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16. Abstract

Based on a recent change in Texas Department of Transportation (TxDOT) policy, frontage roads are not to be included along controlled-access highways unless a study indicates that the frontage road improves safety, improves operations, lowers overall facility costs, or provides essential access. Interchange design options that do not include frontage roads are to be considered for all new freeway construction. Ramps in non-frontage-road settings can be more challenging to design than those in frontage-road settings for several reasons. Adequate ramp length, appropriate horizontal and vertical curvature, and flaring to increase storage area at the crossroad intersection should all be used to design safe and efficient ramps for non-frontage-road settings. However, current design procedures available in standard TxDOT reference documents focus on ramp design for frontage-road settings. The objective of this research project was to develop recommended design procedures for interchange ramps on facilities without frontage roads.

This report describes design procedures for interchange ramps on facilities without frontage roads. The procedures described include models for evaluating the operational and safety benefits of alternative ramp configurations. They also include guidelines for designing ramps of adequate length, appropriate horizontal and vertical curvature, and reasonable accommodation of larger vehicles.

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RECOMMENDED RAMP DESIGN PROCEDURES FOR FACILITIES WITHOUT FRONTAGE ROADS

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data published herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) and/or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation. It is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was James Bonneson, P.E. #67178.

NOTICE

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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CHAPTER 1. INTRODUCTION

OVERVIEW

Based on a recent change in TxDOT policy, frontage roads are not to be included along controlled-access highways unless a study indicates that the frontage road improves safety, improves operations, lowers overall facility costs, or provides essential access. The intent of this policy change is to extend the service life of the freeway and the mobility of the corridor by promoting development away from the freeway. Interchange design options that do not include frontage roads are to be considered for all new freeway construction.

Ramps in non-frontage-road settings can be more challenging to design than those in frontage-road settings for two reasons. First, they must provide drivers a safe transition between the high-speed freeway and the stop condition at the crossroad intersection. Unlike the main lanes, a ramp's design speed changes along its length such that ramp length and design speed change are interrelated. Ramp curves must be carefully sized such that speed changes along the ramp occur in safe and comfortable increments for both cars and trucks.

Second, ramp design for non-frontage-road settings is challenging because the "effective" ramp length (i.e., that portion of the ramp measured from the gore area to the back of queue) can vary based on traffic demands. Thus, during peak demand hours, the speed change may need to occur over a relatively short length of ramp. In contrast, the speed change can occur over the full length of the ramp during low-volume conditions. Sound ramp design should accommodate such variation in effective ramp length by conservatively designing for the high-volume condition. Similar issues exist for entrance ramp design when ramp metering or high-occupancy-vehicle bypass lanes are present.

OBJECTIVE AND SCOPE

Objective

The objective of this report is to describe recommended design procedures for interchange ramps on facilities without frontage roads. The procedures described include models for evaluating the operational and safety benefits of alternative ramp configurations. They also include guidelines for designing ramps of adequate length, appropriate horizontal and vertical curvature, and reasonable accommodation of larger vehicles.

The ramp design guidelines were developed from a variety of sources. These sources include: design guidelines used by several state departments of transportation (DOTs), interviews with TxDOT engineers, analysis of data obtained from field and simulation studies, analysis of crash data for interchanges in Texas, and the guidance in the *Roadway Design Manual* (*1*) and in *A Policy on Geometric Design of Highways and Streets* (*Green Book*) (*2*). Two reports by Bonneson et al.

(*3, 4*) document these development activities. The analytic procedures in this report are implemented in two Excel ® spreadsheets. These spreadsheets are available at http://tti.tamu.edu/documents/0-4538-cma.xls and http://tti.tamu.edu/documents/0-4538-rsa.xls.

Scope

The procedures developed for this research are applicable to the geometric design of interchanges in urban, metropolitan, and rural environments on Texas freeways without frontage roads. The design guidelines address controls and considerations for designing the ramp proper and ramp terminal of both exit and entrance ramps. The guidelines reflect consideration of the performance and physical aspects of both passenger cars and trucks. Finally, they also reflect a sensitivity to safety and operations. The procedures do not address the question of "Where or when should frontage roads be used?" This question is appropriately addressed by the TxDOT administration (and the Texas Transportation Commission) and is a matter of agency policy.

SERVICE INTERCHANGES

This section describes the range of interchange types used with freeway facilities without frontage roads. It provides some guidance on the selection of interchange type and associated ramp configuration based on consideration of traffic demands, topographic features, and right-of-way constraints. It also provides context for the discussion of the ramp evaluation procedures and design guidelines that follow in Chapters 2 and 3, respectively.

There are two basic categories of interchange: system interchanges and service interchanges. System interchanges serve all turning movements without traffic control and, hence, are used for the intersection of two freeway facilities. In contrast, service interchanges have some type of stop or signal control for one or more left-turn movements. These interchanges are most appropriate at locations where the intersecting facility is classified as a local, collector, or arterial roadway. Service interchanges are much more frequent in number than system interchanges and are the subject of this report.

Alternative Interchange Types

The two most common types of service interchange for non-frontage-road settings are the diamond and partial cloverleaf (or "parclo") interchanges. Typical variations of both types are shown in Figure 1-1. The absence of frontage roads allows for the consideration of a wide range of interchange types. This range allows for a more cost-effective ramp configuration to accommodate site-specific traffic demands, topographic features, and right-of-way constraints. The absence of frontage roads eliminates the U-turn traffic movement inherent to frontage road interchanges. A high-volume U-turn movement often causes operational problems at the interchange if it is not provided an exclusive U-turn lane.

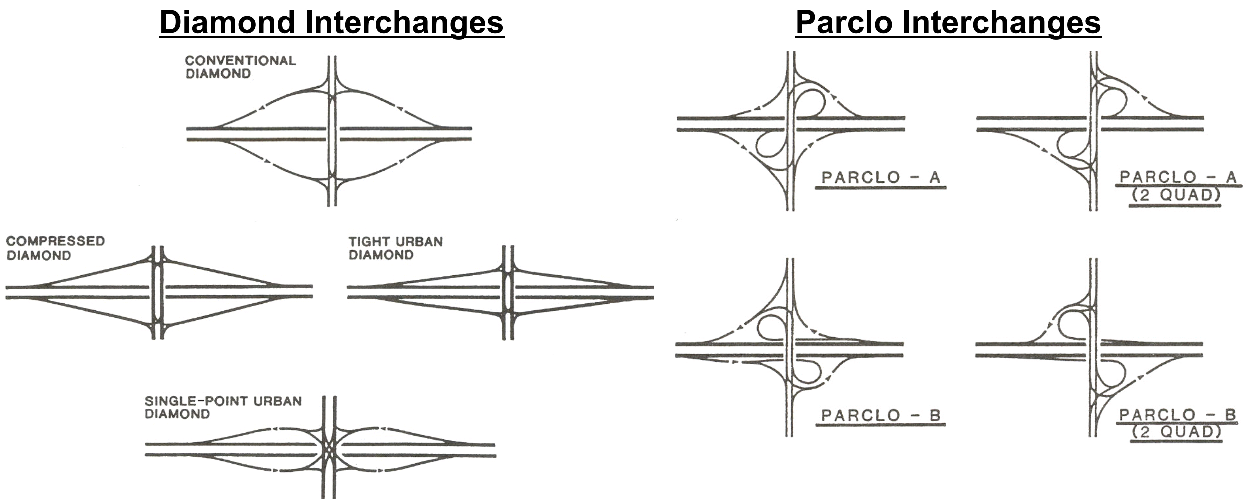


Figure 1-1. Interchange Types Commonly Used in Non-Frontage-Road Settings.

The diamond and parclo interchanges can be further categorized by their ramp separation distance, ramp geometry, ramp control mode, and crossroad cross section. With respect to these categories, the attributes of typical diamond interchanges are listed in Table 1-1; those of typical parclos are listed in Table 1-2.

As indicated in Table 1-1, one advantage of the diamond interchange is that the turn movements from the major road and from the crossroad are "true" to the intended change in direction of travel. In other words, a driver makes a left turn at the interchange when desiring to make a left turn in travel direction. This characteristic is desirable because it is consistent with driver expectancy. Unfamiliar drivers can be confused if a loop ramp configuration requires them to make a right turn at the interchange when they desire to make a left turn in their direction of travel.

In urbanized areas, the tight urban diamond interchange (TUDI) and the single-point urban interchange (SPUI) can provide efficient traffic operation along the crossroad. The TUDI's ramp terminals are easily coordinated using a single signal controller. This ease of coordination is due primarily to the interchange's relatively short ramp separation distance. The efficiency of the SPUI stems from its use of a single signalized junction and non-overlapping left-turn paths. In contrast, the compressed diamond is not as operationally efficient as the TUDI or the SPUI. This characteristic is due to the compressed diamond's wider ramp separation, which is not as conducive to crossroad signal coordination. As a result, the compressed diamond interchange is better suited to rural or suburban settings where traffic demands are low to moderate.

Table 1-1. Characteristics of Typical Diamond Interchanges.

Category		Diamond Interchange Type [1]			
		Conventional	**Compressed**	**TUDI**	**SPUI**
Ramp Separation (centerline to centerline)		800 to 1200 ft	400 to 800 ft	200 to 400 ft	150 to 250 ft (stopline to stopline)
Typical Location		Rural	Suburban	Urban	Urban
Ramp Terminal Control		● 2 stop signs ● 2 actuated signals	● 1 actuated signal ● 2 semi-act. signals	● 1 actuated signal ● 1 or 2 pretimed signals	● 1 actuated signal
Crossroad Left-Turn Bay Geometry	Location	Internal to terminals.	Internal to terminals.	Internal and possibly external, if needed.[2]	**External to intersection.**
	Length	200 to 300-ft bay.	150 to 300-ft bay.	Parallel bays, if needed.[2]	As needed for storage.
Signal Coordination		**Often not essential but can be achieved.**	Often needed but difficult to obtain.	**Needed and easily achieved.**	**Single signal.**
Volume Limits		Moderate.	Moderate.	**Moderate to high.**	**Moderate to high.**
Bridge Width		**Through lanes only.**	Through lanes plus width of median and often part of both left-turn bays.	Through lanes plus both left-turn bays, if needed.[2]	Through lanes plus width of median and wider left-turn lane.
Operational Experience		**Acceptable.**	Acceptable–sometimes the need for progression is a problem.	**Acceptable.**	**Acceptable.**
Signal Phases/Terminal		3, if signalized	3	3	3
Left from Crossroad		**Via left turn.**	**Via left turn.**	**Via left turn.**	**Via left turn.**
Left from Major Road		**Via left turn.**	**Via left turn.**	**Via left turn.**	**Via left turn.**
Right from Crossroad		**Via right turn.**	**Via right turn.**	**Via right turn.**	**Via right turn.**
Right from Major Road		**Via right turn.**	**Via right turn.**	**Via right turn.**	**Via right turn.**
Queues on Exit-Ramp?		Yes	Yes	Yes	Yes

Notes:

1 - Characteristics in **bold** font are generally recognized as advantageous in terms of operations, safety, or cost.

2 - If left-turn and U-turn demands are low to moderate and four-phase operation is provided, then bays are generally not needed. If left-turn or U-turn demands are high or four-phase operation is not provided, then left-turn bays are needed between the ramp terminals. If left-turn bays are provided, then they should extend backward through the upstream ramp terminal.

The parclos shown in Figure 1-1 are most applicable to situations where a specific left-turn movement pair has sufficiently high volume to have a significant negative impact on ramp terminal operation. Both variations of the parclo A provide uncontrolled service for crossroad drivers intending to turn left (in travel direction) at the interchange. Both variations of the parclo B typically provide uncontrolled service for major-road drivers intending to turn left at the interchange.

Table 1-2. Characteristics of Typical Partial Cloverleaf Interchanges.

Category		Parclo Interchange Type [1]			
		Parclo A	Parclo B	Parclo A (2-quad)	Parclo B (2-quad)
Ramp Separation [2] (centerline to centerline)		700 to 1000 ft	1000 to 1400 ft	700 to 1000 ft	1000 to 1400 ft
Typical Location		Suburban	Suburban	Rural	Rural
Ramp Terminal Control		• 2 semi-actuated	• 2 semi-actuated	• 2 stop signs • 2 actuated signals	• 2 stop signs • 2 actuated signals
Crossroad Left-Turn Bay Geometry	Location	**Not applicable.**	Internal to terminals.	**External to terminals.**	Internal to terminals.
	Length	Not applicable.	≤ 40 percent of ramp separation distance.	Based on volume.	≤ 40 percent of ramp separation distance.
Signal Coordination		**Often needed and easily achieved.**	**Not needed as downstream through is unstopped.**	Rarely needed in rural settings.	Rarely needed in rural settings.
Volume Limits		**Moderate to high.**	**Moderate to high.**	Moderate.	Moderate.
Bridge Width		**Through lanes only.**	Through lanes plus width of median.	**Through lanes only.**	Through lanes plus width of median.
Operational Experience		**Acceptable.**	**Acceptable.**	Potential for wrong-way movements.	Potential for wrong-way movements.
Signal Phases/Terminal		2	2	3	3
Left from Crossroad		Via right turn.	**Via left turn.**	Via right turn.	**Via left turn.**
Left from Major Road		**Via left turn.**	Via right turn.	**Via left turn.**	Via right turn.
Right from Crossroad		**Via right turn.**	**Via right turn.**	Via left turn.	**Via right turn.**
Right from Major Road		**Via right turn.**	**Via right turn.**	**Via right turn.**	Via left turn.
Queues on Exit-Ramp?		Yes (on diagonal)	**No**	Yes (on diagonal)	Yes (on loop)

Notes:
1 - Characteristics in **bold** font are generally recognized as advantageous in terms of operations, safety, or cost.
2 - Ramp separation distances listed for the parclo A and parclo A (2-quad) are based on 170-ft loop radii (25-mph design speed). Distances listed for the parclo B and parclo B (2-quad) are based on 250-ft loop radii (30-mph design speed).

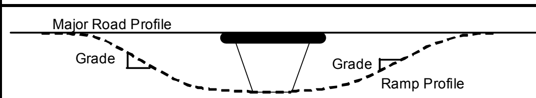
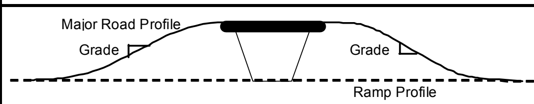
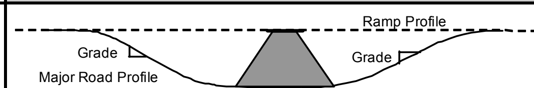
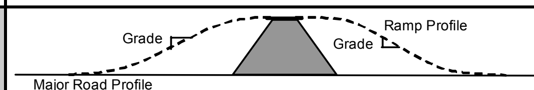
The parclo A and parclo B are more efficient than their "2-quad" counterparts. This feature stems from their elimination of one left-turn movement from the ramp terminal signalization. One advantage of the parclo A is that it satisfies the expectancy of major-road drivers by providing turn movements that are "true" to the driver's intended direction of travel. A second advantage is that it does not require left-turn bays on the crossroad. This advantage can result in a narrower bridge. A third advantage is that its terminal design is not conducive to wrong-way movements. Finally, the speed change from the crossroad to the loop ramp is likely to be relatively small and easy to accommodate with horizontal curves of minimum radius.

The parclo B also has several advantages. One advantage is that its signalized ramp terminals do not require coordination because signal timing for the outbound travel direction can provide a continuous green indication. A second advantage is that it does not require queues to form on the exit ramp because the left-turn and right-turn movements are unsignalized at their intersection with the crossroad. Finally, its ramp terminal design is not conducive to wrong-way movement.

Overpass vs. Underpass

A fundamental consideration in interchange design is whether the major road should be carried over (i.e., an overpass design) or under the crossroad. When topography does not govern, the relative advantages and disadvantages listed in Table 1-3 should be considered when selecting an overpass or underpass design *(2)*. They also provide some insight as to the merit of locating the crossroad below, at, or above the existing ground level.

Table 1-3. Advantages of the Overpass and Underpass Configurations.

Crossroad Location Relative to Existing Ground	**Major Road Location Relative to Crossroad**	
	Overpass	**Underpass**
Below	 ● Offers best sight distance along major road.	Not applicable.
At	 ● Offers best possibility for stage construction. ● Eliminates drainage problems.	 ● Reduced traffic noise to adjacent property. ● Provides best view of ramp geometry.
Above	Not applicable.	 ● Ramp grades decelerate exit-ramp vehicles and accelerate entrance-ramp vehicles. ● Eliminates drainage problems. ● Typically requires least earthwork.
	<u>Other Overpass Advantages:</u> ● Through traffic is given aesthetic preference. ● Accommodates oversize loads on major road.	<u>Other Underpass Advantages:</u> ● Interchange and ramps easily seen by drivers on the major road. ● Bridge size (for crossroad) is smaller.

The information in Table 1-3 identifies advantages of the overpass and underpass designs. However, it appears that the underpass design offers greater benefit when ramp safety and operations are key considerations. The underpass design with the crossroad elevated “above” ground level is often the most advantageous because it provides major-road drivers with: (1) ample preview distance as they approach the interchange, and (2) ramp grades that are helpful in slowing exit-ramp drivers and accelerating entrance-ramp drivers. The one exception to this generalization is the SPUI. For this interchange, the underpass design with the crossroad “at” ground level is preferred because it provides the driver the best view of the ramp geometry in the terminal area. This view is important at a SPUI because of its unusual ramp terminal design.

Selection Considerations

Interchange selection for a specific location generally requires consideration of a wide range of factors. When focusing on operation and right-of-way requirements, the interchange forms most amenable to specific combinations of facility class and area type are listed in Table 1-4.

Table 1-4. Interchange Types Amenable to Various Facility Classes and Area Types.

Intersecting Street or Highway Classification	Area Type		
	Rural	**Suburban**	**Urban**
Local	Conventional Diamond Parclo A (2-quad) Parclo B (2-quad)	Compressed Diamond	Tight Urban Diamond
Collector or Arterial	Parclo A Parclo B	Parclo A Parclo B	Single-Point Urban Diamond Tight Urban Diamond

Only the more common interchange types are listed in Table 1-4. Other interchange types or geometric variations of the types listed may be appropriate in specific situations. Directional ramps may be added to one or more interchange quadrants to serve a specific high-volume turn movement.

Interchange selection should reflect consideration of safety, operation, uniformity of exit patterns (relative to adjacent interchanges), cost, availability of right-of-way, potential for stage construction, and compatibility with the environment *(2)*. The selection of interchange type for rural areas is based primarily on traffic demand, especially turn movement demands. In urban areas, selection is based on traffic demands, interchange spacing, and right-of-way impacts. Interchanges with loop ramps can be very efficient at locations with heavy left-turn volumes; however, their right-of-way requirements can preclude them from consideration in built-up urban environments.

Procedures for evaluating the safety and operation of interchange types are described in Chapter 2. These procedures are sufficiently general that they can be used for the selection of ramp configuration and interchange type for the concept planning and preliminary design stages of the design process.

CHAPTER 2. INTERCHANGE RAMP OPERATION AND SAFETY EVALUATION PROCEDURES

OVERVIEW

This chapter documents the development of two analytic procedures for interchange evaluation. The first procedure presented is intended for evaluating alternative interchange types and ramp configurations based on their operational performance. The second procedure presented is intended for evaluating alternative ramp configurations based on safety, where safety is quantified in terms of expected crash frequency. Both procedures are envisioned to be particularly useful during the concept planning and preliminary design stages of the design process, where they can be used to identify cost-effective interchange design configurations.

This chapter is organized into two main sections. The first section describes the procedure for evaluating the operation of alternative interchange types. The second section describes the procedure for evaluating the safety of alternative ramp configurations and interchange types.

INTERCHANGE OPERATION EVALUATION PROCEDURE

This section describes a procedure for comparing alternative interchange types and ramp configurations. It compares interchange and ramp alternatives on the basis of their impact on traffic operations. The procedure is based on the models developed by Bonneson et al. (*3*). Separate models are provided for signalized and unsignalized interchange types.

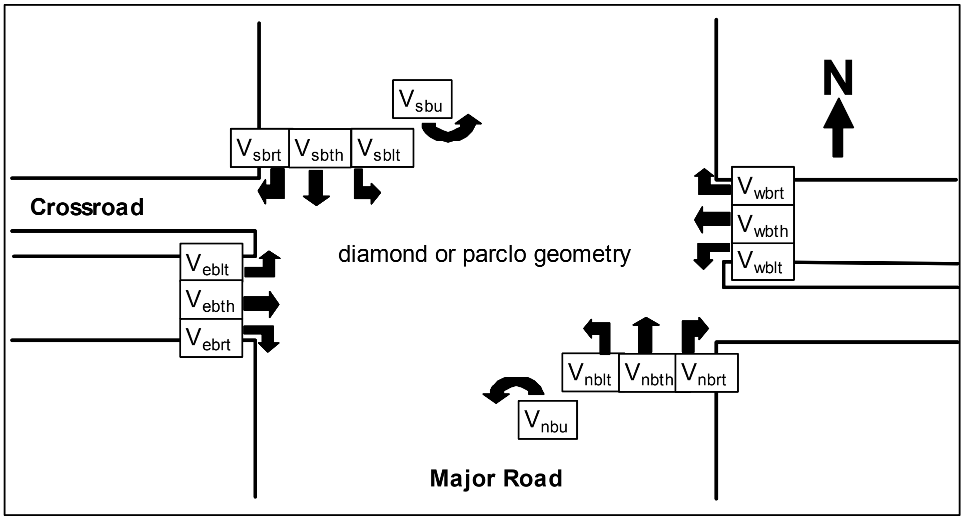
General Procedure

The procedure is suitable for the design concept planning and preliminary design stages of an interchange project. It can be used to obtain a quick estimate of the delay associated with a particular interchange type or ramp configuration. The delay estimate is sensitive to traffic volume, lane configuration, right-turn control mode, saturation flow rate, and ramp separation distances. The procedure consists of three steps. The steps are described in the following sections.

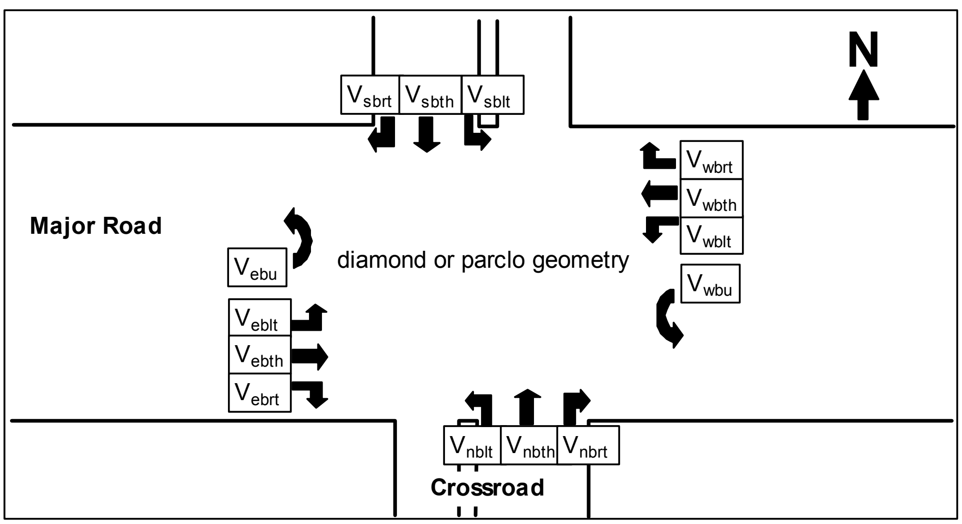
Step 1. Identify Movement Volumes and Lane Assignments

For this step, the design hourly volumes v are identified for the basic movements at the interchange. These movements are identified in Figure 2-1. Of these movements, the through movement and the U-turn movement volumes are negligible for interchanges in non-frontage-road settings. The remaining 10 movements are applicable to the analysis of interchanges without frontage roads.

For signalized interchanges, the basic movement volumes are mapped to the appropriate signal phase. Then, information about the saturation flow rate s and the number of lanes n served by the phase is identified. The saturation flow rate can be estimated as 1900 veh/h/ln for concept planning applications. For preliminary design applications, a refined estimate of this rate can be obtained using an ideal saturation flow rate of 2000 pc/h/ln with the saturation flow adjustment factors described in Chapter 16 of the *Highway Capacity Manual* (5).



a. Major Road Oriented in a North-South Direction.



b. Major Road Oriented in an East-West Direction.

Figure 2-1. Fourteen Basic Traffic Movements at an Interchange.

Step 2. Determine the Controlling Volume Ratio

During this step, the movement volume and other relevant information are used in the appropriate equations to compute the controlling volume "ratio." For signalized interchanges, this ratio is defined to be the sum-of-critical-flow-ratios. Equations for computing this ratio are described in a subsequent section titled "Signalized Interchange Evaluation Procedure."

For unsignalized interchanges, the controlling volume ratio is defined to be the maximum volume-to-capacity ratio of the exit-ramp left-turn movements (crossroad left-turn for the parclo B). Equations for computing this ratio are described in a subsequent section titled "Unsignalized Interchange Evaluation Procedure."

Step 3. Determine Interchange Delay

During this step, the controlling volume ratio is used to determine the expected level of interchange delay for the subject interchange and its corresponding level of service. For signalized interchanges, the sum-of-critical-flow-ratios from Step 2 is used with the appropriate characteristic curve to estimate interchange delay. These curves (and corresponding equations) are described in a subsequent section titled "Signalized Interchange Evaluation Procedure."

For unsignalized interchanges, the maximum volume-to-capacity ratio from Step 2 is used to estimate interchange delay. Characteristic curves (and corresponding equations) for this purpose are described in a subsequent section titled "Unsignalized Interchange Evaluation Procedure."

The interchange delay estimate can be used with Table 2-1 to determine the corresponding level of service provided by the interchange.

Table 2-1. Interchange Level-of-Service Criteria.

Level of Service	Control Delay, s/veh	
	Unsignalized Interchange	Signalized Interchange
A	≤ 10	≤ 10
B	> 10 - 15	> 10 - 20
C	> 15 - 25	> 20 - 35
D	> 25 - 35	> 35 - 55
E	> 35 - 50	> 55 - 80
F	> 50	> 80

Source: Chapter 26 of the *Highway Capacity Manual* (5).

Signalized Interchange Evaluation Procedure

This section describes procedures for evaluating interchanges that are controlled by traffic signals. The following interchange types are addressed:

- SPUI,
- compressed diamond,
- parclo A,
- parclo B, and
- TUDI,
- conventional diamond,
- parclo A (2-quad),
- parclo B (2-quad).

The procedure for evaluating these interchange types was described in a previous section. The information needed to apply this procedure is identified in this section by interchange type.

Single-Point Urban Interchange

Movement Volumes and Lane Assignments. The phase numbering scheme for the SPUI is identified in Figure 2-2. The assignment of the basic movements to each signal phase is shown in Table 2-2.

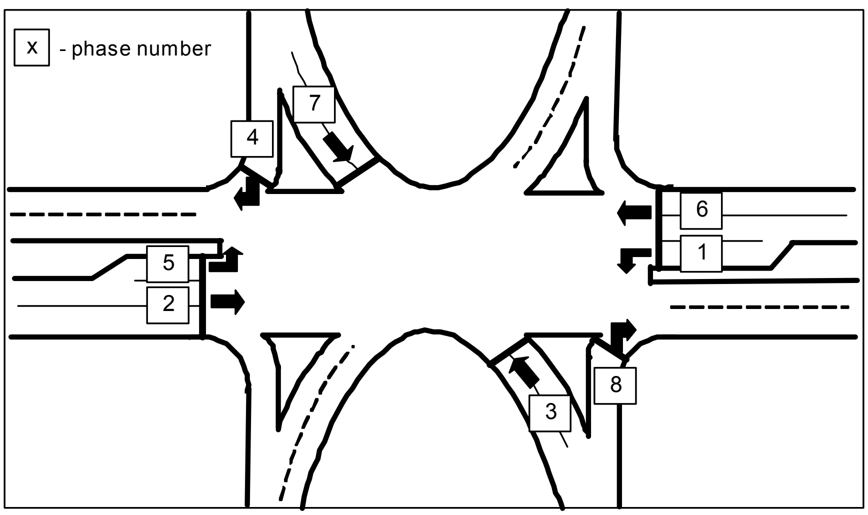


Figure 2-2. Movement and Phase Numbering Scheme for the SPUI.

Sum-of-Critical-Flow-Ratios. The sum-of-critical-flow-ratios associated with the SPUI is computed using the following equation:

$$Y_c = A + B \qquad \textbf{(1)}$$

with,

$$A = Larger\ of: \left[\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2} \ ; \ \frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6} \right] \quad \textbf{(2)}$$

$$B = Larger\ of: \left[\frac{v_3}{s_3 n_3} + \frac{v_4}{s_4 n_4} \ ; \ \frac{v_7}{s_7 n_7} + \frac{v_8}{s_8 n_8} \right] \quad \textbf{(3)}$$

where,

Y_c = sum of the critical flow ratios;
v_i = volume of movement served by phase i (i = 1, 2, ... 8), veh/h;
s_i = saturation flow rate of movement served by phase i (i = 1, 2, ... 8) (default: 1900), veh/h/ln;
n_i = number of lanes serving movement served by phase i (i = 1, 2, ... 8);
A = critical flow ratio for the crossroad movements; and
B = critical flow ratio for the major-road turn movements (i.e., exit-ramp movements).

Table 2-2. Basic Movement Volumes and Phase Numbers at the SPUI.

Major Road Orientation	Phase Number							
	1	2	3	4	5	6	7	8
	Basic Movement Volumes $v_{i,j}$ Associated with Phase [1]							
North-South	v_{wblt}	v_{ebth}	v_{nblt}	v_{sbrt}	v_{eblt}	v_{wbth}	v_{sblt}	v_{nbrt}
East-West	v_{sblt}	v_{nbth}	v_{wblt}	v_{ebrt}	v_{nblt}	v_{sbth}	v_{eblt}	v_{wbrt}

Note:
1 - $v_{i,j}$: traffic volume for direction i and movement j of the 14 basic movements shown in Figure 2-1, where i = nb, sb, eb, wb and j = lt, th, rt. nb: northbound; sb: southbound; eb: eastbound; wb: westbound; lt: left turn, th: through; rt: right turn.

The calculation of A should not include right-turn movements from the crossroad that are served by an exclusive lane. However, if the right-turn movement shares a lane with the through movement, then its volume is added to that of the through movement. The calculation of B should include right-turn movements when they are served by an exclusive phase (i.e., phases 4 or 8). If a right-turn movement is yield-controlled or provides a free-flow right-turn lane, then it should not be included in the calculation (i.e., v_4 or v_8 would equal 0.0).

Interchange Delay. For planning applications, the sum-of-critical-flow-ratios can be used with Figure 2-3 to estimate the associated interchange delay. This figure is applicable to ramp separation distances in the range of 150 to 400 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for a similar range of distances:

$$d_I = 15.1 + (0.010\,D_r + 16.9)\,\frac{Y_c}{1 - Y_c} \quad : \textit{signal-controlled right turn} \qquad \textbf{(4)}$$

$$d_I = 15.1 + (0.008\,D_r + 5.9)\,\frac{Y_c}{1 - Y_c} \quad : \textit{free or yield-controlled right turn} \qquad \textbf{(5)}$$

where,

D_r = ramp separation distance (i.e., the distance between the two ramp center lines, as measured along the crossroad), ft.

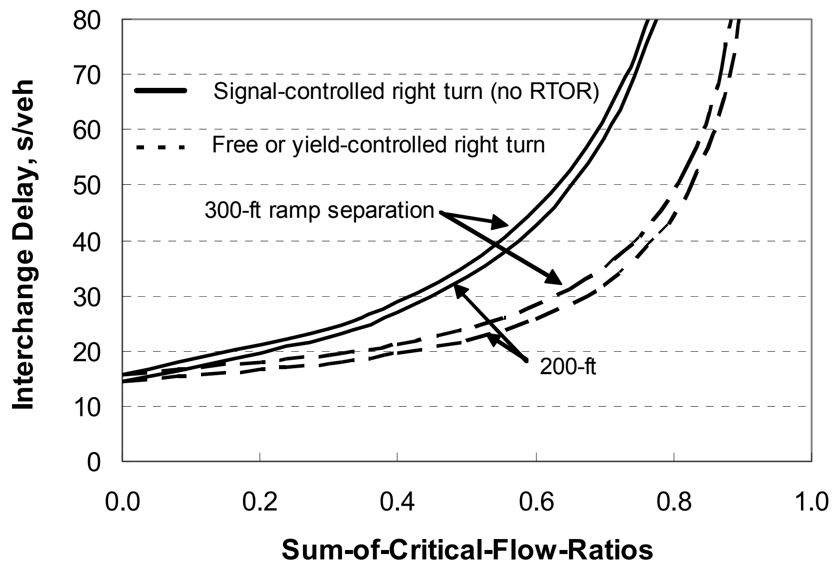


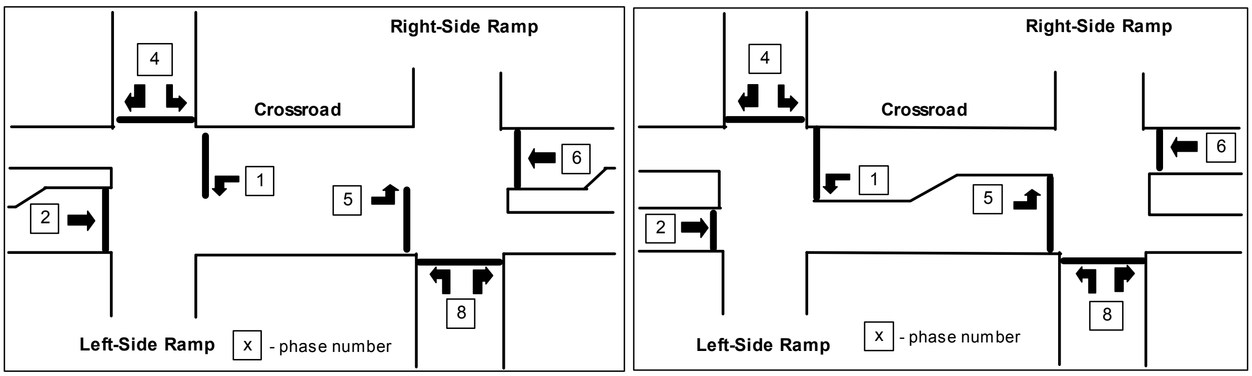
Figure 2-3. SPUI Delay Relationship.

Equation 4 represents a SPUI where right-turn-on-red (RTOR) is not allowed. If RTOR is allowed at the subject SPUI, then the delay can be estimated using both Equations 4 and 5. Specifically, this delay is estimated as a weighted average of the delay obtained from each equation where the weight assigned to Equation 5 is "P_{RTOR}" and that assigned to Equation 4 is "$1 - P_{RTOR}$." The variable "P_{RTOR}" represents the portion of exit-ramp right-turns that turn during the red indication. A logical upper limit of this variable would be equal to "$1 - g/C$" where g is the exit-ramp right-turn phase duration and C is the cycle length. Given that many exit-ramp right-turn vehicles will be served during the corresponding ramp phase, it is appropriate to reduce the upper limit.

Considering the range of other factors that influence P_{RTOR}, it is rationalized that a practical maximum value for P_{RTOR} is about 0.7 $(1-g/C)$. For typical g/C ratios, a default value of 0.50 is suggested for P_{RTOR}.

Tight Urban Diamond Interchange

Movement Volumes and Lane Assignments. The phase numbering scheme for the TUDI is identified in Figure 2-4. The assignment of the basic movements to each signal phase is shown in Table 2-3.



a. Tight Urban Diamond. **b. Compressed Diamond.**

Figure 2-4. Movement and Phase Numbering Scheme for the TUDI and Compressed Diamond.

Table 2-3. Basic Movement Volumes and Phase Numbers at the TUDI and Compressed Diamond.

Major Road Orientation	Ramp Terminal	Phase Number					
		1	2	4	5	6	8
		Basic Movement Volumes $v_{i,j}$ Associated with Phase[1, 2]					
North-South	Left	v_{wblt}	$v_{ebth} + v_{eblt}$	v_{sblt}	--	--	--
	Right	--	--	--	v_{eblt}	$v_{wbth} + v_{wblt}$	v_{nblt}
East-West	Left	v_{sblt}	$v_{nbth} + v_{nblt}$	v_{eblt}	--	--	--
	Right	--	--	--	v_{nblt}	$v_{sbth} + v_{sblt}$	v_{wblt}

Notes:
1 - "--": movement does not exist at this ramp terminal.
2 - $v_{i,j}$: traffic volume for direction i and movement j of the 14 basic movements shown in Figure 2-1, where i = nb, sb, eb, wb and j = lt, th, rt. nb: northbound; sb: southbound; eb: eastbound; wb: westbound; lt: left turn, th: through; rt: right turn.

Sum-of-Critical-Flow-Ratios. The sum-of-critical-flow-ratios associated with the TUDI can be computed using the following equation:

$$Y_c = A + B \tag{6}$$

with,

$$A = Larger\ of\!: \left[\frac{v_2}{s_2 n_2} + \frac{v_4}{s_4 n_4} - y_3 \ ; \ \frac{v_5}{s_5 n_5} + y_7 \right] \tag{7}$$

$$B = Larger\ of\!: \left[y_3 + \frac{v_1}{s_1 n_1} \ ; \ \frac{v_6}{s_6 n_6} + \frac{v_8}{s_8 n_8} - y_7 \right] \tag{8}$$

$$y_3 = Smaller\ of\!: \left[\frac{v_4}{s_4 n_4} \ ; \ y_t \right] \tag{9}$$

$$y_7 = Smaller\ of\!: \left[\frac{v_8}{s_8 n_8} \ ; \ y_t \right] \tag{10}$$

where,

y_3 = effective flow ratio for concurrent (or transition) phase 3;
y_7 = effective flow ratio for concurrent (or transition) phase 7; and
y_t = effective flow ratio for the concurrent phase when dictated by travel time.

The calculation of neither A nor B should include right-turn movements that are served by an exclusive lane. However, if the right-turn movement shares a lane with other movements (i.e., left turn or through) on the approach, then its volume is added to that of the other movement.

The number of lanes n to use in Equations 7 and 8 for phases 2 and 6, respectively, is based on the crossroad left-turn bay design at the interchange. If these left-turn bays extend back from the downstream ramp terminal through the upstream terminal, then the number of lanes available to serve phases 2 or 6 (i.e., n_2 or n_6) should equal the total number of through and left-turn lanes provided on the external approach. For example, consider a left-side ramp terminal with an external crossroad approach having two through lanes. If a single-lane left-turn bay extends back from the right-side terminal through the left ramp terminal (as illustrated in Figure 2-4), then the total number of lanes on the approach is three (= 1 + 2) and the number of lanes served by phase 2 (i.e., n_2) is three.

For concept planning applications, the value of the effective flow ratio y_t should be set to 0.05, 0.07, or 0.085 for ramp separation distances of 200, 300, or 400 ft, respectively. For preliminary design applications, the following procedure can be used to compute Y_c using Figure 2-5 to obtain a more refined estimate of y_t. First, compute the "unadjusted sum-of-critical-flow-ratios" using Equations 6, 7, and 8 with the values of y_3 and y_7 set equal to zero. Then, use this "unadjusted" sum with Figure 2-5 to obtain the effective flow ratio y_t. Next, use y_t in Equations 9 and 10 to obtain y_3 and y_7, respectively. Finally, use y_3 and y_7 in Equations 6, 7, and 8 to compute Y_c.

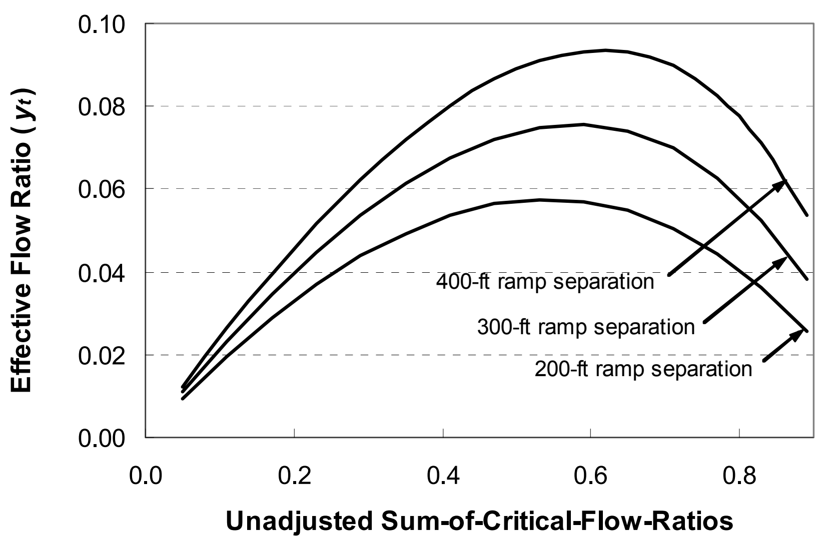


Figure 2-5. Effective Flow Ratio.

Interchange Delay. For concept planning applications, the sum-of-critical-flow-ratios can be used with Figure 2-6 to estimate the associated interchange delay. This figure is applicable to ramp separation distances in the range of 200 to 400 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for a similar range of distances:

$$d_I = 13.4 + 14.2 \frac{Y_c}{1 - Y_c} \quad : \textit{signal-controlled right turn} \qquad \textbf{(11)}$$

$$d_I = 13.4 + 12.8 \frac{Y_c}{1 - Y_c} \quad : \textit{free or yield-controlled right turn} \tag{12}$$

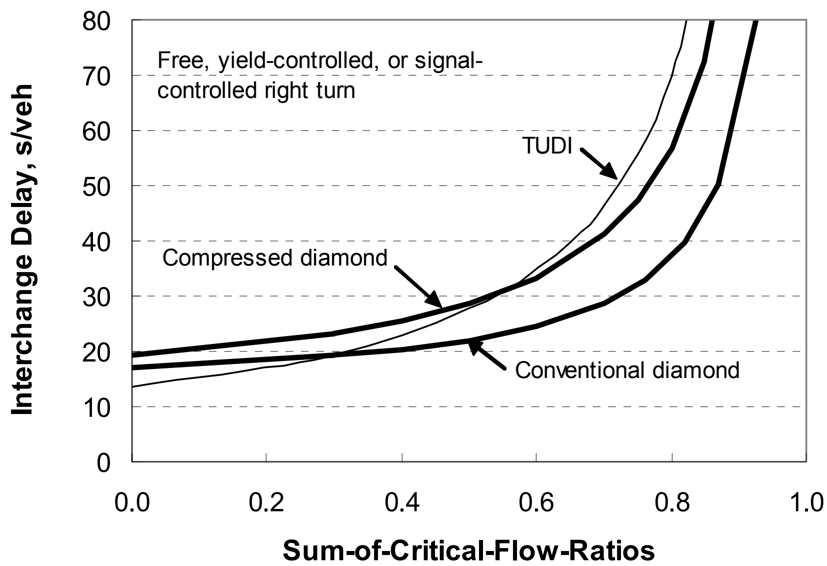


Figure 2-6. Signalized Diamond Delay Relationship.

Compressed Diamond Interchange

Movement Volumes and Lane Assignments. The phase numbering scheme for the compressed diamond is identified in Figure 2-4. The assignment of the basic movements to each signal phase is shown in Table 2-3. As indicated in Figure 2-4, storage for the crossroad left-turn movements is provided between the two ramp terminals in overlapping left-turn bays.

Sum-of-Critical-Flow-Ratios. The sum-of-critical-flow-ratios associated with the compressed diamond can be computed using the following equation:

$$Y_c = A + B \tag{13}$$

with,

$$A = \textit{Larger of:} \left[\frac{v_1}{s_1 n_1} + y_2 \;\; ; \;\; \frac{v_5}{s_5 n_5} + y_6 \right] \tag{14}$$

$$B = Larger\ of: \left[\frac{v_4}{s_4 n_4} \ ; \ \frac{v_8}{s_8 n_8} \right] \quad \textbf{(15)}$$

$$y_2 = Larger\ of: \left[\frac{v_2}{s_2 n_2} \ ; \ \frac{v_5}{s_2} \right] \quad \textbf{(16)}$$

$$y_6 = Larger\ of: \left[\frac{v_6}{s_6 n_6} \ ; \ \frac{v_1}{s_6} \right] \quad \textbf{(17)}$$

where,

y_2 = flow ratio for phase 2 with consideration of prepositioning; and

y_6 = flow ratio for phase 6 with consideration of prepositioning.

Equations 16 and 17 are intended to account for the influence of "driver prepositioning" on the crossroad approaches to each ramp terminal. In this regard, drivers intending to make a left turn at the downstream ramp terminal often position their vehicles in the left-most through lane on the approach to the upstream ramp terminal. This practice can sometimes lead to significant underutilization of the outside through lanes.

The calculation of *A* should not include right-turn movements from the crossroad that are served by an exclusive lane. However, if the right-turn movement shares a lane with the through movement, then its volume is added to that of the through movement. Similarly, the calculation of *B* should not include right-turn movements from the exit ramp that are served by an exclusive lane (i.e., a yield-controlled or a free-flow right-turn lane). However, if the right-turn movement shares a lane with the left-turn movement, then its volume is added to that of the left-turn movement.

Interchange Delay. For concept planning applications, the sum-of-critical-flow-ratios can be used with Figure 2-6 to estimate the associated interchange delay. This figure is applicable to ramp separation distances in the range of 600 to 800 ft. Alternatively, for preliminary design analyses, delay can be estimated using the following equations for a similar range of distances:

$$d_I = 19.2 + (9.4 - 0.011\,[D_r - 700])\,\frac{Y_c}{1 - Y_c} \quad : signal{-}controlled\ right\ turn \quad \textbf{(18)}$$

$$d_I = 19.2 + (8.6 - 0.009\,[D_r - 700])\,\frac{Y_c}{1 - Y_c} \quad : free\ or\ yield{-}controlled\ right \quad \textbf{(19)}$$

Movement Volumes and Lane Assignments. The phase numbering scheme for the conventional diamond is identified in Figure 2-7. The assignment of the basic movements to each signal phase is shown in Table 2-4.

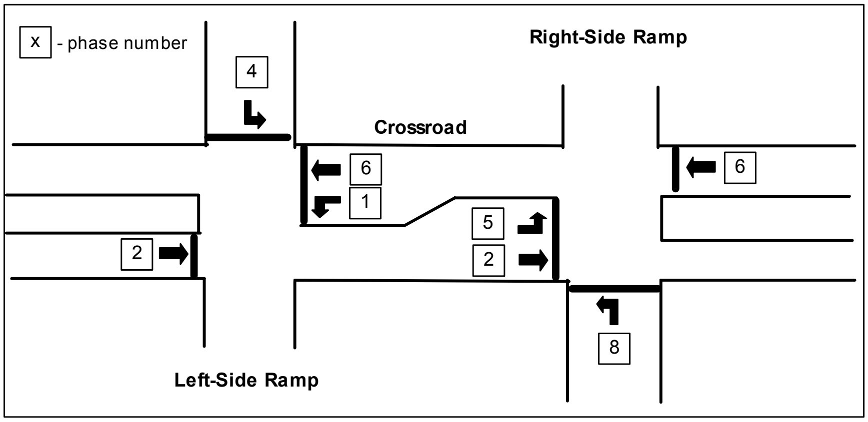


Figure 2-7. Movement and Phase Numbering Scheme for the Conventional Diamond.

Table 2-4. Basic Movement Volumes and Phase Numbers at the Conventional Diamond.

Major Road Orientation	Ramp Terminal	Phase Number					
		1	2	4	5	6	8
		Basic Movement Volumes $v_{i,j}$ Associated with Phase[1, 2]					
North-South	Left	v_{wblt}	$v_{ebth} + v_{eblt}$	v_{sblt}	--	$v_{wbth} + v_{nblt}$	--
	Right	--	$v_{ebth} + v_{sblt}$	--	v_{eblt}	$v_{wbth} + v_{wblt}$	v_{nblt}
East-West	Left	v_{sblt}	$v_{nbth} + v_{nblt}$	v_{eblt}	--	$v_{sbth} + v_{wblt}$	--
	Right	--	$v_{nbth} + v_{eblt}$	--	v_{nblt}	$v_{sbth} + v_{sblt}$	v_{wblt}

Notes:
1 - "--": movement does not exist at this ramp terminal.
2 - $v_{i,j}$: traffic volume for direction i and movement j of the 14 basic movements shown in Figure 2-1, where i = nb, sb, eb, wb and j = lt, th, rt. nb: northbound; sb: southbound; eb: eastbound; wb: westbound; lt: left turn, th: through; rt: right turn.

Sum-of-Critical-Flow-Ratios. The maximum sum-of-critical-flow-ratios associated with a conventional diamond or parclo interchange can be computed using the following equation:

$$Y_{c,max} = Larger\ of\!: \left[Y_{c,left} \ :\ Y_{c,right} \right] \tag{20}$$

with,

$$Y_c = A + B \quad \textbf{(21)}$$

$$A = Larger\ of: \left[\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2} \; ; \; \frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6} \right] \quad \textbf{(22)}$$

$$B = Larger\ of: \left[\frac{v_4}{s_4 n_4} \; ; \; \frac{v_8}{s_8 n_8} \right] \quad \textbf{(23)}$$

where,

$Y_{c,max}$ = largest sum-of-critical-flow-ratios for the two ramp terminals.

Equations 21, 22, and 23 should be applied twice, once for each ramp terminal, to obtain the left-side and right-side sum-of-critical-flow-ratios (i.e., $Y_{c,left}$ and $Y_{c,right}$, respectively). These values are then used in Equation 20 to obtain the maximum sum-of-critical-flow-ratios $Y_{c,max}$.

The basic traffic movements associated with volume variables v_i in Equations 22 and 23 are identified in Table 2-4. If any of the volume variables do not have a corresponding movement identified in this table (i.e., a "--" is used to indicate this condition), then the variable can be assumed to equal 0.0 for the purpose of calculating variables A or B.

The calculation of A should not include right-turn movements from the crossroad that are served by an exclusive lane. However, if the right-turn movement shares a lane with the through movement, then its volume is added to that of the through movement. Similarly, the calculation of B should not include right-turn movements from the exit ramp that are served by an exclusive lane (i.e., a yield-controlled or a free-flow right-turn lane). However, if the right-turn movement shares a lane with the left-turn movement, then its volume is added to that of the left-turn movement.

Interchange Delay. For concept planning applications, the maximum sum-of-critical-flow-ratios can be used with Figure 2-6 to estimate the associated interchange delay. This figure is applicable to ramp separation distances in the range of 1000 to 1200 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for ramp separation distances in the range of 900 to 1300 ft:

$$d_I = 17.1 + (5.0 - 0.011\,[D_r - 1100])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : signal\text{-}controlled\ right\ turn \quad \textbf{(24)}$$

$$d_I = 17.1 + (4.6 - 0.009\,[D_r - 1100])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{free or yield-controlled right} \qquad \textbf{(25)}$$

Parclo A Interchange

Movement Volumes and Lane Assignments. The phase numbering scheme for the parclo A is identified in Figure 2-8. The assignment of the basic movements to each signal phase is shown in Table 2-5.

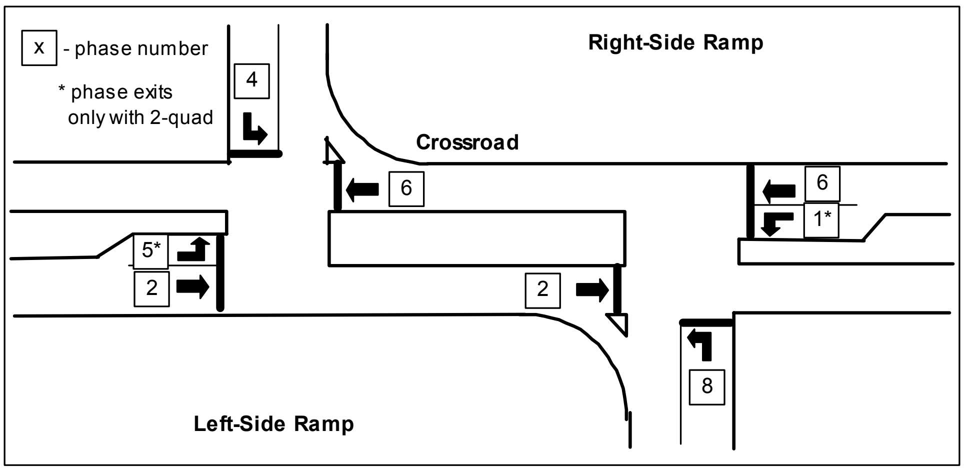


Figure 2-8. Movement and Phase Numbering Scheme for the Parclo A and Parclo A (2-Quad).

Sum-of-Critical-Flow-Ratios. The maximum sum-of-critical-flow-ratios associated with a parclo A interchange can be computed using the equations provided for the conventional diamond.

The basic traffic movements associated with volume variables v_i in Equations 22 and 23 are identified in Table 2-5. If any of the volume variables do not have a corresponding movement identified in this table (i.e., a "--" is used to indicate this condition), then the variable can be assumed to equal 0.0 for the purpose of calculating variables A or B.

The calculation of A should not include right-turn movements from the crossroad that are served by an exclusive lane. However, if the right-turn movement shares a lane with the through movement at the ramp terminal, then its volume is added to that of the through movement. Similarly, the calculation of B should not include right-turn movements from the exit ramp that are served by an exclusive lane (i.e., a yield-controlled or a free-flow right-turn lane). However, if the right-turn movement shares a lane with the left-turn movement at the ramp terminal, then its volume is added to that of the left-turn movement.

Table 2-5. Basic Movement Volumes and Phase Numbers at the Parclo A and Parclo A (2-Quad).

Major Road Orientation	Interchange Type	Ramp Terminal	Phase Number 1	2	4	5	6	8
			Basic Movement Volumes $v_{i,j}$ Associated with Phase [1,2]					
North-South	Parclo A	Left	--	$v_{ebth} + v_{eblt}$	v_{sblt}	--	$v_{wbth} + v_{nblt}$	--
		Right	--	$v_{ebth} + v_{sblt}$	--	--	$v_{wbth} + v_{wblt}$	v_{nblt}
	Parclo A (2-quad)	Left	--	$v_{ebth} + v_{eblt}$	v_{sblt}	v_{ebrt}	$v_{wbth} + v_{nblt}$	--
		Right	v_{wbrt}	$v_{ebth} + v_{sblt}$	--	--	$v_{wbth} + v_{wblt}$	v_{nblt}
East-West	Parclo A	Left	--	$v_{nbth} + v_{nblt}$	v_{eblt}	--	$v_{sbth} + v_{wblt}$	--
		Right	--	$v_{nbth} + v_{eblt}$	--	--	$v_{sbth} + v_{sblt}$	v_{wblt}
	Parclo A (2-quad)	Left	--	$v_{nbth} + v_{nblt}$	v_{eblt}	v_{nbrt}	$v_{sbth} + v_{wblt}$	--
		Right	v_{sbrt}	$v_{nbth} + v_{eblt}$	--	--	$v_{sbth} + v_{sblt}$	v_{wblt}

Notes:
1 - "--": movement does not exist at this ramp terminal.
2 - $v_{i,j}$: traffic volume for direction i and movement j of the 14 basic movements shown in Figure 2-1, where i = nb, sb, eb, wb and j = lt, th, rt. nb: northbound; sb: southbound; eb: eastbound; wb: westbound; lt: left turn, th: through; rt: right turn.

Interchange Delay. For concept planning applications, the maximum sum-of-critical-flow-ratios can be used with Figure 2-9 to estimate the associated interchange delay. This figure is applicable to ramp separation distances in the range of 700 to 900 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for ramp separation distances in the range of 700 to 1000 ft:

$$d_I = 11.7 + (7.8 - 0.011\,[X_r - 800])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{signal-controlled right turn} \qquad \textbf{(26)}$$

$$d_I = 11.7 + (6.6 - 0.009\,[X_r - 800])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{free or yield-controlled right} \qquad \textbf{(27)}$$

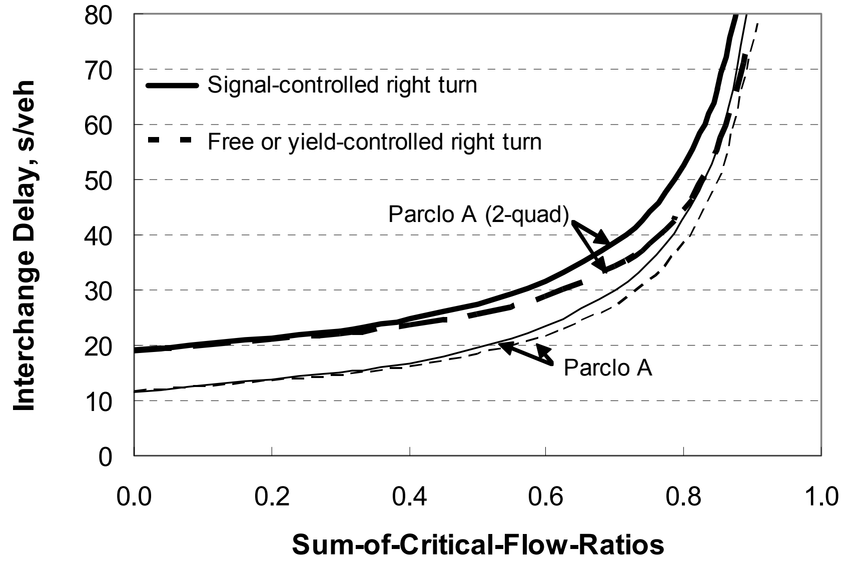


Figure 2-9. Signalized Parclo A and Parclo A (2-Quad) Delay Relationship.

Parclo A (2-Quad) Interchange

Movement Volumes and Lane Assignments. The phase numbering scheme for the parclo A (2-quad) is identified in Figure 2-8. The assignment of the basic movements to each signal phase is shown in Table 2-5.

Sum-of-Critical-Flow-Ratios. The maximum sum-of-critical-flow-ratios associated with a parclo A (2-quad) can be computed using the equations provided for the conventional diamond.

The basic traffic movements associated with volume variables v_i in Equations 22 and 23 are identified in Table 2-5. If any of the volume variables do not have a corresponding movement identified in this table (i.e., a "--" is used to indicate this condition), then the variable can be assumed to equal 0.0 for the purpose of calculating variables *A* or *B*.

With regard to phase 2 for the right-side ramp terminal and phase 6 for the left-side ramp terminal, the calculation of *A* should not include entrance-ramp right-turn movements that are served by an exclusive lane. However, if the right-turn movement shares a lane with the through movement at the ramp terminal, then its volume is added to that of the through movement.

With regard to the exit ramps, the calculation of *B* should not include right-turn movements that are served by an exclusive lane (i.e., a yield-controlled or a free-flow right-turn lane). However, if the right-turn movement shares a lane with the left-turn movement at the ramp terminal, then its volume is added to that of the left-turn movement.

Interchange Delay. For concept planning applications, the maximum sum-of-critical-flow-ratios can be used with Figure 2-9 to estimate the associated interchange delay. This figure is applicable to ramp separation distances in the range of 700 to 900 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for ramp separation distances in the range of 700 to 1000 ft:

$$d_I = 19.1 + (8.3 - 0.011\,[D_r - 800])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{signal-controlled right turn} \tag{28}$$

$$d_I = 19.1 + (6.3 - 0.009\,[D_r - 800])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{free or yield-controlled right turn} \tag{29}$$

Parclo B Interchange

Movement Volumes and Lane Assignments. The phase numbering scheme for the parclo B is identified in Figure 2-10. The assignment of the basic movements to each signal phase is shown in Table 2-6.

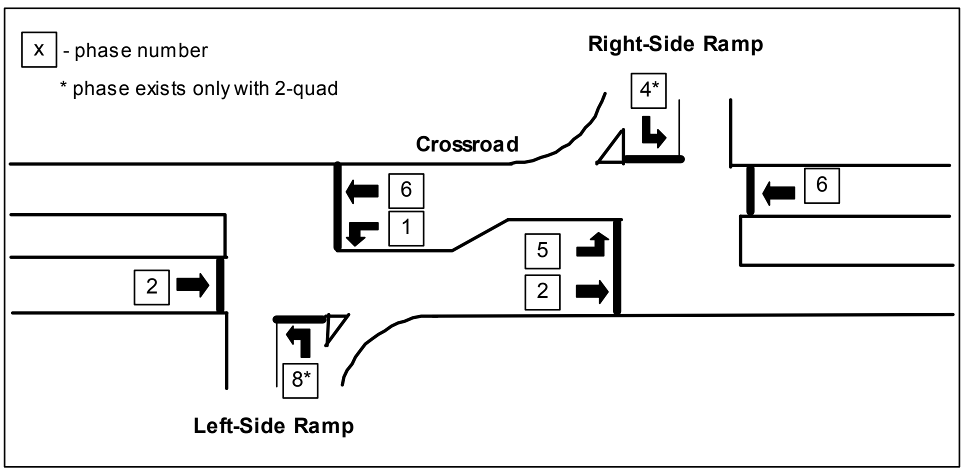


Figure 2-10. Movement and Phase Numbering Scheme for the Parclo B and Parclo B (2-Quad).

Sum-of-Critical-Flow-Ratios. The maximum sum-of-critical-flow-ratios associated with a parclo B interchange can be computed using the equations provided for the conventional diamond.

The basic traffic movements associated with volume variables v_i in Equations 22 and 23 are identified in Table 2-6. If any of the volume variables do not have a corresponding movement identified in this table (i.e., a "--" is used to indicate this condition), then the variable can be assumed to equal 0.0 for the purpose of calculating variables A or B.

Table 2-6. Basic Movement Volumes and Phase Numbers at the Parclo B and Parclo B (2-Quad).

Major Road Orientation	Interchange Type	Ramp Terminal	Phase Number					
			1	2	4	5	6	8
			Basic Movement Volumes $v_{i,j}$ Associated with Phase[1, 2]					
North-South	Parclo B	Left	v_{wblt}	$v_{ebth} + v_{eblt}$	--	--	$v_{wbth} + v_{nblt}$	--
		Right	--	$v_{ebth} + v_{sblt}$	--	v_{eblt}	$v_{wbth} + v_{wblt}$	--
	Parclo B (2-quad)	Left	v_{wblt}	$v_{ebth} + v_{eblt}$	--	--	$v_{wbth} + v_{nblt}$	v_{sbrt}
		Right	--	$v_{ebth} + v_{sblt}$	v_{nbrt}	v_{eblt}	$v_{wbth} + v_{wblt}$	--
East-West	Parclo B	Left	v_{sblt}	$v_{nbth} + v_{nblt}$	--	--	$v_{sbth} + v_{wblt}$	--
		Right	--	$v_{nbth} + v_{eblt}$	--	v_{nblt}	$v_{sbth} + v_{sblt}$	--
	Parclo B (2-quad)	Left	v_{sblt}	$v_{nbth} + v_{nblt}$	--	--	$v_{sbth} + v_{wblt}$	v_{ebrt}
		Right	--	$v_{nbth} + v_{eblt}$	v_{wbrt}	v_{nblt}	$v_{sbth} + v_{sblt}$	--

Notes:
1 - "--": movement does not exist at this ramp terminal.
2 - $v_{i,j}$: traffic volume for direction i and movement j of the 14 basic movements shown in Figure 2-1, where i = nb, sb, eb, wb and j = lt, th, rt. nb: northbound; sb: southbound; eb: eastbound; wb: westbound; lt: left turn, th: through; rt: right turn.

The calculation of A should not include right-turn movements from the crossroad that are served by an exclusive lane. However, if the right-turn movement shares a lane with the through movement at the ramp terminal, then its volume is added to that of the through movement.

Interchange Delay. For concept planning applications, the maximum sum-of-critical-flow-ratios can be used with Figure 2-11 to estimate the associated interchange delay. This figure is applicable to ramp separation distances in the range of 1100 to 1300 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for ramp separation distances in the range of 1000 to 1400 ft:

$$d_I = 9.3 + (3.5 - 0.011\,[D_r - 1200])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{signal-controlled right turn} \qquad \textbf{(30)}$$

$$d_I = 9.3 + (3.4 - 0.009\,[D_r - 1200])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{free or yield-controlled right} \qquad \textbf{(31)}$$

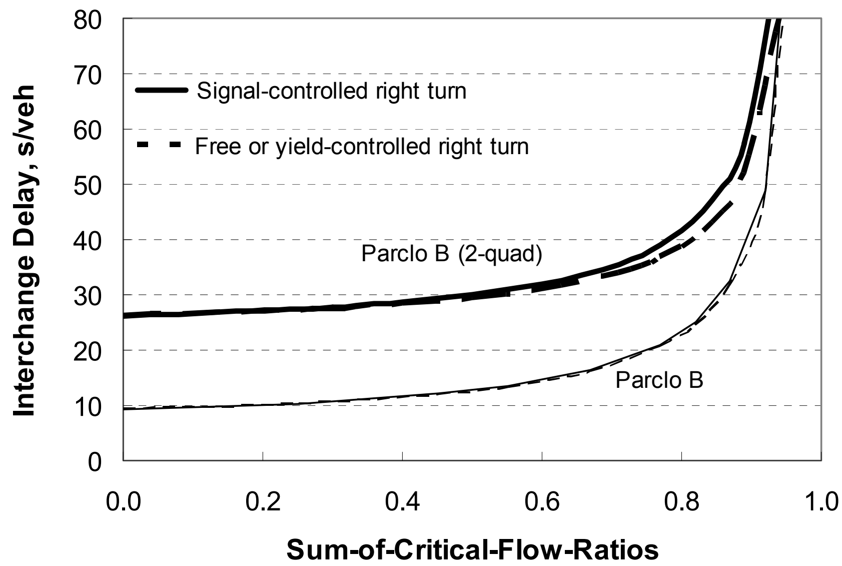


Figure 2-11. Signalized Parclo B and Parclo B (2-Quad) Delay Relationship.

Parclo B (2-Quad) Interchange

Movement Volumes and Lane Assignments. The phase numbering scheme for the parclo B (2-quad) is identified in Figure 2-10. The assignment of the basic movements to each signal phase is shown in Table 2-6.

Sum-of-Critical-Flow-Ratios. The maximum sum-of-critical-flow-ratios associated with a parclo B (2-quad) interchange can be computed using the equations provided for the conventional diamond.

The basic traffic movements associated with volume variables v_i in Equations 22 and 23 are identified in Table 2-6. If any of the volume variables do not have a corresponding movement identified in this table (i.e., a "--" is used to indicate this condition), then the variable can be assumed to equal 0.0 for the purpose of calculating variables A or B.

The calculation of A should not include right-turn movements from the crossroad that are served by an exclusive lane. However, if the right-turn movement shares a lane with the through movement at the ramp terminal, then its volume is added to that of the through movement. Similarly, the calculation of B should not include right-turn movements from the exit ramp that are served by an exclusive lane (i.e., a yield-controlled or a free-flow right-turn lane). However, if the right-turn movement shares a lane with the left-turn movement at the ramp terminal, then its volume is added to that of the left-turn movement.

Interchange Delay. For concept planning applications, the maximum sum-of-critical-flow-ratios can be used with Figure 2-11 to estimate the associated interchange delay. This figure is applicable to ramp separation distances in the range of 1100 to 1300 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for ramp separation distances in the range of 1000 to 1400 ft:

$$d_I = 26.2 + (3.9 - 0.011\,[D_r - 1200])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{signal-controlled right turn} \qquad \textbf{(32)}$$

$$d_I = 26.2 + (3.2 - 0.009\,[D_r - 1200])\,\frac{Y_{c,max}}{1 - Y_{c,max}} \quad : \textit{free or yield-controlled right} \qquad \textbf{(33)}$$

Unsignalized Interchange Evaluation Procedure

This section describes procedures for comparing interchanges that are controlled by stop control on the exit ramps. The following interchange types are addressed:

- TUDI,
- conventional diamond,
- parclo A (2-quad),
- parclo B (2-quad).
- compressed diamond,
- parclo A,
- parclo B, and

As noted in a preceding section, the procedure for evaluating alternative interchange types consists of three steps. The information needed for each of these steps is identified in this section. Unlike the sequence of presentation used for signalized interchanges, a general procedure is described for computing the maximum volume-to-capacity ratio for any interchange type. Then, delay equations are presented separately for each interchange type.

Basic Movement Volumes

The basic movements at a diamond or parclo interchange are identified in Figure 2-1. This figure should be consulted to identify the 10 basic movements at the interchange being evaluated.

Volume-to-Capacity Ratios

With one exception, the larger volume-to-capacity ratio of the two ramp left-turn traffic movements is required to predict delay at an unsignalized intersection. The only exception is the parclo B. Delay prediction at this interchange is based on the larger volume-to-capacity ratio of the two crossroad left-turn movements. The following equations can be used to predict the maximum volume-to-capacity ratio for the ramp and crossroad left-turn movements:

$$X_{max} = Larger\ of: \left[X_{c,left} \ : \ X_{c,right}\right] \ : parclo\ B \quad \textbf{(34)}$$

$$X_{max} = Larger\ of: \left[X_{r,left} \ : \ X_{r,right}\right] \ : other\ interchanges \quad \textbf{(35)}$$

with,

$$X_{r,left} = \frac{v_{r,left}}{1000 - 0.55\,v_{o,r,left}} \times \frac{1}{1 - X_{c,left}} \quad \textbf{(36)}$$

$$X_{r,right} = \frac{v_{r,right}}{1000 - 0.55\,v_{o,r,right}} \times \frac{1}{1 - X_{c,right}} \quad \textbf{(37)}$$

$$X_{c,left} = \frac{v_{c,left}}{1600 - 0.55\,v_{o,c,left}} \quad \textbf{(38)}$$

$$X_{c,right} = \frac{v_{c,right}}{1600 - 0.55\,v_{o,c,right}} \quad \textbf{(39)}$$

where,

$X_{r,\,max}$ = larger of the two exit-ramp volume-to-capacity ratios ($X_{r,\,left}$, $X_{r,\,right}$);
$X_{c,\,max}$ = larger of the two crossroad volume-to-capacity ratios ($X_{c,\,left}$, $X_{c,\,right}$);
$X_{r,\,left}$ = exit-ramp left-turn volume-to-capacity ratio for left-side ramp terminal;
$X_{r,\,right}$ = exit-ramp left-turn volume-to-capacity ratio for right-side ramp terminal;
$X_{c,\,left}$ = crossroad left-turn volume-to-capacity ratio for left-side ramp terminal;
$X_{c,\,right}$ = crossroad left-turn volume-to-capacity ratio for right-side ramp terminal;
$v_{r,\,left}$ = subject exit-ramp left-turn volume for left-side ramp terminal (see Table 2-7), veh/h;
$v_{r,\,right}$ = subject exit-ramp left-turn volume for right-side ramp terminal (see Table 2-7), veh/h;
$v_{c,\,left}$ = subject crossroad left-turn volume for left-side ramp terminal (see Table 2-7), veh/h;
$v_{c,\,right}$ = subject crossroad left-turn volume for right-side ramp terminal (see Table 2-7), veh/h;
$v_{o,\,r,\,left}$ = volume opposing $v_{r,\,left}$ (see Table 2-7), veh/h;
$v_{o,\,r,\,right}$ = volume opposing $v_{r,\,right}$ (see Table 2-7), veh/h;
$v_{o,\,c,\,left}$ = volume opposing $v_{c,\,left}$ (see Table 2-7), veh/h; and
$v_{o,\,c,\,right}$ = volume opposing $v_{c,\,right}$ (see Table 2-7), veh/h.

The volume variables referenced in Equations 36 through 39 represent both the left-turn volumes at the interchange and the volumes that oppose, or conflict with, these left-turn movements. Table 2-7 identifies the basic movement volumes that should be used to obtain the subject left-turn volume and its associated conflicting volume for each of the four left-turn movements.

Table 2-7. Ramp and Crossroad Subject Left-Turn and Opposing Volumes.

Major Road Orientation[3]	Interchange Type	Ramp Terminal	Basic Movement[1,2]: Crossroad Left-Turn Volumes: Subject[4] $v_{c,k}$	Basic Movement[1,2]: Crossroad Left-Turn Volumes: Opposing[4] $v_{o,c,k}$	Basic Movement[1,2]: Ramp Left-Turn Volumes: Subject[4] $v_{r,k}$	Basic Movement[1,2]: Ramp Left-Turn Volumes: Opposing[4] $v_{o,r,k}$
N-S	Diamond[5]	Left	v_{wblt}	$v_{eblt} + v_{ebth} + 2 \times v^*_{ebrt}$	v_{sblt}	$v_{wblt} + v_{wbth} + v_{nblt} + v_{eblt} + v_{ebth}$
		Right	v_{eblt}	$v_{wblt} + v_{wbth} + 2 \times v^*_{wbrt}$	v_{nblt}	$v_{eblt} + v_{ebth} + v_{sblt} + v_{wblt} + v_{wbth}$
	Parclo A	Left	--	--	v_{sblt}	$v_{wblt} + v_{wbth} + v_{nblt} + v_{eblt} + v_{ebth}$
		Right	--	--	v_{nblt}	$v_{eblt} + v_{ebth} + v_{sblt} + v_{wblt} + v_{wbth}$
	Parclo A (2-quad)	Left	v_{ebrt}	$2 \times v_{wblt} + v_{wbth} + v_{nblt}$	v_{sblt}	$v_{wblt} + v_{wbth} + v_{nblt} + v_{eblt} + v_{ebth} + v_{ebrt}$
		Right	v_{wbrt}	$2 \times v_{eblt} + v_{ebth} + v_{sblt}$	v_{nblt}	$v_{eblt} + v_{ebth} + v_{sblt} + v_{wblt} + v_{wbth} + v_{wbrt}$
	Parclo B	Left	v_{wblt}	$v_{eblt} + v_{ebth} + 2 \times v^*_{ebrt}$	--	--
		Right	v_{eblt}	$v_{wblt} + v_{wbth} + 2 \times v^*_{wbrt}$	--	--
	Parclo B (2-quad)	Left	v_{wblt}	$v_{eblt} + v_{ebth} + 2 \times v^*_{ebrt}$	v_{sbrt}	$v_{wblt} + v_{wbth} + v_{nblt} + v_{eblt} + v_{ebth} + v^*_{ebrt}$
		Right	v_{eblt}	$v_{wblt} + v_{wbth} + 2 \times v^*_{wbrt}$	v_{nbrt}	$v_{eblt} + v_{ebth} + v_{sblt} + v_{wblt} + v_{wbth} + v^*_{wbrt}$
E-W	Diamond[5]	Left	v_{sblt}	$v_{nblt} + v_{nbth} + 2 \times v^*_{nbrt}$	v_{eblt}	$v_{sblt} + v_{sbth} + v_{wblt} + v_{nblt} + v_{nbth}$
		Right	v_{nblt}	$v_{sblt} + v_{sbth} + 2 \times v^*_{sbrt}$	v_{wblt}	$v_{nblt} + v_{nbth} + v_{eblt} + v_{sblt} + v_{sbth}$
	Parclo A	Left	--	--	v_{eblt}	$v_{sblt} + v_{sbth} + v_{wblt} + v_{nblt} + v_{nbth}$
		Right	--	--	v_{wblt}	$v_{nblt} + v_{nbth} + v_{eblt} + v_{sblt} + v_{sbth}$
	Parclo A (2-quad)	Left	v_{nbrt}	$2 \times v_{sblt} + v_{sbth} + v_{wblt}$	v_{eblt}	$v_{sblt} + v_{sbth} + v_{wblt} + v_{nblt} + v_{nbth} + v_{nbrt}$
		Right	v_{sbrt}	$2 \times v_{nblt} + v_{nbth} + v_{eblt}$	v_{wblt}	$v_{nblt} + v_{nbth} + v_{eblt} + v_{sblt} + v_{sbth} + v_{sbrt}$
	Parclo B	Left	v_{sblt}	$v_{nblt} + v_{nbth} + 2 \times v^*_{nbrt}$	--	--
		Right	v_{nblt}	$v_{sblt} + v_{sbth} + 2 \times v^*_{sbrt}$	--	--
	Parclo B (2-quad)	Left	v_{sblt}	$v_{nblt} + v_{nbth} + 2 \times v^*_{nbrt}$	v_{ebrt}	$v_{sblt} + v_{sbth} + v_{wblt} + v_{nblt} + v_{nbth} + v^*_{nbrt}$
		Right	v_{nblt}	$v_{sblt} + v_{sbth} + 2 \times v^*_{sbrt}$	v_{wbrt}	$v_{nblt} + v_{nbth} + v_{eblt} + v_{sblt} + v_{sbth} + v^*_{sbrt}$

Notes:

1 - "--": movement does not exist at this ramp terminal.

2 - $v_{i,j}$: cell volumes represent direction i and movement j of the 14 basic movements shown in Figure 2-1, where i = nb, sb, eb, wb and j = lt, th, rt. nb: northbound; sb: southbound; eb: eastbound; wb: westbound; lt: left turn, th: through; rt: right turn.

3 - Major road travel direction. E-W: east and west; N-S: north and south.

4 - $v_{c,k}$: subject crossroad left-turn volume on side k, where k = left, right. $v_{r,k}$: subject ramp left-turn volume on side k. $v_{o,c,k}$: volumes opposing $v_{c,k}$. $v_{o,r,k}$: volumes opposing $v_{r,k}$. Right-turn volume terms denoted by an asterisk (*) should be omitted when right turns are free or yield-controlled.

5 - Includes all diamond interchange configurations (i.e., TUDI, compressed diamond, and conventional diamond).

If the denominator in Equations 36 through 39 is computed as a negative value, then the corresponding volume-to-capacity ratio should be set to 0.95. Moreover, if Equations 38 or 39 yield a value in excess of 0.95, then this value should be set to 0.95.

Interchange Delay

Diamond Interchange. The maximum ramp volume-to-capacity ratio can be used with Figure 2-12 to estimate the associated interchange delay for concept planning applications. This figure is applicable to ramp separation distances in the range of 300 to 1100 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for similar separation distances:

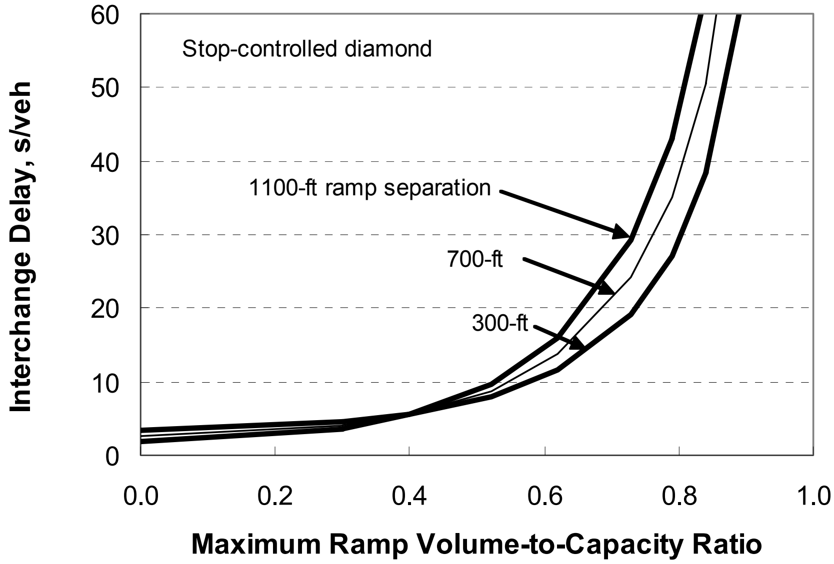
$$d_I = (4.1 - 0.002 D_r) + (5.7 + 0.0074 D_r) \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{stop-controlled right turn} \qquad \textbf{(40)}$$

$$d_I = (4.1 - 0.002 D_r) + (2.9 + 0.0046 D_r) \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{free or yield-controlled right} \qquad \textbf{(41)}$$

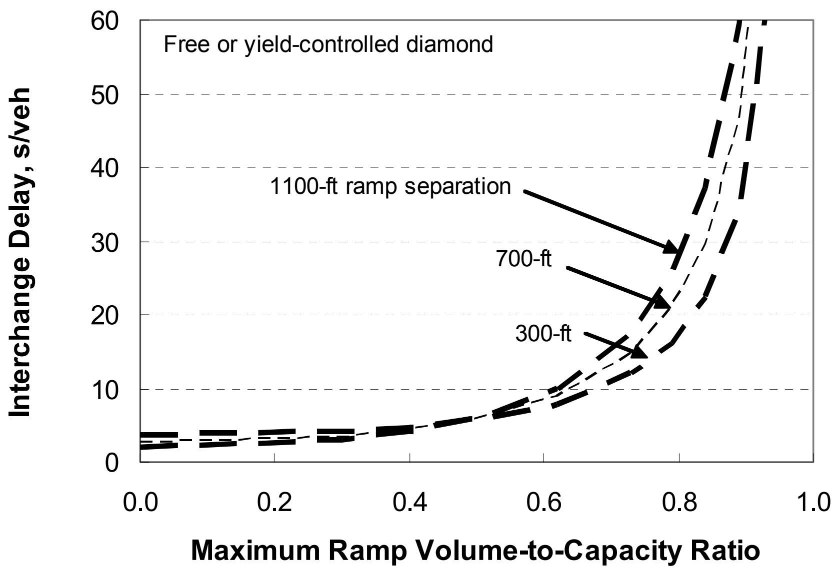
Parclo A Interchange. The maximum ramp volume-to-capacity ratio can be used with Figure 2-13 to estimate the associated interchange delay for concept planning applications. This figure is applicable to ramp separation distances in the range of 700 to 1000 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for similar separation distances:

$$d_I = 7.5 + 2.6 \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{stop-controlled right turn} \qquad \textbf{(42)}$$

$$d_I = 7.5 + 2.5 \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{free or yield-controlled right turn} \qquad \textbf{(43)}$$



a. Stop-Controlled Diamond.



b. Free or Yield-Controlled Diamond.

Figure 2-12. Unsignalized Diamond Delay Relationship.

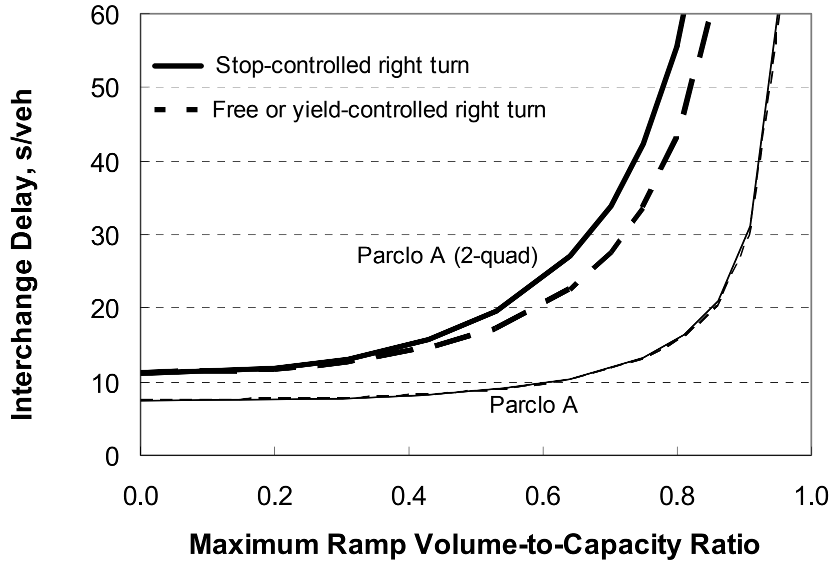


Figure 2-13. Unsignalized Parclo A and Parclo A (2-Quad) Delay Relationship.

Parclo A (2-Quad) Interchange. The maximum ramp volume-to-capacity ratio can be used with Figure 2-13 to estimate the associated interchange delay for concept planning applications. This figure is applicable to ramp separation distances in the range of 700 to 1000 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for similar separation distances:

$$d_I = 11.2 + 13.9 \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{stop-controlled right turn} \tag{44}$$

$$d_I = 11.2 + 10.0 \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{free or yield-controlled right turn} \tag{45}$$

Parclo B Interchange. The maximum *crossroad left-turn* volume-to-capacity ratio can be used with Figure 2-14 to estimate the associated interchange delay for concept planning applications. This figure is applicable to ramp separation distances in the range of 1000 to 1400 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for similar separation distances:

$$d_I = 7.1 + 17.6 \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{stop-controlled right turn} \tag{46}$$

$$d_I = 7.1 + 14.7 \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{free or yield-controlled right turn} \tag{47}$$

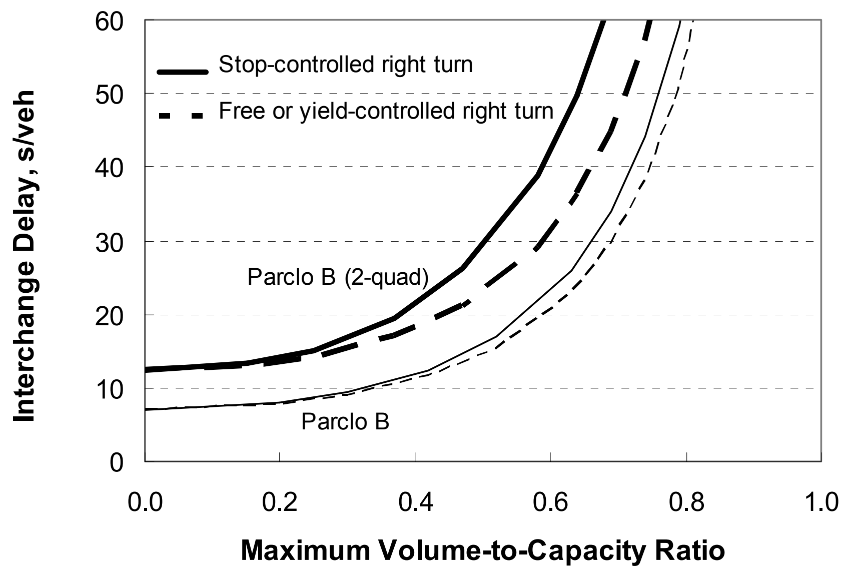


Figure 2-14. Unsignalized Parclo B and Parclo B (2-Quad) Delay Relationship.

Parclo B (2-Quad) Interchange. The maximum ramp volume-to-capacity ratio can be used with Figure 2-14 to estimate the associated interchange delay for concept planning applications. This figure is applicable to ramp separation distances in the range of 1000 to 1400 ft. Alternatively, for preliminary design analyses, interchange delay can be estimated using the following equations for similar separation distances:

$$d_I = 12.4 + 32.9 \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{stop-controlled right turn} \tag{48}$$

$$d_I = 12.4 + 21.0 \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{free or yield-controlled right turn} \tag{49}$$

RAMP SAFETY EVALUATION PROCEDURE

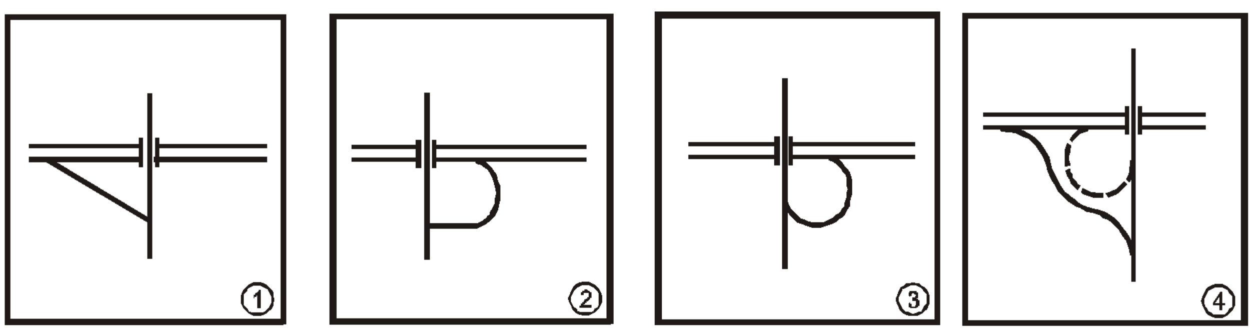
This section describes a procedure for comparing alternative interchange types and ramp configurations in terms of their expected crash frequency. It can be used to estimate the expected annual crash frequency for an individual ramp. These estimates can then be aggregated to obtain an estimate for the entire interchange. The procedure is based on the models developed by Bonneson et al. (*4*). Crashes that occur in the vicinity of the speed-change lanes (i.e., gore area) and those that occur within the ramp terminal conflict area are not considered in this procedure.

General Procedure

The procedure described herein is suitable for the concept planning and preliminary design stages of an interchange project. It can be used to obtain a quick estimate of the expected crash frequency associated with a particular interchange type or ramp configuration for specified ramp annual average daily traffic volumes (AADTs). Four ramp configurations are addressed by this procedure; they are:

- diagonal,
- non-free-flow loop,
- free-flow loop, and
- outer connection.

Exit ramp variations of each configuration are illustrated in Figure 2-15. Entrance ramp versions have a similar alignment.



a. Diagonal. **b. Non-Free-Flow Loop.** **c. Free-Flow Loop.** **d. Outer Connection.**

Figure 2-15. Basic Ramp Configurations at Non-Frontage-Road Interchanges.

The ramp safety evaluation procedure consists of four steps. These steps are described in the following sections.

Step 1. Identify Area Type and Movement AADTs

Area type and interchange turn movement AADTs should be identified in this step. Area type is classified as "urban" or "rural." It is based on the land use surrounding the interchange. Urban land uses are typically characterized as "developed" with access to this development provided on the cross street in the immediate vicinity of a ramp terminal. The development can be residential, commercial, industrial, or business. In urban areas, interchange spacing along the major road is typically less than 2.0 miles. Interchanges in areas that are not consistent with these urban characteristics are considered to be rural interchanges.

The design-year AADT for each interchange turn movement is needed for the analysis. The turn movements of interest are the left-turn and right-turn onto and off of each ramp. This distinction is not essential for some ramp configurations (e.g., diagonal); however, it is needed to evaluate other configurations (e.g., free-flow loop) that serve only one turn movement at the interchange. Whenever possible, design-year turn movement AADTs should be obtained from the planning division. However, if such estimates are unavailable, then the technique described in the section titled "Interchange Turn Movement Estimation Technique" can be used to estimate them.

Step 2. Identify Candidate Ramp Configurations for Each Interchange Quadrant

For this step, the ramp configuration for each quadrant should be identified and matched to the appropriate safety prediction model. The configurations for which a model exists are shown in Figure 2-15.

The location of a ramp's major-road speed-change lane (i.e., gore area) defines the quadrant in which the ramp is located. This definition is intuitive for diagonal and outer connection ramps because their entire length is located in the same quadrant. However, it is not as intuitive in the case of loop ramps. For these ramps, good design practice is to locate the major-road speed-change lane upstream of the interchange for exit ramps and downstream of the interchange for entrance ramps. When this practice is followed, the "loop" portion of a loop ramp is not located in the same quadrant as its speed-change lane.

In most cases, the specification of a ramp configuration for a specific quadrant directly follows from the interchange type being considered (i.e., diamond, parclo A, parclo A [2-quad], parclo B, parclo B [2-quad]). For example, the diagonal exit ramp is typically used for the diamond, parclo A, and parclo A (2-quad) interchange. Similarly, the diagonal entrance ramp is typically used for the diamond, parclo B, and parclo B (2-quad). A non-free-flow loop is used for the parclo B (2-quad) exit ramp and for the parclo A (2-quad) entrance ramp. These ramp configurations serve both a left-turn and a right-turn volume (as identified in Step 1). These two volumes would be summed to estimate the ramp AADT.

For the parclo A entrance ramp and parclo B exit ramp, the outer connection ramp is typically used in combination with a free-flow loop ramp. The outer connection serves the right-turn movement while the free-flow loop serves a left-turn movement (in the context of the change in travel direction made by traveling through the interchange). In both cases, the ramp AADT is the same as the AADT of the one turn movement that it serves. This AADT would be used in the appropriate safety prediction model to estimate ramp crash frequency.

Occasionally, the diamond, parclo A, and parclo A (2-quad) interchange may have a more generous ramp design that resembles the combination of a diagonal ramp and an outer connection. This "combined" ramp design is shown in Figure 2-16. The ramp shown illustrates the geometry of an exit ramp; however, the combined ramp design is also applicable to entrance ramps. A technique for estimating the safety of this ramp configuration is described in the section titled "Combined Ramp Configuration."

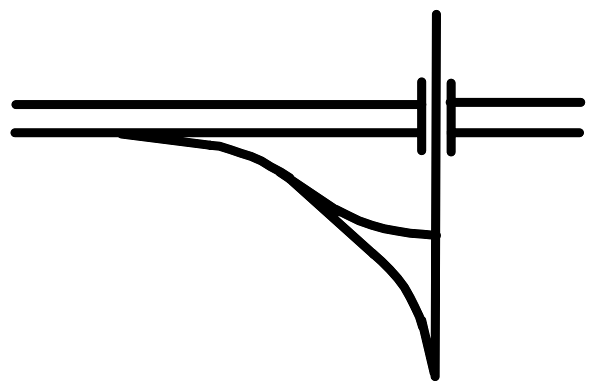


Figure 2-16. Combined Diagonal and Outer Connection Ramp Design.

Step 3. Estimate Annual Crash Frequency for Each Ramp Alternative

For this step, the annual crash frequency is estimated for each ramp alternative considered. The frequency of "all" crashes (i.e., property-damage-only, injury, or fatal) as well as the frequency of severe crashes (i.e., injury or fatal) can be separately computed. Techniques for estimating ramp crash frequency are described in a section titled "Crash Frequency Estimation."

The frequency of severe crashes should be given particular consideration because ramp alternatives that have the fewer severe crashes are likely to be more cost-effective to construct, all other considerations being equal. The frequency of "all" crashes is useful in selecting the safer ramp configuration when two or more ramp configurations have similar severe crash frequencies. In this situation, the ramp configuration with the lower frequency of "all" crashes represents the safer configuration.

Step 4. Estimate Interchange Crash Frequency

For this step, the crash frequencies for selected ramp configurations can be aggregated into an overall interchange crash frequency. This step does not need to be conducted unless the overall crash frequency is desired. The frequency of all crashes, severe crashes, or both can be aggregated.

The aggregation of ramp crash frequencies requires the specification of compatible ramp configurations for each ramp quadrant. For example, the diamond interchange would require specification of diagonal ramps in each of the four quadrants. A parclo A would require specification of diagonal exit ramps in two quadrants and a free-flow loop entrance ramp plus outer connection entrance ramp in the other two quadrants.

The interpretation of the overall interchange crash frequency is consistent with that used for the individual ramps. Interchange alternatives that have fewer severe crashes are likely to be more cost-effective to construct, all other considerations being equal. The frequency of "all" crashes is useful in selecting the safer interchange type when two or more types have similar severe crash frequencies. In this situation, the interchange type with the lower frequency of "all" crashes represents the safer configuration.

Interchange Turn Movement Estimation Technique

If design-year turn movements are not available, they can be estimated using the AADT of the major road and the technique described in this section. The percentages listed in Table 2-8 can be used for this purpose.

Table 2-8. Interchange Turn Movement Volumes as a Percentage of Major-Road AADT.

Ramp Configuration	Turn Movement Percentages by Area Type, %	
	Urban	Rural
Diagonal, Non-free-flow loop [1]	8.0	18.0
Free-flow loop, Outer connection	4.0	9.0

Note:
1 - Percentages listed for the diagonal and non-free-flow loop ramps relate to the total ramp volume (i.e., they include both left-turn and right-turn volumes). For these ramp configurations, the left-turn volume is estimated as 50 percent of the total ramp volume. The remaining volume represents that of the right-turn movement.

The percentages listed in Table 2-8 can be multiplied by the major-road AADT to estimate the corresponding ramp AADT. The diagonal and non-free-flow loop ramps serve both left-turn and right-turn volumes; hence, the estimated ramp AADT must be further multiplied by the left-turn (or right-turn) percentage to determine the left-turn AADT (or right-turn AADT) for the ramp. The left-turn volume is estimated as 50 percent of the total ramp volume. In contrast, the free-flow loop and outer connection ramp configurations serve either a left-turn or a right-turn volume (but not both).

Hence, the ramp AADT estimated from Table 2-8 for free-flow loop and outer connection ramps also represents a turn movement AADT.

To illustrate the technique, consider an interchange located in an urban area with a major-road AADT of 50,000 veh/d. A parclo A is being considered for this location. A diagonal exit ramp is used with this interchange type. Table 2-8 indicates that the exit ramps for this interchange would likely serve about 8.0 percent of the major-road AADT, or 4000 veh/d. Of this amount, 50 percent will turn left at the ramp terminal and the other 50 percent will turn right. Thus, the exit ramp left-turn AADT is 2000 veh/d and the exit ramp right-turn AADT is 2000 veh/d.

Continuing the illustration, the parclo A has a free-flow loop ramp and an outer connection ramp for movements entering the major road. The free-flow loop ramp effectively serves as a left-turn movement at the interchange (i.e., drivers make a left turn in their direction of travel). Its AADT can be estimated as 4.0 percent of the major-road AADT. The outer connection ramp serves as a right-turn movement. Its AADT can also be estimated as 4.0 percent of the major-road AADT. Thus, the cross street left-turn AADT is 2000 veh/d and the cross street right-turn AADT is 2000 veh/d.

Combined Ramp Configuration

The "combined" ramp configuration is occasionally used at some diamond and parclo interchanges. It represents a combination of the outer connection and diagonal ramps. Its geometry was previously shown in Figure 2-16. The geometry and operation of the combined ramp includes the best operational features of the diagonal and outer connector ramps. The left-turn movement is served at the intersection, and the right-turn movement is served by a turning roadway. A disadvantage of this design is that it requires considerable right-of-way and significant distance between the interchange and the nearest downstream intersection on the crossroad. As such, it is best-suited to rural locations.

A safety prediction model is not available for the combined ramp configuration. However, the models developed for the diagonal ramp and the outer connection ramp can be used to estimate crash frequency for the combined ramp. In this application, both models would be used and their estimates of crash frequency combined. For the diagonal ramp model, the AADT used would be that of the left-turn movement for the ramp. For the outer connection model, the AADT used would be that of the right-turn movement. The two crash frequencies (one from each model) would then be summed to obtain an estimate of the combined ramp crash frequency.

Crash Frequency Estimation

Equations are described in this section that can be used to estimate the expected crash frequency for interchange ramps located in Texas. They are applicable only to interchanges in non-frontage-road settings. They can be used to evaluate the safety of alternative ramp configurations for an interchange proposed for construction or an interchange undergoing major reconstruction. The equations should be recalibrated every three years to ensure that they continue to reflect current driver behavior and design practices in Texas.

The simplified safety prediction model for crashes of all severities is:

$$N_t = 0.247\ a_t \left(\frac{V_r}{1000} \right)^{0.76} \qquad \textbf{(50)}$$

where,

N_t = predicted annual number of crashes (of all severities), crashes/yr;
a_t = model calibration coefficient for area type, ramp type, and ramp configuration; and
V_r = average daily traffic on the ramp, veh/d.

The subscript t associated with each model variable in Equation 50 denotes that the variable represents crashes of all severities (including property-damage-only, injury, and fatal crashes). The value of the model calibration coefficient a_t can be obtained from Table 2-9.

Table 2-9. Calibrated Model Coefficients.

Area Type: Rural				**Area Type: Urban**			
Ramp Type	**Ramp Configuration**	**Model Coefficient**		**Ramp Type**	**Ramp Configuration**	**Model Coefficient**	
		a_t	a_{f+i}			a_t	a_{f+i}
Exit	Diagonal	0.83	0.80	Exit	Diagonal	0.57	0.49
	Non-free-flow loop	1.45	1.58		Non-free-flow loop	0.99	0.97
	Free-flow loop	0.52	0.47		Free-flow loop	0.35	0.29
	Outer connection	1.09	1.04		Outer connection	0.74	0.64
Entrance	Diagonal	0.50	0.46	Entrance	Diagonal	0.34	0.28
	Non-free-flow loop	0.88	0.91		Non-free-flow loop	0.60	0.56
	Free-flow loop	0.31	0.27		Free-flow loop	0.22	0.17
	Outer connection	0.66	0.60		Outer connection	0.45	0.37

The simplified safety prediction model for fatal and injury crashes is:

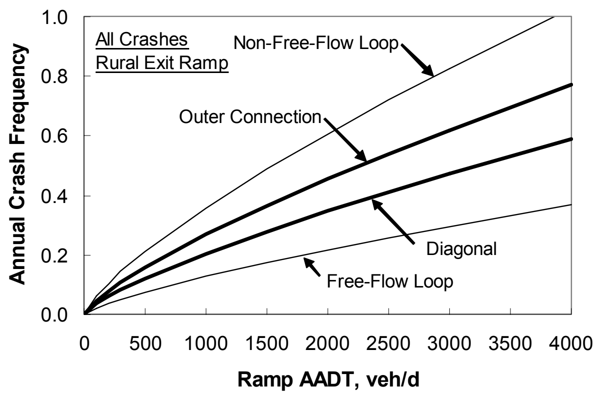
$$N_{f+i} = 0.0957\ a_{f+i} \left(\frac{V_r}{1000} \right)^{0.85} \qquad \textbf{(51)}$$

where,

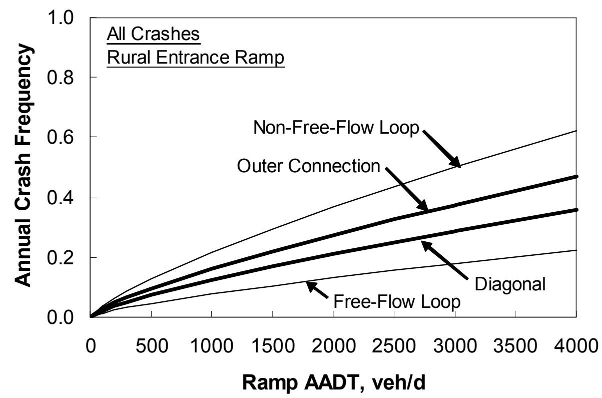
N_{f+i} = predicted annual number of fatal and injury crashes, crashes/yr.

The subscript $f+i$ associated with each model variable in Equation 51 denotes that the variable represents injury and fatal crashes (i.e., property-damage-only crashes are *not* included).

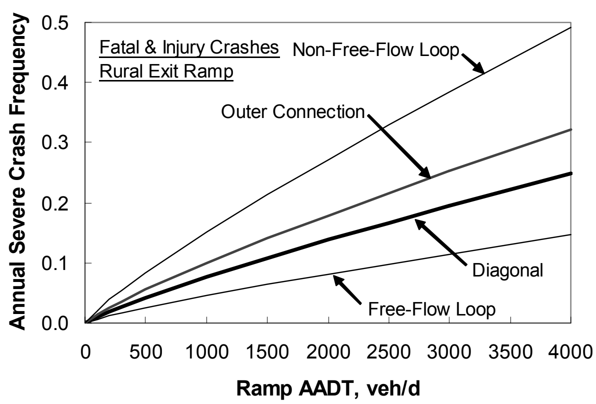
Figures 2-17 and 2-18 can also be used to graphically estimate crash frequency. They were developed using Equations 50 and 51 for a reasonable range of AADTs.



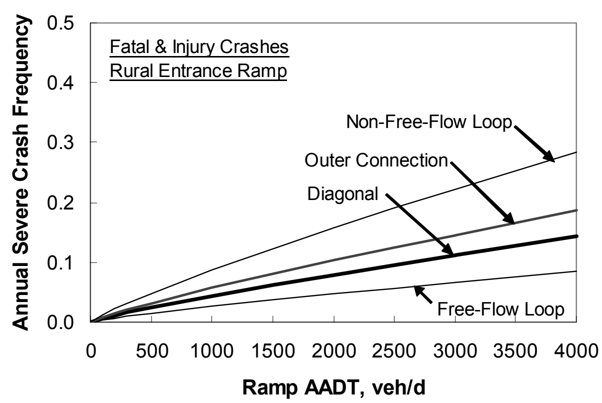
a. All Crashes on Exit Ramps.



b. All Crashes on Entrance Ramps.

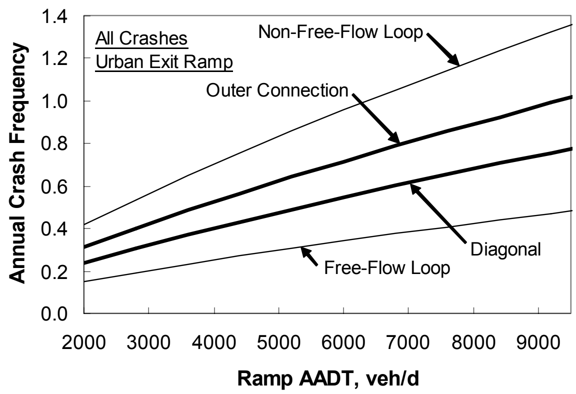


c. Severe Crashes on Exit Ramps.

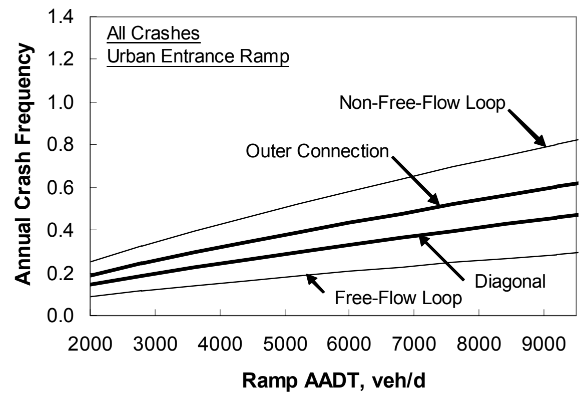


d. Severe Crashes on Entrance Ramps.

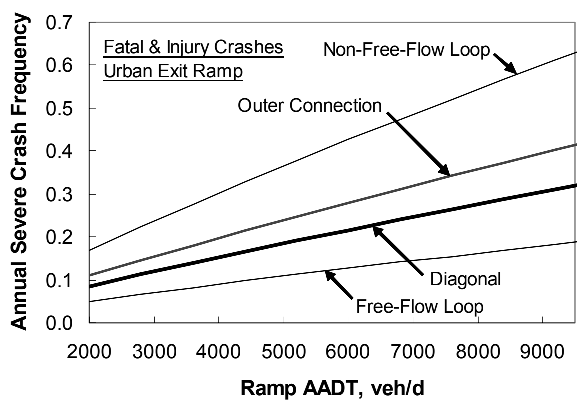
Figure 2-17. Rural Ramp Crash Frequency.



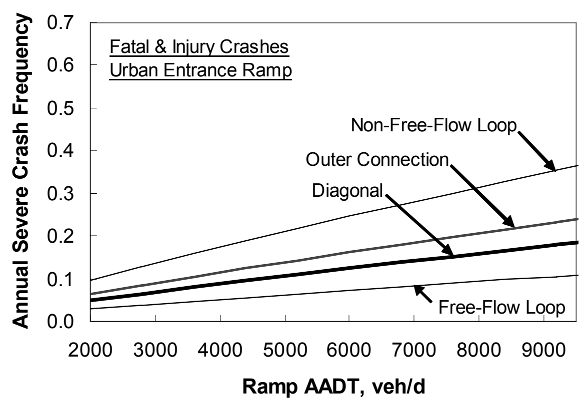
a. All Crashes on Exit Ramps.



b. All Crashes on Entrance Ramps.



c. Severe Crashes on Exit Ramps.



d. Severe Crashes on Entrance Ramps.

Figure 2-18. Urban Ramp Crash Frequency.

CHAPTER 3. RAMP DESIGN GUIDELINES

OVERVIEW

This chapter describes ramp design guidelines for freeway facilities without frontage roads. Design controls and elements routinely considered during the ramp design process are identified. The guidelines are organized to be consistent with Chapter 3, Section 6, of the *Roadway Design Manual* (*1*). The discussion associated with each design control emphasizes conveyance of the information needed to use the control in a design application.

The focus of this chapter is on the design controls and elements applicable to ramps in non-frontage-road settings. Design controls and elements that are common to ramps in both frontage and non-frontage-road settings are addressed in the *Roadway Design Manual* (*1*) and in *A Policy on Geometric Design of Highways and Streets* (*Green Book*) (*2*) and are not repeated herein.

The first section to follow this overview describes the use of design speed to define key elements of ramp design. The approach is tailored to the ramp's configuration and function such that design speed changes are gradual and consistent with driver expectancy. The second section describes the design controls and elements that dictate the horizontal geometry of both entrance and exit ramps. Controls considered include: vertical curvature, maximum superelevation rate, minimum radius, queue storage length, speed-change length, and superelevation transition length. The third section addresses two elements of the ramp cross section. Initially, the controls and considerations that guide in the selection of the number of lanes on the ramp proper are discussed. Thereafter, controls related to the ramp traveled-way width are described. The last section focuses on ramp terminal design. Design controls addressed include: intersection skew angle, approach cross section, and storage length. The selection of an appropriate intersection traffic control mode (i.e., stop or signal control) is also discussed. Access control limits along the crossroad are described.

GENERAL INFORMATION

This section provides standard design detail drawings for four ramp design configurations used at interchanges in non-frontage-road settings. They include: diagonal, non-free-flow loop, free-flow loop, and outer connection. The diagonal ramp is most commonly used in the diamond interchange; however, it can also be used for the parclo A, parclo A (2-quad), parclo B, and parclo B (2-quad) interchanges. The non-free-flow loop is limited to the parclo A (2-quad) and parclo B (2-quad) interchanges. In contrast, the free-flow loop is used at the parclo A and parclo B. The outer connection is used primarily with the parclo A and parclo B.

The design detail drawings provided in Figures 3-1 through 3-5 illustrate each ramp configuration in the context of its use at a diamond or parclo interchange. These drawings are not to scale and the dimensions shown may not reflect the proportions found in actual design.

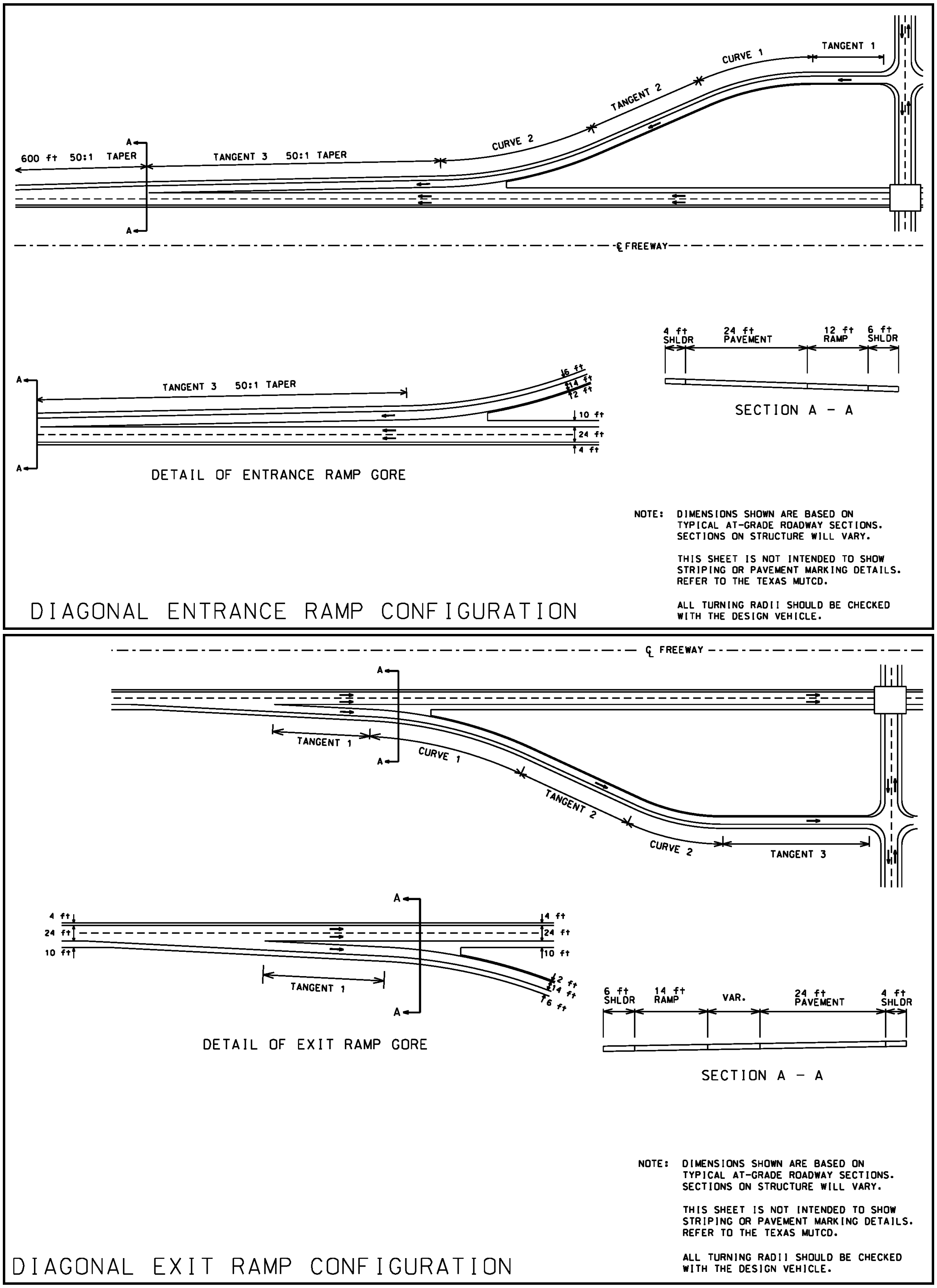
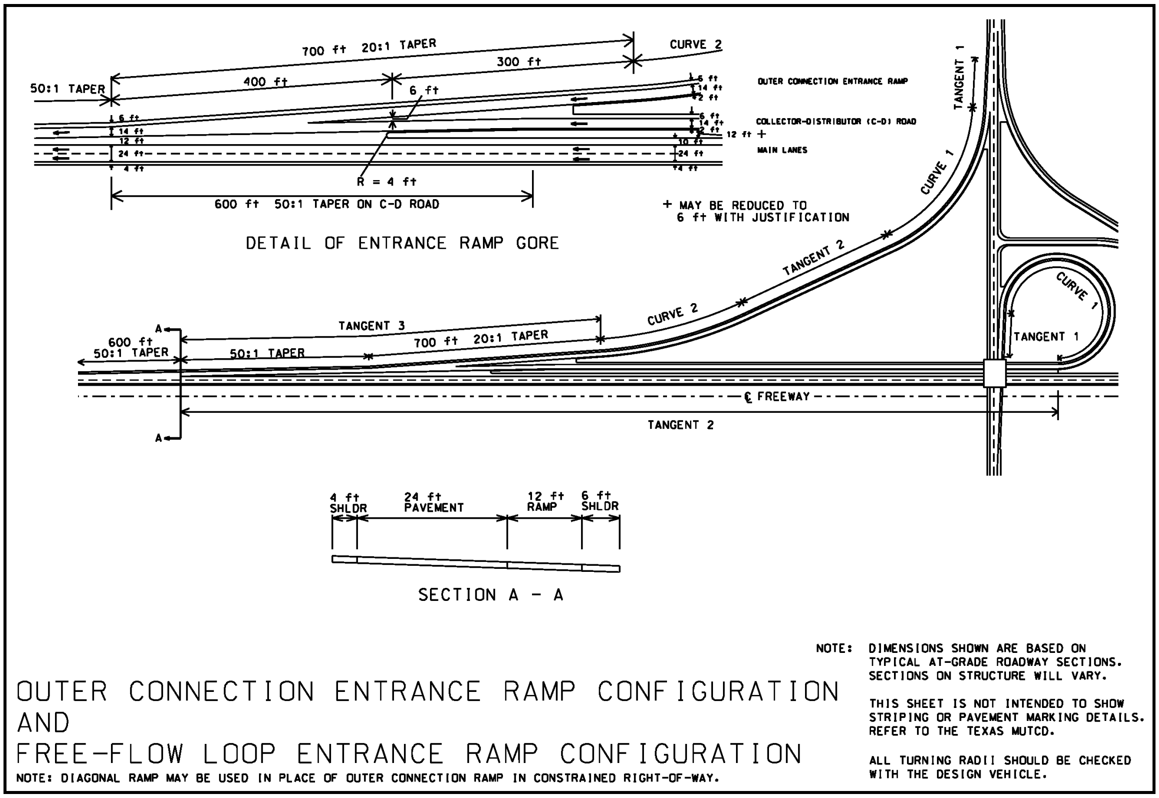


Figure 3-1. Typical Diamond Interchange Entrance and Exit Ramps.



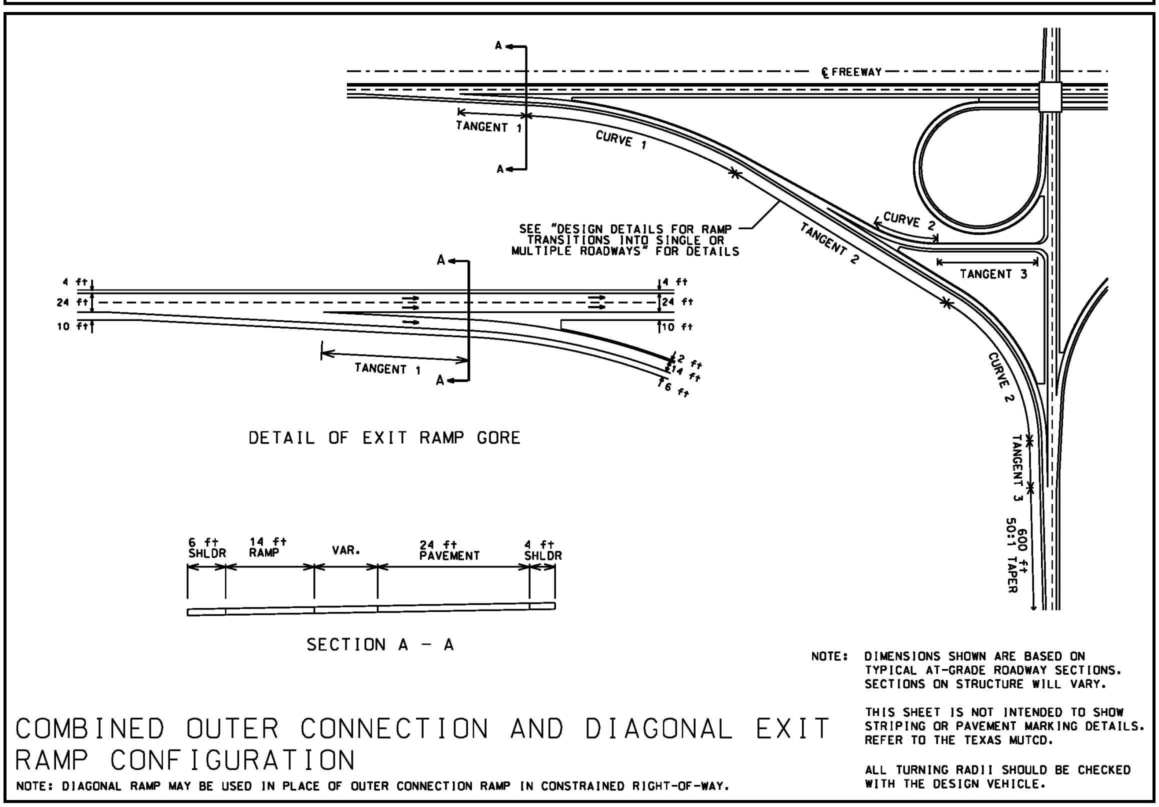
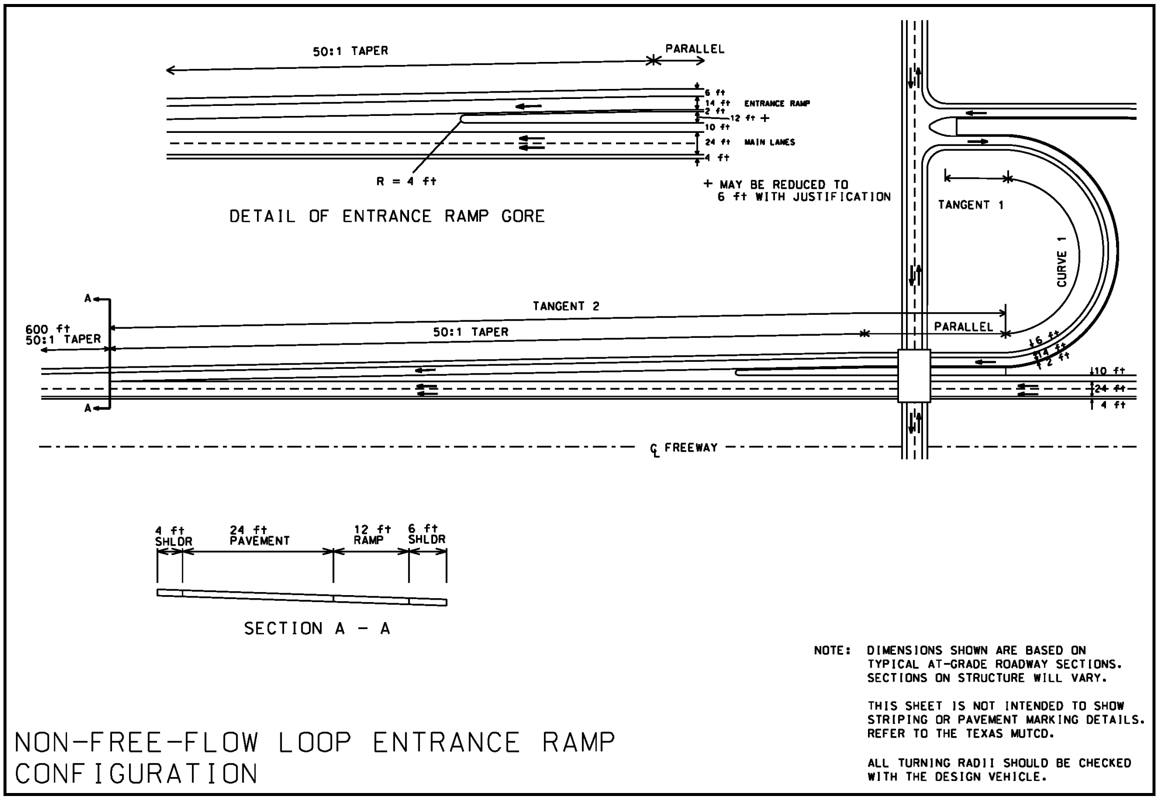


Figure 3-2. Typical Parclo A Interchange Entrance and Exit Ramps.



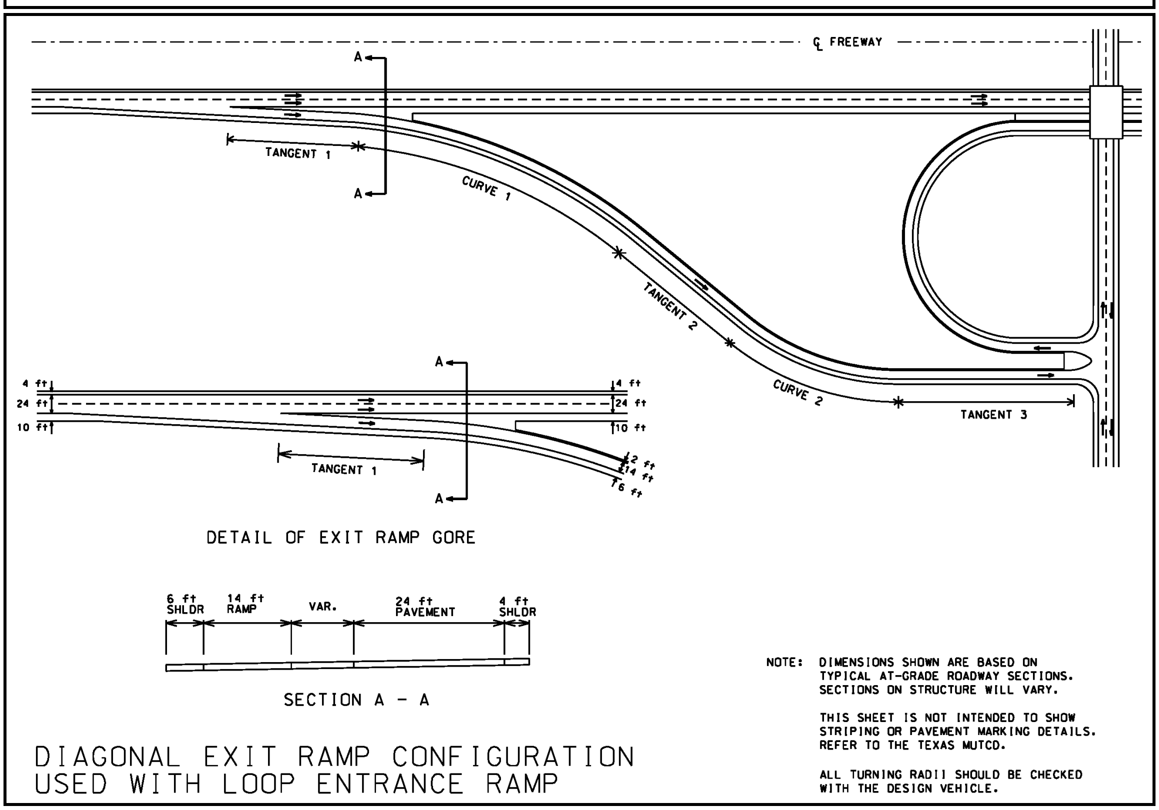
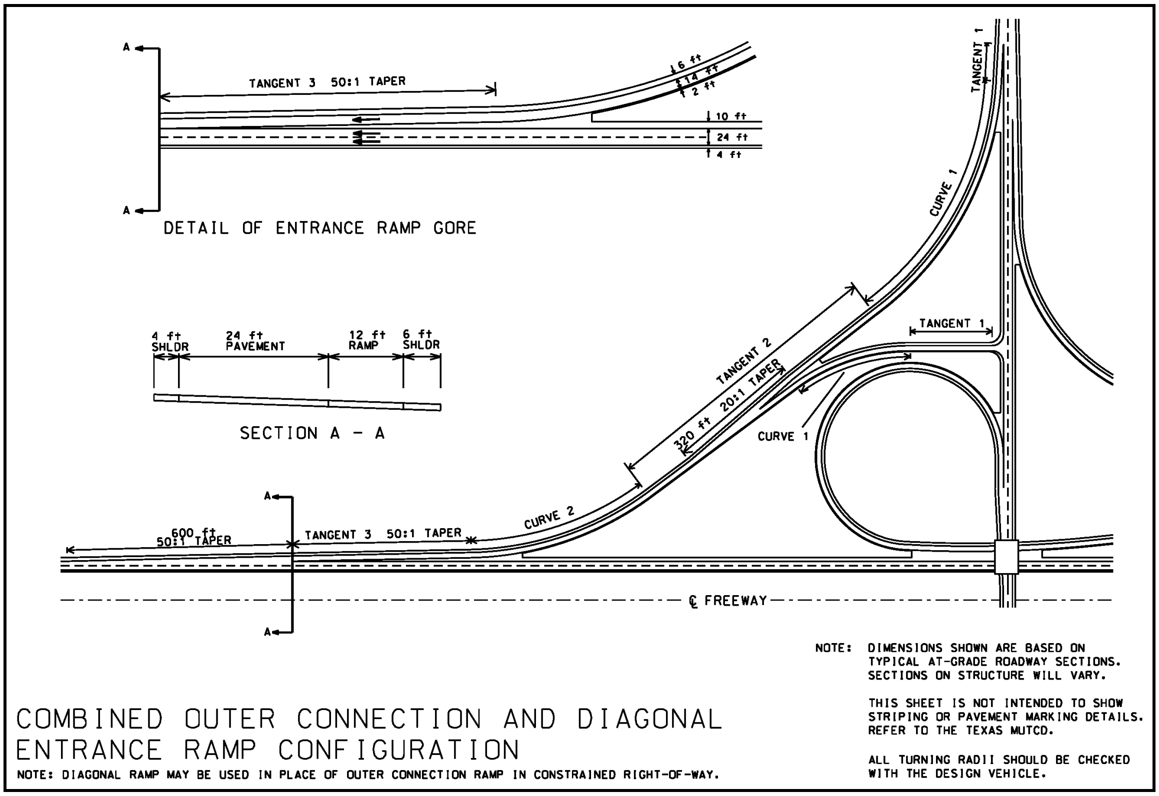


Figure 3-3. Typical Parclo A (2-Quad) Interchange Entrance and Exit Ramps.



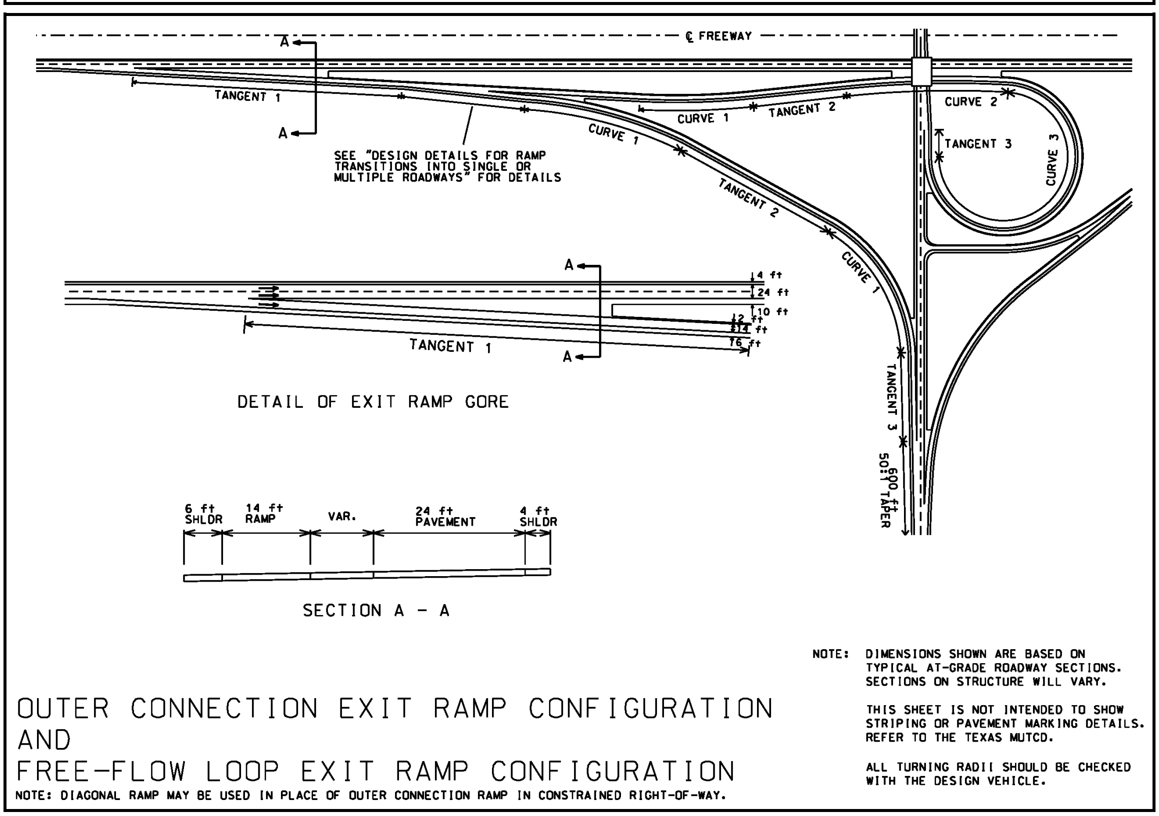
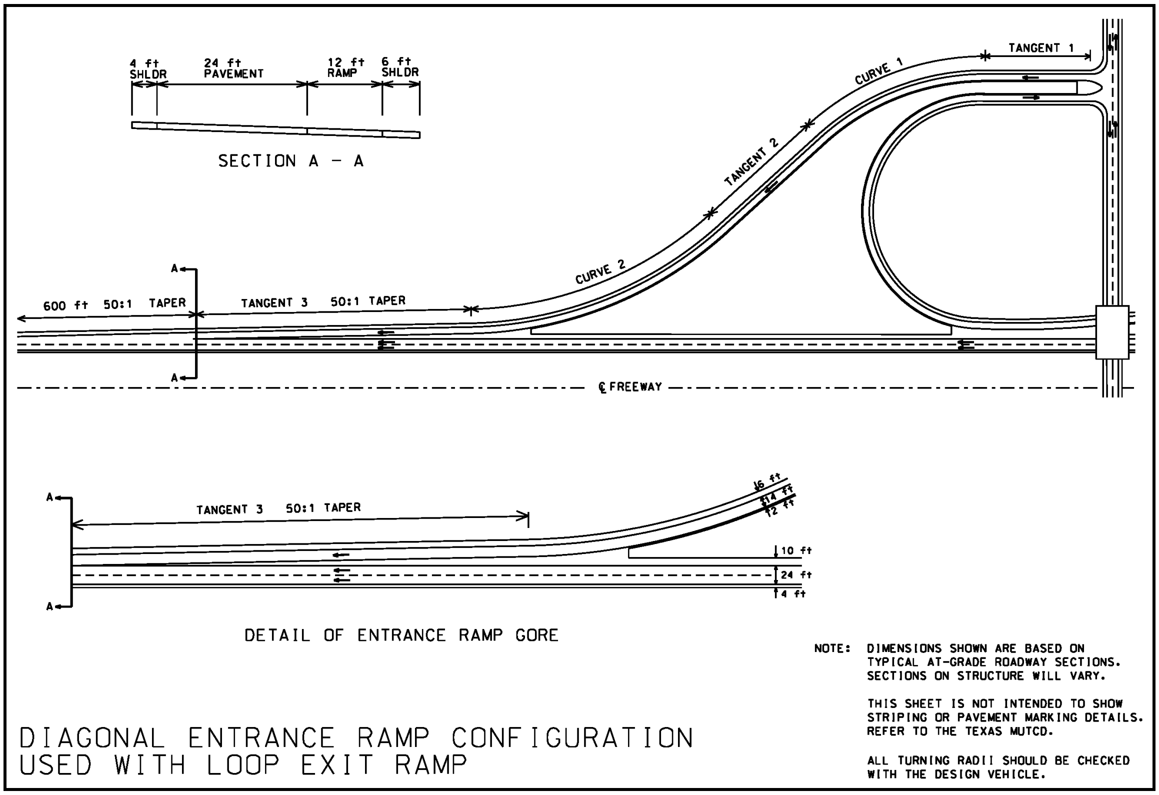


Figure 3-4. Typical Parclo B Interchange Entrance and Exit Ramps.



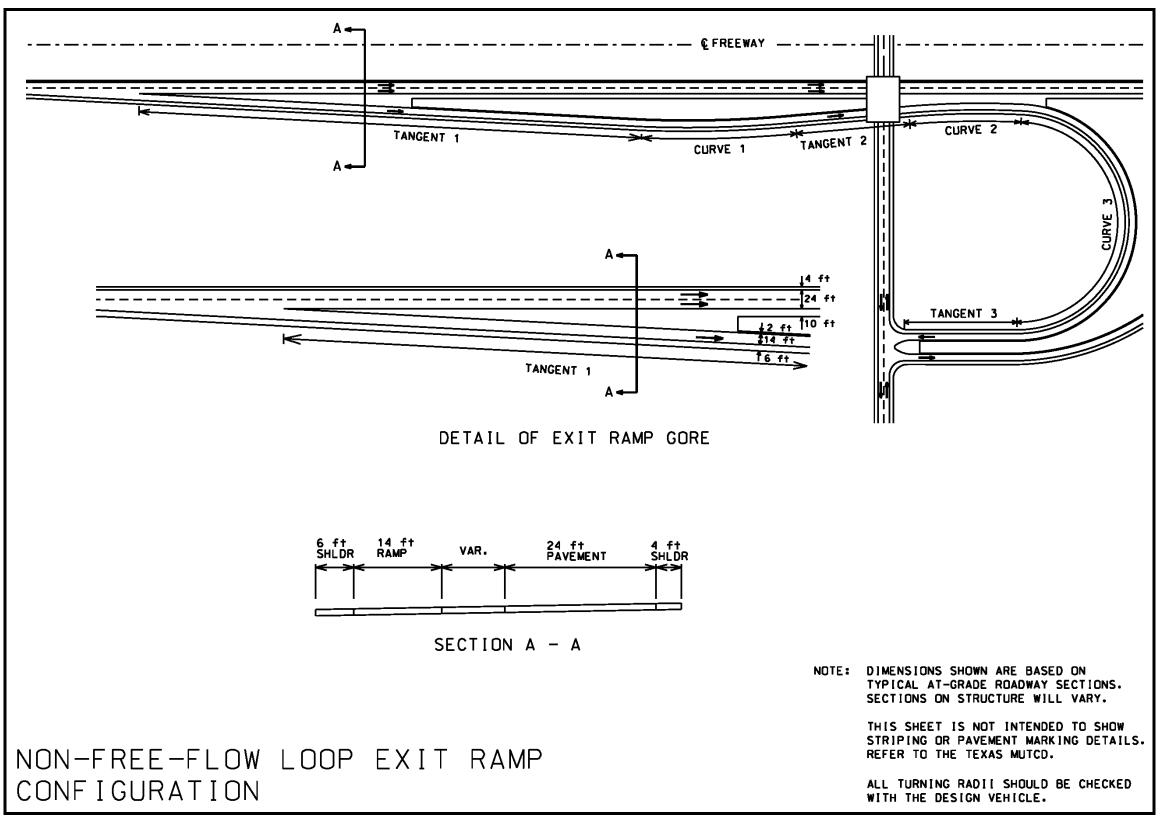


Figure 3-5. Typical Parclo B (2-Quad) Interchange Entrance and Exit Ramps.

DESIGN SPEED

This section describes the use of design speed as a control in ramp design. The approach is tailored to the ramp's configuration such that design speed changes are gradual and consistent with driver expectancy and operational capabilities. It is based on the specification of reasonable speed changes along the various tangents and curves that compose the ramp's horizontal alignment. The total amount of speed change needed is dictated by the design speed of the major road and that of the intersecting crossroad or ramp terminal. This approach is consistent with the ramp design speed guidance provided in the *Green Book* (*2*).

Ramp Segments

This section describes the individual road segments that compose the horizontal alignment for the interchange ramp. These segments consist of both tangents and curves. The segments that compose the diagonal ramp are shown in Figure 3-1. Both the exit and entrance ramp consist of three tangents and two curves. For the exit ramp, Tangent 1 provides for initial vehicle deceleration to the design speed of the controlling curve (i.e., Curve 1). Tangent 2 provides a length for transitioning the superelevation between Curves 1 and 2. Tangent 3 provides a length of roadway for deceleration and storage associated with the ramp terminal. The segments for the entrance ramp perform a similar function but in reverse order to those of the exit ramp.

Segments for a parclo A loop entrance ramp are shown in Figures 3-2 and Figure 3-3. The alignment includes a short segment of tangent to transition from the crossroad (or ramp terminal) design speed to that of the loop ramp. The location of this segment varies depending on whether a parclo A or parclo A (2-quad) is selected for design. The length of Tangent 2 is based on the distance needed to accelerate from the loop design speed to that of the major road. If desired, a spiral curve equal in length to 2.0-s travel time at the design speed can be located between Curve 1 and Tangent 2.

Segments for a parclo B loop exit ramp are shown in Figure 3-4 and Figure 3-5. The alignment shown includes three curves. Curves 1 and 2 are intended to promote driver awareness of the impending loop (i.e., Curve 3) and encourage drivers to gradually reduce speed prior to their arrival to Curve 3. The deflection angles for Curves 1 and 2 are each about two to three times larger than the ramp-to-major-road divergence angle. The location of Tangent 3 varies depending on whether a parclo B or parclo B (2-quad) ramp is selected. If desired, a spiral curve equal in length to 2.0-s travel time at the design speed can be located between Curves 2 and 3.

Segments for an outer connection ramp are shown in Figures 3-2 and 3-4. Similar to the diagonal ramp, three tangents and two curves provide the transition from the major-road design speed to the crossroad design speed. For the exit ramp, the deflection angle for Curve 1 should range from 30 to 45 degrees, with larger values associated with a higher design speed on the crossroad. For the entrance ramp, the deflection angle for Curve 1 should range from 45 to 60 degrees, with larger values associated with a higher design speed on the major road. Some variation from these ranges will occur when the major road and crossroad do not intersect at a right angle.

The exit ramp for the parclo A and the entrance ramp for the parclo B are shown in Figures 3-2 and 3-4 to effectively represent a combination of the outer connection and diagonal ramps. This type of "combined" ramp includes the best operational features of the diagonal and outer connector ramps. The left-turn movement is served at the intersection, and the right-turn movement is served by a turning roadway. A disadvantage of this design is that it requires considerable right-of-way and significant distance between the interchange and the nearest downstream intersection on the crossroad. When these disadvantages are significant, a diagonal ramp is often used in isolation to serve both the left-turn and right-turn movements at the crossroad.

Ramp Curve Design Speeds

This section defines a design speed for each of the ramp segments (i.e., curve or tangent) identified in Figures 3-1 through 3-5. The rules used to define these design speeds are described by Bonneson et al. (*4*). The intent of this specification of design speed by segment is to provide a uniform, gradual transition between the design speed of the major road and that of the crossroad or ramp terminal. To achieve this goal, a design speed is assigned to each successive curve and tangent. The change in design speed among adjacent segments is desirably limited to 5 or 10 mph.

The segment design speeds are based on the design speed of the "controlling" ramp curve. This curve is the first curve encountered on the exit ramp and the last curve encountered on the entrance ramp. The appropriate design speed for this curve is specified in Table 3-1. Column 1 of this table identifies the relationship between ramp configuration and practical design speed values.

Table 3-1. Design Speed for Controlling Ramp Curve.

Ramp Configuration	Major-Road Design Speed, mph										
	30	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**
	Controlling Curve Design Speed, mph										
Outer Connection	25	30	35	40	45	48	50	55	60	65	70
Diagonal	20	25	30	33	35	40	45	45	50	55	60
Loop	20	25	25	25	25	25	25	25	25	25	25

For outer connection ramps, the ramp curve nearest to the crossroad can also be considered as a controlling curve for the purpose of identifying its design speed. The design speed for this curve is also obtained from Table 3-1; however, the column used should correspond to the *crossroad* design speed. For example, consider an outer connection exit ramp merging with a crossroad. If the crossroad has a design speed of 50 mph, Table 3-1 indicates that Curve 2 should have a design speed of 45 mph.

If the tangent intersects the crossroad at a right angle and is part of an exit ramp, then its design speed is specified as a "stop" condition. This specification applies only to the determination of the tangent-length-related control values that are defined by speed and described in this chapter. A design speed that is consistent with that of the upstream curve should be used to define other control values for this tangent (e.g., stopping sight distance, vertical curvature, grade, etc.)

Some segment length controls described in this chapter require specification of a speed at the start and end of the segment. For this purpose, the speed at the end of the segment is considered to be equal to the segment design speed; the speed at the start of the segment is considered to be equal to the design speed of the preceding segment. In this application, "start" and "end" are defined in the direction of travel.

Table 3-2 identifies the recommended design speed for each ramp segment. For each combination of ramp type and configuration, segments are listed from top to bottom in the direction of travel.

HORIZONTAL GEOMETRICS

This section discusses selected design elements that compose the horizontal alignment of the interchange ramp. The topics discussed include: maximum superelevation rate, minimum radius, and minimum length controls for the individual ramp segments.

Maximum Superelevation Rate

The maximum superelevation rate for ramp curves can be either 6.0 or 8.0 percent. Use of a maximum superelevation rate of 8.0 percent allows for smaller curve radii; however, it also tends to increase the minimum length of the tangents by 5 to 8 percent. This increase is due to the additional length needed for superelevation transition. For this reason, a 6.0 percent maximum superelevation rate is preferred.

The one exception to the preference for a 6.0 percent maximum rate is the use of 8.0 percent for the sharpest curve on the loop ramp (i.e., Curve 3 on the loop exit ramp and Curve 1 on the loop entrance ramp). The higher rate of 8.0 percent is recommended for the sharpest curve on the loop ramp because it enables the use of radii of reasonable size. Use of a smaller maximum rate tends to yield radii that need significant right-of-way to construct and that require left-turning drivers to travel considerable extra distance.

Minimum Radius

The minimum radius associated with maximum superelevation rates of 6.0 and 8.0 percent are listed in Table 3-3.

Table 3-2. Ramp Segment Design Speed.

Ramp Type	Ramp Config-uration	Segment [1]	Major-Road Design Speed, mph						
			50	55	60	65	70	75	80
			Ramp Segment Design Speed, mph						
Exit	Diagonal	Tangent 1	40	45	50	55	60	65	70
		Curve 1*	35	40	45	45	50	55	60
		Tangent 2	30	35	40	40	40	45	50
		Curve 2	25	30	35	35	35	40	40
		Tangent 3	stop	stop	stop	stop	stop	stop	stop
	Outer Connection	Tangent 1	45	48	50	55	60	65	70
		Curve 1*	45	48	50	55	60	65	70
		Tangent 2	Average of design speed for Curves 1 and 2.						
		Curve 2	Use Table 3-1 [2].						
		Tangent 3	Use crossroad design speed V_{cr}.						
	Loop	Tangent 1	40	45	50	55	60	65	70
		Curve 1*	35	40	45	45	50	55	60
		Tangent 2	30	35	40	40	40	45	50
		Curve 2	25	30	35	35	35	40	40
		Curve 3	25	25	30	30	30	30	30
		Tangent 3 (Parclo B)	Use crossroad design speed V_{cr}.						
		Tangent 3 (Parclo 2B)	stop	stop	stop	stop	stop	stop	stop
Entrance	Diagonal	Tan. 1 (entry speed = 15 mph)	20	25	30	30	30	35	35
		Curve 1	25	30	35	35	35	40	40
		Tangent 2	30	35	40	40	40	45	50
		Curve 2*	35	40	45	45	50	55	60
		Tangent 3	50	55	60	65	70	75	80
	Outer Connection	Tan. 1 (entry speed = V_{cr})	Same as Curve 1.						
		Curve 1	Use Table 3-1 [2].						
		Tangent 2	Average of design speed for Curves 1 and 2.						
		Curve 2*	45	48	50	55	60	65	70
		Tangent 3	50	55	60	65	70	75	80
	Loop	Tan. 1 (A) (entry speed = V_{cr})	35	35	35	35	35	35	35
		Tan. 1 (2A) (entry speed = 15)	20	20	20	20	20	20	20
		Curve 1*	25	25	25	25	25	25	25
		Tangent 2	50	55	60	65	70	75	80

Notes:

1 - Segment locations are listed in the direction of travel. Segment numbers are shown in Figures 3-1 through 3-5. For computing some control values, design speeds listed are defined to occur at the *end* of the segment.

2 - Curve design speed can be obtained from Table 3-1 by using crossroad design speed (instead of major-road design speed) and selecting a speed from the row labeled "outer connection."

* - Controlling curve.

Table 3-3. Minimum Radius by Curve Design Speed.

Curve Design Speed, mph	Minimum Curve Radius, ft	
	6.0% Maximum Superelevation	8.0% Maximum Superelevation
25	185	170
30	275	250
35	380	350
40	510	465
45	660	600
50	835	760
55	1065	965
60	1340	1205
65	1660	1485
70	2050	1820

Ramp Length Based on Vertical Alignment

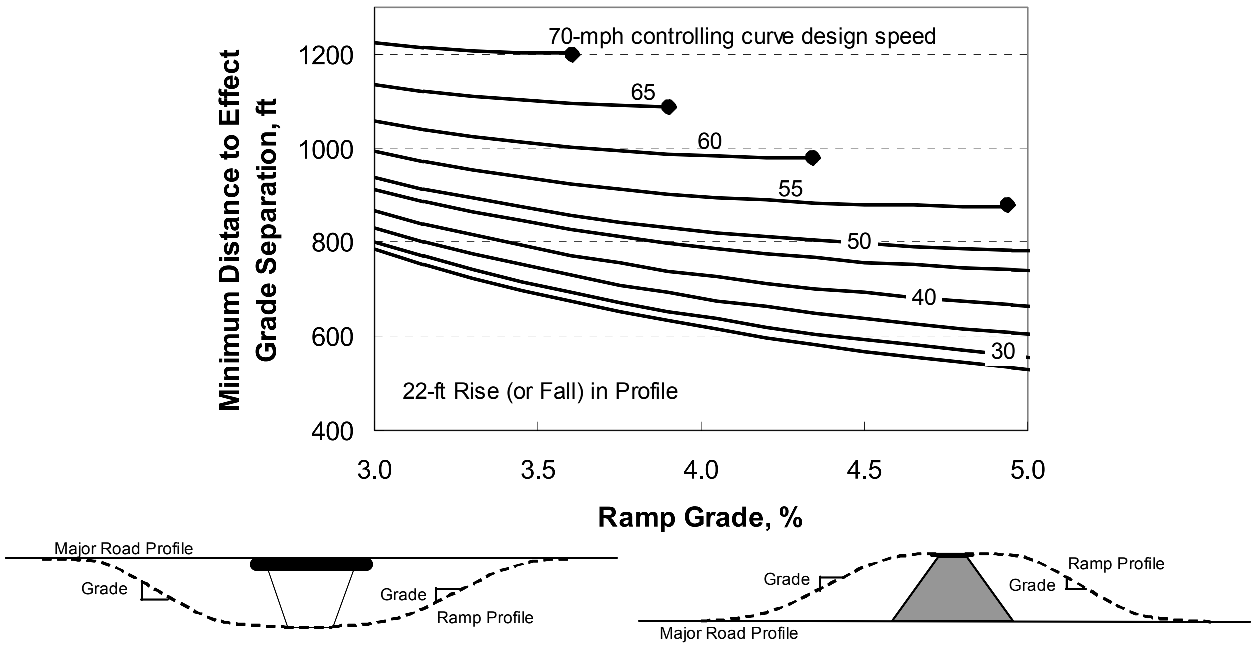
Ramp length is dictated by many controls including those related to: vertical alignment, speed change, and storage requirements. With regard to vertical alignment, ramp length is dictated by stopping sight distance, vertical curve length, grade, and the elevation change needed to vertically separate the major road from the crossroad. In this section, the referenced ramp length is measured from the exit (or entrance) gore on the major road to either: (1) the ramp terminal (if stop-controlled), or (2) the entrance (or exit) gore on the crossroad. The former point applies to the diagonal and 2-quad parclo ramp configurations; the latter applies to the other ramp configurations.

Figure 3-6 illustrates the minimum length dictated by vertical curvature for a 22-ft elevation difference between the major road and crossroad. The circled ends of the lines indicate the point below which the corresponding ramp grade is not feasible. Figure 3-6a applies when the ramp profile undergoes the full elevation change and the major road remains at grade. Figure 3-6b applies when the major road undergoes the elevation change and the ramps remain at grade.

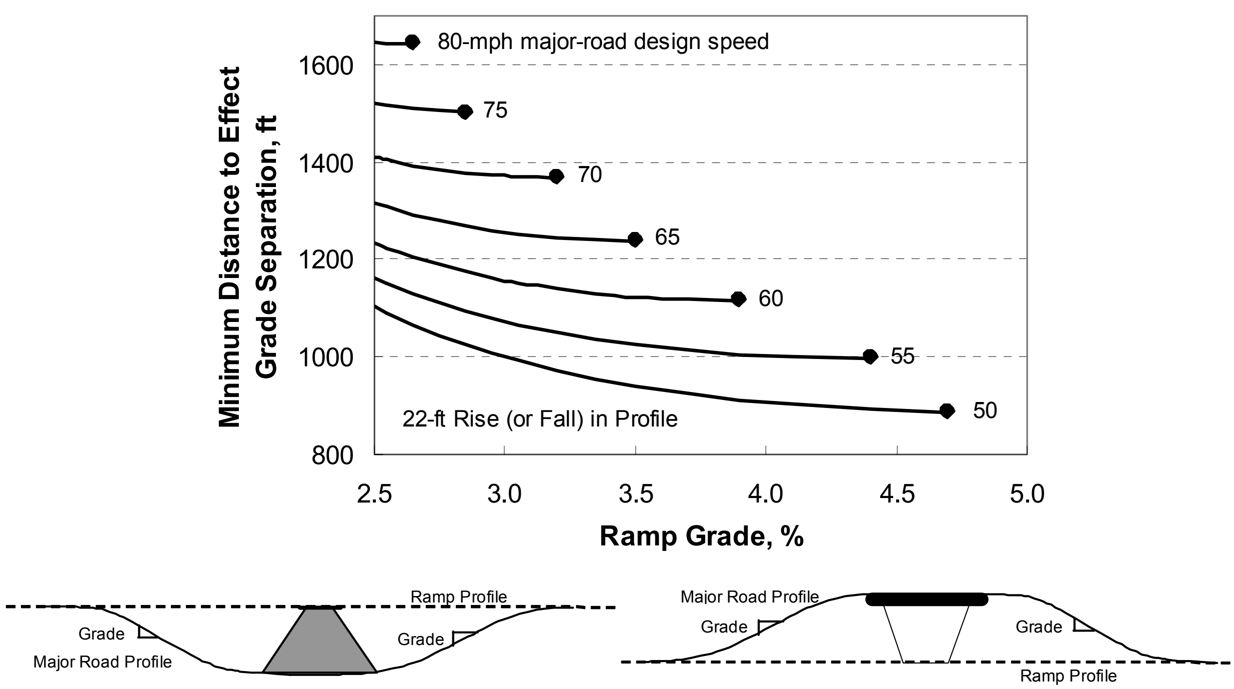
Ramp grades of 4.0 percent or less are preferable (*1*). However, grades up to 5.0 percent do not unduly interfere with truck operation and may be used where appropriate for topographic conditions (*2*). Downgrades on the sharpest curve on the loop ramp (i.e., Curve 1 on the parclo A, Curve 3 of the parclo B) should be limited to a maximum of 4.0 percent (*2*).

Ramp Length Based on Speed Change and Storage

This section describes design controls defining the minimum length of specific portions of the ramp. These controls include: (1) the minimum length required to effect the change in design speed between the major road and ramp junction, and (2) the length needed for queue storage. Considerations of queue storage are appropriate for exit ramps that terminate at a stop- or signal-controlled junction. They are also appropriate for entrance ramps that have a ramp meter.



a. Ramp Profile Undergoes Elevation Change.



b. Major-Road Profile Undergoes Elevation Change.

Figure 3-6. Minimum Ramp Length to Effect Grade Separation.

In this section, the referenced ramp length is measured from the point on the ramp at which a full-width traffic lane is first developed to either: (1) the ramp terminal (if stop-controlled), or (2) the point on the ramp at which the full-width traffic lane ends. The former point applies to the diagonal and 2-quad parclo ramp configurations; the latter applies to the other ramp configurations.

Speed Change and Storage Segments

One minimum ramp length control is based on the sum of the distance needed for speed change and that needed for queue storage. The relationships between ramp length, speed-change length, and storage length are illustrated in Figure 3-7 for diagonal ramps. These relationships also apply to the other ramp configurations.

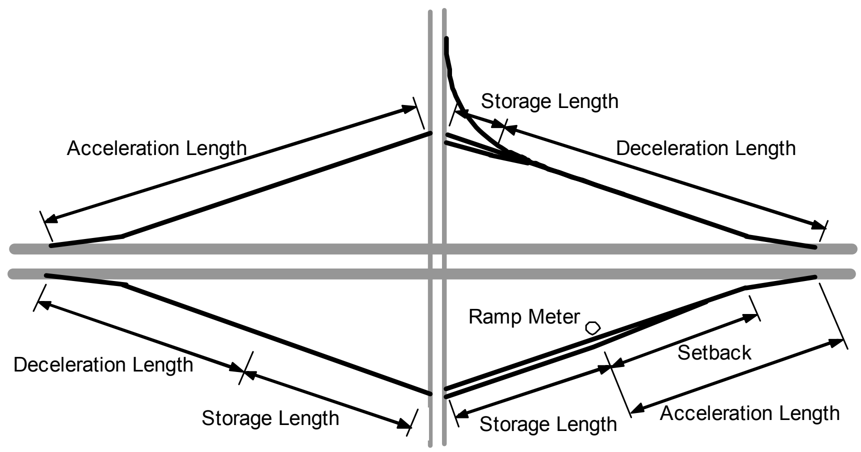


Figure 3-7. Ramp Length Components.

The minimum ramp length for speed change and storage is dependent on ramp type and whether traffic control conditions dictate the need for a storage length component. For exit ramps terminating in a stop condition, the minimum length is based on the distance needed to decelerate from the major-road design speed to a stop condition plus that needed for queue storage. For unmetered entrance ramps, the minimum ramp length is based only on the distance needed to accelerate from the ramp terminal design speed to that of the major road. For metered entrance ramps, the minimum ramp length is based on the distance needed for storage in advance of the meter plus that needed to accelerate from a stop condition to the design speed of the major road.

Component Lengths

This section provides guidance for determining the minimum ramp length needed for speed change and storage. The discussion focuses on the individual components that combine to define this minimum length. Design controls specifying the minimum lengths for the components include:

deceleration, storage, acceleration, and ramp meter setback. Each of these controls is discussed in the remainder of this subsection.

Deceleration Length. The length of ramp needed to decelerate the vehicle from an initial design speed to a final design speed can be obtained from Table 3-4.

Table 3-4. Minimum Length for Deceleration.

Initial Design Speed, mph	Final Design Speed, mph													
	Stop	15	20	25	30	35	40	45	50	55	60	65	70	75
	Minimum Length for Deceleration, ft													
20	150	80	--	--	--	--	--	--	--	--	--	--	--	--
25	190	150	100	--	--	--	--	--	--	--	--	--	--	--
30	235	200	170	140	--	--	--	--	--	--	--	--	--	--
35	280	250	210	185	150	--	--	--	--	--	--	--	--	--
40	320	295	265	235	185	155	--	--	--	--	--	--	--	--
45	385	350	325	295	250	220	140	--	--	--	--	--	--	--
50	435	405	385	355	315	285	225	175	--	--	--	--	--	--
55	480	455	440	410	380	350	285	235	140	--	--	--	--	--
60	530	500	480	460	430	405	350	300	240	130	--	--	--	--
65	570	540	520	500	470	440	390	340	280	220	120	--	--	--
70	615	590	570	550	520	490	440	390	340	280	200	110	--	--
75	660	635	620	600	575	535	490	440	390	330	260	190	100	--
80	720	690	670	640	610	570	530	480	430	370	310	240	170	90

The deceleration lengths in Table 3-4 can be adjusted to account for the effect of grade. The appropriate adjustment factor can be obtained from Figure 3-8. The length obtained from Table 3-4 would be multiplied by this factor to compute a deceleration length adjusted for ramp grade.

Exit Ramp Storage Length. The length of ramp needed for queue storage can be calculated using the following equation:

$$L_{\min,q} = f\ S\ \frac{Q\ r}{3600\ n} \qquad \textbf{(52)}$$

where,

$L_{min,\,q}$ = minimum length of roadway needed to store queued vehicles, ft;

f = adjustment factor to provide for storage of all left-turn vehicles on most cycles (= 2.0);

S = average distance between two queued vehicles (= 25, 30, 35, or 40 ft for truck percentage ranges of 0 to 4, 5 to 9, 10 to 14, or 15 to 19 percent), ft;

Q = ramp design hour left-turn volume, veh/h;
r = time during which vehicles queue (unsignalized: 120 s; signalized: 0.75 C), s;
C = signal cycle length, s; and
n = number of lanes available for queue storage.

As an alternative to Equation 52, the maximum-back-of-queue statistic obtained from an acceptable software traffic model can also be used for design.

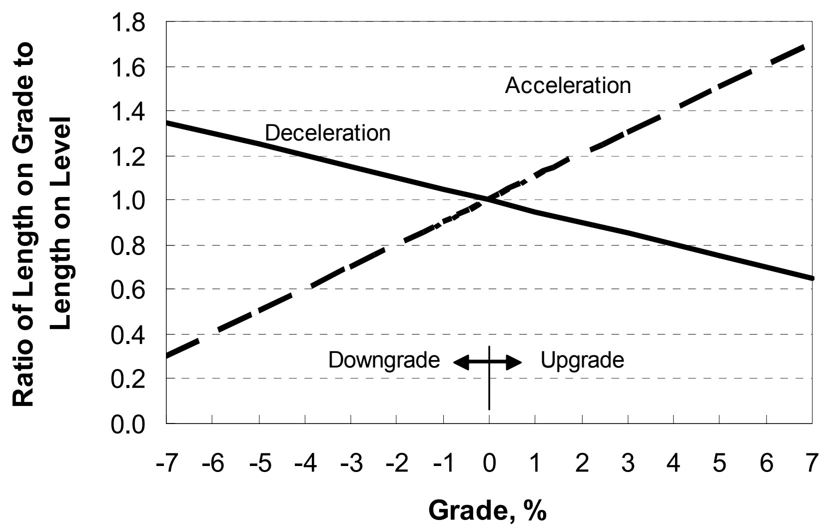


Figure 3-8. Grade Adjustment Factors for Deceleration and Acceleration Lengths.

The storage lengths obtained from Equation 52 for a range of left-turn volumes, left-turn lanes, and ramp terminal control conditions are listed in Table 3-5. The lengths listed in this table reflect a truck percentage in the range of 5 to 9 percent.

For signalized ramp terminals, the trends in Table 3-5 suggest that it is desirable that the ramp design provide 600 ft of queue storage. This storage length would serve left-turn volumes up to 400 veh/h. If left-turn volumes exceed 400 veh/h, a second 600-ft storage lane should be included in the design. Two 600-ft lanes would adequately serve left-turn volumes up to 800 veh/h. If the volume exceeds 800 veh/h, then a third 600-ft storage lane is needed.

For unsignalized ramp terminals (i.e., where the ramp left-turn is stop-controlled), the last two columns of Table 3-5 indicate that it is desirable that the ramp design provide 600 ft of queue storage. This storage length would serve left-turn volumes up to 300 veh/h. At this volume level, signalization of the ramp terminals is likely to be justified.

Table 3-5. Storage Lengths for Signalized and Unsignalized Exit Ramp Terminals.

Signalized Terminals [1]						Unsignalized Terminals [2]	
1 Left-Turn Lane		2 Left-Turn Lanes		3 Left-Turn Lanes		1 Left-Turn Lane	
Ramp Left-Turn Volume, veh/h	Storage Length, ft	Ramp Left-Turn Volume, veh/h	Storage Length, ft	Ramp Left-Turn Volume, veh/h	Storage Length, ft	Ramp Left-Turn Volume, veh/h	Storage Length, ft
100	150	500	375	900	450	50	100
150	225	550	415	950	475	100	200
200	300	600	450	1000	500	150	300
250	375	650	490	1050	525	200	400
300	450	700	525	1100	550	250	500
350	525	750	565	1150	575	300	600
400	600	800	600	1200	600	350	700

Notes:
1 - Lengths are based on an assumed 5 to 9 percent trucks, a 120-s signal cycle, and a 90-s red duration for the ramp.
2 - Lengths are based on an assumed 5 to 9 percent trucks.

Storage Length for Ramp Meter Control. As indicated in Figure 3-7, an entrance ramp controlled by a ramp meter requires a storage area to safely store vehicles between the crossroad ramp terminal and the meter. Guidance for designing an entrance ramp with a ramp meter is provided in *Design Criteria for Ramp Metering* (*6*). This guidance was modified to account for the absence of frontage roads. The recommended minimum storage lengths are listed in Table 3-6.

Table 3-6. Storage Lengths for Entrance Ramps with Ramp Meter Control.

1-Lane Ramp/Single Release		1-Lane Ramp/Multiple Release		2-Lane Ramp/Single Release	
Ramp Volume, veh/h	Storage Length, ft	Ramp Volume, veh/h	Storage Length, ft	Ramp Volume, veh/h	Storage Length, ft
200	310	600	555	1000	725
300	395	700	605	1100	755
400	480	800	650	1200	785
500	560	900	690	1300	805
600	640	1000	730	1400	820
700	725	1100	760	1500	830
800	800	1200	785	1600	840

In general, it is preferable that all entrance ramps in urban areas include storage length for ramp meter control. This practice will ensure adequate ramp length is available in the event that the ramp is metered at some point during its design life.

Based on the trends in Table 3-6, it is desirable that the entrance ramp design include 800 ft for queue storage. Provision of at least 800 ft of storage will allow a single-lane ramp to adequately serve ramp volumes up to 1200 veh/h. If volumes exceed this amount, then a second lane should be added to the ramp. A metered two-lane ramp will serve traffic volumes up to 1600 veh/h.

Acceleration Length. The length of ramp needed to accelerate the vehicle from an initial design speed to a final design speed can be obtained from Table 3-7.

Table 3-7. Minimum Length for Acceleration.

Final Design Speed, mph	Initial Design Speed, mph													
	Stop	15	20	25	30	35	40	45	50	55	60	65	70	75
	Minimum Length for Acceleration, ft													
20	70	10	--	--	--	--	--	--	--	--	--	--	--	--
25	120	60	10	--	--	--	--	--	--	--	--	--	--	--
30	180	140	80	20	--	--	--	--	--	--	--	--	--	--
35	280	220	160	110	20	--	--	--	--	--	--	--	--	--
40	360	300	270	210	120	30	--	--	--	--	--	--	--	--
45	560	490	440	380	280	160	30	--	--	--	--	--	--	--
50	720	660	610	550	450	350	130	30	--	--	--	--	--	--
55	960	900	810	780	670	550	320	150	30	--	--	--	--	--
60	1200	1140	1100	1020	910	800	550	420	180	30	--	--	--	--
65	1410	1350	1310	1220	1120	1000	770	600	370	140	30	--	--	--
70	1620	1560	1520	1420	1350	1230	1000	820	580	370	160	30	--	--
75	1790	1730	1630	1580	1510	1420	1160	1040	780	540	330	90	30	--
80	2000	1920	1860	1790	1690	1580	1360	1180	970	720	510	270	90	30

Table 3-7 can be used to determine the minimum length of ramp needed when a meter is planned. In this application, the acceleration length is provided in addition to the storage length needed for the meter. The initial design speed used with Table 3-7 is the "stop" condition. The final design speed ranges from 50 to 60 mph in correlation with the major-road design speed range of 50 to 80 mph. This final speed reflects a tendency for ramps to be metered only during peak traffic periods when operating speeds in the shoulder lane tend not to exceed 50 mph.

The acceleration lengths in Table 3-7 can be adjusted to account for the effect of grade. The appropriate adjustment factor can be obtained from Figure 3-8. The length obtained from Table 3-7 would be multiplied by this factor to compute an acceleration length adjusted for ramp grade.

Setback For Ramp Meter. To ensure reasonable horizontal clearance between the major road and ramp meter, a nominal setback distance is needed between the ramp meter and ramp gore.

The location of this distance is shown in Figure 3-7. Minimum and desirable values for this distance are 250 and 350 ft, respectively.

Minimum Ramp Length Based on Speed Change and Storage

The minimum length controls described in the previous subsection can be used to examine their impact on ramp length. Controls addressed in this section include: deceleration, storage, and acceleration. The relationship between the associated lengths was shown previously in Figure 3-7. The information in this section is based on the assumption that the ramps are on level terrain. Other lengths will be obtained if the ramps are on grade. The objective of this section is to illustrate how the noted design controls can be used together to assess their impact on overall ramp length.

The discussion in this section focuses on the diagonal and 2-quad parclo ramp configurations because the design speed for the ramp segment adjoining the crossroad is known. The other ramp configurations are not addressed because they require specification of a crossroad design speed. Nevertheless, minimum ramp length based on speed change and storage can be determined for these ramps once the crossroad design speed is specified. The minimum ramp lengths for the diagonal and 2-quad parclo ramps are listed in Table 3-8.

Table 3-8. Minimum Ramp Length Based on Speed Change and Storage.

Ramp Type	Ramp Configuration	Component	Major-Road Design Speed, mph						
			50	55	60	65	70	75	80
			Minimum Ramp Length, ft						
Exit	Diagonal, Parclo B (2-quad)	Storage	600	600	600	600	600	600	600
		Deceleration[1]	435	480	530	570	615	660	720
		Total:	1035	1080	1130	1170	1215	1260	1320
Entrance	Diagonal, Parclo A (2-quad)	Acceleration[1]	660	900	1140	1350	1560	1730	1920
		Total:	660	900	1140	1350	1560	1730	1920
	Metered Ramp	Storage	800	800	800	800	800	800	800
		Acceleration[2]	720	830	900	960	1050	1130	1200
		Total:	1520	1630	1700	1760	1850	1930	2000

Notes:
1 - Lengths are based on the major-road design speed and the design speed of the ramp terminal. This latter speed is defined as “stop” condition and 15 mph for the exit and entrance ramps, respectively.
2 - Lengths are based on the assumption that metering occurs during peak traffic periods. The speed during these periods is estimated to be in the range of 50 to 60 mph and increases with major-road design speed.

The total lengths listed in Table 3-8 are measured from the point on the ramp at which a full-width traffic lane is first developed to the ramp terminal.

Ramp Length Based on Design Speed

This section describes controls that specify the minimum length of individual ramp segments. These controls include: curve travel time, superelevation transition length on tangent, and segment speed change.

Discussion in this section referring to ramp length is based on measurement from the point on the major road at which a full-width traffic lane is provided for diverging (or merging) ramp traffic to either: (1) the ramp terminal (if stop-controlled), or (2) the point on the crossroad where the full-width traffic lane ends for merging (or diverging) ramp traffic. The former point applies to the diagonal and 2-quad parclo ramp configurations; the latter applies to the other configurations.

In some situations, constraints imposed by environmental, cost, or right-of-way considerations result in the ramp being designed to its minimum practical length. In these situations, some of the minimum segment length controls defined in this section are likely to dictate segment length. Rarely, if ever, will all of the segments be at the minimum values specified by these controls. Factors related to the geometry of the ramp alignment, skew angle, and topography will often serve to dictate the length and orientation of the other ramp segments.

Ramp Segments

As shown in Figures 3-1 through 3-5, the ramp horizontal alignment is separated into its curve and tangent segments. Each ramp segment is associated with a different design speed. Design speeds typically change from 5 to 10 mph between adjacent segments. The length of each tangent segment is dependent on considerations of deceleration, acceleration, and superelevation transition. The length of each curve is dependent on consideration of deceleration, acceleration, and travel time.

Component Lengths

Travel Time Length. The *Green Book* (*2*) indicates that highway curves should have a minimum length equal to 15 times the design speed expressed in miles per hour. This control equates to a minimum curve length of 10-s travel time at the design speed. This length is excessive for ramp curve design; however, research on curve driving behavior indicates that 3.0-s travel time is necessary to accommodate the steering maneuver during curve entry and exit. Thus, the minimum length of ramp curve is defined to equal 3.0-s travel time. This length can be computed using the following equation:

$$L_{\min,t} = 4.4\ V_c \qquad \textbf{(53)}$$

where,
$L_{min,\,t}$ = minimum length of ramp curve based on travel time, ft; and
V_c = curve design speed, mph.

Transition Length. This control often dictates the minimum length of tangent between two ramp curves. It is intended to provide sufficient length for superelevation transition from the exited curve to the entered curve. The following equation can be used to calculate this minimum length:

$$L_{\min,r} = w\, p_r \left(\frac{CS_u}{G_u} + \frac{CS_d}{G_d} \right) \tag{54}$$

where,

$L_{min,\,r}$ = minimum length of tangent between two curves based on superelevation transition, ft;
w = width of rotated traffic lane (i.e., ramp traveled-way width) (see Table 3-10), ft;
p_r = portion of transition located on the tangent (use 0.67);
CS_u = change in pavement cross slope from upstream curve to tangent, percent;
CS_d = change in pavement cross slope from tangent to downstream curve, percent;
G_u = maximum relative gradient for upstream curve (see Table 3-9), percent; and
G_d = maximum relative gradient for downstream curve (see Table 3-9), percent.

Variables in Equation 54 indicate that the minimum length of tangent depends on the width of the ramp traveled-way, the design superelevation rate, and the maximum relative gradient. Desirable traveled-way widths for ramps are the subject of discussion in a subsequent section. Equation 54 is formulated for application to tangents between two curves. It can be used for tangents at the beginning and end of the ramp if one of the "change in cross slope" *CS* terms is deleted.

The design superelevation rate needed for Equation 54 can be obtained from the *Roadway Design Manual* (*1*). The maximum relative gradient is dependent on curve design speed. Values of this control are listed in Table 3-9.

Table 3-9. Maximum Relative Gradient.

	Curve Design Speed, mph											
	25	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**
Max. Gradient, %	0.70	0.66	0.62	0.58	0.54	0.50	0.47	0.45	0.43	0.40	0.38	0.35

Deceleration Length. The minimum length of ramp segment needed for deceleration can be obtained from Table 3-4. The "initial" speed is defined as the design speed of the preceding segment. The "final" speed is the design speed of the subject segment. These speeds are identified in Table 3-2 for the various ramp configurations. The lengths in Table 3-4 can be adjusted to account for the effect of grade by using Figure 3-8.

Acceleration Length. The minimum length of ramp segment for acceleration can be obtained from Table 3-7. The "initial" and "final" speeds are defined in the same manner as in the preceding paragraph. The guidance regarding adjustment for ramp grade is also the same.

Balanced Segment Length. Considerations of safety and aesthetics justify the need for balance in the length of the central tangent and its adjacent curves on the outer connection ramp (or any other ramp where two successive curves deflect in the same direction). A tangent that is significantly shorter than that of the adjacent curves results in a "broken-back" arrangement of curves. Such an arrangement is contrary to driver expectancy and can result in an alignment that is not pleasing in appearance. To avoid these complications, Curves 1 and 2 of the outer connection ramp should have about the same length. Moreover, the length of Tangent 2 should equal or exceed the length of Curve 1 (or Curve 2).

Loop Exit Ramp Offset

The loop exit ramp shown in Figures 3-4 and 3-5 has a slight reverse curvature to increase driver awareness and promote a safe reduction in speed prior to the sharp curve at the end of the ramp. The deflection in the ramp alignment should be sufficient to ensure that drivers follow the curved alignment and discourage them from traveling in a straight line through Curves 1 and 2. This can be accomplished by ensuring that Curve 1 is laterally offset from the major road a distance of 40 ft (measured between the nearest edge-of-shoulder for both roadways).

CROSS SECTION

This section discusses issues related to the design of the ramp cross section. These elements include the number of lanes on the ramp and the width of the ramp traveled-way. Design decisions regarding the cross section of the ramp terminal approach are discussed in the next section.

Number of Lanes

The capacity of the ramp terminal or the merge/diverge point often limits ramp volume to values below that of the capacity of a single traffic lane. For this reason, a single-lane ramp cross section should be adequate for most service interchanges. However, a dual-lane ramp proper may be justified if any of the following conditions are expected:

- Design hour ramp volume exceeds the practical capacity of a single traffic lane.
- Ramp is longer than 1400 ft, in which case a two-lane ramp would allow opportunities to pass slower vehicles.
- Ramp is located on a steep upgrade such that a two-lane ramp would allow opportunities to pass vehicles slowed by the grade.
- Ramp has a long, sharp curve such that a two-lane ramp would provide additional accommodation of off-tracking by long vehicles.
- For entrance ramps, design hour ramp volume exceeds 800 veh/h and the ramp will be metered.

For purposes of evaluating the first bullet, the practical capacity for a single-lane diagonal or outer connection ramp is 1550 veh/h. The practical capacity for a loop ramp is 1200 veh/h. The ramp length identified in the second bullet is measured from the exit (or entrance) gore on the major road to either: (1) the ramp terminal (if signal or stop-controlled), or (2) the entrance (or exit) gore on the crossroad.

Ramp Traveled-Way Width

Ramp traveled-way widths should be based on ramp curvature, number of lanes provided, and the portion of trucks in the traffic stream to accommodate truck off-tracking. Desirable traveled-way widths for ramps with curbs or shoulders are provided in Table 3-10. The values shown in the table for the "shoulder" edge treatment are based on left and right shoulder widths of 2.0 and 6.0 ft, respectively. Exhibit 10-67 in the *Green Book* (*2*) provides guidance for determining traveled-way width when the sum of the left and right shoulder widths is less than 8.0 ft.

Table 3-10. Traveled-Way Width for Ramps.

Edge Treatment	Curve Radius, ft	Single-Lane Ramp			Dual-Lane Ramp		
		Truck Percentage			Truck Percentage		
		< 5%	5 to 10%	>10%	< 5%	5 to 10%	>10%
		Traveled-Way Width [1], ft					
Mountable Curb - both sides	150	18	21	23	26	29	32
	200	17	20	22	26	28	30
	300	17	20	22	25	28	29
	400	17	19	21	25	27	28
	500	17	19	21	25	27	28
	Tangent	17	18	20	24	26	26
Vertical Curb - one side [2]	150	18	21	23	27	30	33
	200	17	20	22	27	29	31
	300	17	20	22	26	29	30
	400	17	19	21	26	28	29
	500	17	19	21	26	28	29
	Tangent	17	18	20	25	27	27
Shoulder	150	14	15	17	24	27	30
	200	13	15	16	24	26	28
	300	13	15	15	23	26	27
	400	13	15	15	23	25	26
	500	12	15	15	23	25	26
	Tangent	12	14	14	22	24	24

Notes:
1 - Widths from Reference *2* (Exhibit 10-67) and based on left and right shoulder widths of 2.0 and 6.0 ft, respectively.
2 - For ramps with vertical curb on *both* sides, add 1.0 ft to the widths shown.

RAMP TERMINAL DESIGN

Traffic Control

The traffic control mode used to regulate traffic at the ramp terminals has a significant impact on traffic flow along the crossroad and on the extent of queue growth on the exit ramps. The control mode used for the left-turn movement may not be the same as that used to control the right-turn movements. Possible combinations of control mode are listed in Table 3-11.

Table 3-11. Exit-Ramp Traffic Control Combinations.

Exit-Ramp Left-Turn Traffic Control Modes	Exit-Ramp Right-Turn Traffic Control Modes [1]			
	Signal	Stop	Yield	Merge [2]
Signal	✔	not common	✔	✔
Stop	not common	✔	✔	✔

Notes:
1 - Common control mode combinations are indicated by check (✔).
2 - Free (uncontrolled) right-turn lane with an added lane extending beyond the end of the channelizing island and along the crossroad requiring ramp vehicles to merge with crossroad vehicles.

A decision to use signal control at the ramp terminal should be based on an evaluation of the signal warrants provided in Part 4 of the *Manual on Uniform Traffic Control Devices* (7) and the findings from a capacity analysis. Other criteria may also be considered in the decision of whether to use signal control. In fact, signal control may be helpful under the following conditions:

- to increase ramp capacity and, thereby, prevent spillback from the ramp onto the major road;
- whenever dual left-turn or right-turn lanes are dictated by traffic demand; and
- when sight distance to ramp drivers is restricted along the crossroad.

Both interchange ramp terminals should use the same traffic control mode to regulate the left-turn movements (e.g., both signalized or both unsignalized).

The free-right-turn lane associated with the "merge" design tends to extend a considerable length along the crossroad and can induce intense weaving activity on the crossroad if the adjacent downstream intersection is relatively close to the ramp terminal. For this reason, the "merge" traffic control mode may be best-suited to locations where the distance to the adjacent downstream intersection will exceed 1320 ft for the design life of the interchange.

Traffic control for pedestrians at signalized interchanges often consists of crosswalks across the ramp approach leg, ramp departure leg, and external crossroad leg. A crosswalk across the internal crossroad leg is typically not provided due to complications associated with the ramp terminal signal phasing.

Intersection Skew Angle

Ramp terminal design for non-frontage-road settings can include a discontinuous alignment through the ramp/crossroad junction. This discontinuity allows each ramp junction leg to be skewed in a direction toward the major road, thereby minimizing the curvature on the ramp proper. Figure 3-9 illustrates the discontinuous alignment of the approach and departure legs at a ramp/crossroad junction. The radius, throat width, and taper rate shown in Figure 3-9 are the subject of discussion in the next section.

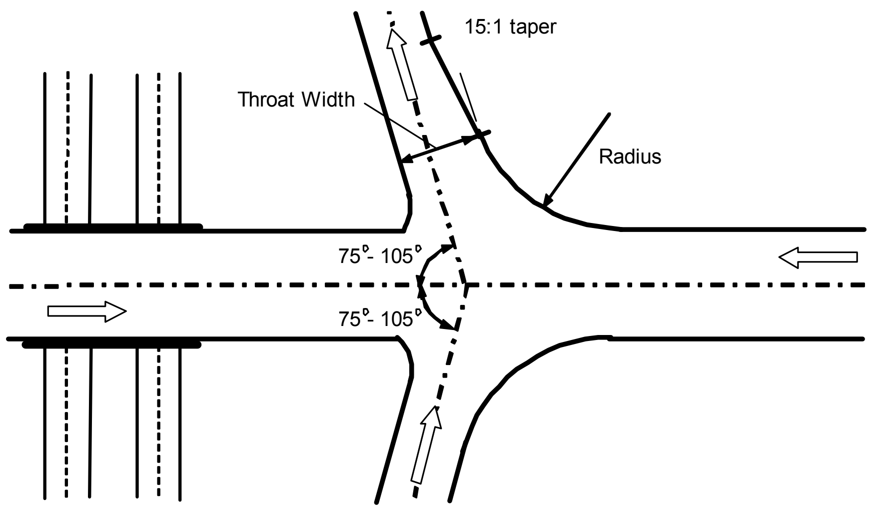


Figure 3-9. Discontinuous Ramp Alignment at Ramp/Crossroad Junction.

Desirably, the alignment of the ramp would be designed such that skew is avoided at the ramp terminal. However, if conditions dictate the use of skew, it is desirable that the skew angle fall within the range of 75 to 105 degrees. Angles larger or smaller than this amount tend to limit the visibility of ramp drivers, increase pedestrian crossing distances, and increase the exposure time for left-turn drivers. In some circumstances, angles in the range of 60 to 120 degrees are acceptable.

Departure Leg Design

The departure leg of the entrance ramp should accommodate the swept path of the design vehicle as it negotiates a left turn or a right turn from the crossroad. The traveled-way within the throat of the departure leg should be sufficiently wide as to allow the design vehicle to turn from the crossroad and enter the ramp without, at any point, encroaching on an adjacent lane, shoulder, or curb. This need can be served by use of a simple corner radius with adequate throat width. It can also be served by use of a three-centered compound curve for the corner radius. Typical three-centered curve designs are provided in Chapter 9 of the *Green Book* (*2*).

If a simple corner radius design is desired, the throat width needed for various radius and design vehicle combinations is provided in Table 3-12. Interpolation is appropriate for intermediate skew angles and radii.

Table 3-12. Ramp Departure Leg Throat Width.

Angle of Intersection, degrees	Corner Radius [1], ft	Throat Width for Selected Design Vehicles [2], ft					
		SU	BUS	WB-40	WB-50	WB-62	WB-67
60	25	17	24	21	28	--	--
	30	16	23	19	25	--	--
	40	14	22	17	22	30	37
	50	14	20	15	20	27	34
	60	14	18	14	18	25	31
90	25	19	30	23	32	--	--
	30	17	25	19	29	--	--
	40	14	22	17	22	39	39
	50	14	19	14	19	33	33
	60	14	16	14	16	29	29
120	25	21	32	24	36	--	--
	30	17	26	19	30	--	--
	40	14	19	17	22	26	30
	50	14	16	14	18	22	25
	60	14	14	14	15	18	21

Notes:
1 - Radii shown represent a simple curve radius design for the edge of traveled way.
2 - Widths from Reference *2* (Exhibit 9-31).
"--" - not applicable.

The throat width identified in Table 3-12 should be provided at the point where the simple radius ends and the tangent portion of the entrance ramp begins. The transition back to the nominal traveled-way width of the ramp, using a 15:1 taper, should also begin at this point. The layout of these design elements is shown in Figure 3-9.

Approach Leg Design

The number of lanes provided on an exit-ramp terminal approach should reflect consideration of the traffic control mode, turn volumes, and crossroad through volume. The guidance provided in a previous section on storage length (see discussion associated with Table 3-5) should be used to determine the minimum number of lanes needed on the ramp terminal approach and their length. An acceptable software traffic model can also be used for making these determinations.

As a minimum, it is preferable to provide at least one approach lane for each traffic movement served at the ramp terminal. This treatment permits the use of separate traffic control modes to regulate each movement and should minimize delays and queues to ramp traffic.

Design to Discourage Wrong-Way Maneuvers

A problem inherent to interchanges is the potential for wrong-way entry into an exit ramp. Several techniques have been used to discourage these maneuvers. One technique is to use a sharp corner radius on the inside of the turn movements that, if completed, would result in a wrong-way maneuver. This technique is illustrated at two locations in Figure 3-10. One location is at the intersection of the left-edge of the exit ramp approach and the right-edge of the crossroad approach. It should discourage improper right turns into the exit ramp. A second location is at the median nose on the external crossroad approach. The sharp radius at this location should discourage improper left turns into the exit ramp.

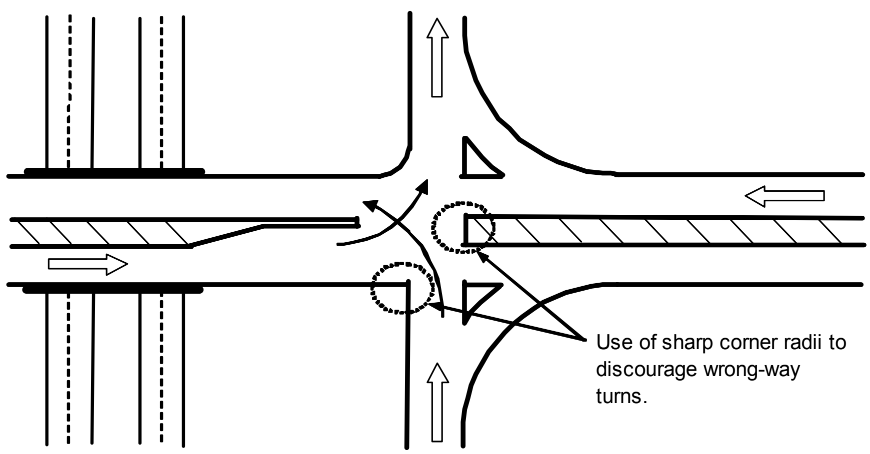


Figure 3-10. Designs to Discourage Wrong-Way Maneuvers.

Another technique to discourage wrong-way maneuvers is to use island channelization within the intersection. If used, this channelization should not be overly complex nor should it obstruct the ramp-to-ramp through traffic movement, if it exists. Although this movement typically has negligible traffic volume, its accommodation in the ramp terminal design is important because it can provide essential capacity during incidents or maintenance activities on the major road.

Some parclo A (2-quad) and parclo B (2-quad) designs have experienced wrong-way maneuvers onto the exit ramps. The potential for this maneuver exists because the ramp junction approach and departure legs are located adjacent to one another on the same side of the crossroad. Separation of these two ramp legs using a median of nominal width can provide for the development of a semicircular crossroad median nose that shadows the exit ramp approach and discourages wrong-way entry via a left turn into the exit ramp.

Access Control on Crossroad

The control of access along the crossroad is essential to the safe and efficient operation of the interchange. The importance of access control is heightened when frontage roads are not provided because of the inherent increase in turning traffic and the focus on development of properties adjacent to the crossroad. Inadequate access control in the vicinity of the interchange can create operational problems on the crossroad that may propagate to the ramps, causing spillback onto the major road. To ensure efficient interchange operation, access rights should be acquired and maintained for a minimum distance along the crossroad, upstream and downstream from the interchange. This "separation distance" is shown in Figure 3-11.

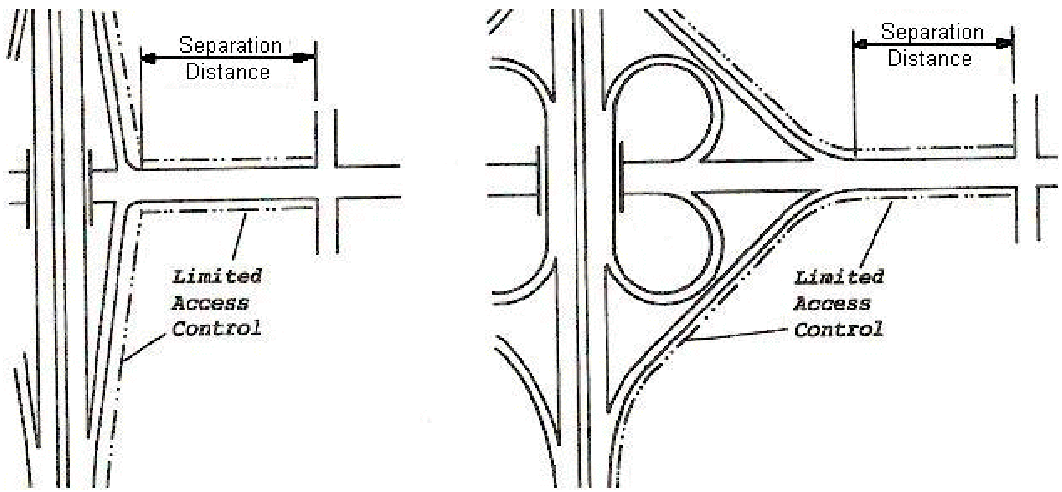


Figure 3-11. Separation Distance for Access Control.

The amount of separation distance needed is dependent on whether the first downstream intersection is signalized or unsignalized. For a signalized intersection, the separation distance should be sufficient for an exit-ramp right-turn vehicle to weave across the crossroad and decelerate into the left-turn bay at the first downstream intersection (or for a vehicle at the intersection to weave across the crossroad and decelerate into the left-turn bay at the interchange ramp terminal). For an unsignalized intersection (or driveway), the separation distance should be sufficient to allow right-turning vehicles to access the adjacent property without disrupting traffic flow through the ramp terminal. The design of the unsignalized access should include channelization to discourage all left-turn movements.

Separation distances for a range of posted speed limits on the crossroad are listed in Table 3-13. The distances shown for unsignalized intersection access are based on the minimum connection spacings listed in the *Access Management Manual* (8). The distances shown for signalized intersection access are consistent with those recommended by other state DOTs.

Table 3-13. Separation Distance Based on Deceleration and Weaving.

Posted Crossroad Speed, mph	Minimum Separation Distance [1], ft	
	First Signalized Intersection	First Unsignalized Intersection [2]
30	450	200
35	500	250
40	550	305
45	600	360
50	650	425
55	700	425

Notes:

1 - Separation distances are measured along the crossroad from the end of the nearside curb return (or taper) on the ramp to the nearside curb line of the adjacent driveway or intersection.

2 - Raised curb median or island channelization should be used to discourage left-turn maneuvers.

CHAPTER 4. REFERENCES

1. *Roadway Design Manual.* Texas Department of Transportation, Austin, Texas, February 2004.

2. *A Policy on Geometric Design of Highways and Streets.* American Association of State Highway and Transportation Officials, Washington, D.C., 2001.

3. Bonneson, J., K. Zimmerman, and M. Jacobson. *Review and Evaluation of Interchange Ramp Design Considerations for Facilities without Frontage Roads.* Report FHWA/TX-04/4538-1. Texas Department of Transportation, Austin, Texas, September 2003.

4. Bonneson, J., K. Zimmerman, C. Messer, D. Lord, and M. Wooldridge. *Development of Ramp Design Procedures for Facilities without Frontage Roads*. Report FHWA/TX-05/0-4538-2. Texas Department of Transportation, Austin, Texas, September 2004.

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6. Chaudhary, N.A. and C.J. Messer. *Design Criteria for Ramp Metering: Appendix to TxDOT Roadway Design Manual*. Report FHWA/TX-01/2121-3. Texas Department of Transportation, Austin, Texas, 2000.

7. *Manual on Uniform Traffic Control Devices–Millennium Edition*. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 2000.

8. *Access Management Manual*. Texas Department of Transportation, Austin, Texas, December 2003.

APPENDIX

RECOMMENDED TEXT FOR THE *ROADWAY DESIGN MANUAL*

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The design guidelines described in Chapter 3 of this report are offered as "recommended" design guidelines in that they represent reasonable ramp design practices. They are based on information gleaned from previous research, a review of authoritative reference documents on ramp design, a review of ramp design practices of other state DOTs, interviews with TxDOT engineers, and the original research of the report authors. Many of the design controls described in Chapter 3 represent minimum values recommended in authoritative documents; others were developed for this research. All controls described herein should be considered to represent reasonable minimum values (or, in some cases, "desirable" minimum values). They should not be construed to represent design standards until adopted by TxDOT and cited in the text of the *Roadway Design Manual.*

It is recommended that TxDOT incorporate by reference the guidelines described in Chapter 3 in a future edition of the *Roadway Design Manual*. This manner of incorporation is recommended for two reasons. First, experience is needed in using the guidelines in Chapter 3 before they are adopted as standards. This experience will provide useful refinement and prioritization of the guideline content. Second, the guidelines are quite extensive and, if wholly incorporated, would constitute a major revision to one chapter of the *Roadway Design Manual*. The implications of the proposed changes on the design of other freeway design elements should be fully evaluated before making the needed major changes to the *Roadway Design Manual*. Hence, the recommended guidelines should undergo a period of review, trial, and refinement by TxDOT engineers before they are added to the *Roadway Design Manual*.

During the review period, the recommended ramp design guidelines should be incorporated by reference in the *Roadway Design Manual*. Text to be added to the *Roadway Design Manual* to facilitate this type of incorporation is highlighted by <u>underline</u> in the following boxed sections.

Chapter 3 - New Location and Reconstruction (4R)

Ramps and Direct Connectors

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◆ Metered Ramps
◆ <u>Ramps at Interchanges without Frontage Roads</u>

Chapter 3 - New Location and Reconstruction (4R)

Metered Ramps

:

Ramps at Interchanges without Frontage Roads

Based on a recent change in TxDOT policy, frontage roads are not to be included along controlled-access highways unless a study indicates that the frontage road improves safety, improves operations, lowers overall facility costs, or provides essential access. Interchange design options that do not include frontage roads are to be considered for all new freeway construction.

Ramps at interchanges without frontage roads can be more challenging to design because of their greater length, combined horizontal and vertical curvature, and potential for limited storage. In some situations, loop ramps and outer connection ramps may offer operational and/or safety benefits beyond those of diagonal ramps. The guidelines provided in *Recommended Ramp Design Procedures for Facilities without Frontage Roads* may be considered when designing ramps for an interchange that does not have frontage roads. This report also describes procedures for evaluating the operational and safety benefits of alternative ramp configurations.