right ramp roadway.

Exhibit 22-32
Default Values for y_t

Distance Between Intersections *D′*	Default Value for y_t
200 ft	0.050
300 ft	0.070
400 ft	0.085

For Phase Movements 2 and 6, the number of assigned lanes (n_2 and n_6) is related to the arterial left-turn bay design. If the left-turn bay extends back to the external approach to the interchange, then the number of lanes on these external approaches is the total number of approaching lanes, including the left-turn bay. If the left-turn bay is provided only on the internal arterial link, n_2 or n_6, or both, would not include this lane.

Compressed Urban Diamond Interchange

Exhibit 22-31 illustrates the phase movement volumes for a CUDI. They are the same as for a TUDI. The sum of critical flow ratios is computed as follows:

Equation 22-28

$$Y_c = A + R$$

$$A = \max\left[\left(\frac{v_1}{s_1 n_1} + y_2\right); \left(\frac{v_5}{s_5 n_5} + y_6\right)\right]$$

$$R = \max\left[\left(\frac{v_4}{s_4 n_4}\right); \left(\frac{v_8}{s_8 n_8}\right)\right]$$

$$y_2 = \max\left[\left(\frac{v_2}{s_2 n_2}\right); \left(\frac{v_5}{s_2}\right)\right]$$

$$y_6 = \max\left[\left(\frac{v_8}{s_8 n_8}\right); \left(\frac{v_1}{s_6}\right)\right]$$

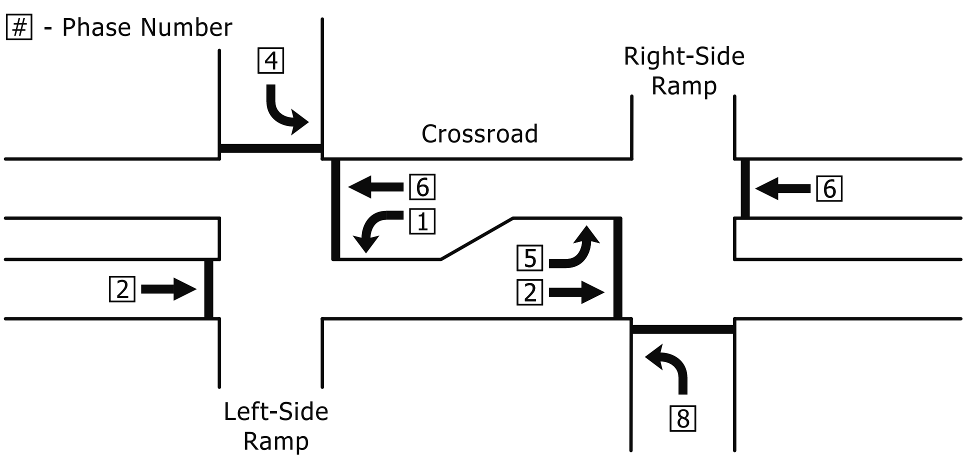
where y_2 and y_6 are the flow ratios for Phases 2 and 6, respectively, with consideration of prepositioning, and all other variables are as previously defined.

All Interchanges with Two Signalized Intersections and Separate Controllers

These interchange types include CDI, Parclo A-4Q, Parclo A-2Q, Parclo B-4Q, and Parclo B-2Q. The computation of the maximum sum of critical volumes is the same for each. Each has two signalized intersections, and each is generally operated with two controllers.

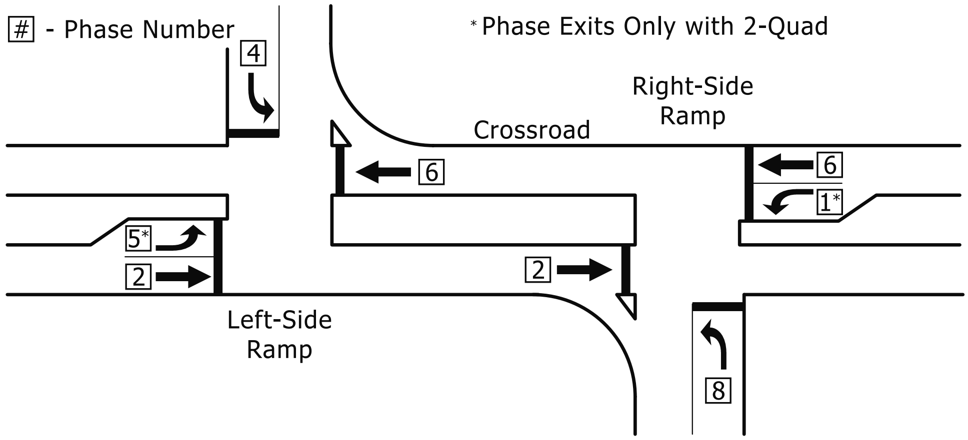
While the equations for estimating the maximum sum of critical volumes are the same, the phase movement volumes differ for each type of interchange, as was indicated in Exhibit 22-29. Exhibit 22-33 through Exhibit 22-35 illustrate the phase movements for each of these interchange types.

Exhibit 22-33
Phase Movements in a CDI



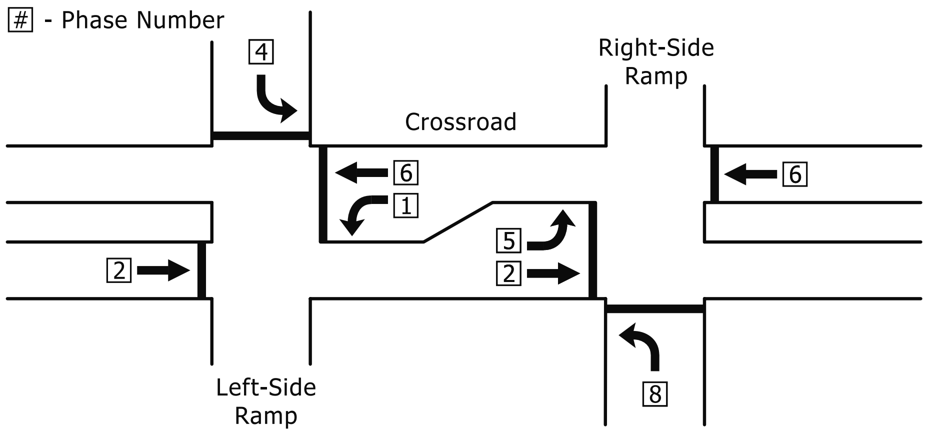
Source: Bonneson et al. (*4*).

Exhibit 22-34
Phase Movements in Parclo A-2Q and A-4Q Interchanges



Source: Messer and Bonneson (*3*).

Exhibit 22-35
Phase Movements in Parclo B-2Q and B-4Q Interchanges



Source: Messer and Bonneson (*3*).

For all conventional diamond, Parclo A, and Parclo B interchanges, the sum of critical flow ratios is computed as follows:

Equation 22-29

$$Y_{c,\max} = \max\left[Y_{c,I};Y_{c,II}\right]$$
$$Y_{c,I} = A_I + R_I$$
$$Y_{c,II} = A_{II} + R_{II}$$
$$A_{I,II} = \max\left[\left(\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2}\right);\left(\frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6}\right)\right]$$
$$R_{I,II} = \max\left[\left(\frac{v_4}{s_4 n_4}\right);\left(\frac{v_8}{s_8 n_8}\right)\right]$$

where all variables are as previously defined.

Note that when values of A_I, A_{II}, R_I, and R_{II} are computed, the movement volumes vary for the Intersections I and II, even though the phase movement designations are the same (Exhibit 22-29).

Some of the phase movement volumes do not exist in either Intersection I or II. A value of 0 is used for the volume in each case where this occurs.

Step 3: Estimation of Interchange Delay

Interchange delay for each interchange type or design is estimated by using regression models that were developed primarily from simulation output but validated with a limited amount of field data (*4*). In each case, two delay estimators are provided on the basis of the control of the off-ramp right-turn movements:

- Case A: Used where the right-turn movements from freeway off-ramps are controlled by the signal.
- Case B: Used where the right-turn movements from freeway off-ramps have a separate lane or lanes that are either free (uncontrolled) or controlled by a YIELD sign.

For SPUIs, a third condition is added. Where the right turns from the freeway ramps are controlled by a signal and right-turn-on-red is allowed, both cases are used, and the results are weighted by the proportions of right turns made during the red and green indications. Since the signal timing is unknown for an interchange type selection application, it is recommended that a 50%/50% split be assumed.

This modification, applied only to SPUIs, is necessary due to difficulties experienced in simulating right-turn-on-red at these interchanges.

Exhibit 22-36 gives the delay equations used to estimate interchange delay for the eight interchange types covered by the interchange type selection procedure. In each case, the variables used are defined as follows:

d = interchange delay (s/veh);

Y_c = critical or controlling flow ratio from Step 1; and

D = distance between the two intersections, measured between the centerlines of the two ramp roadways along the surface arterial (ft).

Exhibit 22-36 also shows the ranges of D' over which these equations are valid. They generally represent the normal design range for these interchange types. These equations should be used with great caution beyond these ranges.

Exhibit 22-36
Estimation of Interchange Delay for Eight Basic Interchange Types

Interchange Type	Valid Range of *D'* (ft)	Interchange Delay (d_I) CASE A: Right Turns Signalized	CASE B: Right Turns Free or YIELD-Controlled
SPUI	150–400	$15.1+(0.01D+16.0)\left(\frac{Y_c}{1-Y_c}\right)$	$15.1+(0.008\,D+5.9)\left(\frac{Y_c}{1-Y_c}\right)$
TUDI	200–400	$13.4+14.2\left(\frac{Y_c}{1-Y_c}\right)$	$13.4+12.8\left(\frac{Y_c}{1-Y_c}\right)$
CUDI	600–800	$19.2+[9.4-0.011\,(D-700)]\left[\frac{Y_c}{1-Y_c}\right]$	$19.2+[8.6-0.009(D-700)]\left[\frac{Y_c}{1-Y_c}\right]$
CDI	900–1,300	$17.1+[5.0-0.011(D-1{,}100)]\left[\frac{Y_c}{1-Y_c}\right]$	$17.1+[4.6-0.009\,(D-1{,}100)]\left[\frac{Y_c}{1-Y_c}\right]$
Parclo A-4Q	700–1,000	$11.7+[7.8-0.011(D-800)]\left[\frac{Y_c}{1-Y_c}\right]$	$11.7+[6.6-0.009\,(D-800)]\left[\frac{Y_c}{1-Y_c}\right]$
Parclo A-2Q	700–1,000	$19.1+[8.3-0.011(D-800)]\left[\frac{Y}{1-Y_c}\right]$	$19.1+[8.3-0.009\,(D-800)]\left[\frac{Y_c}{1-Y_c}\right]$
Parclo B-4Q	1,000–1,400	$9.3+[3.5-0.011(D-1{,}200)]\left[\frac{Y_c}{1-Y_c}\right]$	$9.3+[3.4-0.009\,(D-1{,}200)]\left[\frac{Y_c}{1-Y_c}\right]$
Parclo B-2Q	1,000–1,400	$26.2+[3.9-0.011(D-1{,}200)]\left[\frac{Y_c}{1-Y_c}\right]$	$26.2+[3.2-0.009\,(D-1{,}200)]\left[\frac{Y_c}{1-Y_c}\right]$

Delay estimates can be related to LOS. For consistency, the same criteria as used for the operational analysis methodology (Exhibit 22-11) are applied. Because LOS F is based on a *v/c* ratio greater than 1.00 or a queue storage ratio greater than 1.00, this interchange type selection methodology will never predict LOS F, because it does not predict these ratios. Users should be exceedingly cautious of results when interchange delay exceeds 85 to 90 s/veh.

In evaluating alternative interchange types, the exact distance, *D'*, may not be known for each of the alternatives. It is recommended that all lengths be selected at the midpoint of the range shown in Exhibit 22-36 for this level of analysis.

Interpretation of Results

The output of the interchange type selection procedure for signalized interchanges is a set of delay predictions for (*a*) various interchange types, (*b*) various distances *D* between the two intersections, or (*c*) various numbers and assignments of lanes on ramps and the surface arterials.

While, in general, the lower the interchange delay the better, a final choice must consider a number of other criteria that are not part of this methodology, including the following:

- Availability of right-of-way,
- Environmental impacts,
- Social impacts,
- Construction cost, and
- Benefit–cost analysis.

This methodology provides valuable information that can be used, in conjunction with other analyses, in making an appropriate choice of an interchange type and some of the primary design parameters. The final design, however, will be based on many other criteria in addition to the output of this methodology.

Users are also cautioned that while the definition of interchange delay is similar for both the interchange type selection methodology and the operational analysis methodology, different modeling approaches to delay prediction were taken, and there is no guarantee that the results of the two methodologies will be consistent.

3. APPLICATIONS

DEFAULT VALUES

Agencies that use the methodologies of this chapter are encouraged to develop a set of local default values based on measurements at interchanges in their jurisdiction. Local default values provide the best means of ensuring reasonable accuracy in the analysis results. In the absence of local default values, the values provided in Chapter 18, Signalized Intersections, can be used if appropriate.

TYPES OF ANALYSIS

There are two general types of analysis for signalized interchange ramp terminals: (*a*) final design and traffic operational analysis and (*b*) operational analysis for interchange type selection. Planning applications, as defined in other methodological chapters in the HCM, are not included in this chapter, because signal timing is a critical input to the procedure and is typically not known in the planning stages of a project.

In a final design and operational analysis the interchange configuration is given and detailed traffic control information is available. During the planning and preliminary design analysis stage, however, the engineer typically considers several configurations (e.g., SPUI, diamond), with limited input information available (for example, minimal or no traffic control information). At that stage, the interchange type selection analysis should be used to determine which interchange configurations would be preferable from an operational perspective. These two general types of analysis have different objectives, different requirements, and different levels of required input, and each of them is defined and discussed below.

Final Design and Operational Analysis

Final design and operational analysis for signalized interchanges is to be conducted when the type of interchange is known. The objective is either to provide final design details for LOS or to assess the interchange and provide LOS and other performance measures. Two subcategories are distinguished: (*a*) design analysis (where the input is the desired LOS and the outputs are design elements) and (*b*) operational analysis (where the input is complete design and the output is LOS).

Design analyses include highway design and signal design and are concerned with the physical, geometric, and signal control characteristics of the facility so that it operates at a desired LOS. For those types of analysis, the evaluation is conducted iteratively. The input data typically required for design analysis are fairly detailed and based substantially on design attributes that are being proposed. The objective of the interchange design analysis is to recommend geometric elements such as the number of lanes and storage bay length, or a signal control scheme, to maintain a given LOS. The principal inputs for design analysis are the design hourly volumes and the desired LOS for a given interchange configuration.

The objective of operational analysis is to obtain the LOS of a facility under given traffic, design, and signal control conditions. As in design analysis, the operational analysis is conducted for a given interchange configuration, where the input data include the turning movement demands, number of lanes and their respective lengths and channelization, and traffic control information.

Operational Analysis for Interchange Type Selection

This type of analysis should be used when the type of interchange is not known yet and the engineer is interested in assessing the traffic operations of various alternatives. Detailed information is not known (e.g., signalization information, design details). The principal inputs for an interchange type selection analysis are O-D demands and a list of feasible configurations that can be tested according to site physical and right-of-way conditions. This type of analysis considers signalized interchanges but does not consider unsignalized interchanges or interchanges with roundabouts.

O-D and Turning Movements

Exhibit 22-37 through Exhibit 22-44 illustrate how O-D movements can be obtained from turning movements for each type of interchange considered in this methodology. Exhibit 22-45 through Exhibit 22-52 provide the corresponding calculations for obtaining turning movements from O-D movements.

Exhibit 22-37
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo A-2Q Interchanges

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	EXT-LT		LT		A = (NB LT) – (NB UT)	
	RT		INT-RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	LT		EXT-LT		D = (SB LT) – (SB UT)	
	INT-RT		RT		E = (EB INT-RT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-LT	
Northbound (NB)	LT		LT		G = WB EXT-LT	
	RT		RT		H = (WB INT-RT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (SB UT and NB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-38
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo A-4Q Interchanges

Input					Output	
	Intersection I		**Intersection II**			
Approach	**Turning Move-ment**	**Volume (veh/h)**	**Turning Move-ment**	**Volume (veh/h)**	**O-D Movement Calculation**	**Volume (veh/h)**
Eastbound (EB)	LT		LT		A = (NB LT) – (NB UT)	
	EXT-RT		INT-RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	LT		LT		D = (SB LT) – (SB UT)	
	INT-RT		EXT-RT		E = (EB INT-RT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT		LT		G = WB EXT-RT	
	RT		RT		H = (WB INT-RT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (SB UT and NB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-39
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo AB-2Q Interchanges

Input					Output	
	Intersection I		**Intersection II**			
Approach	**Turning Move-ment**	**Volume (veh/h)**	**Turning Move-ment**	**Volume (veh/h)**	**O-D Movement Calculation**	**Volume (veh/h)**
Eastbound (EB)	LT		LT		A = (NB LT(II)) – (NB UT(II))	
	EXT-RT		INT-RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = NB LT(I)	
Westbound (WB)	INT-LT		EXT-LT		D = (NB RT(I)) – (NB UT(I))	
	RT		RT		E = (EB INT-RT) – (NB UT(I))	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT(I)		LT(II)		G = WB EXT-LT	
	RT(I)		RT(II)		H = (WB INT-LT) – (NB UT(II))	
	TH		TH		I = (EB INT-TH) – (NB RT(I)) + (NB UT(I))	
	UT(I)		UT(II)		J = (WB INT-TH) – (NB LT(II)) + (NB UT(II))	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT(II)	
	UT		UT		N = NB UT(I)	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway [NB UT(I) and NB UT(II)] are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-40
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo AB-4Q Interchanges

Input					Output	
	Intersection I		**Intersection II**			
Approach	**Turning Move-ment**	**Volume (veh/h)**	**Turning Move-ment**	**Volume (veh/h)**	**O-D Movement Calculation**	**Volume (veh/h)**
Eastbound (EB)	LT		LT		A = (NB LT(II)) – (NB UT(II))	
	EXT-RT		INT-RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = SB RT(I)	
Westbound (WB)	INT-LT		LT		D = (NB RT(I)) – (NB UT(I))	
	RT		EXT-RT		E = (EB INT-RT) – (NB UT(I))	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT		LT(II)		G = WB EXT-LT	
	RT(I)		RT(II)		H = (WB INT-LT) – (NB UT(II))	
	TH		TH		I = (EB INT-TH) – (NB RT(I)) + (NB UT(I))	
	UT(I)		UT(II)		J = (WB INT-TH) – (NB LT(II)) + (NB UT(II))	
Southbound (SB)	LT		LT		K	
	RT(I)		RT		L	
	TH		TH		M = NB UT(II)	
	UT		UT		N = NB UT(I)	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway [NB UT(I) and NB UT(II)] are user-specified.
Shading indicates movements that do not occur in this interchange form.

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (SB RT) – (SB UT)	
	EXT-RT		RT		B = SB LT	
	EXT-TH		INT-TH		C = NB LT	
Westbound (WB)	INT-LT		LT		D = (NB RT) – (NB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (NB UT)	
	INT-TH		EXT-TH		F = (EB EXT-RT)	
Northbound (NB)	L T		LT		G = (WB EXT-RT)	
	RT		RT		H = (WB INT-LT) – (SB UT)	
	TH		TH		I = (EB INT-TH) – (NB RT) + (NB UT)	
	UT		UT		J = (WB INT-TH) – (SB RT) + (SB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = SB UT	
	UT		UT		N = NB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-41
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo B-2Q Interchanges

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (SB RT(II)) – (SB UT)	
	EXT-RT		RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = SB RT(I)	
Westbound (WB)	INT-LT		LT		D = (NB RT(I)) – (NB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (NB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	L T		LT		G = WB EXT-RT	
	RT(I)		RT(II)		H = (WB INT-LT) – (SB UT)	
	TH		TH		I = (EB INT-TH) – (NB RT(I)) + (NB UT)	
	UT		UT		J = (WB INT-TH) – (SB RT(II)) + (SB UT)	
Southbound (SB)	LT		LT		K	
	RT(I)		RT(II)		L	
	TH		TH		M = SB UT	
	UT		UT		N = NB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-42
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo B-4Q Interchanges

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (NB LT) – (NB UT)	
	EXT-RT		RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	INT-LT		LT		D = (SB LT) – (SB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	L T		LT		G = WB EXT-RT	
	RT		RT		H = (WB INT-LT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K = NB TH	
	RT		RT		L = SB TH	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-43
Worksheet for Obtaining O-D Movements from Turning Movements for Diamond Interchanges

Exhibit 22-44
Worksheet for Obtaining O-D Movements from Turning Movements for SPUIs

Input			Output	
Approach	**Turning Movement**	**Volume (veh/h)**	**O-D Movement Calculation**	**Volume (veh/h)**
Eastbound (EB)	LT		A = NB LT	
	RT		B = NB RT	
	TH		C = SB RT	
Westbound (WB)	LT		D = SB LT	
	RT		E = EB LT	
	TH		F = EB RT	
Northbound (NB)	LT		G = WB RT	
	RT		H = WB LT	
	TH		I = EB TH	
	UT		J = WB TH	
Southbound (SB)	LT		K = NB TH	
	RT		L = SB TH	
	TH		M	
	UT		N	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through.
The flow of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-45
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo A-2Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	EXT-LT = F		LT	
B			RT		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	LT		EXT-LT = G	
E			INT-RT = H+M		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-46
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo A-4Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	EXT-LT = F		LT	
B			RT		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	LT		EXT-LT = G	
E			INT-RT = H+M		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-47
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo AB-2Q Interchanges

Input		Output				
O-D Movement	Volume (veh/h)	Approach	Intersection I: Turning Movement Calculation	Intersection I: Volume (veh/h)	Intersection II: Turning Movement Calculation	Intersection II: Volume (veh/h)
A		Eastbound (EB)	LT		LT	
B			EXT RT = F		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		EXT-LT = G	
E			RT		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT(I) = C		LT(II) = A+M	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT(I) = N		UT(II) = M	
K		Southbound (SB)	LT		LT	
L			RT		RT	
M			TH		TH	
N			UT		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-48
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo AB-4Q Interchanges

Input		Output				
O-D Movement	Volume (veh/h)	Approach	Intersection I: Turning Movement Calculation	Intersection I: Volume (veh/h)	Intersection II: Turning Movement Calculation	Intersection II: Volume (veh/h)
A		Eastbound (EB)	LT		LT	
B			EXT RT = F		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT(II) = A+M	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT(I) = N		UT(II) = M	
K		Southbound (SB)	LT		LT	
L			RT(I) = C		RT	
M			TH		TH	
N			UT		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-49
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo B-2Q Interchanges

Input		Output				
O-D Movement	Volume (veh/h)	Approach	Intersection I: Turning Movement Calculation	Intersection I: Volume (veh/h)	Intersection II: Turning Movement Calculation	Intersection II: Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT = C		LT	
H			RT = D+N		RT	
I			TH		TH	
J			UT = N		UT	
K		Southbound (SB)	LT		LT = B	
L			RT		RT = A+M	
M			TH		TH	
N			UT		UT = M	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-50
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo B-4Q Interchanges

Input		Output				
			Intersection I		**Intersection II**	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT = N		UT	
K		Southbound (SB)	LT		LT	
L			RT(I) = C		RT(II) = A+M	
M			TH		TH	
N			UT		UT = M	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-51
Worksheet for Obtaining Turning Movements from O-D Movements for Diamond Interchanges

Input		Output				
			Intersection I		**Intersection II**	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH = K	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH = L		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-52
Worksheet for Obtaining Turning Movements from O-D Movements for SPUIs

Input		Output		
O-D Movement	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	LT = E	
B			RT = F	
C			TH = I	
D		Westbound (WB)	LT = H	
E			RT = G	
F			TH = J	
G		Northbound (NB)	LT = A	
H			RT = B	
I			TH = K	
J			UT	
K		Southbound (SB)	LT = D	
L			RT = C	
M			TH = L	
N			UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This section contains specific guidance for the application of alternative tools to the analysis of interchange ramp terminals. Chapter 34, Interchange Ramp Terminals: Supplemental, contains supplemental examples illustrating the use of alternative tools for interchange analysis. Additional information on this topic may be found in the Technical Reference Library in Volume 4.

General guidance on alternative tools is provided in Chapters 6 and 7.

As indicated in Chapter 6, traffic models may be classified in several ways (e.g., deterministic versus stochastic, macroscopic versus microscopic). The alternative tools used for interchange analysis are generally based on models that are microscopic and stochastic in nature. Therefore, the discussion in this section will be limited to microsimulation tools.

Strengths of the HCM Procedure

This chapter offers a comprehensive procedure for analyzing the performance of several types of interchanges. Simulation-based tools offer a more detailed treatment of the arrival and departure of individual vehicles and features of the signal control system, but for most purposes, the HCM procedure produces an acceptable approximation. The HCM procedure offers some advantages over the simulation approach:

- The HCM provides saturation flow rate adjustment factors based on extensive field studies.
- The HCM produces direct estimates of capacity and *v/c* ratio. These measures are much more elusive in simulation.
- The HCM provides LOS by O-D, which facilitates the comparison of operational performance for different interchange configurations.
- It provides deterministic estimates of the measures of effectiveness, which is important for some purposes such as development impact review.
- Simulation tools use definitions of delay (and therefore LOS) different from those of the HCM, especially for movements that are oversaturated at some point during the analysis. Great care must therefore be taken in producing LOS estimates directly from simulation. Chapter 7 discusses simulation-based performance measures in more detail.

Identified Limitations of the HCM Procedure

The identified limitations of the HCM procedure for this chapter cover a number of conditions that are not evaluated explicitly, including the following:

- Oversaturated conditions, particularly cases when the downstream queue spills back into the upstream intersection for long periods of time;
- The impact of spillback on freeway operations (however, the method does estimate the expected queue storage ratio for the ramp approaches);
- Ramp metering and its resulting spillback of vehicles into the interchange;

- Impacts of the interchange operations on arterial operations and the extended surface street network;
- Lane utilizations for interchanges with additional approaches that are not part of the prescribed interchange configuration; and
- Full cloverleaf interchanges (freeway-to-freeway or system interchanges), since the scope of the chapter is limited to service interchanges (e.g., freeway-to-arterial interchanges).

If any of these conditions apply to a particular situation, then alternative tools that recognize them explicitly should be considered to supplement or replace the methodology described in this chapter.

Additional Features and Performance Measures Available From Alternative Tools

This chapter provides a methodology for estimating the capacity, control delay, queue storage ratio, and LOS associated with a given set of traffic, control, and design conditions at an interchange. As with most other procedural chapters in this manual, simulation outputs, especially graphics-based presentations, can provide details on problems at specific elements of the interchange that might otherwise go unnoticed with a macroscopic analysis. For example, problems associated with turn bay overflow or blockage of access to turn bays can be better observed by using microscopic simulation tools. Alternative tools offer additional performance measures such as number of stops, fuel consumption, and pollution. The animated graphics displays offered by many simulation tools are especially useful for observing network operations and identifying problems at specific elements.

Development of HCM-Compatible Performance Measures Using Alternative Tools

Simulation tools provide a wealth of information with regard to performance measures, including queue length, travel time, emissions, and so forth. However, simulation tools often have different definitions for each of these performance measures. General guidance on developing compatible performance measures based on the analysis of individual vehicle trajectories is provided in Chapter 7, with supplemental examples provided in Chapter 24, Concepts: Supplemental. Chapter 18, Signalized Intersections, provides some specific guidance on performance measures for signalized approaches that also applies to this chapter. To obtain LOS for a specific O-D, the analyst will need to obtain the performance measures for the specific approaches using that particular O-D and aggregate them as indicated in the methodology section of this chapter.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

For interchanges, the definitions of delay and queuing are the most significant conceptual differences between the HCM and simulation modeling. Both are measures of effectiveness used to obtain LOS for each O-D, and simulated estimates of them would produce results inherently different from those obtained by the analytical method described in this chapter.

Lane utilization is also treated differently. Simulation tools derive lane distributions and utilization implicitly from driver behavior modeling, while the deterministic model used in this chapter develops lane distributions from empirical models. Differences in the treatment of random arrivals are also an issue in the comparison of performance measures. This topic was discussed in detail in Chapter 18, and the same phenomena apply to this chapter.

In some cases, and when saturation flow rate is not an input, simulation tools do not explicitly account for differences between left-turning, through, and right-turning movements, and all three have very similar saturation headway values. Thus, the left- and right-turn lane capacity would likely be overestimated in those types of tools.

Adjustment of Simulation Parameters to Match the HCM Parameters

Some adjustments will generally be required before an alternative tool can be used effectively to supplement or replace the procedures described in this chapter. For example, the parameters that determine the capacity of a signalized approach (e.g., steady state headway and start-up lost time) should be adjusted to ensure that the simulated approach capacities match the HCM values.

One parameter specific to this chapter is the lane utilization on the approaches within the interchange. Driver behavior model parameters that affect lane choice should be examined closely and modified if necessary to produce better agreement with the lane distributions estimated by the procedures in this chapter.

Simulation tools do not produce explicit capacity estimates. The accepted method of determining the capacity of a signalized approach by simulation is to perform the simulation run(s) with a demand in excess of the computed capacity and use the throughput as an indication of capacity. Chapter 7 provides additional guidance on the determination of capacity in this manner. The Chapter 7 discussion points out the complexities that can arise when self-aggravating phenomena occur as the operation approaches capacity. Because of the interaction of traffic movements within an interchange, the potential for self-aggravating situations is especially high.

In complex situations, conceptual differences between the deterministic procedures in this chapter and those of simulation tools may exist such that the production of compatible capacity estimates is not possible. In such cases, the capacity differences should be noted.

Step-by-Step Recommendations for Applying Alternative Tools

General guidance on selecting and applying alternative tools is provided in Chapters 6 and 7. Chapter 18 provides recommendations specifically for signalized intersections that also apply to interchange ramp terminals.

One step that is specific to this chapter is the emulation of the traffic control hardware. Generally, simulation tools provide great flexibility in emulating actuated control, particularly in the type and location of detectors. In most cases, simulation tools attach a controller to each intersection (or node) in the network. This creates problems for some interchange operations in which a controller at one node must be connected to an approach to another node. A diamond interchange operating with one controller is an example of the complexities that can arise in the emulation of the traffic control scheme.

Some tools are able to accommodate complex schemes more flexibly than others. The ability to emulate the desired traffic control scheme is an important consideration in the selection of a tool for interchange analysis.

Sample Calculations Illustrating Alternative Tool Applications

Supplemental problems involving the use of alternative tools for signalized intersection analysis are presented in Chapter 34.

Example Problem 1 in this chapter involves a diamond interchange that offers the potential for illustrating the use of alternative tools. There are no limitations in this example that suggest the need for alternative tools. It is possible, however, to introduce situations in which alternative tools might be needed for a proper assessment of performance.

Chapter 34 includes supplemental examples that apply alternative tools to deal with two conditions that are beyond the scope of the procedures presented in this chapter.

1. A two-way STOP-controlled intersection in close proximity to the diamond interchange and
2. Ramp metering on one of the freeway entrance ramps connected to the interchange.

In both cases, the demand volumes are varied to examine the self-aggravating effects on the operation of the facility.

4. EXAMPLE PROBLEMS

INTRODUCTION

This part of the chapter describes the application of each of the final design, operational analysis for interchange type selection, and roundabouts analysis methods through the use of example problems. Exhibit 22-53 describes each of the three example problems included in this chapter and indicates the methodology applied. Additional example problems are provided in Chapter 34, Interchange Ramp Terminals: Supplemental.

Exhibit 22-53
Example Problem Descriptions

Problem Number	Description	Application
1	Find the control delay, queue storage ratio, and LOS of a diamond interchange	Operational analysis
2	Find the control delay, queue storage ratio, and LOS of a Parclo A-2Q interchange	Operational analysis
3	Compare eight types of signalized interchanges	Operational analysis for interchange type selection

EXAMPLE PROBLEM 1: DIAMOND INTERCHANGE

The Interchange

The interchange of I-99 (northbound/southbound, NB/SB) and University Drive (eastbound/westbound, EB/WB) is a diamond interchange. Exhibit 22-54 provides the interchange volumes and channelization, while Exhibit 22-55 provides the signalization information.

Exhibit 22-54
Example Problem 1: Interchange Volumes and Channelization

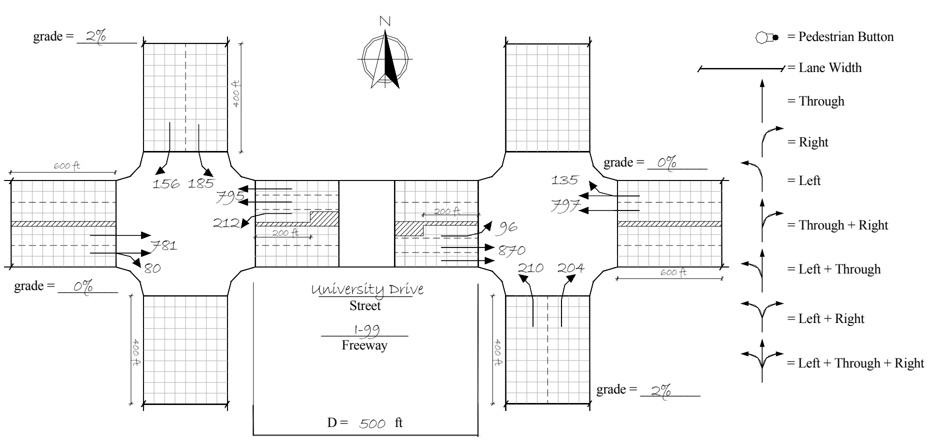


Exhibit 22-55
Example Problem 1: Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (4+8)	Φ (4+7)	Φ (2+5)	Φ (4+8)	Φ (1+6)	Φ (3+8)
Green time (s)	63	43	39	63	53	29
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		19			9	

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. Travel path radii are 50 ft for all right-turning movements and 75 ft for all left-turning movements. Arrival Type 4 is assumed for all arterial movements and Arrival Type 3 for all other movements.

There are 5% heavy vehicles on both the external and internal through movements, and the peak hour factor (PHF) for the interchange is estimated to be 0.90. Start-up lost time and extension of effective green are both 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Outline of Solution

Calculation of O-Ds

O-Ds through this diamond interchange are calculated on the basis of Exhibit 22-21(a). Since all movements utilize the signal and no right-turns-on-red are allowed, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the resulting PHF-adjusted values are presented in Exhibit 22-56.

Exhibit 22-56
Example Problem 1: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	210	233
B	204	227
C	156	173
D	185	206
E	96	107
F	80	89
G	135	150
H	212	236
I	685	761
J	585	650
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

Both external approaches to this interchange consist of a two-lane shared right and through lane group. Use of the two-lane model of Exhibit 22-16 results in the predicted lane utilization percentages for the external through approaches that are presented in Exhibit 22-57.

Exhibit 22-57
Example Problem 1: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	Maximum Lane Utilization	Lane Utilization Factor
Eastbound external	0.5056	0.4944	0.5056	0.9890
Westbound external	0.5181	0.4819	0.5181	0.9651

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hg/ln by using Equation 22-3. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 22-57. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equation 22-6 through Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated as indicated in Chapter 18, Signalized Intersections. The estimated saturation flow rates for the eastbound approaches are shown in Exhibit 22-58.

Calculation of Common Green and Lost Time due to Downstream Queue and Demand Starvation

The duration of common green between various movements is calculated next. Exhibit 22-59 provides the beginning and end of each phase for the two intersections, as well as the calculations of common green between various movements at the two intersections.

Exhibit 22-58
Example Problem 1: Calculation of Saturation Flow Rate for Eastbound Approaches

	Eastbound Turning Movements		
	EXT-TH, EXT-RT	**INT-TH**	**INT-LT**
Base saturation flow (s_0, in pc/hg/ln)	1,900	1,900	1,900
Number of lanes (N)	2	2	1
Lane width adjustment (f_w)	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.952	0.952	1.000
Grade adjustment (f_g)	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.989	0.952	1.000
Left-turn adjustment (f_{LT})	1.000	1.000	0.930
Right-turn adjustment (f_{RT})	0.999	1.000	1.000
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.991	1.000	0.930
Traffic pressure adjustment for lane group (f_v)	1.034	1.036	0.963
Adjusted saturation flow (s, in veh/hg/ln)	3,700	3,568	1,703

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-59
Example Problem 1:
Common Green Calculations

Phase	Intersection I Phase Begin	Intersection I Phase End	Intersection II Phase Begin	Intersection II Phase End
Phase 1	0	63	150	53
Phase 2	68	111	58	111
Phase 3	116	155	116	145

Movement	1st Green Time of Cycle Begin	1st Green Time of Cycle End	2nd Green Time of Cycle Begin	2nd Green Time of Cycle End	Overlap
EB EXT-TH	0	63			53
EB INT-TH	150	53	116	150	
WB EXT-TH	150	53			53
WB INT-TH	0	111			
SB RAMP	116	155			34
EB INT-TH	150	53	116	150	
NB RAMP	58	111			53
WB INT-TH	0	111			
WB INT-LT	68	111			0
EB INT-TH	150	53			
EB INT-LT	116	145			0
WB INT-TH	0	111			

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, INT = internal, EXT = external.

The next step involves the calculation of lost time due to downstream queues. First, the queue at the beginning of the upstream arterial phase and at the beginning of the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. These numbers are then subtracted from the internal link storage length to determine the storage available at the beginning of the respective upstream phase. Exhibit 22-60 presents the calculation of the downstream queues followed by the calculation of the respective lost time due to those downstream queues.

Exhibit 22-60
Example Problem 1:
Calculation of Lost Time due to Downstream Queues

	EB EXT-TH	SB-LT	WB EXT-TH	NB-LT
V_R or V_A (veh/h)	206	868	233	886
N_R or N_A	1	2	1	2
G_R or G_A (s)	39	63	53	63
G_D (s)	97	97	111	111
C (s)	160	160	160	160
CG_{UD} or CG_{RD} (s)	53	34	53	53
Queue length (Q_A or Q_R) (ft)	0.0	4.1	0.0	0.0
G_R or G_A (s)	63	39	63	53
C (s)	160	160	160	160
D_{QA} or D_{QR} (ft)	500	496	500	500
CG_{UD} or CG_{RD} (s)	53	34	53	53
Additional lost time ($L_{D\text{-}A}$ or $L_{D\text{-}R}$) (s)	0.0	0.0	0.0	0.0
Total lost time t'_L (s)	5.0	5.0	5.0	5.0
Effective green time g' (s)	63.0	39.0	63.0	53.0

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, EXT = external.

Calculation of lost time due to demand starvation begins by determining the queue storage length at the beginning of an interval with demand starvation potential by using Equation 22-17. The lost time due to demand starvation is then calculated by using Equation 22-16. The respective calculations are presented in Exhibit 22-61.

Exhibit 22-61
Example Problem 1: Calculation of Lost Time due to Demand Starvation

	EB INT-TH	WB INT-TH
$v_{Ramp\text{-}L}$ (veh/h)	206	233
$v_{Arterial}$ (veh/h)	868	886
C (s)	160	160
$N_{Ramp\text{-}L}$	1	1
$N_{Arterial}$	2	2
CG_{RD} (s)	34	53
CG_{UD} (s)	53	53
H_I	2.02	2.04
$Q_{INITIAL}$ (ft)	0	0
CG_{DS} (s)	0	0
L_{DS} (s)	0	0
t''_L (s)	5	5
Effective green time g'' (s)	97	111

Note: EB = eastbound, WB = westbound, TH = through, INT = internal.

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue divided by the available queue storage by using Equation 31-91. Exhibit 22-62 presents the calculation of the queue storage ratio for all eastbound movements in Example Problem 1. Control delay for each movement is calculated according to Equation 18-47. Exhibit 22-63 provides the control delay for each eastbound movement of the interchange.

Exhibit 22-62
Example Problem 1: Queue Storage Ratio Calculations for Eastbound Movements

	EXT-TH, EXT-RT	INT-LT	INT-TH
Q_{bL} (ft)	0.0	0.0	0.0
v (veh/h/ln group)	957	107	967
s (veh/h/ln)	1850	1703	1784
g (s)	63	29	97
g/C	0.39	0.18	0.61
I	1.00	0.71	0.71
c (veh/h/ln group)	1459	309	2163
$X = v/c$	0.66	0.35	0.45
r_a (ft/s^2)	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0
S_s (mi/h)	5	5	5
S_{pl} (mi/h)	40	40	40
S_a (mi/h)	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04
Rp	1.000	1.333	1.333
P	0.39	0.24	0.81
r (s)	97	131	63
t_f (s)	0.01	0.00	0.00
q (veh/s)	0.27	0.03	0.27
q_g (veh/s)	0.27	0.04	0.36
q_r (veh/s)	0.27	0.03	0.13
Q_1 (veh)	15.22	3.47	3.84
Q_2 (veh)	0.93	0.14	1.37
T	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00
t_A	0	0	0
Q_e (veh)	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00
Q_3 (veh)	0.00	0.00	0.00
Q (veh)	16.15	3.65	3.98
L_h (ft)	25.01	25.00	25.01
L_a (ft)	600	200	500
R_Q	0.67	0.46	0.20

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-63
Example Problem 1: Calculation of Control Delay for Eastbound Movements

	EXT-TH, EXT-RT	INT-LT	INT-TH
g (s)	NA	29	97
g' (s)	63	NA	NA
g/C or g'/C	0.39	0.18	0.61
c (veh/h)	1459	309	2163
$X = v/c$	0.66	0.35	0.45
d_1 (s/veh)	39.6	52.8	7.3
k	0.5	0.5	0.5
d_2 (s/veh)	4.6	2.2	0.5
d_3 (s/veh)	0.0	0.0	0.0
k_{min}	0.04	0.04	0.04
u	0	0	0
t	0	0	0
d (s/veh)	44.1	55.0	7.8

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external, NA = not applicable.

Results

This section discusses the estimation of the delay experienced by each O-D movement and provides the resulting LOS. Delay for each O-D is estimated as a sum of the movement delays for each movement utilized by the O-D, as shown in Equation 22-1. Next, the v/c and queue storage ratios must be checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement will be F. Exhibit 22-64 presents a summary of the results for all O-D movements at this interchange. As shown, neither v/c nor R_Q exceed 1, and all movements have LOS E or better. The LOS is determined by using Exhibit 22-11.

Exhibit 22-64
Example Problem 1: O-D Movement LOS

O-D Movement	Delay (s)	$v/c > 1$?	$R_Q > 1$?	LOS
A	45.6	No	No	C
B	43.7	No	No	C
C	54.6	No	No	C
D	63.6	No	No	D
E	99.2	No	No	E
F	44.2	No	No	C
G	37.5	No	No	C
H	82.7	No	No	D
I	52.0	No	No	C
J	39.8	No	No	C

EXAMPLE PROBLEM 2: PARCLO A-2Q INTERCHANGE

The Interchange

The interchange of I-75 (NB/SB) and Newberry Avenue (EB/WB) is a Parclo A-2Q interchange. Exhibit 22-65 provides the interchange volumes and channelization, while Exhibit 22-66 provides the signalization information.

Exhibit 22-65
Example Problem 2: Intersection Plan View

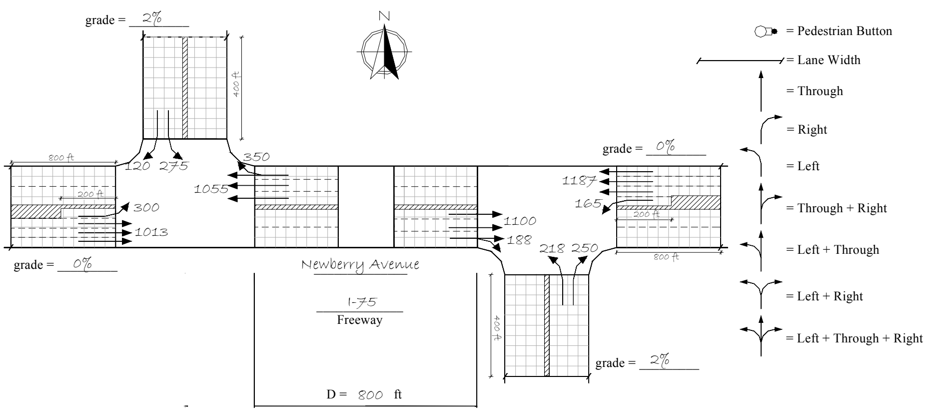


Exhibit 22-66
Example Problem 2: Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (3+8)	Φ (4+8)	Φ (2+5)	Φ (4+7)	Φ (1+6)	Φ (4+8)
Green time (s)	25	60	40	25	35	65
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		0			0	

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. The eastbound and westbound left-turn radii are 80 ft, while all remaining turning movements have radii of 50 ft. The arrival type is assumed to be 4 for arterial movements and 3 for all other movements.

There are 10% heavy vehicles on both the external and internal through movements, and the PHF for the interchange is estimated to be 0.95. Start-up lost time is 3 s for all approaches, while the extension of effective green is 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Outline of Solution

Calculation of O-Ds

O-Ds through this interchange are calculated on the basis of Exhibit 22-21(b). Since all movements utilize the signal and no right-turns-on-red are allowed, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the resulting PHF-adjusted values are presented in Exhibit 22-67.

Exhibit 22-67
Example Problem 2: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	218	229
B	250	263
C	120	126
D	275	289
E	188	198
F	300	316
G	165	174
H	350	368
I	825	868
J	837	881
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

The external approaches to this interchange consist of a three-lane through lane group for the external approaches. Use of the three-lane model from Exhibit 22-17 results in the predicted lane utilization percentages for the external through approaches presented in Exhibit 22-68.

Exhibit 22-68
Example Problem 2: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	V_3	Maximum Lane Utilization	Lane Utilization Factor
Eastbound	0.2660	0.2791	0.4549	0.4549	0.7328
Westbound	0.2263	0.2472	0.5265	0.5265	0.6332

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hg/ln by using Equation 22-3. The lane utilization of the external approaches is obtained as shown above in Exhibit 22-68. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equation 22-6 through Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated according to Chapter 18. The results of these calculations for the eastbound approaches are presented in Exhibit 22-69.

Exhibit 22-69
Example Problem 2: Calculation of Saturation Flow Rates for Eastbound Approaches

	EXT-TH	EXT-LT	INT-TH, INT-RT
Base saturation flow (s_0, in pc/hg/ln)	1,900	1,900	1,900
Number of lanes (N)	3	1	3
Lane width adjustment (f_w)	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.909	1.000	0.909
Grade adjustment (f_g)	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.733	1.000	1.000
Left-turn adjustment (f_{LT})	1.000	0.934	1.000
Right-turn adjustment (f_{RT})	1.000	1.000	0.998
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	1.000	0.934	0.985
Traffic pressure adjustment for lane group (f_v)	0.997	1.013	1.016
Adjusted saturation flow (s, in veh/hg/ln)	3786	1798	5253

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Calculation of Common Green and Lost Time due to Downstream Queue

The duration of common green between various movements is calculated next. Exhibit 22-70 presents the beginning and ending of each phase at the two intersections and the calculations of common green between various movements at the two intersections.

Exhibit 22-70
Example Problem 2: Common Green Calculations

Phase	Intersection I Phase Begin	Intersection I Phase End	Intersection II Phase Begin	Intersection II Phase End	
Phase 1	0	25	0	25	
Phase 2	30	90	30	65	
Phase 3	95	135	70	135	
Movement	**1st Green Time of Cycle Begin**	**1st Green Time of Cycle End**	**2nd Green Time of Cycle Begin**	**2nd Green Time of Cycle End**	**Overlap**
EB EXT-TH	0	90			20
EB INT-TH	70	135			
WB EXT-TH	0	25	70	135	20
WB INT-TH	30	90			
SB RAMP	95	135			40
EB INT-TH	70	135			
NB RAMP	30	65			35
WB INT-TH	30	90			

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, TH = through, INT = internal, EXT = external.

The next step involves the calculation of lost time due to downstream queues. First, the queue at the beginning of the upstream arterial phase and at the beginning of the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. These numbers are then subtracted from the internal link storage length to determine the storage at the beginning of the respective upstream phase. Exhibit 22-71 presents the calculation of the downstream queues followed by the calculation of the respective lost time due to those downstream queues.

Exhibit 22-71
Example Problem 2: Calculation of Lost Time due to Downstream Queues

	EB EXT-TH	SB-LT	WB EXT-TH	NB-LT
V_R or V_A (veh/h)	289	1066	229	1249
N_R or N_A	1	3	1	3
G_R or G_A (s)	40	90	35	95
G_D (s)	65	65	60	60
C (s)	140	140	140	140
CG_{UD} or CG_{RD} (s)	20	40	20	35
Queue length (Q_A or Q_R) (ft)	0.9	48.6	0.0	89.4
G_R or G_A (s)	90	40	95	35
C (s)	140	140	140	140
D_{QA} or D_{QR} (ft)	799	751	800	711
CG_{UD} or CG_{RD} (s)	20	40	20	35
Additional lost time ($L_{D\text{-}A}$ or $L_{D\text{-}R}$) (s)	0	0	0	0
Total lost time t'_L (s)	6	6	6	6
Effective green time g' (s)	89	39	94	34

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, EXT = external.

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue divided by the available queue storage by using Equation 31-91. Exhibit 22-72 presents the calculation of queue storage for all eastbound movements. Control delay for each movement is calculated according to Equation 18-47. Exhibit 22-73 provides the control delay for each eastbound movement of this interchange.

Exhibit 22-72
Example Problem 2: Queue Storage Ratio Calculations for Eastbound Movements

	EXT-TH	EXT-LT	INT-TH, INT-RT
Q_{bL} (ft)	0.0	0.0	0.0
v (veh/h/ln group)	1066	316	1282
s (veh/h/ln)	1262	1798	1751
g (s)	89	24	64
g/C	0.64	0.17	0.46
I	1.00	1.00	0.90
c (veh/h/ln group)	2407	308	2401
$X = v/c$	0.44	1.02	0.54
r_a (ft/s^2)	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0
S_s (mi/h)	5	5	5
S_{pl} (mi/h)	40	40	40
S_a (mi/h)	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04
Rp	1.000	1.000	1.333
P	0.64	0.17	0.61
r (s)	51	116	76
t_r (s)	0.00	0.01	0.00
q (veh/s)	0.30	0.09	0.38
q_g (veh/s)	0.30	0.09	0.50
q_r (veh/s)	0.30	0.09	0.27
Q_1 (veh)	5.36	10.75	6.91
Q_2 (veh)	0.13	4.94	0.33
T	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00
t_A	0	0	0
Q_e (veh)	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00
Q_3 (veh)	0.00	0.00	0.00
Q (veh)	5.49	15.69	7.24
L_h (ft)	25.02	25.00	25.02
L_a (ft)	800	200	800
R_Q	0.17	1.96	0.23

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-73
Example Problem 2: Control Delay for Eastbound Movements

	EXT-TH	INT-LT	INT-TH, INT-RT
g (s)	NA	24	64
g' (s)	89	NA	NA
g/C or g'/C	0.64	0.17	0.46
c (veh/h)	2407	308	2401
$X = v/c$	0.44	1.02	0.56
d_1 (s/veh)	12.9	58.0	18.81
k	0.5	0.5	0.5
d_2 (s/veh)	0.6	57.7	1.53
d_3 (s/veh)	0.0	0.0	0.0
k_{min}	0.04	0.04	0.04
u	0	0	0
t	0	0	0
d (s/veh)	13.5	115.7	20.34

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external, NA = not applicable.

Results

This section discusses the estimation of delay experienced by each O-D movement and provides the resulting LOS. Delay for each O-D is estimated as a sum of the movement delays for each movement utilized by the O-D, as shown in Equation 22-1. Next, the v/c and queue storage ratios are checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement will be F. Exhibit 22-74 presents the resulting delay, v/c, and R_Q for each O-D movement. As shown, O-D Movement F has a v/c and R_Q ratio greater than 1, resulting in a LOS of F.

Exhibit 22-74
Example Problem 2: O-D Movement LOS

O-D Movement	Delay (s)	$v/c > 1$?	$R_Q > 1$?	LOS
A	78.9	No	No	D
B	55.7	No	No	D
C	41.1	No	No	C
D	70.0	No	No	D
E	33.9	No	No	C
F	115.7	Yes	Yes	F
G	61.6	No	No	D
H	40.0	No	No	C
I	33.9	No	No	C
J	40.0	No	No	C

EXAMPLE PROBLEM 3: OPERATIONAL ANALYSIS FOR INTERCHANGE TYPE SELECTION

The Interchange

An interchange is to be built at the junction of I-83 (NB/SB) and Archer Road (EB/WB) in an urban area. The interchange type selection methodology is used.

The Question

Which interchange type is likely to operate better under the given demands?

The Facts

This interchange will have two-lane approaches with single left-turn lanes on the arterial approaches. Freeway ramps will consist of two-lane approaches with channelized right turns in addition to the main ramp lanes. Default saturation flow rates for use in the interchange type selection analysis are given in Exhibit 22-28. The O-Ds of traffic through this interchange are shown in Exhibit 22-75.

Exhibit 22-75
Example Problem 3: O-D Demand Information for the Interchange

O-D Movement	Volume (veh/h)
A	400
B	350
C	400
D	550
E	150
F	200
G	225
H	185
I	600
J	800
K	2,500
L	3,200
M	0
N	10

Outline of Solution

Mapping O-D Flows into Interchange Movements

The primary objective of this example is to compare up to eight interchange types against a given set of design volumes. The first step is to convert these O-D flows into movement flows through the signalized interchange. The interchange type methodology uses the standard NEMA numbering sequence for interchange phasing, and Exhibit 22-29 demonstrates which O-Ds make up each NEMA phase at the eight interchange types. Exhibit 22-76 shows the corresponding volumes for this example on the basis of the O-Ds from Exhibit 22-75. Since this interchange has channelized right turns, Exhibit 22-77 shows only the NEMA phasing volumes utilizing the signals.

Exhibit 22-76
Example Problem 3: NEMA Flows (veh/h) for the Interchange

Interchange Type	NEMA Phase Movement Number							
	1	2	3	4	5	6	7	8
SPUI	185	800	400	400	150	1,025	560	350
TUDI /CUDI	185	950	--	960	160	1,210	--	750
CDI (I)	185	950	--	960	--	1,200	--	--
CDI (II)	--	1,150	--	--	160	1,210	--	750
Parclo A-4Q (I)	--	750	--	960	--	1,385	--	--
Parclo A-4Q (II)	--	1,310	--	--	--	985	--	750
Parclo A-2Q (I)	--	750	--	960	200	1,385	--	--
Parclo A-2Q (II)	225	1,310	--	--	--	985	--	750
Parclo B-4Q (I)	185	950	--	--	--	1,200	--	--
Parclo B-4Q (II)	--	1,150	--	--	160	1,210	--	--
Parclo B-2Q (I)	185	950	--	--	--	1,200	--	400
Parclo B-2Q (II)	--	1,150	--	350	160	1,210	--	--

Note: (I) and (II) indicate the intersections within the interchange type; -- indicates that the movement does not exist in this interchange type.

Interchange Type	NEMA Phase Movement Number							
	1	2	3	4	5	6	7	8
SPUI	185	600	400	0	150	1,025	560	350
TUDI /CUDI	185	750	--	560	160	1,210	--	750
CDI (I)	185	750	--	560	--	1,200	--	--
CDI (II)	--	1,150	--	--	160	1,210	--	750
Parclo A-4Q (I)	--	750	--	560	--	1,385	--	--
Parclo A-4Q (II)	--	1,150	--	--	--	985	--	750
Parclo A-2Q (I)	--	750	--	560	200	1,385	--	--
Parclo A-2Q (II)	225	1,150	--	--	--	985	--	750
Parclo B-4Q (I)	185	750	--	--	--	1,200	--	--
Parclo B-4Q (II)	--	1,150	--	--	160	1,210	--	--
Parclo B-2Q (I)	185	750	--	--	--	1,200	--	400
Parclo B-2Q (II)	--	1,150	--	350	160	1,210	--	--

Exhibit 22-77
Example Problem 3: NEMA Flows for the Interchange Without Channelized Right Turns

Computation of Critical Flow Ratios

Comparison between the eight intersection types begins with computation of the critical flow ratio at each interchange type. The first intersection type to be calculated is the SPUI by using Equation 22-26. On the basis of the default saturation flow rate for a SPUI and the values for the NEMA phases, Exhibit 22-78 shows the output from these calculations for a SPUI.

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements, A	0.368	0.306
Critical flow ratio for the ramp movements, R	0.350	0.156
Sum of critical flow ratios, Y_c	0.718	0.462

Exhibit 22-78
Example Problem 3: SPUI Critical Flow Ratio Calculations

The TUDI critical flow ratios are calculated by using Equation 22-27. Exhibit 22-79 shows these calculations for a 300-ft distance between the two TUDI intersections.

	Signalized Right Turns	Channelized Right Turns
Effective flow ratio for concurrent phase when dictated by travel time, y_t	0.070	0.070
Effective flow ratio for concurrent Phase 3, y_3	0.070	0.070
Effective flow ratio for concurrent Phase 7, y_7	0.070	0.070
Critical flow ratio for the arterial movements, A	0.461	0.294
Critical flow ratio for the ramp movements, R	0.474	0.315
Sum of critical flow ratios, Y_c	0.935	0.609

Exhibit 22-79
Example Problem 3: TUDI Critical Flow Ratio Calculations

The CUDI critical flow ratios are calculated by using Equation 22-28. Exhibit 22-80 shows these calculations for a CUDI with the given O-D flows.

Exhibit 22-80
Example Problem 3: CUDI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Flow ratio for Phase 2 with consideration of prepositioning, y_2	0.264	0.208
Flow ratio for Phase 6 with consideration of prepositioning, y_6	0.208	0.208
Critical flow ratio for the arterial movements, A	0.373	0.332
Critical flow ratio for the ramp movements, R	0.267	0.156
Sum of critical flow ratios, Y_c	0.640	0.488

The CDI, Parclo A-4Q, Parclo A-2Q, Parclo B-4Q, and Parclo B-2Q all use separate controllers. For these interchanges the critical flow ratios are calculated for each intersection, and then the maximum is taken for the overall interchange critical flow ratio. These numbers are all calculated by using Equation 22-29 and the default saturation flows. Exhibit 22-81 through Exhibit 22-85 show the calculations for these interchanges utilizing two controllers.

Exhibit 22-81
Example Problem 3: CDI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.165
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.655	0.498
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.221	0.118
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.651	0.486
Maximum sum of critical flow ratios, Y_c	0.655	0.498

Exhibit 22-82
Example Problem 3: Parclo A-4Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.385	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.282
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.667	0.615
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.364	0.333
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.208	0.111
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.572	0.444
Maximum sum of critical flow ratios, Y_c	0.667	0.615

Exhibit 22-83
Example Problem 3: Parclo A-2Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.502	0.451
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.165
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.784	0.616
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.452
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.221	0.111
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.651	0.563
Maximum sum of critical flow ratios, Y_c	0.784	0.616

Exhibit 22-84
Example Problem 3: Parclo B-4Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.000	0.000
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.373	0.333
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.000	0.000
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.430	0.368
Maximum sum of critical flow ratios, Y_c	0.430	0.368

Exhibit 22-85
Example Problem 3: Parclo B-2Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.111	0.111
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.484	0.444
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.103	0.103
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.533	0.471
Maximum sum of critical flow ratios, Y_c	0.533	0.471

Estimation of Interchange Delay

Estimation of interchange delay is the final step when interchange types are compared. On the basis of the critical flow ratios calculated previously, Exhibit 22-36 can be used to calculate the delay at the eight interchange types. Exhibit 22-86 shows the solutions to these calculations.

Exhibit 22-86
Example Problem 3:
Interchange Delay for the Eight Interchange Types

Intersection Type	Interchange Delay d_I (s) Right Turns Signalized	Interchange Delay d_I (s) Right Turns Free or YIELD-controlled
SPUI	62.9	22.0
TUDI	217.7	33.3
CUDI	35.9	27.4
CDI	26.6	21.7
Parclo A-4Q	26.2	21.6
Parclo A-2Q	47.4	29.0
Parclo B-4Q	11.9	11.3
Parclo B-2Q	30.7	29.0

Results

As demonstrated by Exhibit 22-86, a Parclo B-4Q would be the best interchange type to select in terms of operational performance for the given O-D flows at this interchange. For the final interchange type selection, however, additional criteria should be considered, including those related to economic, environmental, and land use concerns.

5. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

1. Elefteriadou, L., C. Fang, R. P. Roess, E. Prassas, J. Yeon, X. Cui, A. Kondyli, H. Wang, and J. M. Mason. *Capacity and Quality of Service of Interchange Ramp Terminals.* Final Report, National Cooperative Highway Research Program Project 3-60. Pennsylvania State University, University Park, March 2005.
2. Elefteriadou, L., A. Elias, C. Fang, C. Lu, L. Xie, and B. Martin. *Validation and Enhancement of the Highway Capacity Manual's Interchange Ramp Terminal Methodology.* Final Report, National Cooperative Highway Research Program Project 3-60A. University of Florida, Gainesville, 2009.
3. Messer, C. J., and J. A. Bonneson. *Capacity of Interchange Ramp Terminals.* Final Report, National Cooperative Highway Research Program Project 3-47. Texas A&M Research Foundation, College Station, April 1997.
4. Bonneson, J., K. Zimmerman, and M. Jacobson. *Review and Evaluation of Interchange Ramp Design Considerations for Facilities Without Frontage Roads.* Research Report 0-4538-1. Cooperative Research Program, Texas Transportation Institute, Texas A&M University System, College Station, 2004.
5. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets,* 5th ed. Washington, D.C., 2004.