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16. Abstract

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 in non-frontage-road settings are more complicated 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 are focused on ramp design for frontage-road settings. The objective of this research project is to develop recommended design procedures for interchange ramps on facilities without frontage roads.

This report describes the findings from the first year of a two-year project. Current ramp design practices were reviewed, and the operational features of interchange ramps in non-frontage-road settings were evaluated. Both the diamond and partial cloverleaf interchange forms were considered. The findings from the evaluation of the field and simulation data indicate that a rational approach to interchange type selection and operational evaluation is feasible using the critical movement analysis approach. A procedure for using this approach is documented in the Appendix.

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REVIEW AND EVALUATION OF INTERCHANGE RAMP DESIGN CONSIDERATIONS FOR FACILITIES WITHOUT FRONTAGE ROADS

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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 (Minute Order 108544), frontage roads are not to be included along controlled-access highways (i.e., freeways) 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 are more complicated to design than those in frontage-road settings for several reasons. First, they must be designed to provide a safe transition in vehicle speed and acceleration 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 potion 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 (HOV) bypass lanes are present.

In summary, adequate ramp length, appropriate horizontal and vertical curvature, and flaring to increase storage area at the crossroad intersection might be used to design operationally superior ramps for non-frontage-road settings. However, current design procedures available in standard TxDOT reference documents are focused on ramp design for frontage-road settings. Therefore, a need exists to develop interchange ramp design procedures (or modify existing guidelines) that address the aforementioned concerns and that can be used to construct safe and efficient interchange ramps for facilities without frontage roads.

RESEARCH OBJECTIVE

The objective of this research project is to develop recommended design procedures for interchange ramps on facilities without frontage roads. These procedures should describe the operational and safety benefits of alternative ramp configurations and offer guidelines that help engineers select the most appropriate configuration for design-year traffic. The procedures should be based on the findings from a review of the ramp design guidelines used by other state DOTs, the analysis of traffic data obtained from field and simulation studies, and the analysis of crash data for interchanges in Texas.

RESEARCH SCOPE

The recommended guidelines developed for this research will be applicable to the geometric design of interchanges in urban, metro, and rural environments on Texas freeways without frontage roads. The guidelines will address design controls, design criteria, and design procedures for exit and entrance ramps, as they are affected by the exclusion of the frontage road. They will reflect consideration of the needs of both cars and trucks. Finally, they will also reflect a reasonable sensitivity to safety and operations. The research does 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.

RESEARCH APPROACH

The project's research approach is based on a two-year program of field study, evaluation, and development. This approach will ultimately yield a guideline document to assist in the design of interchange ramps on facilities without frontage roads. During the first year of the project, the state-of-the-practice were documented, and the operational effects of ramp design configurations were evaluated. In the second year, the safety effects of ramp design configurations will be evaluated, and the findings will be incorporated into a recommended ramp design procedures document.

The main product of this research will be a document titled *Recommended Ramp Design Procedures for Facilities without Frontage Roads*. This document will provide technical guidance for engineers who desire to design safe and efficient interchange ramps on facilities without frontage roads. It will also provide guidelines for: (1) selecting the most appropriate software product for modeling ramp traffic operations, and (2) using this product to evaluate alternative ramp designs. This document will be developed in the second year of the research project.

CHAPTER 2. REVIEW OF RAMP DESIGN PRACTICE

OVERVIEW

This chapter describes the more common interchange types, and associated ramp configurations, used on facilities without frontage roads. The focus of the chapter is the operational and safety effects of interchange ramp design alternatives. The chapter also includes a review of the design standards used by several state departments of transportation (DOTs) and the design policy documents produced by the American Association of State Highway and Transportation Officials (AASHTO) and by the Transportation Association of Canada (TAC). The chapter concludes with a synthesis of TxDOT ramp design practice, as ascertained during a series of meetings held at selected TxDOT district offices.

INTERCHANGE TYPES

The AASHTO document, *A Policy on Geometric Design of Highways and Streets* (*1*) (*Green Book*) identifies several viable interchange types that can be used when frontage roads are not present. Commonly used types are illustrated in Figure 2-1. Of these types, only the diamond and the single-point urban interchange (SPUI) have been adapted to one-way frontage-road facilities.

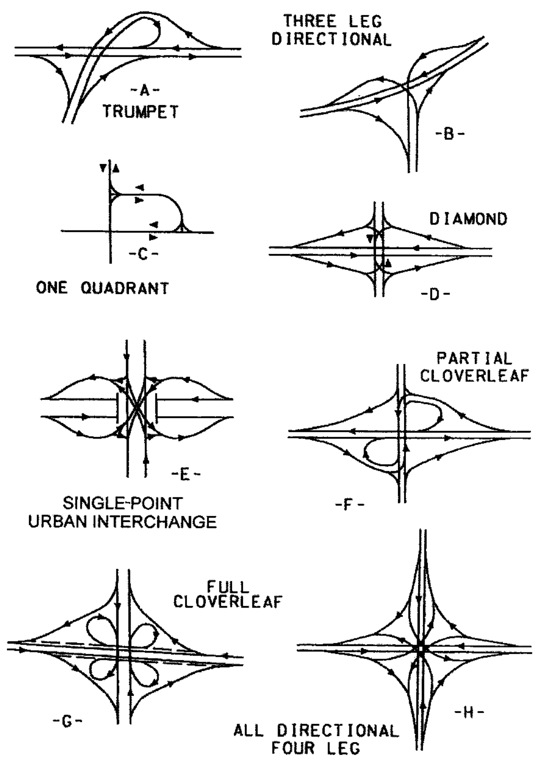


Figure 2-1. Typical Interchange Types. (*1*)

The interchange types shown in Figure 2-1 can be categorized by their functional category, the type of intersecting facility, and the number of legs at the interchange. Table 2-1 identifies these categorizations for the interchanges listed in Figure 2-1. In general, service interchanges have some type of stop or signal control on their ramp terminals. These interchanges are most appropriate at locations where the intersecting facility is classified as a local, collector, or arterial. System interchanges serve all turning movements without traffic control and, hence, are used for the intersection of two freeway facilities.

Table 2-1. Interchange Classification.

Functional Category	Type of Intersecting Facility	Interchange Legs	
		Three-Leg Interchange	**Four-Leg Interchange**
Service Interchange	Local Collector Arterial	Trumpet Diamond (Directional occasionally used)	One quadrant Diamond SPUI Partial cloverleaf (parclo)
System Interchange	Freeway	Directional (Trumpet occasionally used)	Full cloverleaf (rarely used today) All directional without loops All directional w/loops (not shown)

A survey of state DOTs was conducted by Garber and Fontaine *(2)* for the purpose of developing guidelines for interchange selection. They received completed surveys from 36 of the 50 state DOTs. One survey question related to the types of interchanges being used in the respondent's jurisdiction. The responses to this question are summarized in Figure 2-2.

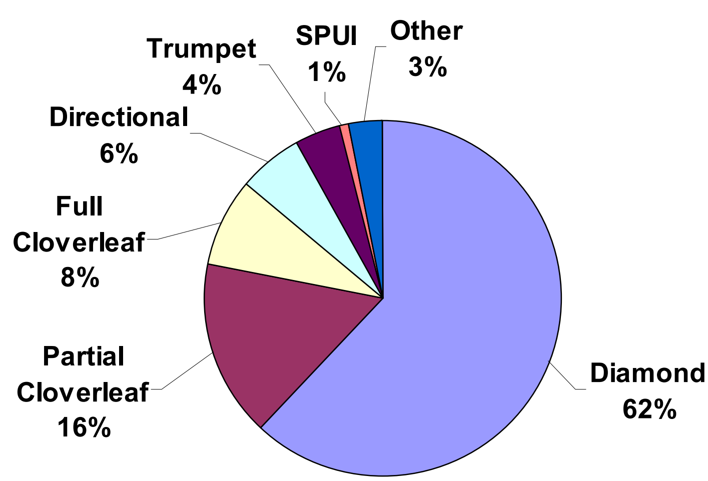


Figure 2-2. Distribution of Interchange Types Used in the U.S.

The trends in Figure 2-2 indicate that the diamond is the most widely used interchange type, followed by the partial cloverleaf (or "parclo"). Together, these two interchange types account for 78 percent of all interchanges. In contrast, the SPUI represents the interchange type that is least used. It is likely that this trend reflects the fact that the SPUI configuration is relatively new and that it is most cost-effective in urban areas with dense development.

Diamond and Parclo Interchange Types

As shown previously in Figure 2-2, the two most common types of service interchange are the diamond and parclo. Typical variations of both types are shown in Figure 2-3.

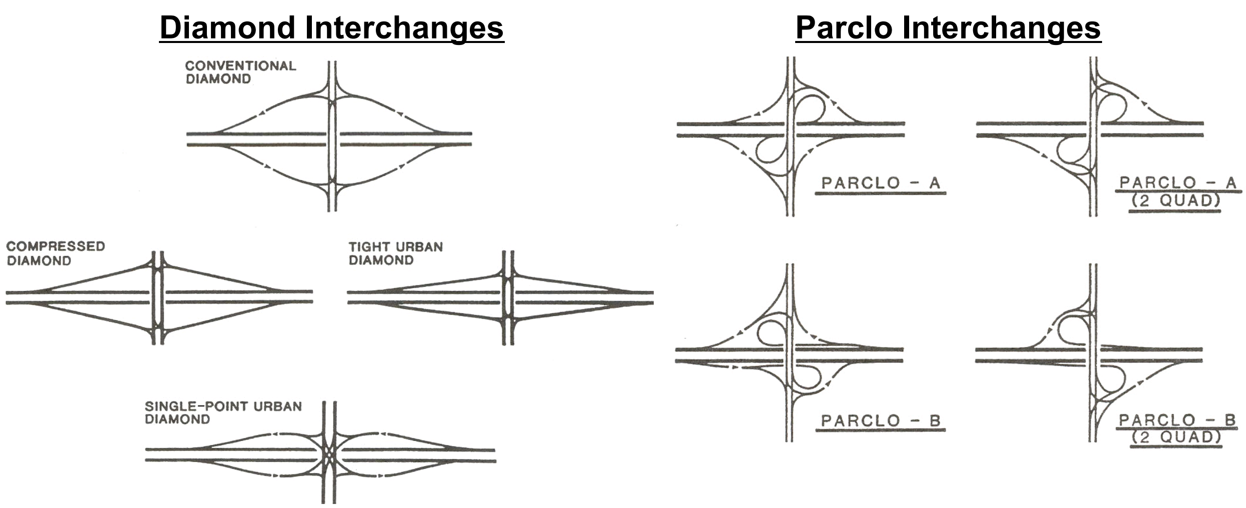


Figure 2-3. Interchange Types Commonly Used in Non-Frontage-Road Settings. (*3*)

The diamond and parclo interchanges can be further categorized by their ramp separation distance, ramp geometry, ramp control mode, and crossroad cross section. A report by the Transportation Research Board (*3*) describes these characteristics for typical interchange types. The characteristics offered in this document for the diamond interchange are included in Table 2-2. Characteristics offered by Messer et al. (*4*) and by Copas and Pennock (*5*) are also included.

As indicated in Table 2-2, 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.

Table 2-2. Characteristics of Typical Diamond Interchanges.

Category		Diamond Interchange Type [1, 2]			
		Conventional	Compressed	Tight Urban	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.[3]	**External to intersection.**
	Length	200 to 300 ft bay.	150 to 300 ft bay.	Parallel bays, if needed.[3]	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 - Table content based on information provided in References *3*, *4*, and *5*.

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

3 - 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.

In urbanized areas, the tight urban diamond interchange (TUDI) and the SPUI can provide very 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 (*4*). As a result, the compressed diamond interchange is better suited to rural or suburban settings where traffic demands are low to moderate.

Table 2-2 indicates that typical ramp separation distances for the TUDI range from 200 to 400 ft. However, the TxDOT *Roadway Design Manual* (*6*, p. 3-82) recommends a minimum ramp separation distance of 300 ft for the TUDI. This recommendation effectively eliminates ramp separations between 200 and 300 ft—distances that have been found to yield very efficient TUDI operation.

The parclos shown in Figure 2-3 are most applicable to situations where a specific left-turn movement pair has sufficiently high volume as to have a significant negative impact on ramp terminal operation (*3*). 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 provide uncontrolled service for major-road drivers intending to turn left at the interchange.

The parclo A and parclo B are more efficient forms than their "2-quad" counterparts. This feature is due to their elimination of one left-turn movement from the ramp terminal signalization. In fact, the *Geometric Design Guide for Canadian Roads* (*7*, p. 2.4.5.9) indicates that the parclo A is "the most effective interchange between a freeway and an arterial road. It has superior operational qualities and...is preferable to all others."

The parclo A has several advantages over other interchange types. These advantages have been documented in the *Geometric Design Guide for Canadian Roads* (*7*). 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, which can reduce the width of the bridge deck. 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 (*7*). One advantage is that its signalized ramp terminals do not require coordination because the outbound travel direction can be designed to receive 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 movements.

Overpass vs. Underpass

The elimination of frontage roads increases the range of options available when developing the major-road and crossroad profiles through the interchange. This added flexibility can translate into more cost-effective designs because the designer can tailor the ramp configuration solely to the topography and the interchanging traffic movements. 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 2-4 should be considered when selecting an overpass or underpass design. They also provide some insight as to the merit of locating the crossroad below, at, or above the existing ground level.

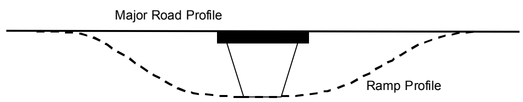
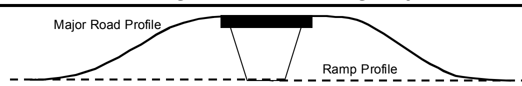
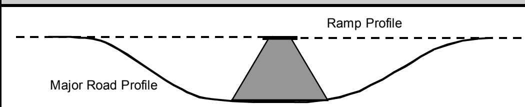
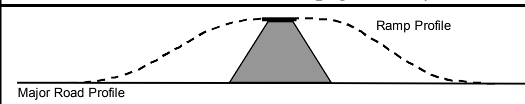
Table 2-3. Characteristics of Typical Partial Cloverleaf Interchanges.

Category		Parclo Interchange Type[1,2]			
		Parclo A	**Parclo B**	**Parclo A (2-quad)**	**Parclo B (2-quad)**
Ramp Separation[3] (centerline to centerline)		700 to 1000 ft	700 to 1000 ft	700 to 1000 ft	700 to 1000 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 - Table content based on information provided in References *3* and *5*.
2 - Characteristics in **bold** font are generally recognized as advantageous in terms of operations, safety, or cost.
3 - Ramp separation distances listed are based on 170-ft loop radii (25-mph design speed). Distances shown should be increased by 300 ft if a 250-ft loop radii (30-mph design speed) is used for the loop ramps.

The information in Table 2-4 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.

Table 2-4. Advantages of the Overpass and Underpass Configurations.[1]

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.

Notes:
1 - Advantages cited in Reference *1*, pp. 763 to 764.

Operational Comparison of Alternative Types

As noted previously, the elimination of the frontage road alignment increases the designer's options in terms of choosing the most appropriate interchange type. This flexibility is helpful *if* the designer takes the initiative to consider the full range of interchange options and to determine the most cost-effective type. Unfortunately, quantitative guidance to aid in this selection is not readily available in standard reference documents.

Several researchers (*2, 3, 4, 8, 9*) have developed useful characterizations about the operational performance of several commonly used service interchanges. Some of these characterizations are based on the researcher's experience while others are based on the use of analytic or simulation-based models. The characterizations based on the use of models tend to quantify the performance of alternative interchange types over a range of traffic volumes.

The characterizations offered by Bonneson and Lee (*8*) are shown in Figure 2-4. The trends shown relate the sum-of-critical-flow-ratios to delay for interchanges in frontage-road settings. A "critical flow ratio" is defined as the largest ratio of volume to saturation flow rate for each traffic movement served during a signal phase. The critical flow ratio for each phase is then added to obtain the sum-of-critical-flow-ratios. An equation for computing this sum is provided by Bonneson and Lee (*8*). Interchange delay is the total delay to all traffic movements at both interchange ramp terminals divided by just the 12 basic movements entering the interchange (i.e., left turn, through, and right turn for each of four external approaches). Bonneson and Lee indicate that this statistic is unbiased by interchange type or ramp separation distance.

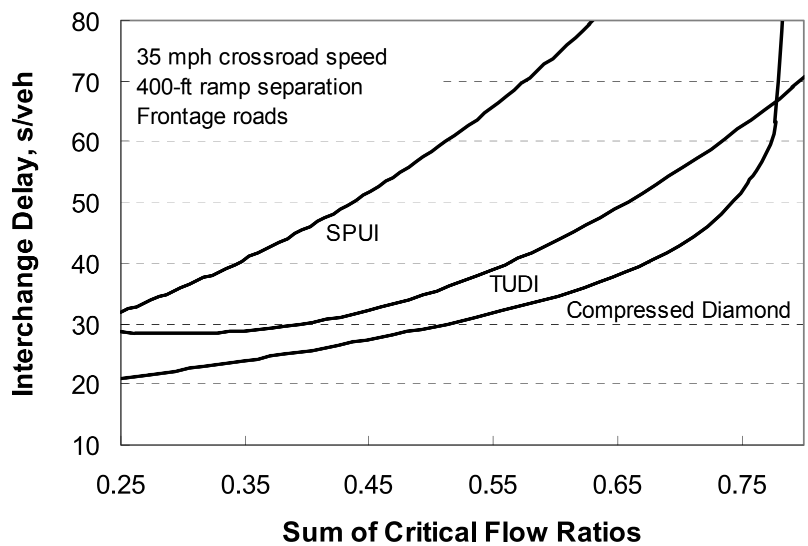


Figure 2-4. Operational Performance Comparison of Interchanges with Frontage Roads.

The trends in Figure 2-4 suggest that the TUDI and compressed diamond operate more efficiently than the SPUI when frontage roads are present. However, the opposite trend was found by Messer et al. (*4*) for interchanges in non-frontage-road settings. Specifically, they found that the SPUI was more efficient than other diamond interchanges in non-frontage-road settings. Garber and Fontaine (*2*) also found that the SPUI was more efficient than other diamond interchanges in non-frontage-road settings. They also found that the parclo was slightly more efficient than the SPUI, regardless of traffic volume.

Selection Considerations

Interchange selection for a specific location generally requires consideration of a wide range of factors. Leisch (*9*) indicates these factors include:

- classification of intersecting facilities,
- volume and pattern of existing and future traffic,
- environmental requirements,
- local access and circulation considerations,
- physical constraints and right-of-way considerations,
- construction and maintenance costs, and
- road-user costs (i.e., costs related to safety and operations).

The first two factors provide direction on the basic interchange forms that are viable for specific facility classes and area types. The manner by which they can be considered is shown in Figure 2-5. For each interchange shown, the major road is shown with a horizontal orientation.

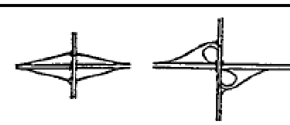
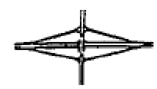
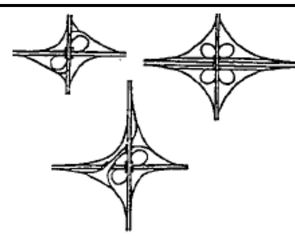
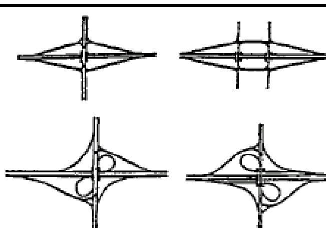
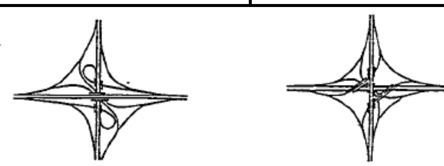
Functional Category	Intersecting Facility	Area Type	
		Rural	**Urban**
Service Interchange	Local Road or Street		
	Collectors and Arterials		
System Interchange	Freeway		

Figure 2-5. Interchange Types Amenable to Various Facility Classes and Area Types. (*9*)

The full range of interchange types is not shown in Figure 2-5. Other interchange types or geometric variations of the types shown in the figure may be appropriate in specific situations. For example, the SPUI is most appropriate for the urban area type where the intersecting facility is an arterial street. The parclo A (2-quad) is probably best suited to rural settings where the intersecting facility is a local road.

Right-of-way, construction cost, safety, and operations are likely to dictate interchange type selection for any specific location. In fact, in most instances only two or three interchange types are likely to satisfactorily accommodate these factors (*9*). Of the three, Holzmann and Marek (*10*, p. 93) indicate that "right-of-way restrictions are some of the most common conditions that influence the ultimate interchange layout" in urban areas. They also note that the "presence or absence of frontage roads within the right-of-way may also have a major influence on the final layout."

In rural areas, the *Green Book* (*1*, p. 807) indicates that interchange type selection is based primarily on traffic demand, especially turn movement demands. However, it is implied in the text of the *Green Book* that safety and cost are also important considerations.

RAMP CONFIGURATIONS

Alternative Ramp Configurations

This section describes the various ramp configurations that are available for interchanges in non-frontage-road settings. Figure 2-6 illustrates five basic ramp configurations within which most ramps can be broadly classified (*1*, p. 827). The ramp configurations used in frontage road settings are not shown, but can be described as slip, buttonhook, and scissor (*11*).

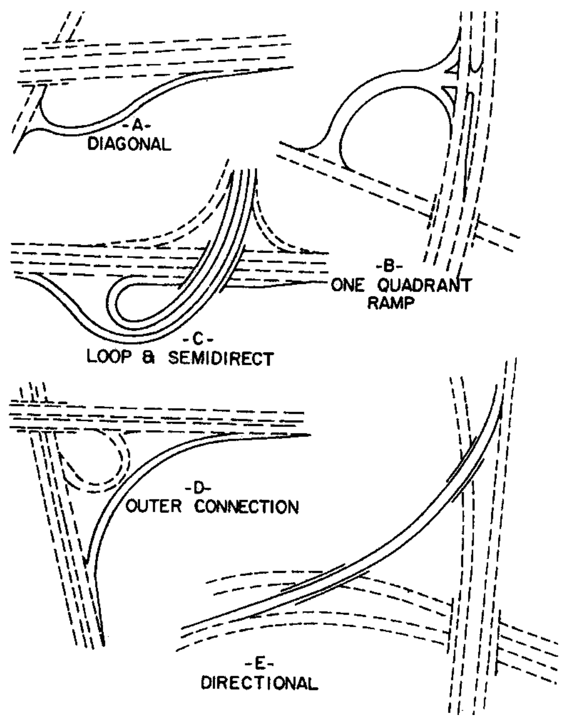


Figure 2-6. Alternative Ramp Configurations. (*1*)

When frontage roads are not present, the designer has a wider selection of alternative ramp configurations to consider. This diversity results because several restrictions posed by frontage roads (e.g., the width of the interchange, the need to provide a through traffic connection at the crossroad, etc.) are removed. Ultimately, it can benefit motorists because the designer is afforded additional flexibility in adapting the ramp to the topography and to traffic demands.

Ramp configuration is defined by the following characteristics: major-road access orientation (i.e., entrance or exit), controlling curvature, turn movements supported, alignment deflection, and capacity. These characteristics are described in Table 2-5 for several ramp configurations used with service interchanges. In general, the diagonal and outer connection ramps have the desirable features of a higher design speed, larger controlling radius, smaller total alignment deflection angle, and higher capacity.

Table 2-5. Characteristics of Typical Ramps Used with Service Interchanges.[1]

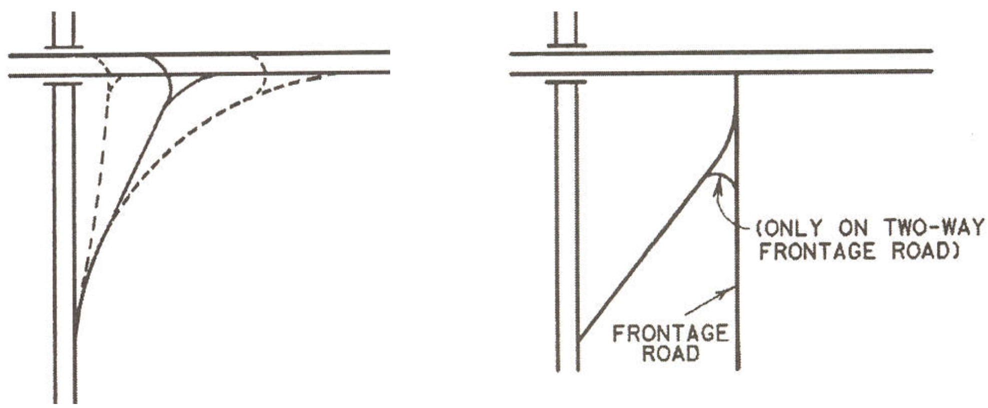
Characteristic		Ramp Configuration					
		Diagonal		Outer Connection		Loop	
		Exit	Entrance	Exit	Entrance	Exit	Entrance
Interchange Type	Diamond	✔C	✔M				
	Parclo A	✔C			✔M		✔M
	Parclo A (2-quad)	✔C					✔M
	Parclo B		✔M	✔M		✔M	
	Parclo B (2-quad)		✔M			✔C	
Ramp Design Speed (% of highway design speed)		70%		85%		50%	
Ramp Design Speed Based on a 65-mph Highway Design Speed		45 mph		55 mph		25, 30 mph	
Controlling Radius[2] (for ramp design speed listed above)		540 ft (Exhibit 3-43)		965 ft (Exhibit 3-14)		150, 230 ft (Exhibit 3-43)	
Total Alignment Deflection,[3] deg		90°		90°		270°	
Capacity of Ramp Proper,[4] veh/h/ln		2100		2200		1800 to 1900	

Notes:
1 - ✔ denotes the typical ramp configuration for the exit and entrance ramp associated with a specific interchange type. "M" denotes ramps whose capacity is dictated by the merge area and, thereby, limited to 1000 to 1200 veh/h/ln. "C" denotes ramps whose capacity is dictated by the control mode at the ramp's intersection with the crossroad.
2 - Controlling radii obtained from exhibits indicated in Reference *1*. A maximum superelevation rate of 8.0 percent was used with Exhibit 3-14 to obtain the controlling radius for the outer connection ramp.
3 - Sum of curve deflections along ramp proper.
4 - Capacity values are based on Exhibit 25-3 from Reference *12*.

The capacity of the ramp proper for each configuration is shown in the last row of Table 2-5. The capacity values shown are based on the assumption that at least one curve on the ramp has a radius equal to the controlling radius. The capacity of the ramp proper rarely controls the capacity of the overall ramp. Rather, the capacity of the diagonal exit ramp is typically dictated by the control mode at the ramp's intersection with the crossroad. Similarly, the loop exit ramp capacity of the parclo B (2-quad) is dictated by the control mode at the ramp intersection. The capacity of all entrance ramps is dictated by the capacity of the ramp merge area. Typically, the capacity of this merge limits ramp capacity to a value of 1000 to 1200 veh/h/ln (*5*, *13*). The loop exit ramp at the parclo B is also dictated by the capacity of the merge area.

Diagonal Configuration

Figure 2-7a illustrates the diagonal ramp configuration commonly used with the diamond interchange. It also illustrates the flexibility this ramp has with regard to the location of the ramp terminal at the crossroad (relative to the frontage road ramp in Figure 2-7b). Ramp location can be described in terms of its "offset" distance, which is the distance between the ramp terminal and the major road as measured along the crossroad. Wider offset distances are associated with longer ramps, flatter angles of entry for the right-turn movement onto the crossroad, flatter grades, and improved traffic flow between the two interchange ramp intersections.



a. Diagonal Ramp. **b. Diagonal (Slip) Ramp at Frontage Road.**

Figure 2-7. Alternative Diagonal Ramp Configurations. (*1*)

The diagonal (or slip) ramp in a frontage road situation is shown in Figure 2-7b. This ramp ends at its junction with the frontage road. The distance between this junction and the crossroad is dictated by considerations of queue storage and weaving on the frontage road. Research by Fitzpatrick et al. (*14*) and by Jacobson et al. (*15*) indicates that such distances should range from 300 to 1500 ft for storage and 300 to 500 ft for weaving. During the highest demand hours of the day, the weave distance may actually be used as additional queue storage and may prevent spillback onto the freeway.

Significant weaving does not occur on exit ramps in non-frontage-road settings. As a result, these exit ramps can have a shorter overall length (as measured from exit gore to crossroad) than an exit ramp in a frontage-road setting. While there can be advantages derived by this reduced length, the designer must be mindful to ensure that the non-frontage-road exit ramp has sufficient length to prevent spillback onto the major road during peak traffic hours.

In a non-frontage-road setting, the ramp terminal legs can be designed with or without skew at the crossroad intersection. As shown in Figure 2-8, a straight alignment can be used with a taper-type exit ramp with a 5-degree skew angle at the ramp terminal. This small amount of skew can eliminate curvature in the ramp, simplify the design, and eliminate curve-related crashes.

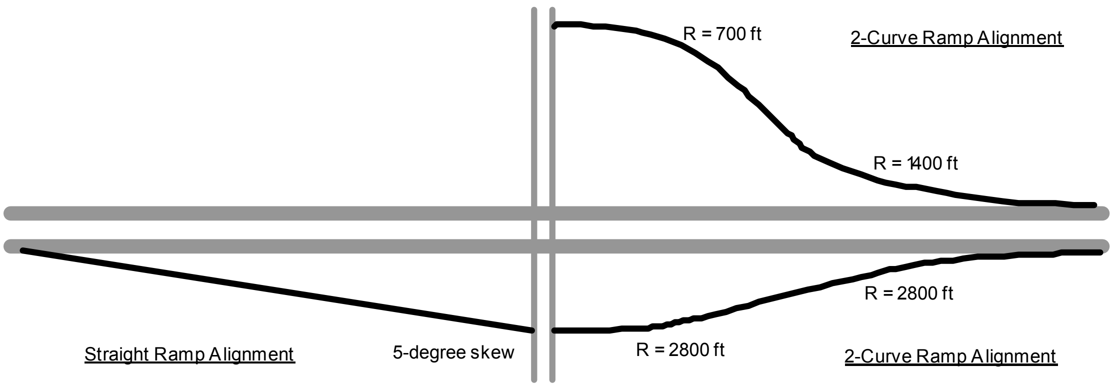


Figure 2-8. Three Diagonal Ramp Alignment Options.

Skew in the intersecting alignments at an intersection can increase crash frequency by making it more difficult for ramp drivers attempting to yield or right-turn-on-red to view crossroad traffic. As a result, a “2-curve” ramp alignment is often used to eliminate skew at the intersection. Two variations of the “2-curve” design exist. One variation uses large radii (e.g., 2800 ft radius) and a minimal ramp offset distance. A second variation uses sharper radii and a wide ramp offset distance. This variation is often used in rural areas to separate the ramp terminals and, sometimes, to accommodate a loop ramp in the same quadrant.

Loop Configuration

Figure 2-6c (and 2-6d with dashed lines) illustrates the loop ramp configuration. Loop ramps are often used at cloverleaf interchanges. They can be designed using a constant radius for the entire length of curve, or a 400- to 500-ft spiral can be used to transition to and from a 150- to 240-ft curve of constant radius (*13*). Loop ramps are often used in conjunction with collector-distributor roads along the major road to ensure that the design is consistent with driver expectancy, to minimize weaving on the major road, and to provide for a safe speed change for vehicles traveling between the intersecting streets.

The loop ramp configuration is a viable alternative when sufficient right-of-way is available to accommodate the loop roadway. The loop design converts a stop or signal-controlled left-turn movement at the ramp terminal into a merge or yield-controlled movement on a loop ramp. This "control-and-movement reversal" can be beneficial when left-turn volumes from either the major road or the crossroad are high. Moreover, when used in combination with outer connection ramps, the loop ramp can eliminate one signal phase at the ramp terminal intersection with a corresponding benefit to traffic operation.

Operational Comparison of Diagonal and Loop Ramps

Quantitative guidelines indicating when the diagonal ramp configuration is operationally superior to the loop ramp configuration are not available in the literature. The capacity of the two configurations is quite similar (see Table 2-5). However, the operating speed tends to differ significantly among the two configurations with loop ramps tending to be associated with lower speeds than diagonal ramps. This lower speed combines with the loop's longer length (i.e., the loop is longer than the diagonal ramp by the length of the loop) to yield a longer running time on a loop ramp than on a diagonal ramp (*1*, p. 792). This trend is illustrated in Figure 2-9. It is based on the relationship between speed and radius shown in Exhibit 3-44 of the *Green Book* (*1*, p. 202).

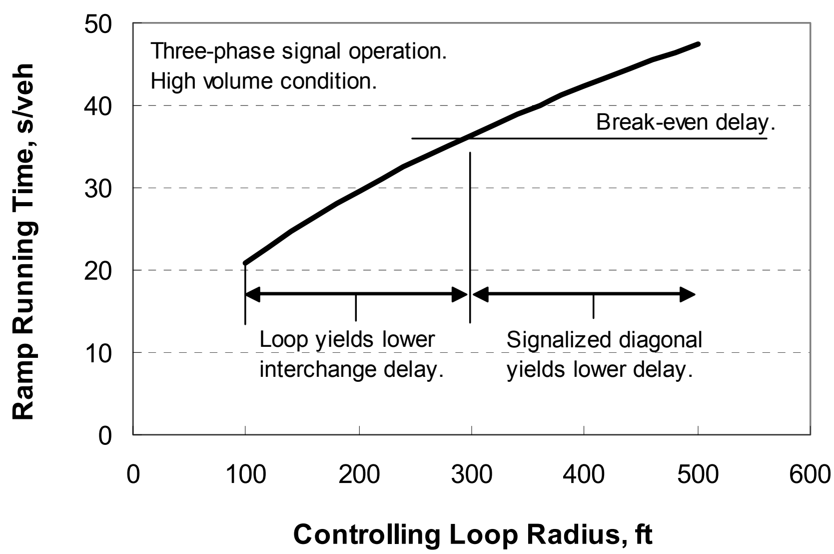


Figure 2-9. Relationship between Loop Radius and Running Time.

Although the trend in Figure 2-9 indicates that running time increases with radius, the most equitable comparison of the two ramp configurations is based on "travel" time wherein both the ramp running time and the delay incurred at the ramp terminal are considered. Specifically, comparisons between a diagonal ramp and a loop ramp should also include the additional delay incurred if one configuration has a change in control mode at the ramp terminal.

To illustrate the use of Figure 2-9, consider a comparison of diagonal ramps with loop ramps in the context of a conversion from the diamond to a parclo B. At the parclo, the ramp conversion eliminates the delay due to a stop- or signal-controlled left-turn at the diagonal ramp terminal but it adds the running time incurred on the loop. In other words, the travel time increase on the loop ramp is "traded" for elimination of delay at the signal. For typical three-phase signal operation under relatively high-volume conditions, the signal-related delay incurred on the diagonal ramp is about 30 to 40 s. This delay represents the "break-even" delay that defines the largest radius for which an operational benefit is derived by the use of a loop ramp. The use of a larger loop radius would yield more delay than if a diagonal ramp were used. According to Figure 2-9, the corresponding break-even radius is 300 ft. The *Green Book* (*1*, p. 792) indicates that a 250-ft break-even radius is often found when the cost of right-of-way is also considered.

Safety Comparison of Alternative Configurations

Several previous studies have examined the frequency and severity of crashes at interchanges. In general, these examinations focused on the individual ramps as opposed to the overall interchange. This focus is due to the unique influence of individual ramp element design on crash potential. The findings of many of these studies are summarized by Twomey et al. (*16*). For example, they reported crash rates comparing "curved" and "straight" ramp alignments. Their data indicate that curved ramps have 14 percent more crashes than straight ramps.

Twomey et al. (*16*) also summarized crash rates for several ramp configurations. The relationships between volume and crash frequency implied by these rates are illustrated in Figure 2-10. The trends in this figure are applicable to exit ramp configurations. They indicate that the buttonhook ramp design used with frontage roads has the highest crash rate. In contrast, the loop ramp with a free-flow left-turn movement has the lowest crash rate. It should be noted that the trend line labeled "free-flow loop" is actually identified as "cloverleaf loops with collector-distributor roads" in the Twomey et al. report. However, the label "free-flow loop" is believed to be equivalent given the discussion in the report.

Crash rates for entrance ramps were also reported by Twomey et al. (*16*). These rates indicate that exit ramps have about 35 percent more crashes than entrance ramps. Alternatively, the rates indicate that the ratio of entrance to exit ramp crashes is 0.74:1. This value is not consistent among the alternative ramp configurations; however, all entrance ramps experienced fewer crashes than similarly shaped exit ramps. Specifically, the diagonal ramp has an entrance-to-exit ratio of 0.60:1; the button hook has a ratio of 0.67:1; the outer connection has a ratio of 0.73:1; and the free-flow loop is relatively indifferent with a ratio of 0.95:1.

A more recent study of ramp crash frequency was completed by Bauer and Harwood (*11*). They used rigorous statistical modeling techniques to quantify the relationship between crash frequency and ramp volume. Their models revealed a non-linear relationship between volume and crash frequency. The crash prediction equation they developed for exit ramps in rural settings was used to develop the trend lines shown in Figure 2-11.

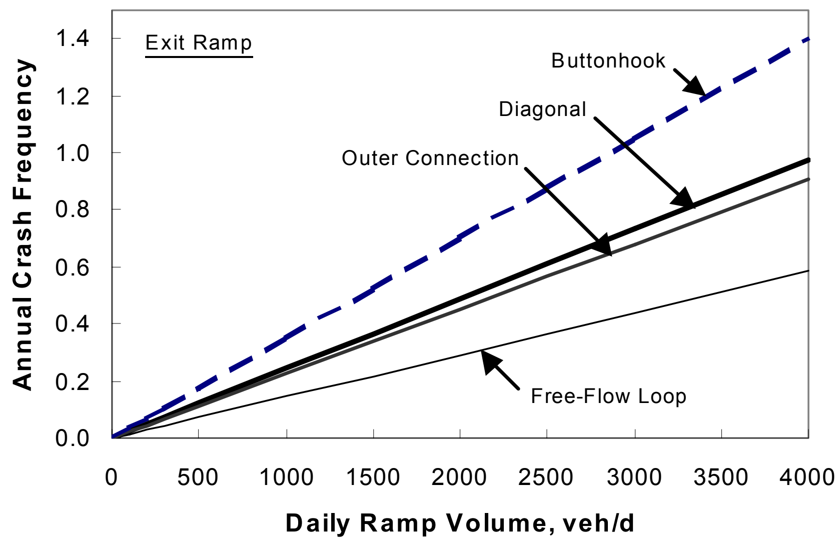


Figure 2-10. Effect of Ramp Volume on Crash Frequency at Exit Ramps.

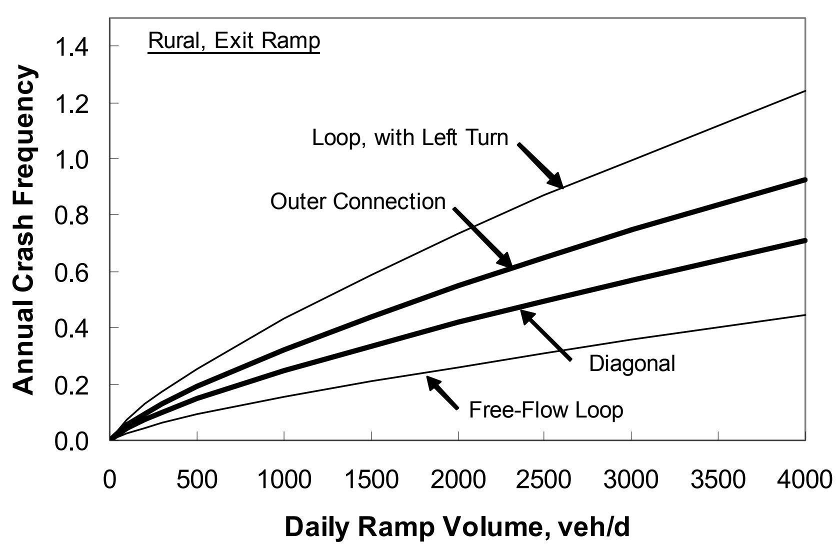


Figure 2-11. Effect of Ramp Volume on Crash Frequency at Rural Exit Ramps.

With the exception of the non-linear relationship between volume and crash frequency, the trends shown in Figure 2-11 are very consistent with those shown in Figure 2-10. The free-flow loop ramp has the lowest crash frequency, regardless of volume. The loop with a controlled left-turn movement (as would be found at a two-quadrant parclo) has the highest crash frequency. Comparing Figures 2-10 and 2-11, it appears that the "loop with left turn" is associated with fewer crashes than the buttonhook ramp.

The models reported by Bauer and Harwood (*11*) indicate that ramps in urban areas have 40 percent more crashes than those in rural areas, given the same volume level and configuration. Bauer and Harwood also found that entrance ramps were associated with fewer crashes than exit ramps. This trend is consistent with that found in the crash rates reported by Twomey et al. (*16*). Specifically, Bauer and Harwood found that exit ramps have about 65 percent more crashes than entrance ramps. This percentage is equivalent to a 0.61:1 ratio of entrance to exit ramp crashes.

NON-FRONTAGE-ROAD RAMP DESIGN ISSUES

The objective of this section is to: (1) identify the elements of ramp design that may vary, depending on whether frontage roads do or do not exist, and (2) synthesize the design guidelines reported in authoritative reference documents and the issues addressed in research journals. The issues raised in this section will be used in the second year of this research project to guide the development of ramp design procedures for facilities without frontage roads.

Review of the *Roadway Design Manual*

The TxDOT *Roadway Design Manual* (*6*) was reviewed to identify areas where additional guidance could be provided to facilitate ramp design for non-frontage-road facilities. This review focused on Chapter 3 of the *Roadway Design Manual*; however, there were some references to guidance provided in Chapter 2. The findings from this review are summarized in Table 2-6.

The check marks in column 2 of Table 2-6 suggest that new content would be useful in several sections of Chapter 3 of the *Roadway Design Manual*. The titles of these sections are listed in column 1. In general, additional design detail sheets could be added to illustrate the design considerations for a wider range of ramp configurations. Also, discussion is needed regarding the design of the ramp's horizontal geometry to accommodate peak period traffic queues and ramp metering operations. Finally, more information is needed in Chapter 3 regarding ramp terminal design for non-frontage-road applications. The nature of the information in each of these topic areas is discussed in the remainder of this chapter.

General Information

The example ramp design detail sheets in Chapter 3 of the *Roadway Design Manual* address slip ramp, buttonhook, and scissor ramp configurations. There is also one diagonal ramp detail sheet. Detail sheets for the one-quadrant, loop, and outer connection configurations should be added for use in non-frontage-road situations.

Table 2-6. Ramp Design Guidelines in the *Roadway Design Manual*.

Topic Area[1]	New Content May Be Needed for NFR Interchanges[2]	Type of NFR Content that Could be Added[2]
General Information	✔	Need typical detail figures for one-quadrant, loop, and outer connection ramps.
Design Speed		Coverage is consistent with the *Green Book* and the design manuals of other DOTs.
Horizontal Geometrics	✔	Need information about total length of NFR ramp and its length components (deceleration to controlling curve, deceleration to stop, and storage).
Distance between Successive Ramps		Coverage is consistent with the *Green Book* and the design manuals of other DOTs.
Cross Section and Cross Slopes	✔	Superelevation rates in Table 3-21 should be updated to be consistent with the current edition of the *Green Book*.
Sight Distance		Coverage is consistent with the *Green Book* and the design manuals of other DOTs.
Grades and Profiles		Coverage is consistent with the *Green Book* and the design manuals of other DOTs.
Metered Ramps	✔	Need information about total length of NFR entrance ramp with meter and its length components (decelerate to stop, storage, accelerate to freeway speed)
Collector-Distributor Roads		Coverage is consistent with the *Green Book* and the design manuals of other DOTs.
Frontage Road Turnarounds and Intersection Approaches	✔	More information about NFR ramp terminal design would be beneficial. Topics could include: turning roadway design, storage length, access control, design to discourage wrong-way entry.

Notes:
1 - Topic areas are based on section headings in Chapter 3 of the *Roadway Design Manual* (*6*, p. 3-93 to 3-114).
2 - NFR: non-frontage-road interchanges.

An example of the type of detail that could be provided for the loop and outer connection ramps is shown in Figure 2-12, as adapted from the *Geometric Design Guide for Canadian Roads* (*7*). Design details for both ramps are shown in the context of a parclo A design. The details of ramp terminal design are also shown. This detail is not provided in the *Roadway Design Manual.* Additional detail describing the typical ramp cross section would need to be added to the detail sheet before its incorporation into the *Roadway Design Manual*.

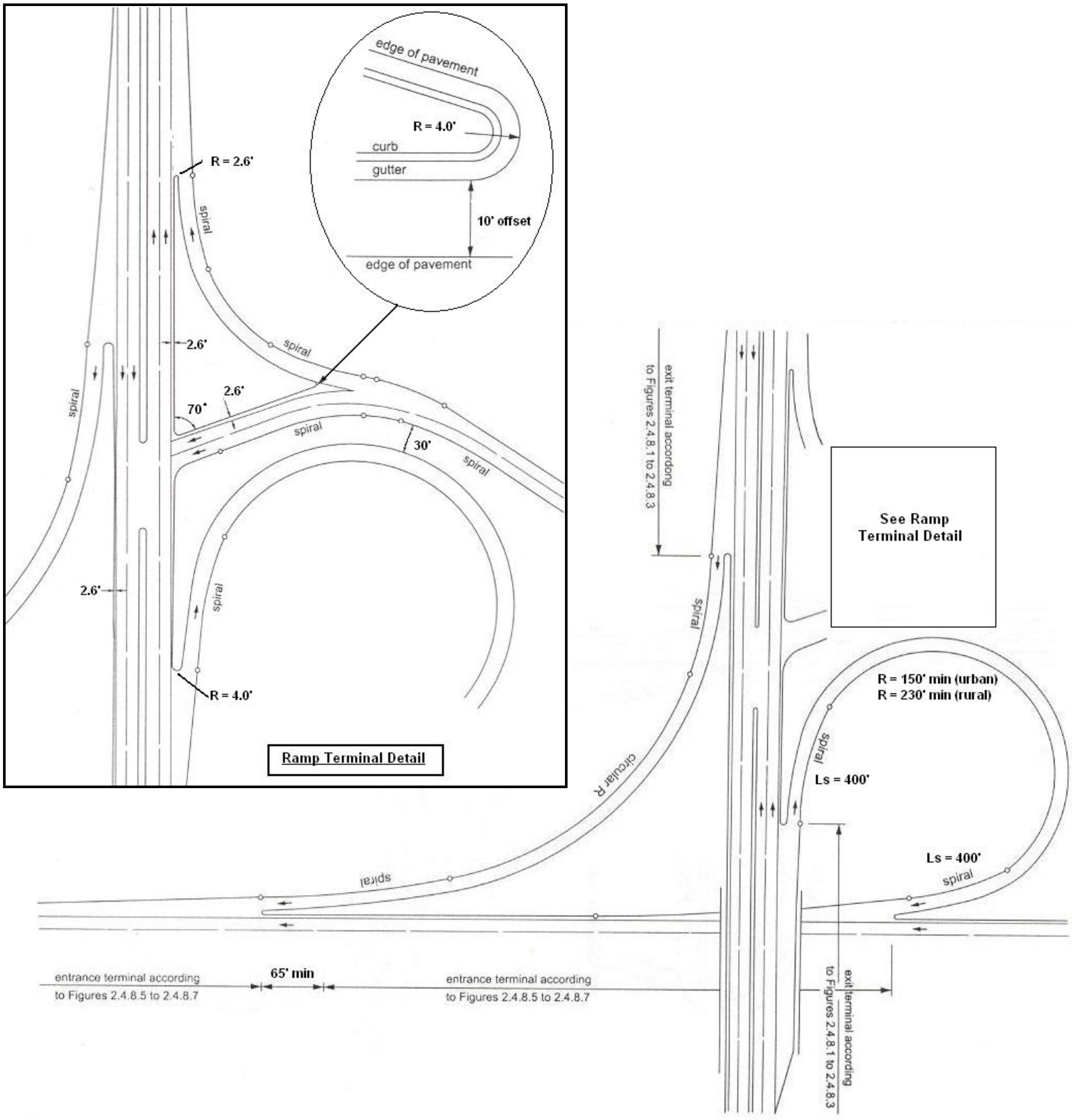


Figure 2-12. Illustrative Parclo A Design Detail. (*7*)

Ramp Proper Design

This section describes the design guidance found in the literature related to the design of the ramp proper. Design topics addressed in this section include: design speed, horizontal geometrics, and cross section. Design guidance for the ramp terminal area is provided in a subsequent section.

Design Speed

The guidance provided in the *Green Book* (*1*, p. 829) for ramp design is summarized in this section, as it relates to the design of ramps for non-frontage-road facilities. This guidance references a table describing the relationship between highway design speed and ramp design speed. This relationship is repeated in Table 3-20 of the *Roadway Design Manual*, which is shown herein as Table 2-7. The *Green Book* recommends the use of "mid" to "upper range" design speeds for outer connection ramps, "middle" range speeds for diagonal ramps, and "lower range" speeds for loop ramps. It recognizes that "upper range" speeds are desirable for all ramps but that cost and operational considerations rarely justify loop design speeds higher than the lower range.

Table 2-7. Ramp Design Speed Guidelines in the *Roadway Design Manual*. (*6*)

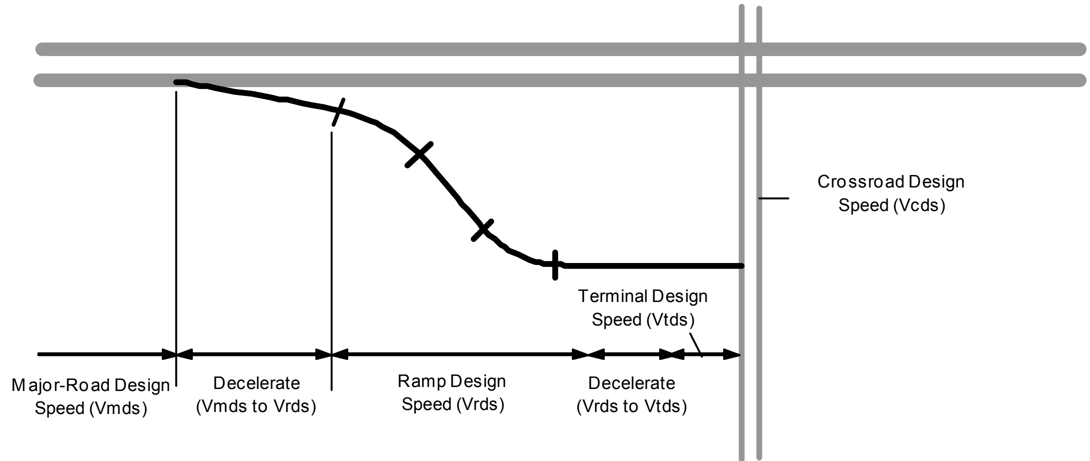
Guide Values for Ramp/Connection Design Speed as Related to Highway Design Speed												
Highway Design Speed, mph:		**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**
Ramp Design Speed[1], mph	Upper Range (85%)	25	30	35	40	45	48	50	55	60	65	70
	Middle Range (70%)	20	25	30	33	35	40	45	45	50	55	60
	Lower Range (50%)	15	18	20	23	25	28	30	30	35	40	45

Note:

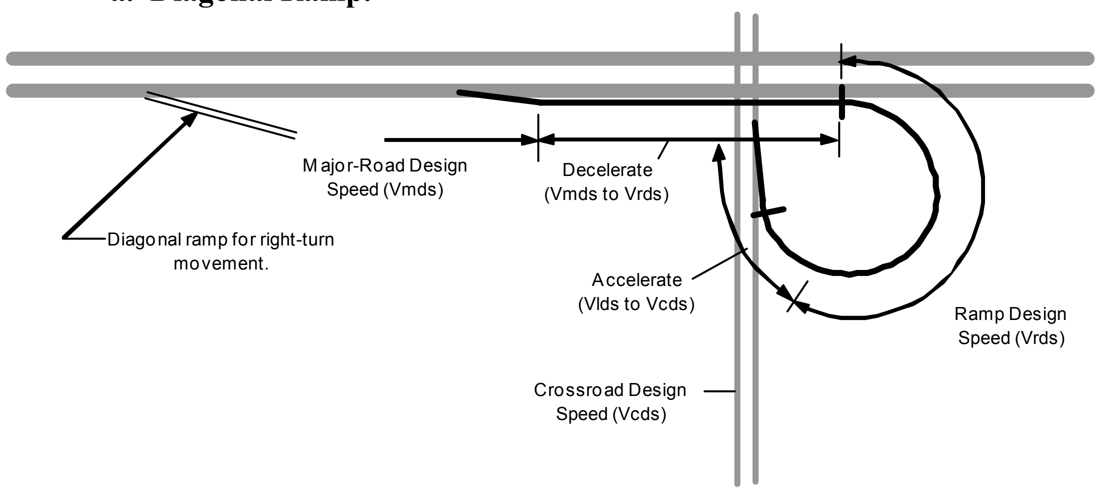
1 - Loops: Upper and middle range values of design speed generally do not apply. The design speed on a loop is usually 25 mph (185-ft minimum radius) based on a maximum superelevation rate e_{max} of 6.0 percent. Particular attention should be given to controlling superelevation on loops due to the tight turning radii and speed limitations.

The *Green Book* (*1*, p. 829-834) also provides general guidance regarding the transition between the design speed of the major road and that of the crossroad or ramp terminal. This guidance indicates that the ramp exit should include an initial length where the driver can decelerate from the major-road design speed to the ramp design speed. The ramp design speed is then maintained until it needs to be transitioned to that of the terminal or, in the case of a free-flow movement, to the design speed of the crossroad. These types of design speed transitions are illustrated in Figure 2-13 for the diagonal and loop exit ramp configurations. Similar relationships would be used for entrance ramps.

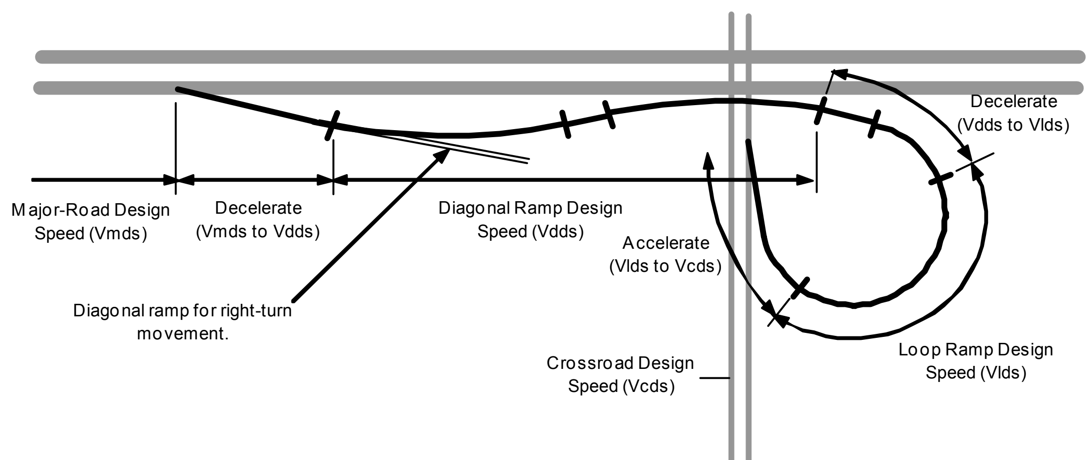
On exit loop ramps, the provision of an adequate length of roadway for deceleration to the ramp design speed is particularly important when the "lower range" design speed is used for the loop ramp. The appropriateness of this range has been examined extensively in light of concerns about truck safety on loop ramps. One examination of ramp speeds by Hunter et al. (*17*) revealed that the 50th percentile ramp speed is consistently greater than the values listed in Table 2-7. Based on this finding, they recommended that TxDOT delete the "lower range" of design speeds from Table 3-20 of the *Roadway Design Manual*.



a. Diagonal Ramp.



b. Free-Flow Loop For Dual-Exit Ramp.



c. Free-Flow Loop For Single-Exit Ramp

Figure 2-13. Design Speed Application to Exit Ramps.

Harwood and Mason (*18*) also evaluated the adequacy of the ramp design speeds provided in Table 2-7. Specifically, they investigated the margin of safety provided by the "lower range" design speeds and associated curve radii. Based on their investigation, they suggested that the "lower range" speeds should be avoided in ramp design if substantial truck volumes are anticipated. Their motivation was based on observations similar to those of Hunter et al. (*17*) (i.e., that most ramp drivers did not drive at, or below, the "lower range" speeds).

Walker (*13*) recommends the use of a single-exit ramp with a two-stage ramp design speed technique to overcome the problems noted by Hunter et al. (*17*) and by Harwood and Mason (*18*). This technique is shown in Figure 2-13c. It would be used instead of the geometry shown in Figure 2-13b. Following this technique, Table 2-7 would be used to define the diagonal ramp design speed based on the middle range speeds and the loop ramp design speed based on the lower range speeds. In this manner, the mild curve initially encountered along the ramp would encourage the driver to slow to the lower range speed before reaching the loop ramp.

Horizontal Geometrics

Ramp Curvature. The *Green Book* (*1*, p. 830) provides recommendations for minimum radii for use with ramp curves. These recommendations are separated into low-speed (45 mph or less) and high-speed design categories. For low-speed design, minimum curve radii on ramps follow the guidelines for turning roadways. For high-speed design, minimum curve radii on ramps follow the guidelines for rural highways and high-speed urban streets.

The *Roadway Design Manual* (*6*) deviates from the aforementioned *Green Book* guidance. Specifically, the *Roadway Design Manual* recommends that minimum radii for ramp design follow that for rural highways and high-speed urban streets, regardless of the design speed category (see the footnote to Table 3-20 of Reference *6*). It does not make the distinction between low- and high-speed design. The result is that the minimum radii recommended in the *Roadway Design Manual* for low-speed ramp design are about 20 percent larger than those recommended by the *Green Book.*

There are three approaches to configuring the loop curve geometry: simple curve radius, flat-sharp-flat compound curve radii, and sharp-flat-sharp compound curve radii. Of these, the flat-sharp-flat design is the most widely used, followed closely by the simple curve design (*19*). The sharp-flat-sharp design is attractive because it can reduce right-of-way requirements; however, it is prone to safety problems because the resulting curve speed sequence is contrary to driver expectancy. Spiral transition curves bracketing the simple curve radius are also available for loop ramp design, as shown in Figure 2-12. However, Keller (*19*) indicates that only about 12 percent of the state DOTs use spiral curves for this purpose.

Exit-Ramp Length. The length of the exit ramp is dictated by many factors including: vertical curvature, design speed transition, and storage requirements. The effect of vertical curvature was noted in a previous section titled Overpass vs. Underpass. In that section, it was stated that provision of stopping sight distance along the ramp or major-road alignments would dictate ramps of 900 to 1100 ft in length, should either the major road or ramps remain at grade. Shorter ramp lengths could be used if the elevation change were partially accommodated by both alignments.

The relationship between ramp length, design speed transition, and storage is illustrated in Figure 2-14 for diagonal ramps with a variety of terminal treatments. The minimum exit-ramp length for the diagonal ramp shown is dependent on the distance needed to decelerate from the major-road design speed (*Vmds*) to a stop condition. Thereafter, an additional length is needed to store the queue (queue storage length is described in a later section titled Ramp Terminal Design). These considerations are summarized in Table 2-8.

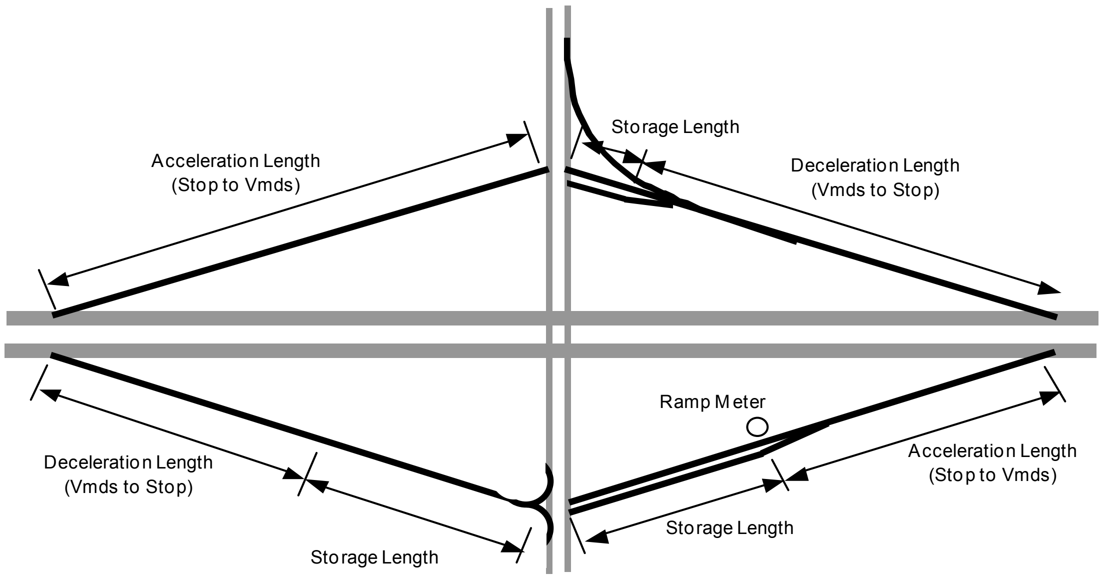


Figure 2-14. Ramp Length Components.

Table 2-8 also lists the minimum ramp length components associated with a free-flow loop ramp and an outer connection ramp. For these two ramp configurations, deceleration length is based on deceleration to the ramp design speed and acceleration (or deceleration as appropriate) to the crossroad speed. Storage is not a consideration for these ramp configurations.

The deceleration lengths recommended in the *Green Book* (*1*) and the *Roadway Design Manual* (*6*) are shown in Table 2-9. These lengths are also consistent with lengths recommended in the *Geometric Design Guide for Canadian Roads* (*7*). However, it should be noted that this guide provides a range of deceleration lengths for which the values in Table 2-9 represent the lower value in each range. This finding suggests that longer lengths (longer by as much as a factor of 2.0) may be desirable, if available. Recent research by Koepke (*20*) also concurs with the need for longer deceleration lengths than those listed in Table 2-9.

Table 2-8. Minimum Ramp Length Based on Speed Change and Storage.

Major-Road Access Orientation	Length Component[1]	Ramp Configuration[2]			
		Diagonal	Outer Connection	Stop or Signal-Controlled Loop	Free-Flow Loop
Exit Ramp	Deceleration	Vmds to Stop	Vmds to Vrds	Vmds to Stop	Vmds to Vrds
	Acceleration	n.a.	Vrds to Vcds	n.a.	Vrds to Vcds
	Storage[3]	Yes	No	Yes	No
Entrance Ramp with Ramp Meter	Acceleration	Stop to Vmds	Stop to Vmds	Stop to Vmds	Stop to Vmds
	Storage[4]	Yes	Yes	Yes	Yes
Entrance Ramp without Meter	Acceleration	Vtds to Vmds	Vcds to Vmds	Vtds to Vmds	Vcds to Vmds
	Storage	No	No	No	No

Notes:

1 - Deceleration and acceleration lengths are listed in Tables 2-9 and 2-10, respectively.

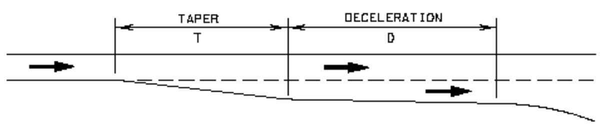
2 - Vmds: major-road design speed; Vtds: terminal design speed; Vrds: ramp design speed; Vcds: crossroad design speed; Stop - stop condition; n.a.: not applicable.

3 - Storage lengths for exit ramps are obtained from Figures 2-16 or 2-17.

4 - Storage lengths for entrance ramps with a ramp meter are listed in Table 2-11.

Table 2-9. Recommended Deceleration Length. *(6)*

Highway Design Speed, mph	Minimum Length of Taper (T), ft	Deceleration Length (D), ft								
		Exit Curve Design Speed, mph								
		Stop Condition	15	20	25	30	35	40	45	50
30	150	235	200	170	140	--	--	--	--	--
35	165	280	250	210	185	150	--	--	--	--
40	180	320	295	265	235	185	155	--	--	--
45	200	385	350	325	295	250	220	--	--	--
50	230	435	405	385	355	315	285	225	175	--
55	250	480	455	440	410	380	350	285	235	--
60	265	530	500	480	460	430	405	350	300	240
65	285	570	540	520	500	470	440	390	340	280
70	300	615	590	570	550	520	490	440	390	340
75	330	660	635	620	600	575	535	490	440	390

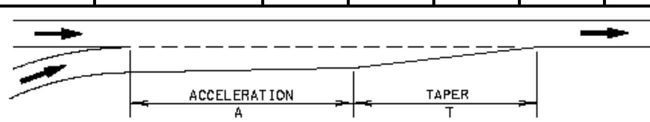


Entrance Ramp Length. As shown in Figure 2-14, the minimum entrance ramp length is dependent on whether the ramp is controlled by a ramp meter. If there is no ramp meter, the minimum entrance ramp length is dependent solely on the distance needed to accelerate the vehicle to the major-road design speed. If there is a ramp meter, then additional storage distance is needed between the meter and the crossroad. The appropriate length components needed to compute the entrance ramp length are listed in Table 2-8.

The acceleration lengths recommended in the *Green Book* (*1*) and the *Roadway Design Manual* (*6*) are shown in Table 2-10. These lengths are also consistent with lengths recommended in the *Geometric Design Guide for Canadian Roads* (*7*). However, it should be noted that this guide provides a range of acceleration lengths for which the values in Table 2-10 tend to be about midway between the upper and lower values in each range. This finding suggests that longer lengths (longer by as much as a factor of 1.3) may be desirable, if available. Recent research by Koepke (*20*) also concurs with the need for longer acceleration lengths than those listed in Table 2-10.

Table 2-10. Recommended Acceleration Length. (*6*)

Highway Design Speed, mph	Minimum Length of Taper (T), ft	Acceleration Length (A), ft								
		Entrance Curve Design Speed, mph								
		Stop Condition	15	20	25	30	35	40	45	50
30	150	180	140	--	--	--	--	--	--	--
35	165	280	220	160	--	--	--	--	--	--
40	180	360	300	270	210	120	--	--	--	--
45	200	560	490	440	380	280	160	--	--	--
50	230	720	660	610	550	450	350	130	--	--
55	250	960	900	810	780	670	550	320	150	--
60	265	1200	1140	1100	1020	910	800	550	420	180
65	285	1410	1350	1310	1220	1120	1000	770	600	370
70	300	1620	1560	1520	1420	1350	1230	1000	820	580
75	330	1790	1730	1630	1580	1510	1420	1160	1040	780



Storage lengths for ramp meters on non-frontage-road facilities was the subject of a survey by Lomax and Fuhs (*21*). Their findings are summarized in columns 2 and 3 of Table 2-11. Examination of this table indicates that the range of distances among state DOTs is quite varied. Desirable storage lengths range from 500 to 1400 ft. In retrofit situations, the range is from 300 to 760 ft. It should be noted that the distance between the meter stop bar and the freeway gore is not directly comparable to the acceleration distance because some acceleration occurs in the speed-change lane associated with the ramp entrance.

Table 2-11. Queue Storage Lengths for Ramp Meter Applications.

State DOT	Section of Entrance Ramp			
	Queue Storage, ft		Stop Bar to Freeway Gore, ft	
	Retrofit	Desirable	Retrofit	Desirable
Arizona[1]	760	1010	340 ft at 40 mph; 450 ft at 45 mph	
California[2, 3]	--	1000	--	660
Colorado[4, 5]	see note 4	see note 4	see note 5	see note 5
Illinois	--	--	300	300
Michigan[6]	--	--	250	250
Minnesota[7]	300	500	250 to 300	400
Oregon	--	--	250	250
Virginia	400	1400	300 to 350	300 to 350
Washington	500	1000	--	700

Notes:
1 - Arizona DOT notes a desirable total distance from the centerline of the crossroad to the freeway gore of 1400 ft.
2 - For three-lane ramps and ramps with peak-hour volume exceeding 1500 vehicles, use a 1000-ft acceleration lane.
3 - Recommends study of individual ramp volumes to determine storage needs; allows use of crossroad for storage.
4 - Provide maximum storage length for each site by putting stop bar as close to the freeway gore as possible.
5 - Put stop bar as close to freeway gore as possible and extend acceleration lane on freeway as needed.
6 - Metered ramps are on a depressed freeway (ramps are on a downgrade approach).
7 - If HOV lane is provided, provide 500 to 600 ft between stop bar and freeway gore (15:1 taper).

Cross Section and Cross Slopes

The *Highway Capacity Manual 2000* (*12*, p. 13-24) indicates that a two-lane ramp proper is needed when the design-hour ramp volume exceeds 1500 veh/h. An earlier edition of the *Highway Capacity Manual* (*22*) provided additional guidelines as to conditions that justify the use of two lanes along the ramp. Specifically, satisfaction of any one of the following three conditions may justify a two-lane ramp, but would not necessarily require a two-lane ramp terminal:

- The ramp is longer than 1000 ft, in which case a two-lane ramp would allow opportunities to pass stalled or slow-moving vehicles.
- Queues are expected to form on the ramp from a signal or stop-controlled ramp terminal, in which case a two-lane ramp would provide additional queue storage.
- The ramp is located on a steep grade or has minimal geometrics, in which case the two-lane ramp would allow opportunities to pass vehicles slowed by the grade or additional accommodation of off tracking by long vehicles.

Ramp Terminal Design

Design Speed

The design of a stop- or signal-controlled ramp terminal is based on the guidelines used for intersection design. In non-frontage-road applications where the through movement does not exist, the ramp terminal design speed is dictated by near-minimum turning speeds. The slowest design speed for right-turning vehicles is 10 mph (*1*, p. 587). Higher speeds can be used for left-turn movements and turning roadways; however, the *Green Book* does not offer specific recommendations on design speed for these conditions.

Traffic Control

The traffic control mode used to regulate traffic at the ramp intersections 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 2-12.

Table 2-12. Exit-Ramp Traffic Control Combinations.

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

Note:
1 - 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.

The design of the ramp terminal approach is influenced by the type of control used for the left- and right-turn movements. The control mode affects queue length, number of lanes, and right-turn design (e.g., stop control–simple radius and no channelizing island, merge control–free right-turn lane with large channelizing island).

Copas and Pennock (*5*) conducted a survey of city and state transportation agencies to identify methods used to determine the most appropriate traffic control for exit-ramp terminals. The response to the survey indicated the unanimous use of the signal warrants provided in Part 4 of the *Manual on Uniform Traffic Control Devices* (*23*). However, other criteria were also cited as reasons for considering signal control. These criteria include:

- to increase ramp capacity and, thereby, prevent spillback from the ramp onto the major road;
- desirably, whenever ramp left-turn volumes exceed 250 veh/h but definitely when they exceed 500 veh/h;

- 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.

Copas and Pennock (*5*) reported that two state DOTs usually install signals at both ramp terminals even if a signal is justified at only one terminal. Finally, they noted that most agencies surveyed consider ramp terminals to be unique types of intersections and that signal warrants should be developed to specifically address interchanges.

Intersection Skew

Ramp terminal design for non-frontage-road settings can have a discontinuous ramp 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. It also introduces skew in the intersecting alignments. Figure 2-15 illustrates the discontinuous alignment of the approach and departure legs at a ramp/crossroad junction.

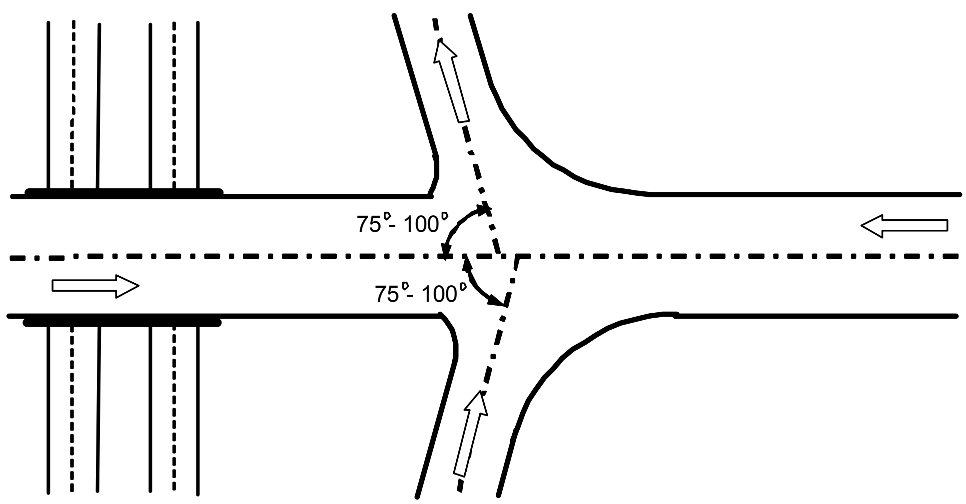


Figure 2-15. Discontinuous Ramp Alignment at Ramp/Crossroad Junction.

The Green Book (*1*, p. 585) recommends that the intersection angle fall in the range of 60 to 120 degrees. Angles larger or smaller than this amount tend to limit the visibility of ramp drivers (especially truck drivers), increase pedestrian crossing distance, and increase the exposure time for left-turn drivers. In recognition of the negative aspects of skew, the California and the Washington DOTs limit intersection angles to a range of 75 to 105 degrees for newly constructed intersections and those undergoing major reconstruction (*24, 25*).

Approach Leg Design

Cross Section. The number of lanes provided on an exit ramp-terminal approach should reflect consideration of traffic control mode, turn volumes, and crossroad through volume. Capacity analysis techniques, such as given in the *Highway Capacity Manual* (*12*) can be used to determine of the number of lanes at both signalized and stop-controlled ramp terminals. The objective when using these techniques is to provide enough lanes on the ramp approach to provide reasonable, and equitable, service to motorists on each approach leg.

Research by Bonneson and Messer (*26*) indicate that driver behavior at interchanges in urban areas is sufficiently aggressive as to yield saturation flow rates (and corresponding capacities) that match those at the highest-type at-grade intersections. Their findings indicated that interchange traffic movements exhibit an ideal saturation flow rate of 2000 pc/h/ln, which is larger than the 1900 pc/h/ln recommended by the *Highway Capacity Manual* (*12*).

Storage Length. Storage length needed on a controlled exit-ramp terminal approach is determined by the control mode, left-turn volumes, crossroad through volume, number of approach traffic lanes, signal timings (if applicable), and desired probability that the storage length will not be exceeded. With regard to the latter probability, the Florida DOT (*27*) recommends defining the minium storage length for design based on a 90-percent probability that the storage length will not be exceeded. Guidelines in Chapter 17 of the *Highway Capacity Manual* (*12*) suggest that storage length should be based on a 95-percent probability level.

The *Green Book* (*1*, p. 718) recommends that storage length at unsignalized intersections should be based on the average number of turning vehicles that arrive in a two-minute period. For signalized intersections, the *Green Book* further recommends a length equal to 1.5 to 2.0 times the average number of vehicles that would store per signal cycle. For both situations, there should always be storage for at least two motorized vehicles. The probability of exceeding the resulting storage length is not specified, nor is the basis for the two-minute period.

Neuman (*28*) offers a variation of the *Green Book* recommendations. For "desirable" designs, he recommends a length sufficient to store 2.0 times the average arrivals per cycle (or per two minutes for an unsignalized intersection). For "minimum" design, he recommends 1.0 times the average arrivals per cycle (or two minutes).

For stop-controlled approaches, Chapter 17 of the *Highway Capacity Manual* (*12*) recommends the use of Figure 2-16. The trends shown in this figure can be used to determine the number of queued vehicles on the ramp approach. These vehicles would be distributed equally among the available ramp lanes to determine the furthest upstream reach of the queue. The count of queued vehicles is converted to a minimum storage length by multiplying it by the average length of lane occupied by a queued vehicle (i.e., 25 ft).

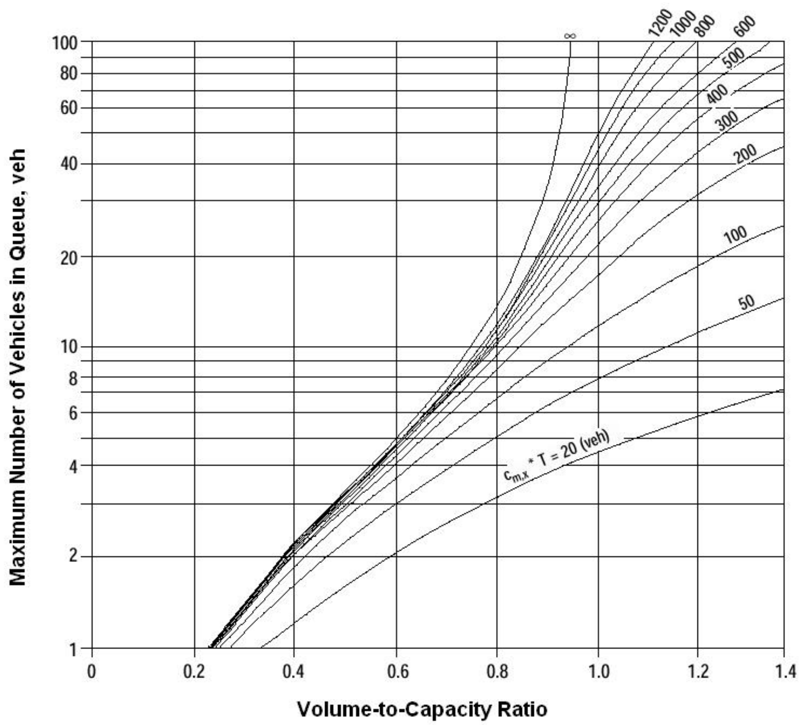


Figure 2-16. 95th Percentile Queue Length for a Stop-Controlled Approach. (*12*)

For signalized intersections, Neuman (*28*) recommends the use of Figure 2-17. This figure was originally developed by Leisch (*29*). It accepts as input, the average volume per lane, cycle length, and truck percentage. The output from the figure is used to directly estimate the minimum storage length for design.

Right-Turn Channelization. At diamond and parclo interchanges, a key design decision is whether to provide right-turn channelization at the ramp terminal. Properly designed channelization can increase capacity and improve safety (*1*). It is even more important at non-frontage-road interchanges because they do not have through traffic lanes that can be shared with the right-turn movement.

From the standpoint of desirable vehicle operation, a turning roadway with free-flow right turns provides the most efficiency. This type of geometry is shown in Figure 2-18 as an "added lane" design. If this design is not provided, then a "merge" design would provide good vehicle operation, provided that the crossroad does not have excessive traffic volume. If some pedestrian activity is present or the crossroad volume is excessive, then the "yield" design can provide acceptable operation and safely accommodate pedestrians. Finally, if pedestrian volumes are significant, then the "stop" design shown in Figure 2-18 is preferred. If vehicular volumes are also high, then a dual right-turn lane with signal control may be needed.

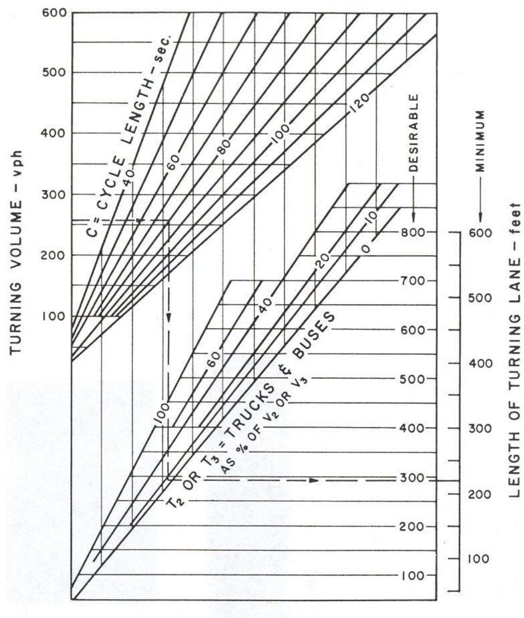


Figure 2-17. Minimum Storage Length for a Signalized Approach. (*29*)

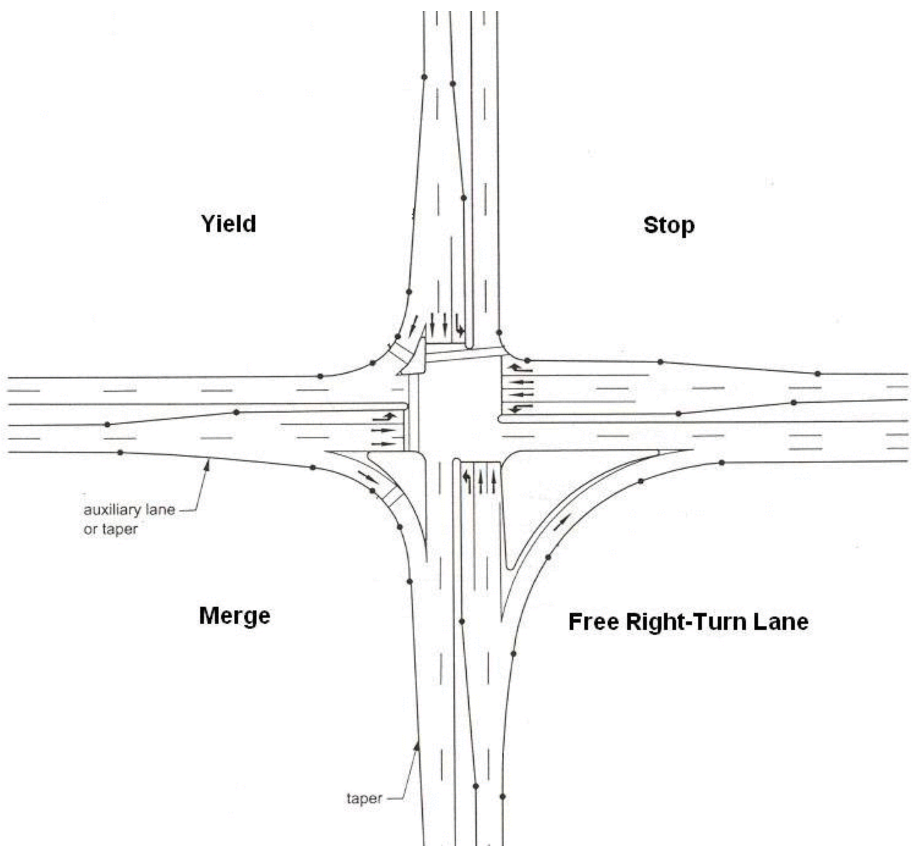


Figure 2-18. Alternative Right-Turn Channelization Designs. (*7*)

A survey conducted by Copas and Pennock (*5*) revealed that a majority of state DOTs favor the use of a turning roadway with free-flow right turns. The most frequently cited reason is to increase the capacity of the ramp terminal. However, it was noted that the left- and right-turn movements must be separated sufficiently far back on the ramp approach leg to ensure that neither movement has an adverse effect on the operation of the other. There was some disagreement as to whether the channelizing island should be painted or curbed in rural and suburban areas.

Design to Discourage Wrong-Way Maneuvers

A problem inherent to service interchanges is the potential for wrong-way entry into an exit ramp (*1*, p. 683). While the maneuver is not frequent, it has the potential to result in a severe crash. A study by Cirillo et al. (*30*) indicates that about five percent of all fatalities on the interstate system are attributable to crashes resulting from wrong-way movements. The parclo A (2-quad) and parclo B (2-quad) designs are particularly susceptible to wrong-way movements because the ramp approach and departure legs are located on the same side of the crossroad and are typically located very close to one another.

Several techniques have been used in combination to minimize wrong-way maneuvers. One technique is to use island channelization to prohibit, or restrict, wrong-way turns. Figure 2-19 illustrates how median channelization on the crossroad can be extended slightly into the intersection to provide shadowing of the approach leg to discourage improper left turns into the exit ramp.

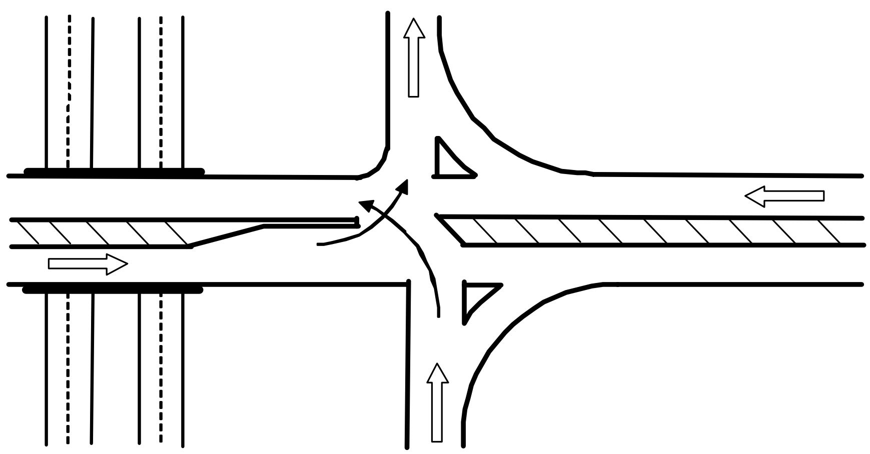


Figure 2-19. Designs to Discourage Wrong-Way Maneuvers. (*1*)

Another technique to prevent wrong-way maneuvers is to use a short-radius curve, or angular break, at the intersection of the left-edge of the exit ramp approach with the right edge of the crossroad approach. This technique is also shown in Figure 2-19. It should discourage improper right-turns into the exit ramp.

Access Control on the 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 on 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. This "separation distance" is shown in Figure 2-20.

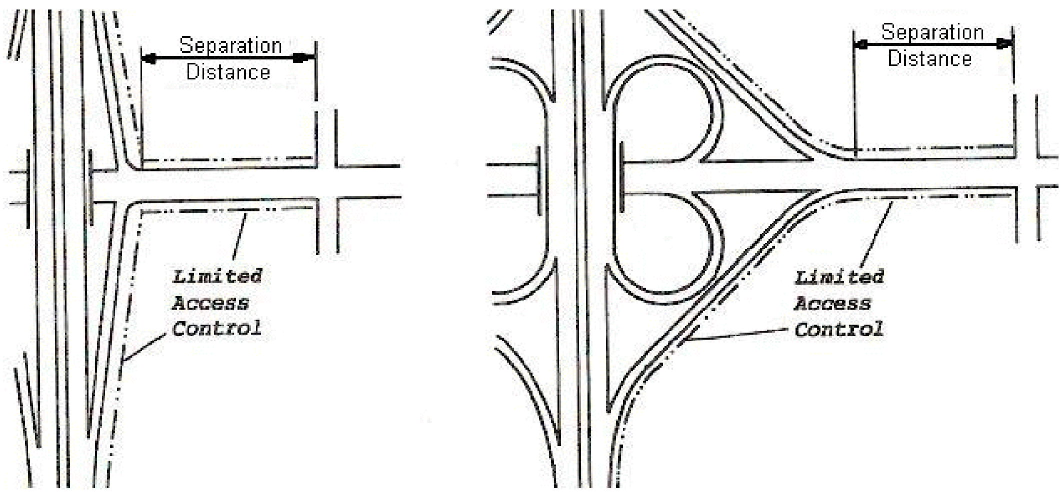


Figure 2-20. Separation Distance for Access Control. (*5*)

The *Green Book* (*1*, p. 753) indicates that the separation distance identified in Figure 2-20 should include sufficient length for an exit-ramp vehicle to weave across the crossroad and decelerate into the left-turn bay at the first downstream intersection. It suggests that this distance should be at least 500 ft (*1*, p. 797) but otherwise does not provide additional guidance as to conditions where longer or shorter distances may be needed.

A more formal analysis of the distances needed for deceleration and weaving is shown in Table 2-13. The deceleration lengths listed in this table were obtained from Table 2-9, and the weaving lengths were obtained from Jacobson et al. (*15*). The values in this table suggest that separation distance for access control may vary from 495 to 990 ft, depending on crossroad design speed and cross section. These values are similar to those used by the Illinois DOT. They recommend a minimum of 500 ft in urban areas and 700 ft in rural areas (*5*).

Table 2-13. Separation Distance Based on Deceleration and Weaving.

Design Speed, mph	Deceleration Length, ft	Total Separation Distance, ft Number of Lanes in Weaving Section: 2	3	4
30	235	495	595	695
35	280	540	640	740
40	320	580	680	780
45	385	645	745	845
50	435	695	795	895
55	480	740	840	940
60	530	790	890	990
Minimum Weaving Distance, ft:		260	360	460

SYNTHESIS OF CURRENT RAMP DESIGN PRACTICE

Engineers in four TxDOT districts were interviewed to identify ramp design issues and challenges associated with non-frontage-road (NFR) interchanges. The discussions addressed the design guidance provided in the *Roadway Design Manual* (*6*), additional guidance that would be useful if available, operational problems associated with the ramp terminals, and safety problems associated with alternative interchange ramp configurations. The districts visited include: Austin, Bryan, Dallas, and Fort Worth. The meeting participants represented engineers from the design, traffic, and planning (i.e., TP&D) divisions.

In preparation for each meeting, maps and aerial photographs from the U.S. Geological Survey (USGS) Bureau were reviewed to determine the number and location of NFR interchanges in Texas. Maps for 12 TxDOT districts were carefully reviewed for this purpose. The results are shown in Figure 2-21. Each dot shown in the figure indicates the location of one NFR interchange. All total, 160 NFR interchanges were identified in this manner. By extrapolation, it is believed that there are about 300 NFR interchanges in Texas.

During each meeting, the participants were given a document that described NFR interchange design issues. These issues were presented using graphical or tabular methods. Aerial photos of several NFR interchanges in the participants' district were also provided. These issues and images were accompanied with a series of questions that probed into the adequacy of existing ramp design guidance, the need for additional guidance, ramp terminal operation, or interchange safety. The responses to these questions were recorded and are summarized in the remainder of this section.

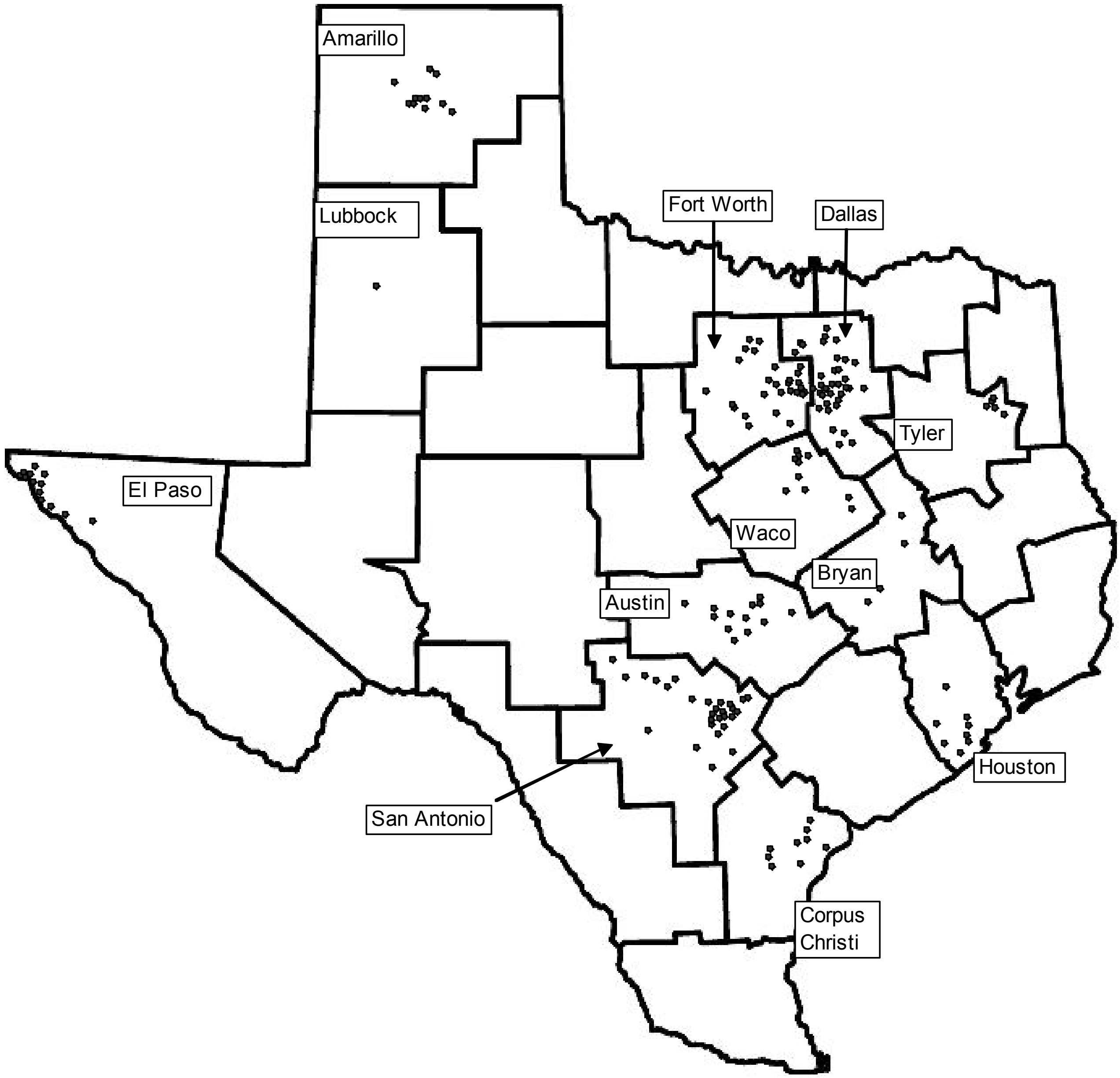


Figure 2-21. Non-Frontage-Road Interchanges in 12 TxDOT Districts.

Interchange Types

Initially, the participants were asked broadly about the judgment, or guidelines, they used to select an interchange form for a specific location. The responses focused on driver expectancy and the quality of interchange operation.

Driver Expectancy

Driver expectancy is a dominant consideration in the selection of interchange type. Ramp configurations that are consistent with driver expectation are highly desirable. Specifically, of the two basic types of interchanges shown in Figure 2-3, the diamond interchange is rationalized to be more consistent with driver expectation in terms of the turn maneuvers required for traffic entering or exiting the crossroad ramp terminals. Specifically, the diagonal ramps associated with the diamond require left turns (and right turns) for what would be effectively a left turn (and right turn) in the direction of travel. This pattern is desirable because it is consistent with driver expectation. In contrast, some turn movements at the parclo interchange are contrary to driver expectation (e.g., a turn to the right is required to effect the equivalent of a left turn in the direction of travel).

In addition, driver expectancy is believed to be largely satisfied by using the same interchange form along a highway and within a district. In this manner, drivers become accustomed to interchange driving during their journey, which makes interchange navigation more a matter of routine than experimentation. Design consistency is more important in rural settings due to the high speeds and long distances between interchanges inherent to rural highways. Interchanges in areas having a large number of older drivers were cited as being locations where design consistency was very important.

Within the diamond interchange category, the SPUI is believed to be contrary to driver expectancy in Texas (it was acknowledged that some states have been using the SPUI design for a sufficient number of years and that it is becoming less of a novelty to drivers in those states). The SPUI's offset, opposing left-turn paths, and large intersection conflict area were noted to be the main features that are contrary to driver expectancy.

Interchange Operation

Operational benefits were cited as factors considered when selecting among alternative interchange types. Operational problems were found most frequently at interchanges in urban areas where traffic demands are high and right-of-way is constrained. The TUDI is frequently used in urban areas; however, queuing between the closely-spaced ramp terminals is a problem. The TUDI requires special signal phasing to minimize queue spillback on the crossroad between the ramp terminals.

The parclo A and parclo B are attractive in high-volume situations because they have a higher overall capacity than the diamond interchanges. This capacity benefit stems from the use of loop ramps with this interchange, thereby eliminating one signal phase at the ramp terminal. The primary operational disadvantage of the loop ramp is that it does not offer a capacity benefit relative to a signalized diagonal ramp. Other disadvantages of the loop ramp were also noted. They are documented in a subsequent section titled Ramp Configurations.

The SPUI is believed to offer operational benefits relative to the other diamond or parclo forms. This benefit is due to the SPUI's use of one signalized junction to control all intersecting ramp and crossroad traffic movements. The SPUI is believed to work best when the junction is located above the major road (i.e., a major-road underpass structure) because this configuration provides drivers a good view of the intersection conflict area (relative to an overpass structure). The SPUI's large-radius left-turn paths were also noted to be well-suited to routes with high truck volumes.

Operational Comparison of Alternative Types

Several engineers reported the occasional use of capacity analysis or simulation to evaluate the operation of alternative interchange types. Software products (e.g., CORSIM, PASSER III, and Synchro) were cited as being used to automate the analysis process. Operational analyses are conducted most frequently during the development stages of an urban project to facilitate the evaluation of alternative interchange types. The level of detail required by the analysis is typically significant in urban areas with high traffic volumes and the potential for congestion. In contrast, the analysis is greatly simplified for rural, low-volume locations. Hence, simulation is rarely used for rural interchange projects.

Ramp Configurations

The discussion of ramp configuration focused primarily on those configurations used at service interchanges and include: diagonal, loop, and outer connection ramps. Relative to an interchange in a frontage road setting, the NFR interchange was presented as being more flexible to design because of the wider range of ramp configurations that could be considered. This flexibility stems from the lack of a through movement at the ramp terminal and the elimination of the weaving section on the frontage road between the ramp and the crossroad.

The diagonal ramp (shown previously in Figure 2-6) is preferred by all meeting participants. It is believed to offer the best combination of operations, safety, and cost. The loop ramp is believed to have similar capacity, but more frequent crashes and greater right-of-way need. However, it was acknowledged that the loop did eliminate a signal phase at the crossroad ramp terminal—a feature that indirectly benefits the crossroad traffic movements.

One safety problem noted with loop ramps is that drivers often travel along them at a speed in excess of that which is safe. This behavior is especially critical when the driver is operating a heavy vehicle. Superelevation at the maximum rate of 6.0 percent can be used to mitigate this problem; however, it can cause other safety problems if traffic queues form within the superelevated section of the loop (e.g., tip over).

Non-Frontage-Road Ramp Design Issues

Ramp Proper Design

The discussion of ramp proper design focused on horizontal geometry and cross section. With regard to horizontal geometry, issues related to ramp curvature and ramp length were addressed. The discussion of cross section was directed toward conditions that indicate when a two-lane ramp proper is appropriate. The details of this discussion are provided in subsequent paragraphs.

Horizontal Geometrics. For NFR interchange ramps, it was posed that there was a wide range of alignments available with the diagonal ramp. However, this range presented several design issues, including: (1) the amount of skew to include in the ramp/crossroad intersection, and (2) the number of curves in the alignment. Figure 2-8 (previously shown) illustrates the range of alignment options available.

The "flat-radius 2-curve" ramp alignment, shown in the lower right quadrant of Figure 2-8, is generally preferred over the "sharp-radius 2-curve" alignment or the "straight ramp" alignment. The "straight ramp" is considered acceptable if the right-turn movement from the exit ramp is the primary traffic movement. However, the skewed intersection that results from the use of this alignment is considered a negative feature. The "sharp-radius 2-curve" alignment was also noted to require more right-of-way and to have greater likelihood of run-off-the-road crashes (resulting from the sharp curvature).

Ramp Curvature. Questions were posed regarding ramp curvature, the use of spiral transition curves, and the superelevation rates used on ramps. It was consistent among the meeting participants that spiral transition curves are not used for ramp alignments. However, compound curves are used for loop ramp design to provide the proper speed transition.

Superelevation on ramp curves is generally kept to minimum levels. Meeting participants commented that a 6.0 percent superelevation rate is a preferred upper limit. The rates shown in Table 3-21 of the *Roadway Design Manual* are specifically identified in the *Roadway Design Manual* as being applicable to interchange ramps. These rates are reproduced in Table 2-14. However, the superelevation rates listed in Table 2-6 of the *Roadway Design Manual* (*6*) were consistently identified as being used for interchange ramp design. The rates obtained from this table are generally within the ranges listed in Table 3-21, but are not exactly the same.

The perceived reason for the difference between rates in Tables 2-6 and 3-21 of the *Roadway Design Manual* was explored, but the responses were clearly speculation on the part of the participants. In fact, the meaning of the superelevation ranges listed in Table 3-21 does not appear to be well known. (It should be noted that the information in Table 3-21 was provided in previous editions of the *Green Book* [*1*] but that it is not provided in the current edition.)

Table 2-14. Ramp Superelevation Rates in the *Roadway Design Manual.*

Table 3-21: Superelevation Range for Curves on Connecting Roadways						
Radius (ft)	**Range in Superelevation Rate (percent) For Connecting Roadways with Design Speed, mph, of:**					
	20	**25**	**30**	**35**	**40**	**45**
90	2-10					
150	2-8	4-10				
230	2-6	3-8	6-10			
310	2-5	3-6	5-9	8-10		
430	2-4	3-5	4–7	6-9	9-10	
540	2-3	3-5	4-6	6-8	8-10	10-10
600	2-3	2-4	3-5	5-7	7-9	8-10
1000	2	2-3	3-4	4-5	5-6	7-9
1500	2	2	2-3	3-4	4-5	5-6
2000	2	2	2	2-3	3-4	4-5
3000	2	2	2	2	2-3	3-4

Exit-Ramp Length. It was posed that the length of the exit ramp, as measured from the gore to the crossroad, includes a segment for deceleration and storage. If a frontage road exists, then a third segment is needed to provide for weaving between exit ramp and frontage road traffic. Thus, this weaving section represents a length that is not needed for exit ramps at NFR interchanges. These points were illustrated for the meeting participants using drawings like that shown in Figure 2-22. The participants agreed that the shorter exit-ramp length for the NFR interchange is likely to result in a lower cost and safer operation because the weaving segment is eliminated. It was also noted that the ramp location for the NFR interchange was not dictated by access point location along the frontage road.

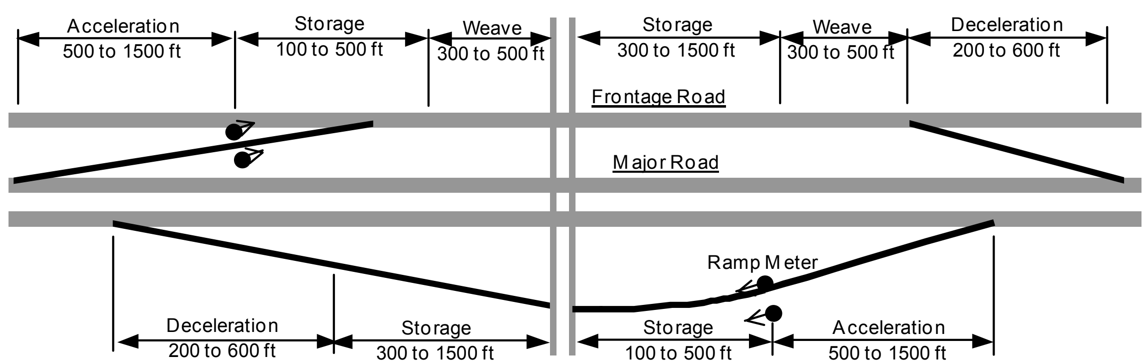


Figure 2-22. Ramp Length Components at Interchanges with and without Frontage Roads.

Entrance Ramp Length. The length of the entrance ramp was also discussed. As shown in Figure 2-22, this length includes a segment for storage behind the ramp meter (if a meter is used) and a length for acceleration. If a frontage road exists, a third segment is needed to provide for weaving between the entrance ramp and frontage road traffic. This weaving section is not needed for entrance ramps at NFR interchanges.

Cross Section. The discussion of ramp cross section dealt with the issue of the conditions for which a single-lane cross section should be widened to include a second lane. Specific guidelines indicating when a two-lane cross section is appropriate were not identified. However, situations that are believed to benefit from a second lane include:

- when storage in one lane is inadequate;
- when it is desirable to separate ramp traffic movements because of different control modes (e.g., signalized left turns and yield-controlled right turns);
- when volumes exceed the capacity of a single lane;
- when there are significant truck volumes (i.e. slow moving vehicles); and
- when the ramp is on a steep upgrade and there are significant truck volumes.

It was also noted that a two-lane ramp may invite requests for access to the ramp because it would more nearly resemble the cross section of a frontage road.

Grade. The meeting participants pointed out that the *Roadway Design Manual* (*6*) recommends that the controlling grade on ramps should "preferably" be limited to 4.0 percent or less. The reaction of the meeting participants to this citation was that 4.0 percent seemed limiting and appeared to be more stringent than guidance provided in the *Green Book* (*1*) for interchange ramps.

Ramp Terminal Design

Traffic Control. The issue of ramp traffic control was discussed. It was noted that the decision to signalize the ramp terminals was based, in part, on an evaluation of traffic signal warrants. The warrants were evaluated for both ramp terminals. If it was determined that a signal was needed for either terminal, then both terminals were signalized. It was also noted that the same traffic control was always used for both ramp terminals, regardless of whether the ramp was controlled using two-way stop control, all-way stop control, or traffic signal control.

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.

Skew Angle. It was posed that the lack of a frontage road might result in the more frequent inclusion of skew at the ramp terminal. This skew was previously discussed with regard to Figure 2-15. The participants felt that a small skew angle is acceptable but that an unskewed ramp terminal (i.e., a 90-degree intersection of the two alignments) is preferred.

The participants pointed out that, as at a standard four-legged intersection, skew has several negative impacts to ramp terminal operation and safety. Specifically, a skewed approach can create sharper turn movement travel paths with larger deflection angles, which can significantly reduce turn movement capacity. Skew can degrade safety because some turn movements will require a difficult head rotation by the driver to verify the safety of the turn maneuver. Also, skew often increases the size of the intersection conflict area and creates the need for island channelization (especially when there are many pedestrians). Lastly, a skewed approach also adds complexity to the ramp terminal design.

Approach Leg Design. The meeting participants were asked about their practices related to the design of the ramp terminal approach leg. The discussion focused on the need for (and design of) a free-flow right-turn lane. Drawings of alternative right-turn lanes and associated channelization, like that shown in Figure 2-18, were provided to promote the discussion of this topic.

The stated "preferred" approach leg design is to keep the left- and right-turn movements together on the exit-ramp terminal approach, consistent with the geometry labeled "stop" or "yield" in Figure 2-18. This preferred design is intended to ensure that the terminal has the appearance of one "normally sized" intersection. This configuration also provides better "target" value to unfamiliar motorists approaching the interchange on the crossroad. In contrast, the "added lane" design in Figure 2-18 results in two, smaller points of intersection with the crossroad.

The "stop" and "yield" designs were stated as being more desirable for urban environments due to concerns about speed, access control, and pedestrian safety. The "merge" or "added lane" designs are better suited to rural settings where higher speeds are expected, downstream intersections are distant, pedestrians are rare, and right-of-way is relatively inexpensive. The "merge" or "added lane" design is sometimes dictated by severe skew in the intersecting alignments or when the right-turn volume is exceptionally high.

The operational benefits of the "merge" and "added lane" designs were recognized in the discussion as were several potential safety issues and design challenges. For example, it was noted that these designs may be associated with more frequent wrong-way movements because of the two points of intersection. The large radii of the right-turn path requires the use of island channelization. This channelization provides refuge for pedestrians, however, the higher turn speeds associated with these designs still reduce the safety of pedestrians crossing the ramp terminals. Also, the geometric design of the turning roadway is more complicated and the cost of its right-of-way higher.

The distance between the gore of the "merge" or "added lane" design and a downstream intersection was stated to be of critical importance to traffic flow efficiency along the crossroad. Both designs require sufficient distance for the ramp right-turn vehicle to weave to the crossroad left-turn lane to complete a left turn at the downstream intersection. It was also noted that these two designs may precipitate high right-turn speeds and last-minute lane changes on the crossroad. These factors may lead to an increase in crash frequency relative to the "stop" or "yield" designs.

U-Turn Lanes. It was posed that NFR interchanges would not need a separate U-turn lane at the interchange. There was general agreement on this point as U-turns were typically precipitated by businesses accessed by the frontage road. However, it was noted that a U-turn lane may be needed in special situations (e.g., at a downstream NFR interchange when the full complement of turn movements were not supported at the upstream interchange).

Access Control

The control of access along the ramp and crossroad was discussed during each meeting. The conversation tended to focus on access along the crossroad because access to the ramp was never allowed. With regard to access along the crossroad, a rule-of-thumb used by engineers in one district is to limit access within 200 ft of the ramp terminal for new construction. It was noted that this criteria, combined with a "merge" or "added lane" right-turn design, could result in the need for a large distance (e.g., 400 ft) between the ramp terminal and the nearest adjacent intersection. For an existing alignment, it was noted that it is rarely possible to limit access within 200 ft of the ramp terminal. In fact, driveways to commercial properties are routinely found within 200 ft of the ramp in built-up areas.

CHAPTER 3. OPERATIONAL EFFECTS DATA COLLECTION PLAN

OVERVIEW

The chapter describes the development of a plan for collecting the data needed to (1) evaluate the effect of alternative interchange ramp configurations on traffic operations, and (2) develop simple guidelines for identifying the more efficient configurations. Initially, a model is developed for relating basic ramp design variables to the delay incurred by motorists traveling through the interchange. Then, a plan is described for assembling the data needed to calibrate this model using simulation software. Finally, a plan is described for collecting the field data needed to verify the accuracy of the simulation software.

CRITICAL FLOW RATIO MODEL DEVELOPMENT

The models developed for this research are based on the "critical-movement analysis" (CMA) approach traditionally used for intersection analysis. This approach forms the basis for signalized intersection analysis in the *Highway Capacity Manual* (*HCM*) (*12*). The CMA approach is described in Chapter 16 of the *HCM*. Variations of this method have previously been applied to interchange ramp terminal capacity analysis (*8, 25*).

The CMA approach combines the influence of an intersection's signal phase sequence, traffic volume, traffic distribution, and number of lanes for each movement into a single number (i.e., the sum-of-critical-volumes). An additional refinement is to incorporate the saturation flow rate of each traffic lane into the computation and compute the sum-of-critical-flow-ratios. Bonneson and Lee (*8*) have demonstrated that this sum is highly correlated with delay at intersections and at interchanges where both ramps are controlled by one signal controller. The relationship between this sum and the delay at interchanges controlled by two signal controllers (i.e., one controller for each ramp terminal) has not been tested and is the subject of this research. If such a relationship holds for all interchange types, regardless of whether one or two signal controllers is used, the sum-of-critical-flow-ratios can form the basis of a quantitative procedure for comparing alternative interchange ramp configurations.

Bonneson and Lee (*8*) found that the sum-of-critical-flow-ratios is correlated with delay when the cycle time is allocated to each phase in proportion to its critical flow ratio (see the discussion associated with Figure 2-4). This condition is satisfied when a full-actuated signal control is used, provided that each phase has a reasonably short minimum green (e.g., 15 s or less) and a large maximum green setting (e.g., 50 s or more). It also closely approximated when pretimed control is used and cycle time is explicitly allocated to each phase in proportion to its critical flow ratio.

This section describes the development of a sum-of-critical-flow-ratios model for each of the more common signalized interchange types and associated ramp configurations. In the next chapter, these models will be used to relate the sum to interchange delay. The graphic portrayal of this relationship is referred to as a "characteristic curve" because of its ability to relate a wide range of input variables to an interchange performance. In combination, the models for computing the sum-of-critical-flow-ratios and the corresponding characteristic curves will form the basis for evaluating alternative interchange ramp configurations.

Model Development for the SPUI

Signal Phase Sequence

This section describes a model for computing the sum-of-critical-flow-ratios for the SPUI. The typical signal phase sequence for this interchange is shown in Figure 3-1.

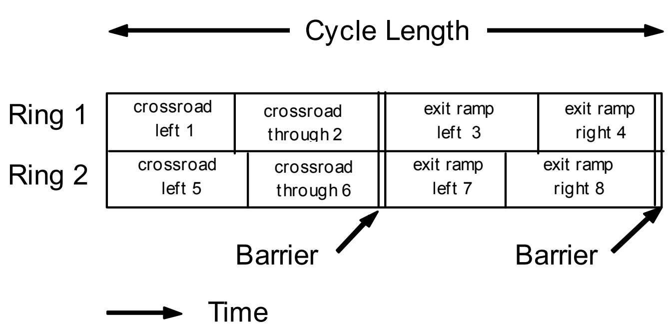


Figure 3-1. Phase Sequence and Ring Structure for the SPUI.

The sequence shown in Figure 3-1 is represented using the dual-ring structure common to current signal controllers. This structure allows for the separate service of four left-turn movements and four through movements. Because of this characteristic, this structure is sometimes called "quad left phasing."

The assignments shown in Figure 3-1 indicate that the SPUI phase sequence has leading left turns on both approaches; however, the model (described in the next paragraph) makes no distinction between leading and lagging left-turn phasing. The barriers shown indicate that phases 2 and 6 (and 4 and 8) must end together but that phases 1 and 5 (and 3 and 7) can end independently of one another. Phases 4 and 8 are not used at SPUIs with free or yield-controlled ramp right-turn movements. Figure 3-2 shows the relationship between these phases and the traffic movements at the SPUI.

Critical Flow Ratio Model

The sum-of-critical-flow-ratios associated with the phase sequence shown in Figure 3-1 can be computed using the following equation:

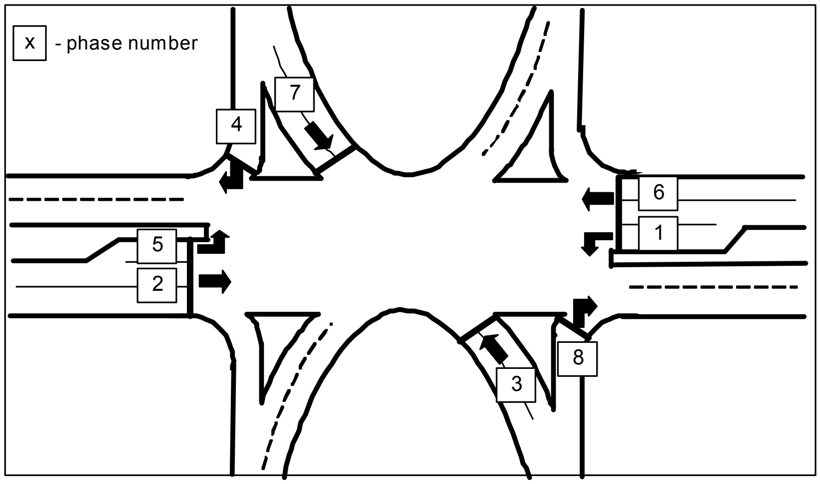


Figure 3-2. Movement and Phase Numbering Scheme for the SPUI.

$$Y_c = A + B \tag{1}$$

with,

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

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

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), veh/h/ln;
n_i = number of lanes served by phase i (i = 1, 2, ... 8);
A = critical flow ratio for the crossroad movements; and
B = critical flow ratio for the exit-ramp movements.

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 provided 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).

Model Development for the TUDI

Signal Phase Sequence

This section describes a model for computing the sum-of-critical-flow-ratios for the TUDI. A "four-phase" phase sequence is generally used for the TUDI because of its narrower ramp separation distances. It is particularly efficient when one or more of the left-turn movements has a relatively high volume. The "four-phase" phase sequence is illustrated in Figure 3-3.

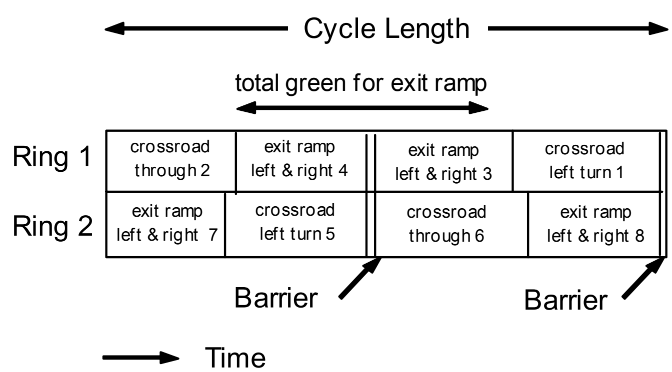


Figure 3-3. Phase Sequence and Ring Structure for the TUDI.

The term "four-phase" is a legacy term that is used here for consistency with the literature. It refers to the exclusive, sequential service provided by the controller to each of the four external approaches to the interchange. It should not be construed to mean that there are only four phases serving interchange traffic movements. Unless noted otherwise, all references to the term "phase" in this section refer to the movement numbering convention established by the National Electrical Manufacturers Association (NEMA).

The four-phase sequence shown in Figure 3-3 uses Ring 1 to serve traffic at the "left-side" ramp terminal and Ring 2 to serve traffic at the "right-side" ramp terminal. Ring 1 allocates two phases to the left-side ramp movements (i.e., 4 and 3). Similarly, Ring 2 allocates two phases to the right-side ramp movements (i.e., 8 and 7). For each phase pair, the phase that occurs second is concurrent with an upstream through phase (e.g., the left side phase 3 is concurrent with the right side through phase 6). The "concurrent phase" (i.e., 3 or 7) always follows the "primary phase" (i.e., 4 or 8) and has a *fixed* duration equal to the interchange travel time minus two seconds.

In Figure 3-3, phase 4 is a primary phase and phase 3 is its concurrent phase. Together, these two intervals define the duration of the left-side ramp phase. Phases 8 and 7 share a similar relationship for the right-side ramp phase. Phases 3 and 7 are also referred to as “transition” intervals in the literature. Figure 3-4 shows the relationship between the phases identified in Figure 3-3 and interchange traffic movements at the TUDI.

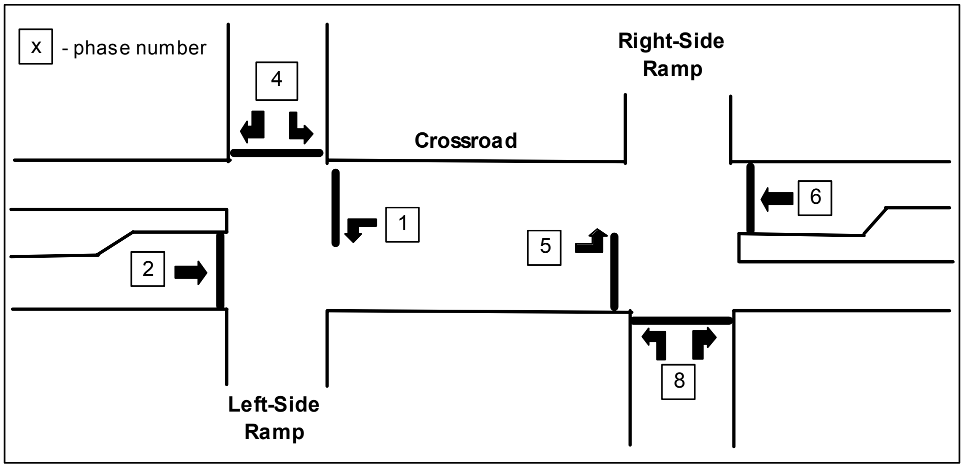


Figure 3-4. Movement and Phase Numbering Scheme for the TUDI.

Critical Flow Ratio Model

The sum-of-critical-flow-ratios associated with the phase sequence shown in Figure 3-3 can be computed using the following equation:

$$Y_c = A + B \tag{4}$$

with,

$$A = Larger\ of: \left[\frac{v_2}{s_2 n_2} + \frac{v_4}{s_4 n_4} - y_3 \quad ; \quad \frac{v_5}{s_5 n_5} + y_7 \right] \tag{5}$$

$$B = Larger\ of: \left[y_3 + \frac{v_1}{s_1 n_1} \quad ; \quad \frac{v_6}{s_6 n_6} + \frac{v_8}{s_8 n_8} - y_7 \right] \tag{6}$$

$$y_3 = Smaller\ of\!: \left[\frac{v_4}{s_4 n_4} \quad ; \quad y_t \right] \qquad \textbf{(7)}$$

$$y_7 = Smaller\ of\!: \left[\frac{v_8}{s_8 n_8} \quad ; \quad y_t \right] \qquad \textbf{(8)}$$

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 (see Figure 3-5).

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 5 and 6 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 3-4), 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.

The effective flow ratio for the concurrent phase (when its duration is dictated by travel time) y_t can be derived as:

$$y_t = T_t \frac{Y_c}{C - L} \qquad \textbf{(9)}$$

where:

T_t = travel time through the interchange minus two seconds, s.

Equation 9 converts interchange travel time into an equivalent flow ratio so that the impact of the fixed-duration concurrent phases (i.e., 3 and 7) can be properly reflected in the sum-of-critical-flow-ratios. The use of this equation with Equations 4 through 8 leads to a computational circularity relative to the variable Y_c. To overcome this limitation, Figure 3-5 was developed to provide a graphic means of defining the value of y_t. This figure was developed using Equations 4 through 9 in an iterative manner. The cycle length used for the analysis was set to a value 30 percent larger than the minimum-delay cycle length, as suggested by Bonneson and Lee (*8*).

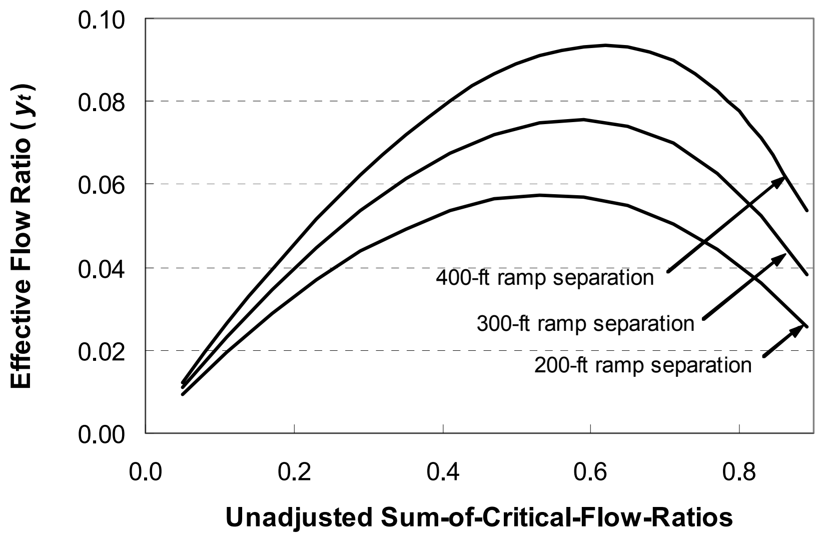


Figure 3-5. Effective Flow Ratio.

With the help of Figure 3-5, the following procedure can be used to compute Y_c for the TUDI. First, compute the "unadjusted sum-of-critical-flow-ratios" using Equations 4, 5, and 6 with the values of y_3 and y_7 set equal to zero. Then, use this "unadjusted" sum with Figure 3-5 to obtain the effective flow ratio, y_t. Next, use y_t in Equations 7 and 8 to obtain y_3 and y_7, respectively. Finally, use y_3 and y_7 in Equations 4, 5, and 6 to compute Y_c.

Alternatively, it is sufficiently accurate to assume that the unadjusted sum-of-critical-flow-ratios is in the range of 0.4 to 0.7 for design concept planning applications. In this case, Equations 4 through 8 can be used directly with the value of y_t set to 0.05, 0.07, or 0.085 for ramp separation distances of 200, 300, or 400 ft, respectively.

Model Development for the Compressed Diamond

Signal Phase Sequence

This section describes a model for computing the sum-of-critical-flow-ratios for the compressed diamond. A "three-phase" phase sequence is often used for this interchange when it has a ramp separation distance in the range of 400 to 800 ft. Compressed diamonds with ramp separation distances in excess of 800 ft are often signalized using two controllers (one for each ramp terminal). This type of control is the subject of a subsequent section. The "three-phase" sequence is illustrated in Figure 3-6.

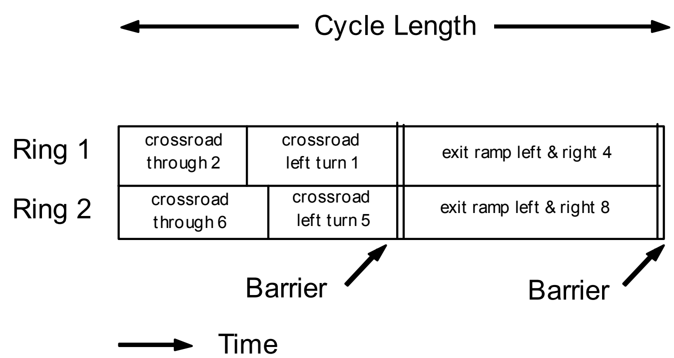


Figure 3-6. Phase Sequence and Ring Structure for the Compressed Diamond.

The term "three-phase" is a legacy term that is used here for consistency with the literature. It refers to the concurrent service provided by the controller to both crossroad approaches, to both internal left-turn movements, and to both exit-ramp approaches. Unless noted otherwise, all references to the term "phase" in this section refer to the movement numbering convention established by NEMA.

Figure 3-7 shows the relationship between the phases identified in Figure 3-6 and the traffic movements at the compressed diamond. As with the TUDI, this phase sequence uses Ring 1 to serve traffic at the left-side ramp terminal and Ring 2 to serve traffic at the right-side ramp terminal.

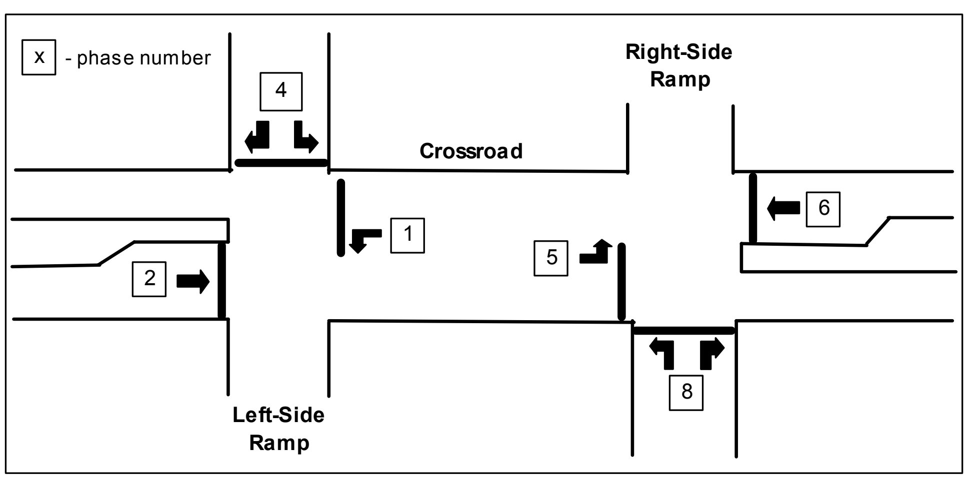


Figure 3-7. Movement and Phase Numbering Scheme for the Compressed Diamond.

Critical Flow Ratio Model

The sum-of-critical-flow-ratios associated with the phase sequence shown in Figure 3-6 can be computed using the following equation:

$$Y_c = A + B \tag{10}$$

with,

$$A = Larger\ of\!: \left[\frac{v_1}{s_1 n_1} + y_2 \ ; \ \frac{v_5}{s_5 n_5} + y_6 \right] \tag{11}$$

$$B = Larger\ of\!: \left[\frac{v_4}{s_4 n_4} \ ; \ \frac{v_8}{s_8 n_8} \right] \tag{12}$$

$$y_2 = Larger\ of\!: \left[\frac{v_2}{s_2 n_2} \ ; \ \frac{v_5}{s_2} \right] \tag{13}$$

$$y_6 = Larger\ of\!: \left[\frac{v_6}{s_6 n_6} \ ; \ \frac{v_1}{s_6} \right] \tag{14}$$

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 13 and 14 account for the tendency of drivers to position their vehicles in the inside lane at the upstream ramp terminal when attempting a left-turn maneuver at the downstream ramp terminal. This tendency is more prevalent at interchanges with shorter ramp separation distances because the difficulty of changing lanes is greater on shorter segments. Equations 13 and 14 provide a means of checking whether left-turn drivers will "preposition" in the inside lane and create unbalanced lane usage on the crossroad approach to the interchange.

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.

Model Development for Interchanges with Two Signal Controllers

This section describes a general model for computing the sum-of-critical-flow-ratios for interchanges that have one signal controller at each of the two ramp terminals. Each of the following interchange types typically have two signal controllers and are addressed in this section:

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

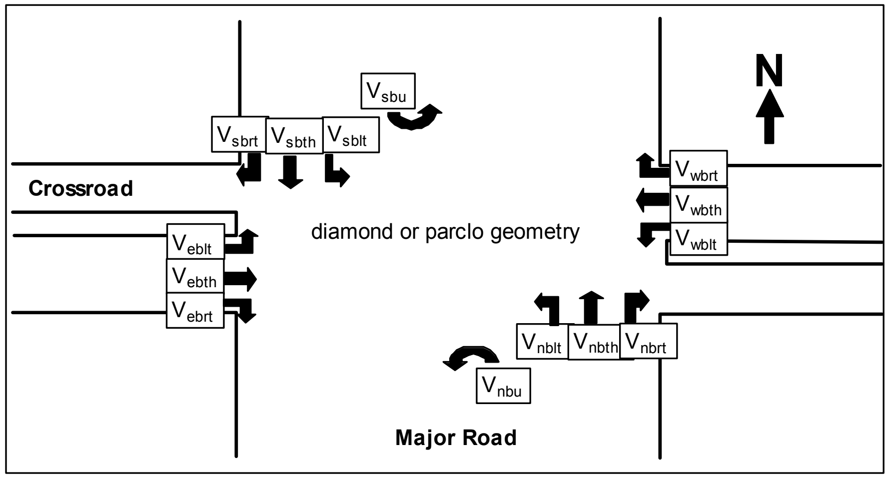
Basic Traffic Movements

The development of a general model for two-controller interchanges is based on the following observations: (1) each controller has a unique sum-of-critical-flow-ratios, and (2) there are 14 basic traffic movements that travel through the interchange. The first observation dictates the need to compute a sum-of-critical-flow-ratios for each ramp terminal. The latter observation simplifies the analysis such that only the basic movements need to be identified. The traffic movements executed internally to the interchange can be computed from these basic movements.

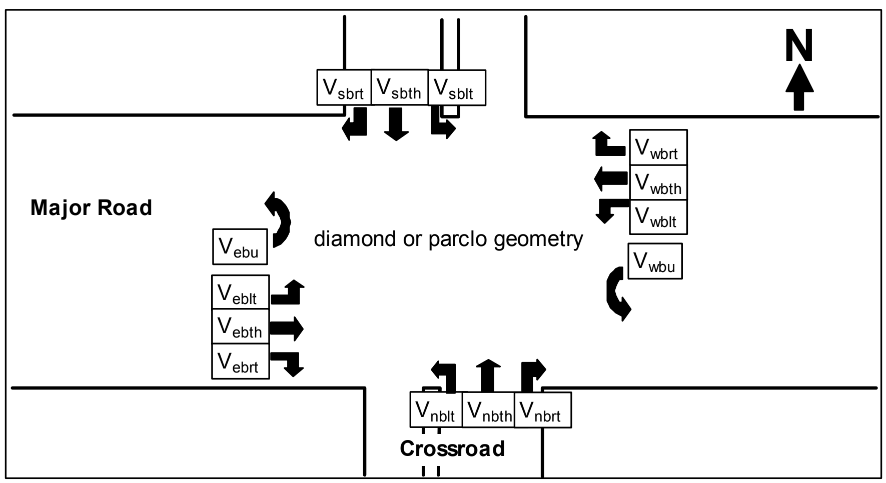
The 14 basic movements are identified in Figure 3-8. Of these movements, the through movement and the U-turn movement from the ramps can be ignored for interchanges in non-frontage-road settings. The remaining 10 movements are applicable to the analysis of interchanges without frontage roads.

Signal Phase Sequence

The phase sequence and corresponding phase numbering scheme for each ramp terminal is shown in Figure 3-9. As suggested by this figure, both sequences indicate that the crossroad left-turn phases lead the conflicting through phase; however, a lagging left-turn arrangement is also used for some interchange types. The model (described in the next section) makes no distinction between leading and lagging left-turn phasing and works equally well for both arrangements. Figure 3-10 shows the relationship between the phases identified in Figure 3-9 and the interchange traffic movements at the conventional diamond and the parclo interchanges.



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



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

Figure 3-8. Fourteen Basic Traffic Movements at an Interchange.

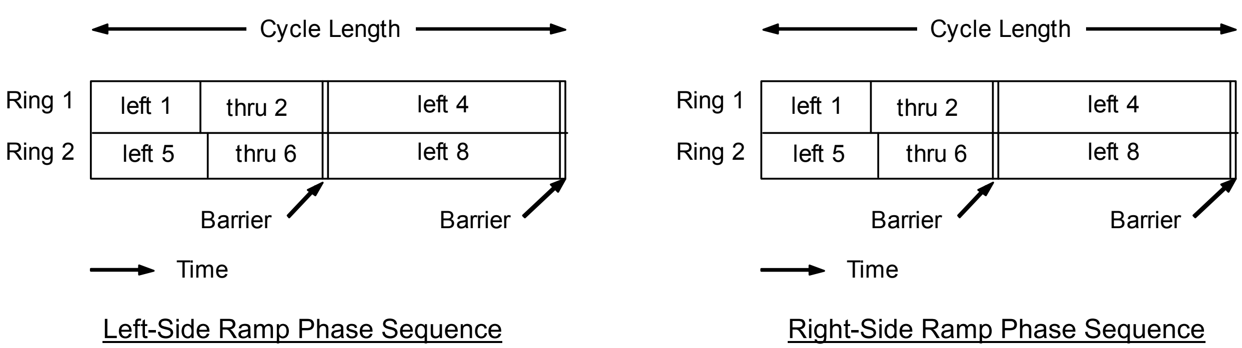
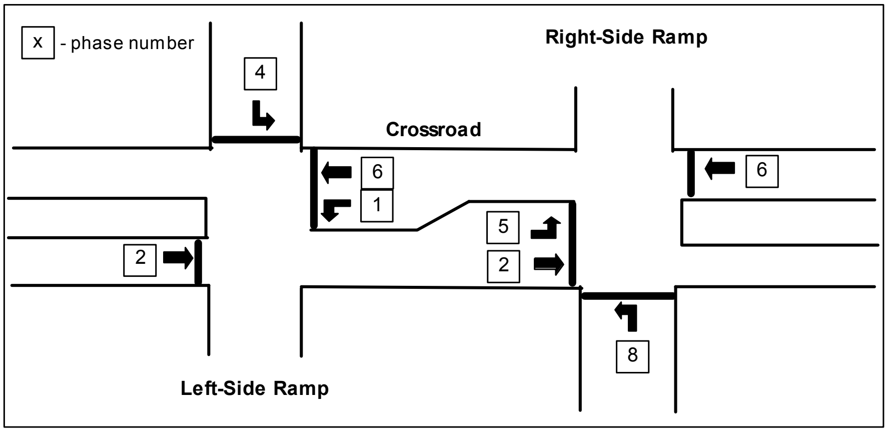
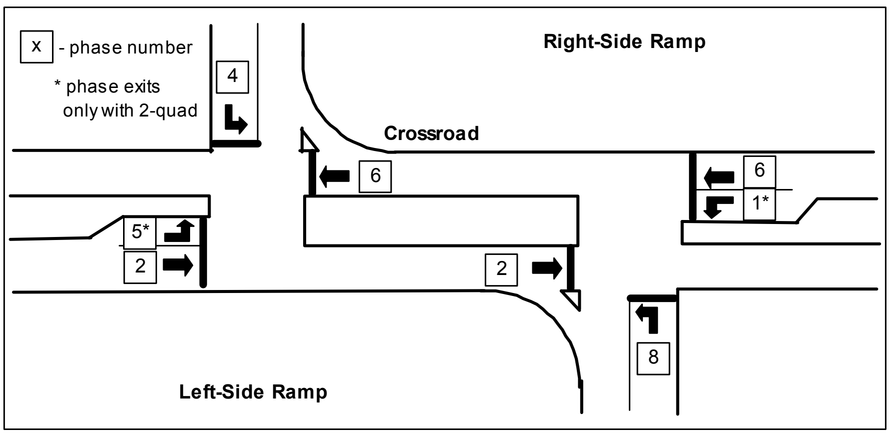


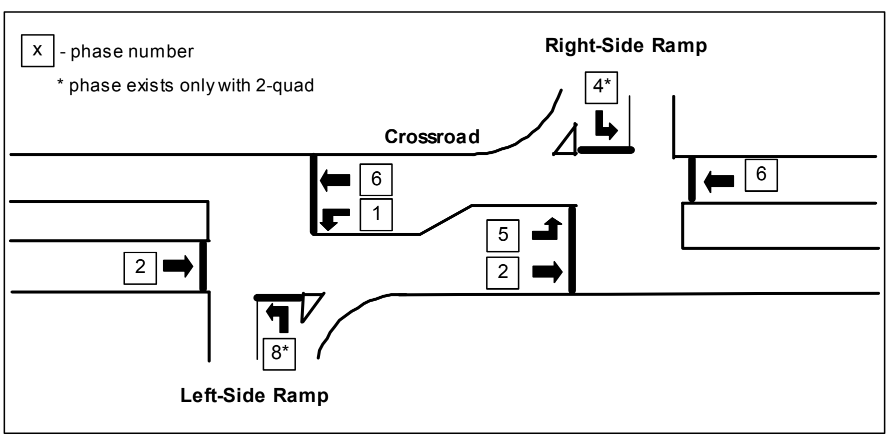
Figure 3-9. Phase Sequence and Ring Structure for Interchanges with Two Controllers.



a. Conventional Diamond.



b. Parclo A and Parclo A (2-Quad).



c. Parclo B and Parclo B (2-Quad).

Figure 3-10. Movement and Phase Numbering for Interchanges with Two Controllers.

The relationship between the basic traffic movements and the phase numbering shown in Figure 3-10 is provided in Table 3-1. The information in this table highlights the similarities among the five interchange types listed. For example, it indicates that phases 2 and 6 always serve the same traffic movements. Hence, the volume served during phase 2 and phase 6 is the same for all interchange types.

Table 3-1. Relationship between Basic Movement Volumes and Phase Number at Interchanges with Two Signal Controllers.

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	Conventional Diamond	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}
	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}
	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	Conventional Diamond	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}
	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}
	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 3-8, 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 information in Table 3-1 also indicates that the parclo A phases have nearly the same volume components as the conventional diamond. The one exception is that the parclo A serves the crossroad left-turn movements with a free-flow loop ramp and, thereby, removes these movements from the signalization. The parclo B phases also have nearly the same volume components as the conventional diamond. The one exception here is that the ramp left-turn movements are served by a free-flow loop ramp. The 2-quad parclos have the same number of signalized movements as the conventional diamond. However, each 2-quad parclo substitutes one right-turn movement for one left-turn movement served by the diamond.

The sum-of-critical-flow-ratios associated with either phase sequence shown in Figure 3-9 can be computed using the equations below, with the appropriate phase volumes obtained from Table 3-1.

$$Y_c = A + B \tag{15}$$

with,

$$A = Larger\ of\!: \left[\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2} \quad ; \quad \frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6} \right] \tag{16}$$

$$B = Larger\ of\!: \left[\frac{v_4}{s_4 n_4} \quad ; \quad \frac{v_8}{s_8 n_8} \right] \tag{17}$$

These equations should be applied twice, once for each ramp terminal, to obtain the left-side and right-side sum-of-critical-flow-ratios. If any of the volume variables v_i in Equations 16 or 17 do not have a corresponding movement identified in Table 3-1 (i.e., a "--" is used to indicate this condition), then the volume variable can be assumed to equal 0.0 for the purpose of calculating 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.

To illustrate the use of Equations 16 and 17 with Table 3-1, consider their application to the left-side ramp terminal at a conventional diamond with a north-south major-road orientation. The guidance in Table 3-1 indicates that the following two equations will be obtained:

$$
\begin{aligned}
A &= Larger\ of\!: \left[\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2} \ ; \ \frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6} \right] \\
&= Larger\ of\!: \left[\frac{v_{wblt}}{s_{wblt} n_{wblt}} + \frac{v_{ebth} + v_{eblt}}{s_{ebth} n_{ebth}} \ ; \ 0.0 + \frac{v_{wbth} + v_{nblt}}{s_{wbth} n_{wbth}} \right]
\end{aligned}
\qquad \textbf{(18)}
$$

$$
\begin{aligned}
B &= Larger\ of\!: \left[\frac{v_4}{s_4 n_4} \ ; \ \frac{v_8}{s_8 n_8} \right] \\
&= Larger\ of\!: \left[\frac{v_{sblt}}{s_{sblt} n_{sblt}} \ ; \ 0.0 \right]
\end{aligned}
\qquad \textbf{(19)}
$$

As indicated by the equations above, the saturation flow rate s_i and the number of lanes n_i for phases 2 and 6 should always represent the flow rate and lanes associated with the through movement served by the phase. The flow rate and lanes provided to the left-turn movement (which are also served during phases 2 and 6) are not applicable when computing the flow ratio for phase 2 or phase 6.

DELAY PREDICTION MODEL DEVELOPMENT

This section describes the development of simple models for predicting interchange delay. For signalized interchanges, the delay model is related to the sum-of-critical-flow-ratios for the ramp junction. For unsignalized interchanges, the delay model is related to the volume-to-capacity ratio of the minor movements at the interchange. Initially, the term "interchange delay" is defined and an equation for computing it is described. Then, the delay prediction models are developed. The calibration of these models using simulation data is described in Chapter 4.

Interchange Delay

The delay incurred by motorists traveling through an interchange includes control delay, traffic delay, and geometric delay. Control delay is the delay incurred as a result of a control device (e.g., stop sign or signal). Traffic delay is the increase in running time that is incurred when the interaction of vehicles causes drivers to reduce their speed below the free-flow speed. Geometric delay is the increase in running time that is incurred when interchange geometry causes drivers to reduce speed to negotiate the facility. Of these delay statistics, the *HCM* (*12*) bases its level of service thresholds for intersections and interchanges on control delay.

The comparison of alternative interchange types and associated ramp configurations requires that the delay statistic used is unbiased by type or configuration. Of the three delay components, control delay and geometric delay satisfy this requirement; however, traffic delay does not. Traffic delay is influenced by interchange type because it is positively correlated with travel distance. As a result, wider interchanges will be associated with more traffic delay on the crossroad segment between the ramp terminals than narrow interchanges. However, this delay is due to travel along a busy road, regardless of whether the road is located between ramp terminals or external to the ramp terminals. For this reason, traffic delay is not a viable delay statistic for comparing alternative interchange types.

The most equitable delay statistic for comparing the operation of alternative interchange types is one that combines control delay and geometric delay. The estimation of geometric delay at interchanges requires the calculation of any increase in running time that is due to a speed change caused by geometry. By this definition, geometric delay is incurred by all motorists traveling along uncontrolled ramps (e.g., right-turn via an outer connection ramp or left-turn via a loop ramp). For typical speed changes on outer connection ramps, geometric delay amounts to only a few additional seconds of running time. This amount of delay is negligible when compared to the magnitude of control delay incurred. In contrast, the geometric delay incurred by motorists on loop ramps is not negligible and must be considered. An equation for computing this delay is the subject of a subsequent section.

Definition of Interchange Delay

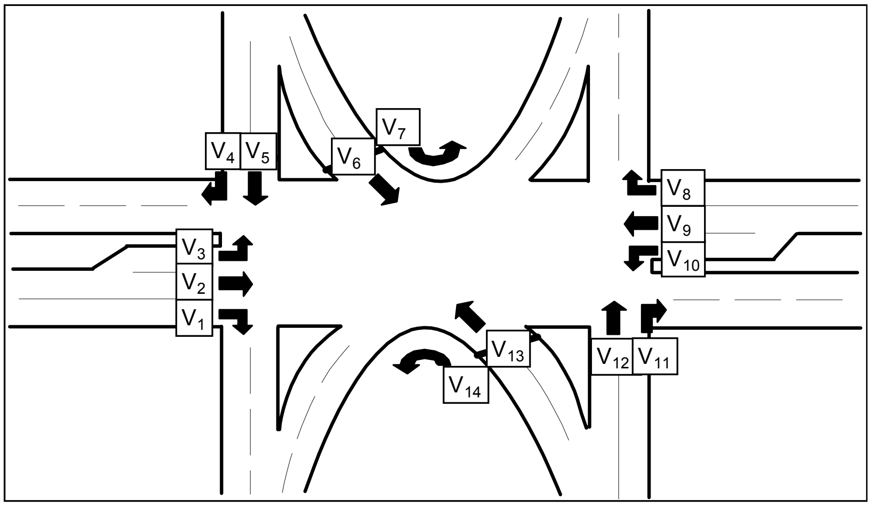
A useful delay statistic for comparing the performance of alternative interchange types is described in this section. It is referred to as "interchange delay." This statistic is derived to represent the average delay incurred by a motorist when traveling through the interchange (excluding motorists on the major roadway). It is computed as the total delay (in veh-hr) incurred by all vehicles using the interchange divided by the volume of vehicles on the *external* approaches to the interchange. Interchange delay is computed using the following equation:

$$d_I = \frac{\sum (d_i\, v_i) + \sum (d_j\, v_j) + \sum (t_i\, v_i)}{\sum v_i} \qquad \textbf{(20)}$$

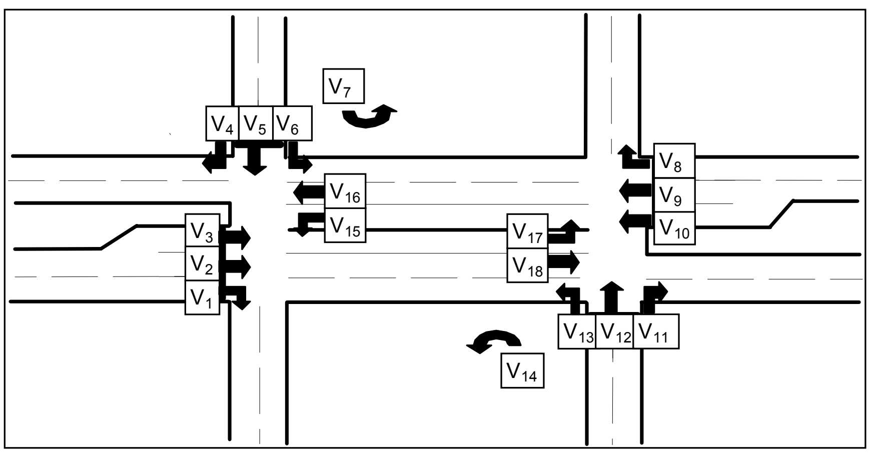
where:

- d_I = interchange delay, s/veh;
- d_i = control delay for external movement i (i = 1, 2, ... , 14), s/veh;
- d_j = control delay for internal movement j (j = 15, 16, 17, 18), s/veh;
- t_i = geometric delay for external turn movement i, s/veh;
- v_i = volume for external movement i, veh/h; and
- v_j = volume for internal movement j, veh/h.

The traffic movement numbers identified in Equation 20 (by subscript) are defined in Figures 3-11a and 3-11b for the SPUI and TUDI, respectively. The external movements are those that correspond to the 14 basic movements identified in Figure 3-8.



a. Single-Point Urban Interchange.



b. Tight Urban Diamond Interchange.

Figure 3-11. Traffic Movement Numbers Used for Delay Calculation.

Equation 20 is defined so that its denominator is constant for all interchange types. This denominator represents the total volume entering the interchange. It represents the sum of the basic traffic movements. This is another reason that the delay obtained from Equation 20 is unbiased and can be used to compare alternative interchange types without bias.

Loop Running Time

As discussed previously, the running time along a loop ramp constitutes a geometric delay to the motorist. The length of ramp along the loop represents an additional travel distance for the motorist, relative to the length of a diagonal ramp. Thus, the running time along the loop represents a tangible geometric delay that should be included in Equation 20. It can be estimated as:

$$t = \frac{L_r}{V_r} + d_{ad} \tag{21}$$

with,

$$L_r = 1.5\,\pi\,R \tag{22}$$

$$V_r = 3.28\,R^{0.452} \tag{23}$$

where:
t = loop running time, s/veh;
L_r = length of circular portion of loop ramp, ft;
R = radius of loop ramp, ft;
V_r = running speed on loop ramp, ft/s; and
d_{ad} = delay due to speed change to and from loop running speed V_r (use 3.0 s/veh), s/veh.

The "delay due to speed change"used in Equation 21 represents the additional travel time incurred due to the change in speed as a vehicle enters and then leaves the loop ramp. For typical speeds, this delay is relatively small and can be approximated as 3.0 s/veh for typical speed changes associated with loop ramps.

Figure 3-12 illustrates the relationship between running time and loop radius, as obtained from Equation 21. For typical loop radii that range from 150 to 250 ft, the running time varies from 26 to 33 s/veh.

Interchange Delay Prediction Model

This section describes the development of simple models for predicting interchange delay as a function of interchange type, traffic volume, control mode, and number of traffic lanes. With regard to control mode, separate models are developed for signalized and unsignalized interchanges.

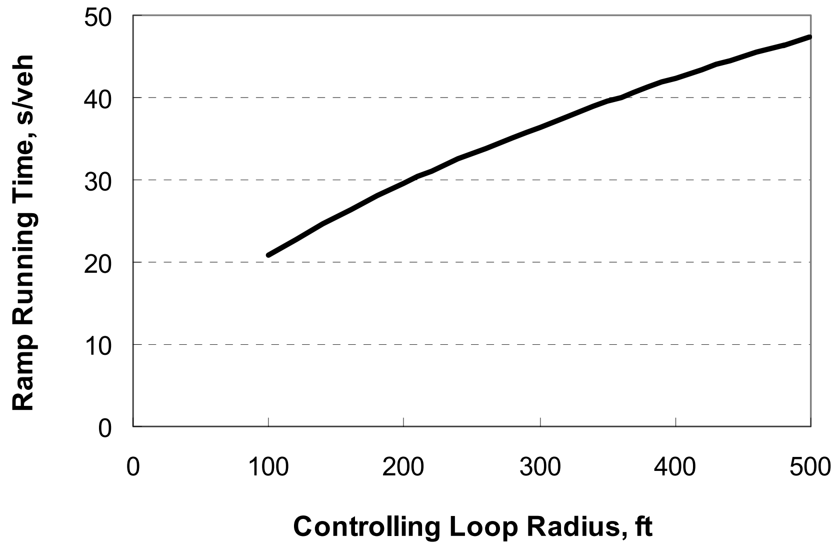


Figure 3-12. Running Time on Circular Portion of Interchange Loop Ramp.

Signalized Interchange Delay

The models for predicting delay at a signalized interchange are based on interchange type, number of signal controllers (i.e., one or two controllers), and sum-of-critical-flow-ratios. This latter attribute reflects the combination of volume and the number of traffic lanes for each of the critical traffic movements.

Interchanges with One Signal Controller. For interchanges that use one signal controller (i.e., the SPUI, TUDI, and compressed diamond), the following model form was developed:

$$d_I = b_0 + b_1 \frac{Y_c}{1 - Y_c} \tag{24}$$

where:
d_I = interchange delay, s/veh;
Y_c = sum of the critical flow ratios; and
b_0, b_1 = regression coefficients.

The regression coefficients included in the model are intended to be calibrated using simulation data.

The formulation of Equation 24 is based on the relationship between the sum-of-critical-flow-ratios and interchange delay reported by Bonneson and Lee (*8*), as shown in Figure 2-4. The first term in Equation 24 (i.e., the regression constant b_0) represents the minimum delay incurred by the average vehicle negotiating the interchange. It is caused by the signal control of the ramp terminals, the loop running time, or both. The second term is loosely based on queuing theory and is intended to represent the delay caused by random arrivals.

Figure 3-13 illustrates the delay relationship predicted by Equation 24 when b_0 equals 25 s/veh and b_1 has a value of 9.0. These values were selected to yield a trend line that compares favorably with that shown for the TUDI in Figure 2-4.

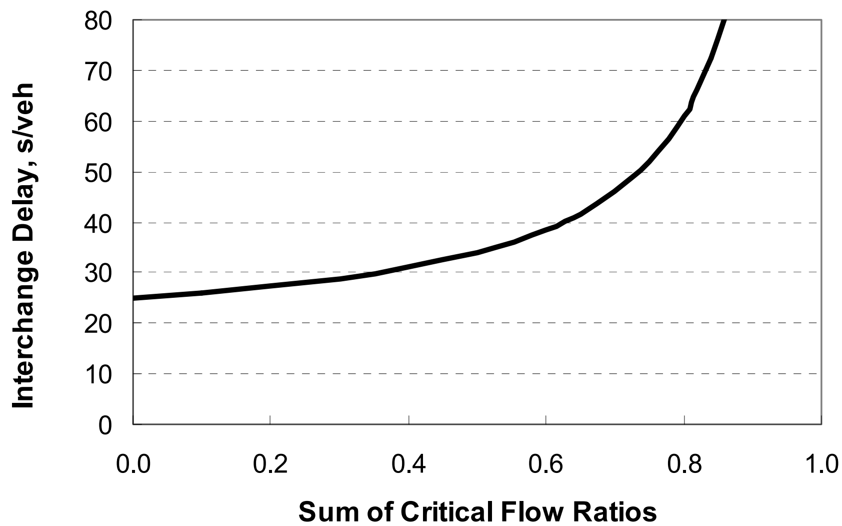


Figure 3-13. Relationship between Sum-of-Critical-Flow-Ratios and Delay.

Interchanges with Two Signal Controllers. When an interchange has a signal controller at each ramp terminal there is a unique sum-of-critical-flow-ratios for each ramp terminal. A delay prediction model was developed that included a term for the critical flow ratio at each terminal. The equation representing this model is:

$$d_I = b_0 + b_1 \frac{Y_{c,\max}}{1 - Y_{c,\max}} + b_2 \frac{Y_{c,\min}}{1 - Y_{c,\min}} \qquad \textbf{(25)}$$

where:
$Y_{c,max}$ = largest sum-of-critical-flow-ratios for the two ramp terminals;
$Y_{c,\ min}$ = smallest sum-of-critical-flow-ratios for the two ramp terminals; and
b_i = regression coefficients, $i = 0, 1, ..., n$.

The regression coefficients included in the model are intended to be calibrated using simulation data.

The formulation of Equation 25 follows the same rationale as used in the development of Equation 24. However, there are two terms included to explain the random delay that occurs at each of the two ramp terminals. The term associated with the b_1 coefficient is intended to explain the delay that occurs at the one terminal having the largest sum-of-critical-flow-ratios. The term associated with the b_2 coefficient would explain the delay that occurs at the other ramp terminal.

Unsignalized Interchange Delay

The model for predicting delay at a two-way stop-controlled interchange is based on interchange type, interchange volumes, and the volume-to-capacity ratio of the minor traffic movements. Only the minor movements are considered in the model terms in recognition of the fact that the crossroad through movements are not delayed at a two-way stop-controlled interchange. The following model form was developed to predict interchange delay at two-way stop-controlled interchanges:

$$d_I = b_0 + b_1 \frac{X_{r,\max}^2}{1 - X_{r,\max}} + b_2 \frac{X_{r,\min}^2}{1 - X_{r,\min}} + b_3 \frac{X_{c,\max}^2}{1 - X_{c,\max}} + b_4 \frac{X_{c,\min}^2}{1 - X_{c,\min}} \tag{26}$$

with,

$$X_{c,left} = \frac{v_{c,left}}{1600 - 0.55\, v_{o,c,left}} \tag{27}$$

$$X_{c,right} = \frac{v_{c,right}}{1600 - 0.55\, v_{o,c,right}} \tag{28}$$

$$X_{r,left} = \frac{v_{r,left}}{1000 - 0.55\, v_{o,r,left}} \times \frac{1}{1 - X_{c,left}} \tag{29}$$

$$X_{r,right} = \frac{v_{r,right}}{1000 - 0.55\, v_{o,r,right}} \times \frac{1}{1 - X_{c,right}} \tag{30}$$

where:

$X_{r,\, max}$ = larger of the two exit-ramp volume-to-capacity ratios ($X_{r,\, left}$, $X_{r,\, right}$);
$X_{r,\, min}$ = smaller 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 ($X_{c,\, left}$, $X_{c,\, right}$);
$X_{c,\, min}$ = smaller 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 3-2), veh/h;
$v_{r,\, right}$ = subject exit-ramp left-turn volume for right-side ramp terminal (see Table 3-2), veh/h;
$v_{c,\, left}$ = subject crossroad left-turn volume for left-side ramp terminal (see Table 3-2), veh/h;
$v_{c,\, right}$ = subject crossroad left-turn volume for right-side ramp terminal (see Table 3-2), veh/h;
$v_{o,\, r,\, left}$ = volume opposing $v_{r,\, left}$ (see Table 3-2), veh/h;
$v_{o,\, r,\, right}$ = volume opposing $v_{r,\, right}$ (see Table 3-2), veh/h;
$v_{o,\, c,\, left}$ = volume opposing $v_{c,\, left}$ (see Table 3-2), veh/h; and
$v_{o,\, c,\, right}$ = volume opposing $v_{c,\, right}$ (see Table 3-2), veh/h.

The formulation of Equation 26 is based on the assumption that interchange delay consists largely of the delay to the two left-turn movements at each ramp terminal. The terms in the model are intended to relate the delay to these four movements to overall interchange delay.

The volume variables referenced in Equations 27 through 30 represent both the left-turn volumes at the interchange and the volumes that oppose, or conflict with, these left-turn movements. Table 3-2 identifies the 14 basic movement volumes (as identified in Figure 3-8) that should be used to obtain the subject left-turn volume and its associated conflicting volume for each of the four left-turn movements. For example, Table 3-2 indicates that Equation 27 would be modified as follows to compute the volume-to-capacity ratio of the crossroad left-turn movement at the left-side terminal of a diamond interchange:

$$X_{c,left} = \frac{v_{wblt}}{1600 - 0.55\,(v_{eblt} + v_{ebth} + 2\, v_{ebrt})} \tag{31}$$

The denominator in Equations 27 through 30 represents the capacity of the associated left-turn movement. These capacity equations are based on linear approximations to the capacity relationships shown in Exhibit 17-7 of the *HCM* (*12*). The "1 - *X*" term in the denominator of Equations 29 and 30 is an adjustment to account for the portion of the ramp capacity consumed by the crossroad left-turn movement. This adjustment is consistent with the methodology described in Chapter 17 of the *HCM* (*12*).

Table 3-2. 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		Ramp Left-Turn Volumes	
			Subject[4] $v_{c,k}$	Opposing[4] $v_{o,c,k}$	Subject[4] $v_{r,k}$	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 A-7, 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 27 through 30 is computed as a negative value, then the corresponding volume-to-capacity ratio should be set to 0.95. Moreover, if Equations 29 or 30 yield a value in excess of 0.95, then this value should be set to 0.95. These checks will prevent unrealistically large delay estimates from Equation 26.

In Table 3-2, the right-turn volumes opposing a crossroad left-turn movement are inflated by a factor of 2.0. This adjustment is needed to reflect the fact that a decelerating right-turn vehicle effectively blocks a conflicting left-turn movement about twice as long as a through vehicle.

The right-turn movements conflicting with the ramp left-turn movements are also inflated by a factor of 2.0 in Table 3-2. However, Chapter 17 of the *HCM* (*12*) indicates that only one-half of the volume of these right-turn movements should be included in the opposing volume sum. Thus, these two adjustments effectively offset each other (i.e., $2 \times 0.5 = 1$) and, as a result, no net adjustment is applied to the right-turn volumes opposing a ramp left-turn movement.

DATA COLLECTION PLAN

This section describes a data collection plan that was developed to provide the data needed to calibrate the delay models described in the previous section. The plan is based on the use of simulation software to evaluate the effect of alternative ramp configurations on interchange delay. The plan also addresses the collection of field data to calibrate the simulation software and to validate its performance predictions. Initially, the plan for conducting the simulation experiments is described. Then, the field data collection plan is outlined.

Simulation Data Collection Plan

The development of geometric design guidelines requires the consideration of a wide range of ramp configurations, traffic volumes, and control device combinations. In this manner, the resulting guidelines are comprehensive in that the resulting design will be able to provide satisfactory service under a wide range of operating conditions. Simulation software is ideally suited to the controlled development of a database reflecting the factorial combination of interchange types, traffic volumes, and control devices found at interchanges in Texas.

Simulation Software Selection

Several simulation software products have the potential to be used for analyzing traffic operations at interchanges. Ramp configuration can influence traffic operation on the ramp, the crossroad, and the major road. In addition, the traffic control used at the ramp terminal significantly impacts interchange operation. Therefore, it is imperative that the simulation software can model both the crossroad and the major road as a system and that it can accurately replicate the signalized and unsignalized control strategies used at many interchanges.

Some software products, such as PASSER II and TRANSYT-7F, were developed for the analysis of arterial intersections. However, these models do not accurately model the operation of an interchange operated by a single controller. Other software products, such as PASSER III and the TEXAS model, were developed explicitly for the analysis of signalized diamond interchanges. However, neither of these products can accurately model a parclo or a two-way stop-controlled diamond.

Simulation software products that are capable of modeling a variety of interchange types and control modes include CORSIM, Synchro/SimTraffic, and VISSIM. These three products also have the ability to model the complexities of actuated interchange signal operation. The CORSIM and VISSIM programs are microscopic, stochastic models that provide a graphical environment for coding and analyzing the simulation runs. Syncho is a combination macroscopic network analysis tool and optimization tool. SimTraffic is a microscopic simulation model extension of Synchro. The capabilities of each of these models is presented in Table 3-3.

Table 3-3. Input and Output Capabilities of Selected Simulation Software Products.

Variable			Simulation Software Capabilities [1]		
			CORSIM (Version 5.0)	**Synchro/ SimTraffic**	**VISSIM**
Input	Traffic characteristics	Vehicular volume	✔	✔	✔
		Truck volume	✔	✔	✔
		Pedestrian volume	✔	✔	✔
		Peak-hour factor		✔	
	Traffic control modes	Pretimed	✔	✔	✔
		Actuated	✔	✔	✔
		Single-controller operation	✔ [2]	✔	✔
		Stop, Yield	✔	✔	✔
	Geometry	Ramp length	✔	✔	✔
		Free right-turn lane	✔	✔	✔
		Loop ramp	✔	✔	✔
Performance measures	Delay		Control	Control	Total
	Probability of spillback	Onto major road	✔	✔	✔
		Onto crossroad	✔	✔	✔
	Average queue length		✔	✔	✔
	Volume-to-capacity ratio			✔	

Notes:
1 - ✔: simulation software product accepts the input or produces the output indicated.
2 - Hardware-in-the-loop using an external signal controller is needed to model an interchange with a single-controller.

CORSIM. The CORSIM simulation model was developed by the FHWA and has been upgraded and enhanced a number of times throughout the past 20 years. The latest version includes the TRAFED graphical model input development, which greatly improves the ease of model development. CORSIM combines two other models (NETSIM for network modeling and FREESIM for freeway modeling) such that an entire travel corridor can be simulated.

CORSIM has two limitations that could affect the results of the analysis of some interchange types. First, it does not allow one traffic controller to regulate two ramp terminals, as is often the case for some interchange types. This limitation mainly effects operations when simulating a signalized TUDI or compressed diamond with actuated control. This limitation can be overcome by using a “hardware-in-the-loop” extension to CORSIM so that an external signal controller can be used to regulate the signal operation.

The second major limitation of CORSIM is the erratic lane change behavior that it often exhibits when it is used to simulate a short road segment, such as that on the crossroad between two ramp terminals. The erratic lane change behavior occurs because CORSIM does not consider vehicles that are on the road segment downstream of the subject road segment. As a result, simulated drivers on the subject segment are often required to make unrealistic lane changes just as they enter the downstream segment (and are “told,” for the first time, by the simulator that they will be turning at the end of the downstream segment). This limitation can increase the simulated delay beyond that which would truly be incurred.

This second limitation can be avoided by ensuring that all modeled road segments are relatively “long” (experience indicates that lengths of 800 ft or more are adequate for typical volume conditions). If short segments must be included, then a special modeling technique can be used to ensure drivers are prepositioned in the proper lane well in advance of their turn (thereby eliminating the need for last-minute lane changes).

Syncho/SimTraffic. Synchro is a macroscopic analysis tool that is used to analyze and optimize traffic signal timings along an arterial or a network of arterials. The graphical user interface makes it very easy to draw the network and input volumes, lane geometry, and signal timing information. For each intersection, a capacity analysis is performed providing volume-to-capacity ratios, delays, and levels of service for each approach and for the entire intersection.

Synchro allows a single controller to control more than one intersection. In addition, custom phasing patterns that do not follow the traditional NEMA design can also be modeled. The same data files created for Synchro can be used in the SimTraffic microscopic simulation model to examine vehicle interaction throughout the network. This simulator can model a wide range of geometry, traffic, and control conditions; however, it does so using relatively simple models and with limited detail in the performance measure reporting.

VISSIM. VISSIM is a microscopic, stochastic simulation software product that can analyze freeways, arterials, pedestrian paths, and several forms of mass transit facilities. One advantage of VISSIM is that vehicle paths (or routes) can be specified in advance of the interchange. This ensures realistic lane changing behavior because drivers have adequate advance notice of the need to turn and can preposition their vehicle in the proper lane well in advance of this turn. VISSIM allows a single controller to control more than one intersection using either a pretimed and an actuated control mode.

Software Selection. The CORSIM software product was selected for use in generating the model calibration database. It was selected because (1) it provides a control delay estimate, (2) it has been widely used and shown to provide reasonable modeling accuracy, (3) it was believed to have the best balance between complexity and ease-of-use, and (4) the research team was most familiar with its capability having used it recently on other projects.

Experimental Design

A series of simulation scenarios were developed to facilitate the evaluation of alternative ramp configurations and the development of ramp design guidelines. The development of this experimental plan includes consideration of the following factors:

- interchange ramp geometry,
- traffic characteristics,
- traffic control modes, and
- performance measures.

The values or conditions represented by these factors are described in the subsequent paragraphs.

Interchange Ramp Geometry. Interchange ramp geometry factors include interchange type (and associated ramp configuration), ramp separation distance, number of approach lanes, turn bays, and right-turn channelization. Table 3-4 lists the interchange types that were simulated as well as the associated ramp separation distances and control modes.

As indicated in Table 3-4, eight interchange types were identified for simulation. All eight were simulated with signal-controlled ramp terminals. However, only seven types were simulated with two-way stop-controlled ramps because the SPUI is not designed to operate as an unsignalized interchange. The ramp separation distances shown were selected to be representative of each interchange type. For the signalized interchanges, the number of signal controllers used as well as the selection of actuated or pretimed control was based on typical interchange signalization practices. All total, there are 16 unique simulation scenarios represented by this approach.

Two approach cross section categories were considered for each scenario in Table 3-4. The first category represents a "large" interchange. It has two left-turn lanes for the crossroad and ramp left-turn movements. It also has three through lanes on each crossroad approach to a ramp terminal. The second category represents a "small" interchange. It has one lane for each left-turn movement and two through lanes on each crossroad approach. Both categories include an exclusive right-turn lane on each ramp and on each crossroad approach. The lane allocations for both categories are described in Table 3-5.

Table 3-4. Interchange Type and Control Mode Scenarios.

Scenario	Control Mode	Interchange Type	Separation Distance, ft	Signal Controllers
1	Two-Way Stop [1] (unsignalized)	TUDI	300	not applicable
2		Compressed Diamond	700	
3		Conventional Diamond	1100	
4		Parclo A	800	
5		Parclo A (2-quad)	800	
6		Parclo B	1200	
7		Parclo B (2-quad)	1200	
8	Signalized	SPUI	200	1 actuated (quad-left phasing)
9			300	1 actuated (quad-left phasing)
10		TUDI	300	1 actuated (four-phase w/transition)
11		Compressed Diamond	700	1 actuated (three-phase)
12		Conventional Diamond	1100	2 coordinated, pretimed
13		Parclo A	800	2 coordinated, pretimed
14		Parclo A (2-quad)	800	2 coordinated, pretimed
15		Parclo B	1200	2 actuated (outbound 100% green)
16		Parclo B (2-quad)	1200	2 coordinated, pretimed

Note:
1 - Left-turn movement from each exit ramp is stop controlled.

Table 3-5. Interchange Cross Section and Right-Turn Control Scenarios.

Scenario		Right-Turn Control [3]	Number of Lanes by Approach and Basic Movement [1, 2]											
Cat-egory	No.		Northbound			Southbound			Eastbound			Westbound		
			L	T	R	L	T	R	L	T	R	L	T	R
Large	1	Stop/signal	2	2	1	2	2	1	2	3	1	2	3	1
	2	Yield												
	3	Free												
Small	4	Stop/signal	1	2	1	1	2	1	1	2	1	1	2	1
	5	Yield												
	6	Free												

Notes:
1 - L: left turn; T: through; R: right turn.
2 - Eastbound and westbound movements are on the crossroad; northbound and southbound turn movements are on the ramps; northbound and southbound through movements are on the major road. All turn lanes are exclusive.
3 - Right-turn control also dictates right-turn geometry. Stop or signal control: simple, 25-ft radius curve. Yield control: three-centered curve with small island channelization. Free (uncontrolled): free-right-turn lane with 300-ft merge lane on intersected road (exit-ramps use 230-ft radius right-turn lane, entrance ramps use a 150-ft right-turn lane).

As indicated in the footnotes to Table 3-5, the right-turn lane geometry varied with the right-turn control mode. Specifically, it was varied from "simple curve with stop (or signal) control," to "three-centered curve with yield control," to "free right-turn lane with no control." The latter type of control has an added lane extending beyond the end of the channelizing island and along the crossroad or entrance ramp.

A total of six scenarios were identified in Table 3-5 reflecting two cross-section categories and three right-turn control modes. For any one scenario, the same right-turn geometry and control mode was used for all four right-turn movements. Exceptions to this rule include: (1) right-turns to or from the loop ramps associated with the parclo A and parclo B were always uncontrolled, and (2) the exit-ramp right-turn movements at the parclo B were yield controlled for the "stop/signal" control mode because a signal phase was not needed for the exit-ramp left-turn movement.

Left-Turn Bays on the Crossroad. Left-turn bays on the crossroad were included in the design of the crossroad cross section. The length of these left-turn bays was dependent on the type of interchange. Parallel left-turn bays were used for the TUDI. These bays extended back through the upstream ramp terminal for a distance of 300 ft (as measured from the point where the full-width bay began to the *upstream* ramp terminal stop line). In this manner, the total bay storage was equal to 300 ft *plus* the ramp separation distance. The left-turn bays were overlapped and of equal length for the compressed diamond, the parclo B (2-quad), and the parclo B. The full-width left-turn bay length was set to 40 percent of the ramp separation distance. The left-turn bays were external to the interchange for the SPUI and parclo A (2-quad). The full-width bay length was 400 ft.

Turn Lanes on the Exit Ramp. Left-turn lanes on the exit ramp at the approach to the exit-ramp terminals were developed as separate lanes from the right-turn movements. The development length was 400 ft (as measured from the point where the full-width bay began to the ramp terminal stop line). Right-turn bays were used with the "yield" and "stop/signal" controlled right-turn movements. These bays were 200 ft in length.

Traffic Characteristics. To facilitate the examination of traffic demand patterns on interchange operation, a total of 30 volume scenarios were developed. These volumes are listed in Table 3-6 for the signalized scenarios and in Table 3-7 for the unsignalized scenarios. As indicated by the table footnotes, through traffic volumes listed for the northbound and southbound directions apply to the grade-separated major road (i.e., the through movements shown do not travel on the ramps). All scenarios included 10 percent heavy vehicles in the traffic mix.

The volumes in Tables 3-6 and 3-7 represent "basic" movements, as identified in Figure 3-8. They were patterned after the volumes used by Garber and Smith *(31)* in their operational evaluation of alternative interchange types. The set of volumes used consists of three groups of 10 volume patterns. One group represents low volumes, another group represents moderate volumes, and a third group represents high volumes. Within each group, there are five volume patterns for the crossroad combined with two volume patterns for the exit ramps. The crossroad (i.e., eastbound and westbound) volume patterns have the following attributes:

Table 3-6. Signalized Interchange Basic Movement Volume Scenarios.

Scenario		Volume by Approach and Basic Movement,[1,2] veh/h											
Level	No.	Northbound			Southbound			Eastbound			Westbound		
		L	T	R	L	T	R	L	T	R	L	T	R
High	1	400	1925	350	600	3000	400	800	1000	300	800	1000	300
	2	300	1350	350	700	3575	400	800	1000	300	800	1000	300
	3	400	1925	350	600	3000	400	400	1200	300	1000	800	300
	4	300	1350	350	700	3575	400	400	1200	300	1000	800	300
	5	400	1925	350	600	3000	400	400	800	300	1000	1200	300
	6	300	1350	350	700	3575	400	400	800	300	1000	1200	300
	7	400	1925	350	600	3000	400	800	800	300	800	1200	300
	8	300	1350	350	700	3575	400	800	800	300	800	1200	300
	9	400	1925	350	600	3000	400	400	1000	300	1000	1000	300
	10	300	1350	350	700	3575	400	400	1000	300	1000	1000	300
Mod-erate	1	350	1750	225	450	2250	300	475	795	215	475	795	245
	2	250	1200	225	550	2825	300	475	795	215	475	795	245
	3	350	1750	225	450	2250	300	350	825	215	550	850	245
	4	250	1200	225	550	2825	300	350	825	215	550	850	245
	5	350	1750	225	450	2250	300	350	850	215	550	825	245
	6	250	1200	225	550	2825	300	350	850	215	550	825	245
	7	350	1750	225	450	2250	300	475	625	215	475	1050	245
	8	250	1200	225	550	2825	300	475	625	215	475	1050	245
	9	350	1750	225	450	2250	300	350	800	215	575	800	245
	10	250	1200	225	550	2825	300	350	800	215	575	800	245
Low	1	300	1600	100	300	1500	200	150	590	130	150	590	190
	2	200	1025	100	400	2075	200	150	590	130	150	590	190
	3	300	1600	100	300	1500	200	300	450	130	100	900	190
	4	200	1025	100	400	2075	200	300	450	130	100	900	190
	5	300	1600	100	300	1500	200	300	900	130	100	450	190
	6	200	1025	100	400	2075	200	300	900	130	100	450	190
	7	300	1600	100	300	1500	200	150	450	130	150	900	190
	8	200	1025	100	400	2075	200	150	450	130	150	900	190
	9	300	1600	100	300	1500	200	300	600	130	150	600	190
	10	200	1025	100	400	2075	200	300	600	130	150	600	190

Notes:

1 - L: left turn; T: through; R: right turn.

2 - Eastbound and westbound movements are on the crossroad; northbound and southbound turn movements are on the ramps; northbound and southbound through movements are on the major road.

Table 3-7. Unsignalized Interchange Basic Movement Volume Scenarios.

Scenario		Volume by Approach and Basic Movement,[1, 2] veh/h											
Level	No.	Northbound			Southbound			Eastbound			Westbound		
		L	T	R	L	T	R	L	T	R	L	T	R
High	1	200	950	175	300	1500	200	400	500	150	400	500	150
	2	150	675	175	350	1775	200	400	500	150	400	500	150
	3	200	950	175	300	1500	200	200	600	150	500	400	150
	4	150	675	175	350	1775	200	200	600	150	500	400	150
	5	200	950	175	300	1500	200	200	400	150	500	600	150
	6	150	675	175	350	1775	200	200	400	150	500	600	150
	7	200	950	175	300	1500	200	400	400	150	400	600	150
	8	150	675	175	350	1775	200	400	400	150	400	600	150
	9	200	950	175	300	1500	200	200	500	150	500	500	150
	10	150	675	175	350	1775	200	200	500	150	500	500	150
Mod-erate	1	151	750	97	194	975	129	204	342	92	204	342	105
	2	108	525	97	237	1225	129	204	342	92	204	342	105
	3	151	750	97	194	975	129	151	355	92	237	366	105
	4	108	525	97	237	1225	129	151	355	92	237	366	105
	5	151	750	97	194	975	129	151	366	92	237	355	105
	6	100	475	90	220	1125	120	140	340	86	220	330	98
	7	140	700	90	180	900	120	190	250	86	190	420	98
	8	100	475	90	220	1125	120	190	250	86	190	420	98
	9	140	700	90	180	900	120	140	320	86	230	320	98
	10	100	475	90	220	1125	120	140	320	86	230	320	98
Low	1	165	875	55	165	825	110	83	325	72	83	325	105
	2	110	575	55	220	1125	110	83	325	72	83	325	105
	3	165	875	55	165	825	110	165	248	72	55	495	105
	4	110	575	55	220	1125	110	165	248	72	55	495	105
	5	165	875	55	165	825	110	165	495	72	55	248	105
	6	100	525	50	200	1025	100	150	450	65	50	225	95
	7	150	800	50	150	750	100	75	225	65	75	450	95
	8	100	525	50	200	1025	100	75	225	65	75	450	95
	9	150	800	50	150	750	100	150	300	65	75	300	95
	10	100	525	50	200	1025	100	150	300	65	75	300	95

Notes:
1 - L: left turn; T: through; R: right turn.
2 - Eastbound and westbound movements are on the crossroad; northbound and southbound turn movements are on the ramps; northbound and southbound through movements are on the major road.

- Scenarios 1 & 2: equal through volumes and equal left-turn volumes.
- Scenarios 3 & 4: unequal left-turn volumes and unequal through volumes where the heavier through volume opposes the heavier left-turn volume.
- Scenarios 5 & 6: unequal left-turn volumes and unequal through volumes where the heavier through volume opposes the lighter left-turn volume.
- Scenarios 7 & 8: equal left-turn volumes and unequal through volumes.
- Scenarios 9 & 10: unequal left-turn volumes and equal through volumes.

The exit-ramp volumes within each of the crossroad scenario pairs in the list above were also varied. In general, the volumes reflect unequal left-turn volumes; however, the degree to which the left-turn volumes are unequal in any pair ranges from negligible to more than a factor of two. This range is intended to reflect off-peak and peak-hour traffic demand patterns on the exit ramps.

The average free-flow traffic speed used in the simulation was set as 84 percent of the design speed (which was assumed to represent the 95th percentile speed). Ramp design speeds and associated controlling radii are listed in Table 3-8. The major-road design speed was defined to be 65 mph. The crossroad design speed was defined to be 45 mph. A range of crossroad speeds was considered in the development of the simulation scenarios. However, Bonneson and Lee (*8*) found that speeds in the range of 35 to 45 mph did not have a significant effect on interchange delay.

Table 3-8. Characteristics of Typical Ramps Used with Service Interchanges.

Ramp Function	Characteristic	Ramp Configuration		
		Diagonal	Outer Connection	Loop
	Interchange Type	Diamond	Parclo	Parclo[1]
Exit	Ramp Design Speed (based on a 65-mph major road design speed)	45 mph	55 mph	30 mph
	Controlling Radius[2] (for above ramp design speed)	540 ft (Exhibit 3-43)	965 ft (Exhibit 3-14)	230 ft (Exhibit 3-43)
Entrance	Ramp Design Speed (based on a 45-mph crossroad design speed)	35 mph	40 mph	25 mph
	Controlling Radius[2] (for above ramp design speed)	310 ft (Exhibit 3-43)	465 ft (Exhibit 3-14)	150 ft (Exhibit 3-43)

Notes:
1 - Exit loop ramp speed and radius applies to parclo B and parclo B (2-quad). Entrance loop ramp speed and radius applies to parclo A and parclo A (2-quad).
2 - Controlling radii obtained from exhibits indicated in Reference *1*. A maximum superelevation rate of 8.0 percent was used with Exhibit 3-14 to obtain the controlling radius for the outer connection ramp.

Table 3-9 describes the traffic characteristics used for the signal-controlled movements. The saturation flow rate was set to 2000 veh/h/ln based on the recommendations of Bonneson and Messer (*32*). The start-up lost time for the first vehicle was set at 2.0 s. The adaptation of these base values to the actual discharge rates for the turn movements was achieved by adjusting the average turn speed. In this regard, the speed shown for the SPUI reflects left-turn and right-turn radii of 300 ft and 50 ft, respectively. For the other interchange types, a left-turn radius of 75 ft was assumed.

Table 3-9. Traffic Characteristics for Signal-Controlled Movements.

Traffic Characteristic	Interchange Type	Traffic Movement		
		Left Turn	Through	Right Turn
Average Turn Speed, ft/s	SPUI	44	not applicable	19
	All (except SPUI)	24	not applicable	19
Saturation Flow Rate, veh/h/ln	All	2000	2000	2000
Start-Up Lost Time, s	All	2.0	2.0	2.0

Signal Control. As indicated in Table 3-4, both pretimed and actuated control modes are considered for the signalized interchanges. The details describing how these modes are modeled in the simulation software is the subject of this section.

Signal Timing. In general, the signal phase sequence and controller type selection was tailored to each of the interchange types. The objective in making these selections was to provide the most efficient signal phasing possible and to remain consistent with typical practice regarding interchange signalization. The signal control characteristics used in the simulation experiments are listed in Table 3-10.

Table 3-10. Signal Control Characteristics.

Interchange Type	Signal Controllers		Coordination Reference Phases [1,2]	Phase Sequence Description	Left-Turn Phase Order [1]
	Number	Type			
SPUI	1	Actuated	n.a.	Quad-left phasing (Fig. 3-1)	Leading
TUDI	1	Actuated	n.a.	Four-phase w/transition (Fig. 3-3)	Leading
Compressed Diamond	1	Actuated	n.a.	Three-phase (Fig. 3-6)	Lagging
Conventional Diamond	2	Pretimed	Through	Three-phase	Lagging
Parclo A	2	Pretimed	Through	Two-phase	n.a.
Parclo A (2-quad)	2	Pretimed	Exit ramp	Three-phase	Leading
Parclo B	2	Actuated [3]	n.a.	Two-phase	n.a.
Parclo B (2-quad)	2	Pretimed	Through	Three-phase	Lagging

Notes:
1 - n.a.: not applicable.
2 - Common reference phase at each ramp terminal.
3 - Coordination between ramp terminal signals is not required as the "outbound" crossroad through movement is provided a green indication throughout the signal cycle.

As indicated in Table 3-10, the pretimed interchanges required the identification of a coordination reference phase. One reference phase was established at each ramp terminal. The start of the reference phase at one terminal was coincident with the start of the reference phase at the other terminal. The reference phases always served the same traffic movement at each terminal (e.g., the two "inbound" crossroad through movements, the two exit-ramp left-turn movements, etc.).

In general, crossroad left-turn phases lagged (or followed) the crossroad through phases at interchanges with three-phase operation. One exception to this rule was at the parclo A (2-quad). At this interchange, the crossroad left-turn phases led the through phases because of the benefit it offered in terms of progression efficiency. Drivers turning left from the crossroad were served by a "protected" signal phase. Permitted or protected-permitted left-turns were not allowed.

The duration of the yellow and all-red intervals was computed using the procedure described in a report published by the Institute of Transportation Engineers (ITE) (*33*). The computed intervals are listed in Table 3-11.

Table 3-11. Yellow and All-Red Interval Duration.

Interchange Type	Interval Duration by Approach and Movement,[1] s					
	Exit-Ramp Left Turn		Crossroad Left Turn		Crossroad Through	
	Yellow	All-Red	Yellow	All-Red	Yellow	All-Red
SPUI - 200-ft ramp sep.[2, 3]	4	5	4	4	4	4
SPUI - 300-ft ramp sep.[2, 3]	4	5	4	5	4	6
Diamond	4	2	4	2	4	1
Parclo A	4	2	n.a.	n.a.	4	1
Parclo A (2-quad)	4	2	4	2	4	1
Parclo B	n.a.	n.a.	4	2	4	1
Parclo B (2-quad)	3	2	4	2	4	1

Note:
1 - n.a.: not applicable.
2 - All-red intervals shown are based on clearance path lengths for a SPUI having yield-controlled exit-ramp right-turn movements. For signal controlled exit-ramp right-turns, it could be rationalized that the all-red intervals for the exit-ramp left-turn and the crossroad through movements should be 1 to 2 s longer than those shown in the table. This increase was not applied in this research because it is believed that the resulting all-red intervals would be perceived as unreasonably long and not a reflection of actual practice.
3 - Signalized exit-ramp right-turn movements at the SPUI had yellow and all-red intervals of 4.0 and 0.0 s, respectively.

An analysis of CORSIM output indicated that CORSIM does not properly model right-turn-on-red (RTOR) during the opposing left-turn phase. During this interval, RTOR drivers turn with little regard for the presence of crossing left-turning vehicles. In contrast, RTOR is properly modeled during the crossroad through phase.

All phases can be coded in CORSIM as allowing RTOR or not allowing RTOR. Phases that were not opposed by a left-turn phase were coded as "RTOR allowed." RTOR was also allowed for phases that were opposed by a left-turn phase *provided* that: (1) the departure leg had more traffic lanes than the opposing left-turn movement, (2) left-turn vehicles turned into the inner-most lanes on the departure lane, and (3) the right-turn was associated with a lengthy green phase. The first two criteria were satisfied by all interchange types; however, only the diamond and parclo types satisfied the third criterion. The "opposed" right-turn movements at these two interchange types turn from the crossroad and thereby, have a lengthy green phase. It was decided to "disallow RTOR" for the SPUI exit-ramp right-turns as they are not associated with a lengthy green phase.

Details of Actuated Signal Control. The minimum green intervals varied by interchange type and phasing. For the SPUI and parclo B, a minimum green interval of 8.0 s was used for all phases. For the TUDI, the minimum green intervals were defined using the "rules-of-thumb" coined by de Camp (*34*). Specifically, the two crossroad through phases each have minimums equal to the larger of 8.0 s or *twice* the travel time between ramps less 6.0 s. The concurrent, or transition, phase (i.e., phase 3 and phase 7 in Figure 3-4) serving the ramp left-turn movement has a minimum green equal to the travel time less 7.0 s. Similarly, the primary phase (i.e., phase 4 and phase 8) serving the ramp left-turn movement has a minimum green equal to the travel time less 4.0 s. Finally, both internal left-turn phases (i.e., phases 1 and 5) have minimums equal to 8.0 s.

For the compressed diamond, the minimum green intervals were defined using the techniques recommended by Bonneson and Lee (*35*). In this regard, the exit-ramp minimums are set equal to 8.0 s. The crossroad through phase minimums are set equal to the larger of 8.0 s or the travel time between ramps plus 1.0 s. Finally, the crossroad left-turn phase minimum green intervals are set equal to the larger of 8.0 s or the travel time minus 10 s.

A maximum green interval of 50 s is used for all phases at all interchange types. For all signalized lanes, a stop line detection zone of 40 ft was used with a passage time of 2.0 s. No advance detection was used on any approach except on the exit-ramp approaches for the TUDI. On these approaches, an advance detector was used to ensure efficient use of the concurrent phase. The details of this detection design are described by Venglar et al. (*36*).

Details of Pretimed Signal Control. The green interval durations for the pretimed phases were determined using the procedure described in Appendix B of Chapter 16 of the *HCM* (*12*). This procedure allocates the cycle length among the phases such that the resulting volume-to-capacity ratios of the critical phases are equal. In some instances, this procedure yields unrealistically short green interval durations. In these instances, a practical minimum green of 8 s was imposed.

The cycle length was estimated using an equation derived by Webster (*37*). This equation estimates the cycle length that yields minimal delay. In some instances, this equation predicts unrealistic cycle lengths. Hence, practical limits were imposed such that no cycle length was less than 60 s or more than 125 s.

Some additional constraints on phase duration were used in order to maintain efficient coordination between the two ramp terminals. These constraints are summarized in Table 3-12. Whenever these constraints were not satisfied, the cycle length was adjusted (away from the optimal cycle length obtained from the Webster equation) and the green intervals re-computed using the *HCM* allocation procedure such that the resulting green intervals satisfied the additional constraints.

Table 3-12. Constraints Used to Determine Optimal Pretimed Phase Duration.

Interchange Type	Constraint	Green Interval Constraints by Approach and Movement, s		
		Exit Ramp	**Crossroad**	
		Left Turn	**Left Turn**	**Through**
Conventional Diamond	Maximum	Only via maximum cycle of 125 s.	Less than travel time.	Only via maximum cycle of 125 s.
	Minimum	8 s or more.	8 s or more.	Sufficiently long to clear queue upstream, allow it to travel to the downstream terminal, and clear this terminal before the end of its crossroad left-turn phase.
Parclo A	Maximum	not applicable.	Less than travel time.	Only via maximum cycle of 125 s.
	Minimum		8 s or more.	Sufficiently long to clear queue upstream, allow it to travel to the downstream terminal, and clear this terminal before the end of its crossroad through phase.
Parclo A (2-quad)	Maximum	Sum of both greens is less than travel time.		Only via maximum cycle of 125 s.
	Minimum	8 s or more.	8 s or more.	Sufficiently long to clear queue upstream, allow it to travel to the downstream terminal, and clear this terminal before the end of its crossroad through phase.
Parclo B (2-quad)	Maximum	Same as conventional diamond.		
	Minimum	Same as conventional diamond.		

The constraints listed in Table 3-12 are intended to ensure efficient traffic progression between the ramp terminals. The exit-ramp green intervals for the conventional diamond, parclo A, and parclo B (2-quad) were checked to ensure that they did not exceed the travel time between ramp terminals. This approach is intended to progress ramp traffic through the downstream ramp terminal without stopping. The crossroad through phase of these same interchange types was also checked to ensure that it was long enough to allow through traffic to progress through the downstream ramp terminal. Finally, the green intervals of the exit-ramp and crossroad left-turn phases at the parclo A (2-quad) were checked to ensure that their sum did not exceed the travel time between ramp terminals. This constraint was needed to ensure efficient progression for ramp traffic.

Green interval durations were computed (using the aforementioned procedure) for each of the 30 volume levels listed in Table 3-6. Then, for each of the three volume levels (i.e, high, moderate, and low), one average green interval was computed for each signal phase. This average green was then used during the simulation for the corresponding 10 volume scenarios within each volume level. In this manner, the rigidity of pretimed control was replicated in the simulation experiments and was reflected in the resulting delays.

Performance Measures. Two key performance measures were extracted from the simulation output. One measure was control delay for each traffic movement traveling through the interchange. This delay was combined with loop travel time to compute interchange delay (using Equation 20). The second measure extracted was average back of queue for the exit-ramp movements.

Simulation Scenarios

The previous section described an experimental design that consists of 30 volume scenarios, 6 geometry scenarios, and 16 interchange type scenarios. The product of these scenarios yields 2880 unique factor combinations to be simulated. For each combination, one 1-hour simulation was conducted to produce one interchange delay estimate and two queue length estimates (one for each exit ramp). The resulting delay statistics produced by the 1-hour simulations were found to be sufficiently precise as to yield accurate predictive relationships relating interchange type, right-turn control mode, traffic volume, and performance.

Field Data Collection Plan

A field data collection plan was developed for the purpose of acquiring the data needed to calibrate the simulation software prior to its use in the simulation experiments. The data identified in this plan included traffic volume, signal phase duration, and delay. These data were collected with video camcorders located discretely in the vicinity of each of three interchanges. A survey of each interchange's geometric layout, lane markings, speed limit, and signing was also conducted prior to the field data collection. A description of the data collected and the location of the interchanges studied is provided in the following paragraphs.

Field Data

Several types of data were needed to calibrate the simulation software model. Data that served as inputs to the software included the geometry, traffic characteristics, and traffic control mode present at the interchange. Delay data were needed to confirm the performance prediction accuracy of the simulation software. The specific data collected during the field studies, and their method of collection, are listed in Table 3-13.

Also listed in Table 3-13 is the method used to collect the corresponding data. Traffic volume, yellow duration, phase green duration, and delay data were extracted from videotape recordings of each external interchange approach. The methods described in Appendix A of Chapter 16 of the *HCM* (*12*) were used to guide the delay study. All videotape-related data were measured during one hour of peak traffic demand. Prior to conducting the field studies, each interchange study site was surveyed to gather some basic information about relevant geometric details and traffic control devices. These data were supplemented with agency records to obtain a complete description of the interchange geometry and signalization.

Table 3-13. Field Data Types and Collection Method.

Category	Data Type	Data Collection Method			
		Field Study		Site Survey	Agency Files
		Videotape	Manual		
Geometry	Number and width of traffic lanes				✔
	Ramp separation distance				✔
	Turn bay length				✔
	Photo log			✔	
	Horizontal layout in plan view				✔
Traffic characteristics	Traffic counts by movement	✔			
	Percent heavy vehicles	✔			
Traffic control	Approach speed limit			✔	
	Phase sequence		✔		✔
	Yellow warning interval duration	✔			
	All-red clearance interval duration		✔		
Performance measures	Green interval duration	✔			
	Delay by movement	✔			

Site Selection

Three interchanges were selected for field study. These interchanges were selected to collectively represent urban and rural conditions, signalized and unsignalized operation, and diagonal and loop ramp geometry. Collectively, the interchanges exhibited moderate-to-high traffic volumes during the study hour. The interchanges selected for field study are identified in Table 3-14.

Table 3-14. Interchange Study Sites.

District	County	Interchange Address	Nearest City	Area Type	Interchange Type	Traffic Control
Bryan	Brazos	F.M. 60 & F.M. 2154	College Station	Suburban	Parclo A/B (2-quad)	Signalized
		F.M. 2818 & F.M. 60	College Station	Rural	Diamond	Unsignalized
Austin	Travis	Loop 360 & R.M. 2222	Austin	Urban	Diamond	Signalized

Data Collection Plan

During the study of each interchange, traffic conditions were recorded on four approach legs—two exit-ramp approaches and two external crossroad approaches. Videotape recorders and manual observers will be the primary data collection methods. Following the study, the videotape was replayed in the laboratory and data were extracted from it. These data were then combined with the manual observations and used to calibrate the simulation software.

Each field study took place during one hour of one weekday at each interchange. More specifically, the traffic and performance data for all four interchange approaches were collected simultaneously during the peak traffic hour. Data describing interchange geometry were collected before the traffic and performance data collection activities. One or more plan sheets showing the horizontal and vertical geometry of each interchange were obtained from the appropriate TxDOT district headquarters.

During each field study, one camcorder was positioned on each of four interchange approaches. The clock on each camcorder was set to a common time to facilitate the synchronization of event times during data extraction. Each camcorder was mounted on a tripod and located behind the curb. Whenever possible, the camera was placed in such a manner as to minimize any influence that its presence had on driver behavior. Each camcorder field of view was adjusted such that it afforded a view of both the traffic events and the signal indication on one ramp terminal approach. The location of each camera for the study of a diamond interchange is shown in Figure 3-14. Similar locations were used at the parclo A/B.

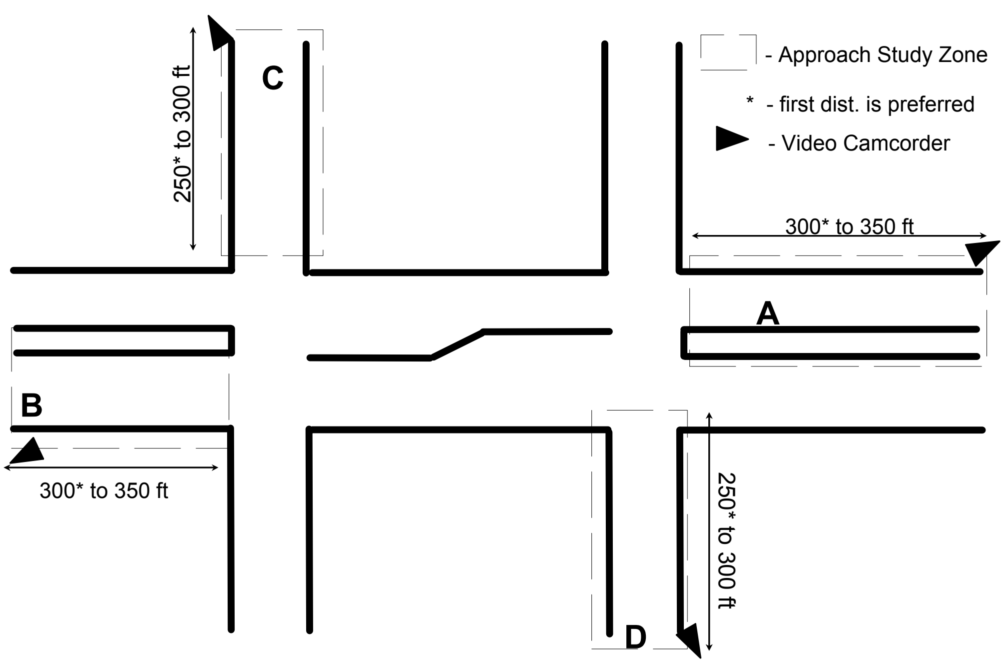


Figure 3-14. Typical Camcorder Locations during the Field Study.

CHAPTER 4. DATA ANALYSIS

OVERVIEW

This chapter summarizes an analysis of the factors influencing interchange delay. The factors considered include: interchange type, cross section, right-turn control mode, and traffic volume. The analysis of these factors is based on the critical flow ratio and delay prediction models described in Chapter 3. The calibration of these models and their use in evaluating alternative ramp configurations is the subject of separate sections in this chapter.

SIMULATION SOFTWARE CALIBRATION

Field Data Summary

This section summarizes the data collected at the interchange field study sites. Initially, the geometric characteristics of each interchange are described. Then, the traffic flow, traffic control, and performance characteristics of each interchange are summarized.

Geometric Characteristics

Table 4-1 lists the geometric characteristics of the field study sites. As the information in the table indicates, both the parclo and the diamond interchange forms were included in the study. The interchange of F.M. 60 and F.M. 2154 is a parclo A/B (2-quad). This interchange provides a loop ramp for vehicles exiting the major road in one travel direction and a diagonal ramp for vehicles exiting from the other direction. This ramp configuration is dictated by the close proximity of a railroad track to the crossroad.

The interchange at F.M. 2818 and at Loop 360 both have a diamond interchange configuration. The interchange at F.M. 2818 was unique in that it served left turns from the major road at two locations. Left turns from the major road were served via a left turn at the intersection of the exit ramp with the crossroad. This turn location is provided at all diamond interchanges. However, left turns from the major road could also be completed by a direct left turn off of the major road (across the median) and onto the *far-side* exit ramp. In this manner, the left-turning driver would ultimately make a right turn at the intersection of the exit ramp with the crossroad. This dual left-turn capability tended to increase the exit-ramp right-turn volumes and decrease the exit-ramp left-turn volumes, as measured at the ramp terminals.

Traffic Volume

Calibration data were collected during the peak traffic hour at each of the three study sites. The volumes observed during the study hour are replicated in Table 4-2. It should be noted that the volumes listed represent the basic movement volumes, as previously depicted in Figure 3-8.

Table 4-1. Study Site Geometric Characteristics.

Location [1]	Major Road Dir. [2]	Inter-change Type	Ramp Separ-ation, ft	Number of Lanes by Approach and Basic Movement [3, 4]											
				Northbound			Southbound			Eastbound			Westbound		
				L	T	R	L	T	R	L	T	R	L	T	R
F.M. 60 & F.M. 2154	E-W	Parclo A/B	480	1	2	1	1	2	0	1	n.a.	0	1	n.a.	1
F.M. 2818 & F.M. 60	N-S	Diamond	800	1	n.a.	1	1	n.a.	1	1	2	free	1	2	free
Loop 360 & R.M. 2222	N-S	Diamond	500	2	n.a.	1	2	n.a.	1	1	2	free	1	2	free

Notes:

1- Major road listed first and crossroad listed second.

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

3 - L: left turn; T: through, R: right turn. Basic movements for interchanges are identified in Figure 3-8.

4 - Free: free right-turn lane with an added lane extending beyond the end of the channelizing island and along the crossroad. "0": no right-turn lane provided. n.a.: not available. Lane assignments were not surveyed for the major-road through movements.

Table 4-2. Study Site Traffic Volumes.

Location	Major Road Dir. [1]	Volume by Approach and Basic Movement, [2,3] veh/h											
		Northbound			Southbound			Eastbound			Westbound		
		L	T	R	L	T	R	L	T	R	L	T	R
F.M. 60 & F.M. 2154	E-W	86	508	414	81	367	11	72	n.a.	77	310	n.a.	100
F.M. 2818 & F.M. 60	N-S	25	n.a.	237	17	n.a.	475	125	295	285	431	488	247
Loop 360 & R.M. 2222	N-S	528	n.a.	363	132	n.a.	1015	430	556	451	351	1118	131

Notes:

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

2 - L: left turn; T: through, R: right turn. Basic movements for interchanges are identified in Figure 3-8.

3 - n.a.: not available. Volumes were not collected for the major-road through movements.

Traffic Control Characteristics

Table 4-3 lists the traffic control characteristics at the two signalized interchanges. These characteristics include the speed limit, phase sequence, yellow interval duration, and all-red interval duration. Both interchanges used one actuated controller to control both ramp terminals simultaneously. With one exception, protected-only left-turn operation was used for all crossroad left-turn movements. At F.M. 60 and F.M. 2154, the southbound left-turn movement from the crossroad used protected-permitted left-turn operation. The interchange at Loop 360 was operated with a single-controller using a four-phase signal sequence. However, the sequence was not observed to employ a concurrent (or fixed transition) interval during the study hour.

Table 4-3. Study Site Traffic Control Characteristics.

Location	Speed Limit,[1] mph	Phase Sequence	Ramp Terminal	Phase Number					
				1	2	4	5	6	8
				Yellow Interval (All-Red Interval),[2,3] s					
F.M. 60 & F.M. 2154	40	Three phase (leading lefts)	Left (B)	4 (1.5)	4 (2)	--	--	4 (2)	4 (1.5)
			Right (A)	4 (1.5)	4 (2)	--	--	4 (2)	4 (1.5)
Loop 360 & R.M. 2222	45	Four phase (no transition)	Left	4 (0)	4 (0)	4 (0)	--	4 (0)	--
			Right	--	4 (0)	--	4 (0)	4 (0)	4 (0)

Notes:
1 - Speed limit on the crossroad.
2 - "--": movement does not exist at this ramp terminal.
3 - Phase numbers coincide with the phase sequence and numbering identified in Figures 3-9 and 3-10. Phases 1 and 5 serve crossroad left-turn movements. Phases 2 and 6 serve crossroad through movements. Phases 4 and 8 serve ramp left-turn movements.

The interchange at F.M. 2818 & F.M. 60 was not signalized. The exit-ramp left-turn movements were stop controlled. The speed limit on the crossroad (i.e., F.M. 60) is 55 mph.

Operational Characteristics

The operational characteristics for the three field study sites are summarized in Table 4-4. These characteristics include the cycle length, green interval duration, and control delay for the interchanging traffic movements. Their measurement was based on event times extracted from the videotapes recorded during the field study. Each characteristic was measured once during each signal cycle (or every 90-s interval at the unsignalized interchange). The delay data were collected and reduced using the technique described in Appendix A of Chapter 16 of the *HCM* (*12*). The green interval statistics listed in the table represent averages of the individual cycle observations.

As indicated by the data in Table 4-4, a wide range in green interval duration and control delay was found in the collective set of interchange sites. The delays range from level of service (LOS) A to LOS F (*12*). In fact, the collective set of traffic movements at the first two interchanges listed exhibit delays ranging from LOS A to LOS E. The third interchange exhibited demands that exceeded capacity for the westbound crossroad through movement during most of the study hour. The resulting delay of 160 s/veh was the largest observed among all study sites and traffic movements.

CORSIM Adaptation

Several techniques were used to adapt the CORSIM software so that it could accurately model signalized and unsignalized interchanges. For example, it was learned that CORSIM (Version 5.0) was not able to accurately model unsignalized intersection operations. As a result, it was necessary to use Version 5.1 for these interchanges because this version included an improved algorithm for modeling gap acceptance by drivers turning left at stop-controlled approaches. This modeling deficiency did not pose any restriction to the use of Version 5.0 to model signalized interchanges. Version 5.0 was preferred for modeling signalized interchanges because it had previously been adapted by the researchers to operate in a hardware-in-the-loop environment (a capability that was needed to model the single-controller interchanges).

Table 4-4. Study Site Operational Characteristics.

Location	Cycle Length, s	Character-istic	Ramp Terminal	Phase (or Movement) Number[1, 2, 3]					
				1	2	4	5	6	8
F.M. 60 & F.M. 2154	120	Green Interval, s	Left (B)	9	82	--	--	98	12
			Right (A)	2	75	--	--	83	26
		Control Delay, s/veh	Left (B)	10	5	--	--	1	32
			Right (A)	60	5	--	--	7	31
F.M. 2818 & F.M. 60	n.a.	Control Delay, s/veh	Left	14	0	49	--	0	--
			Right	--	0	--	10	0	47
Loop 360 & R.M. 2222	150	Green Interval, s	Left	70	56	12	--	130	--
			Right	--	106	--	72	30	36
		Control Delay, s/veh	Left	n.a.	160	71	--	0	--
			Right	--	0	--	n.a.	63	44

Notes:

1 - Phase numbers coincide with the phase sequence identified in Figures 3-9 and 3-10. For unsignalized interchanges, phase numbers are effectively "movement" numbers. Phases 1 and 5 serve crossroad left-turn movements. Phases 2 and 6 serve crossroad through movements. Phases 4 and 8 serve ramp left-turn movements.

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

3 - n.a.: not available. Data not collected during field study.

The delay statistic obtained from CORSIM was "queue delay." This delay reflects time spent in queue or moving-up while in a queue. Data published by Zhang et al. (*38*) were used to develop a relationship between queue delay and control delay, the latter being the delay computed by the evaluation model. This analysis indicated that control delay could be estimated using the following relationship:

$$d = 0.74 + 1.04\, d_q \tag{32}$$

where:

d = average control delay, s/veh; and

d_q = average queue delay, s/veh.

This equation is based on 52 delay observations and has a coefficient of determination R^2 of 0.99.

As noted previously, experience with CORSIM indicated that it was unable to accurately model traffic behavior related to gap acceptance on "short" road segments. This limitation affects simulated drivers who desire to change lanes or turn left through a gap in oncoming traffic. The fundamental problem is that CORSIM is programed to assess the adequacy of gaps by scanning all gaps on the subject road segment. If one of the two vehicles that form a "gap" is traveling on the preceding road segment, then CORSIM does not recognize this vehicle. In this situation, the gap is incorrectly defined by CORSIM as being infinitely long.

To overcome the aforementioned limitation, a minimum length was imposed on several road segments in the coded software representation (i.e., input file) of each interchange. Specifically, the minimum length for the crossroad segment external to the ramp terminal was set at 610 ft. This distance was determined to be adequate for gap acceptance by left-turning vehicles and by those turning right on a red indication. Given the relatively high number of lane changes that occur on the crossroad segment between the two ramp terminals, a minimum length of 800 ft was established for this segment (exceptions to this minimum are noted in the next paragraph). Finally, a minimum length of 300 ft was set for any segment within which a merge operation occurred (e.g., the segment just downstream of a free right-turn lane).

The CORSIM limitation associated with short lengths was a significant concern with regard to the simulation of the TUDI and the compressed diamond. The crossroad segment internal to the two ramps is inherently less than 800 ft for these two interchange forms. To overcome this limitation, the segment length was represented accurately in the coded input file; however, a special modeling technique was used to ensure that the through and left-turning drivers were prepositioned in the proper lane *prior* to reaching the internal segment.

Calibration Results

Calibration Procedure

The calibration activity consisted of a comparison of the operational characteristics measured in the field with those obtained from the simulations. The intent of this exercise was to adjust key parameters in the CORSIM model for the purpose of reconciling differences observed between the field-measured characteristics and those obtained from the simulation. After several iterations in this process, it was determined that the more significant differences were due to the following two causes:

1. Significant deviations in any one pair of characteristics that were explained by the occurrence of random events, as opposed to an input error or inappropriate parameter setting.

2. Consistent deviations in overall average delay (or green interval duration) at any one interchange.

In general, the initial calibration activities focused on improving the software representation of the simulated interchange (i.e., minimize errors due to Cause 2 above). In many instances, the deviations were often caused by subtle differences between the actual interchange's signal operation (or geometry) and the representation of this operation (or geometry) as coded in the simulation software. Model parameters were not adjusted when deviations resulted from Cause 1.

If a deviation was due Cause 2 but no further improvements could be made in the software representation of the existing conditions, then an adjustment to a CORSIM input parameter was considered. Adjustments of this type that were considered include: average queue discharge headway, average start-up lost time, acceptable left-turn gap distribution, average free-flow speed, and free-flow speed distribution.

Analysis and Findings

The operational characteristics obtained from the simulation of each interchange are shown in Table 4-5. The statistics listed represent an average for the simulated hour.

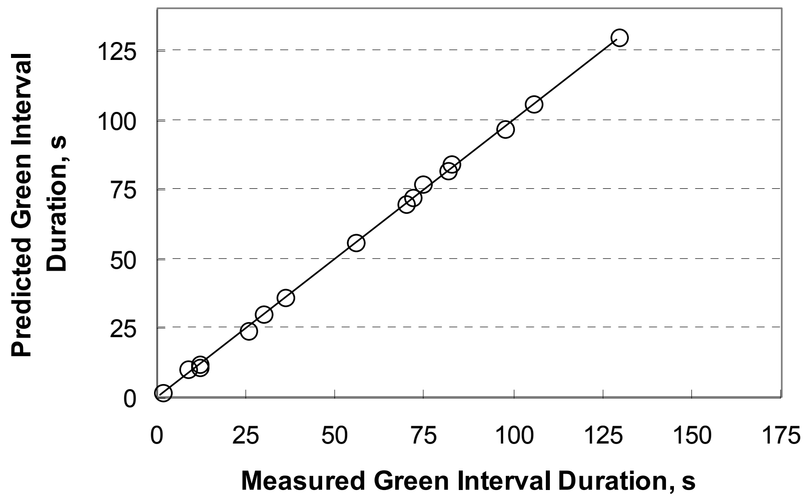
Table 4-5. Simulated Interchange Operational Characteristics.

Location	Cycle Length, s	Character-istic	Ramp Terminal	Phase (or Movement) Number [1,2,3]					
				1	2	4	5	6	8
F.M. 60 & F.M. 2154	120	Green Interval, s	Left (B)	10	82	--	--	97	11
			Right (A)	2	77	--	--	84	24
		Control Delay, s/veh	Left (B)	5	3	--	--	1	25
			Right (A)	61	2	--	--	4	35
F.M. 2818 & F.M. 60	n.a.	Control Delay, s/veh	Left	5	0	25	--	0	--
			Right	--	0	--	14	0	53
Loop 360 & R.M. 2222	150	Green Interval, s	Left	70	56	12	--	130	--
			Right	--	106	--	72	30	36
		Control Delay, s/veh	Left	n.a.	150	65	--	0	--
			Right	--	0	--	n.a.	55	45

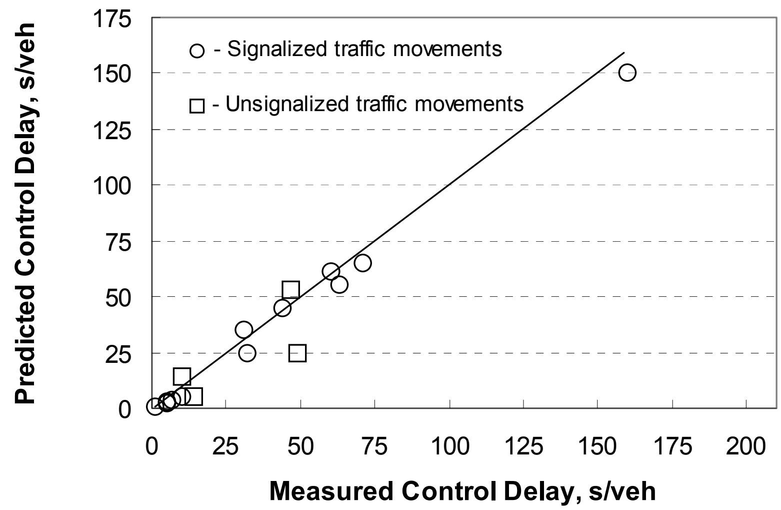
Notes:
1 - Phase numbers coincide with the phase numbering identified in Figures 3-9 and 3-10. For unsignalized interchanges, phase numbers are effectively "movement" numbers. Phases 1 and 5 serve crossroad left-turn movements. Phases 2 and 6 serve crossroad through movements. Phases 4 and 8 serve ramp left-turn movements.
2 - "--": movement does not exist at this ramp terminal.
3 - n.a.: not available.

The statistics listed in Table 4-5 were obtained by carefully coding the input file associated with each interchange. In this regard, several iterations were required to eliminate coding inaccuracies. Further adjustments to key CORSIM parameters (by varying these parameters from their default values) did not lead to significant additional improvement in terms of agreement between the predicted and observed values.

A comparison of the statistics listed in Table 4-5 with those in Table 4-4 indicated fairly good agreement. A graphical comparison of these statistics are shown in Figure 4-1. The line shown is *not* a line of "best fit" to the data shown. Rather, it represents a line where measured values would be equal to predicted values. Any deviation from this line is an indication of less than perfect agreement between the simulation output and the field observations.



a. Green Interval Duration.



b. Control Delay.

Figure 4-1. Comparison of Simulation Predictions with Field-Measured Data.

The trends in Figure 4-1 indicate very good agreement between the measured and predicted values using CORSIM's default parameter values. From this analysis, it was concluded that CORSIM (with its default values) was able to predict green interval duration and control delay at signalized and unsignalized interchanges with reasonable accuracy.

DELAY MODEL CALIBRATION

This section describes the calibration of the delay models developed in Chapter 3. The data obtained from the simulation software were used for this purpose. One delay model was calibrated for each interchange-type and control-mode combination. The signalized interchange models and the unsignalized interchange models are discussed in separate sections due to their differences in delay model structure.

Signalized Interchange Model Calibration

The model calibration activity for the signalized interchange consisted of computing the sum-of-critical-flow-ratios for each of the simulation scenarios. Then, interchange delay was computed using Equation 20. The control delay contribution to interchange delay was obtained from the simulation output. The geometric delay associated with the loop ramps was computed using Equation 21. Regression analysis was used to calibrate the delay model. The delay data are shown in Figure 4-2 for the SPUI with a 300-ft ramp separation distance. Similar relationships were found for the other interchange types evaluated.

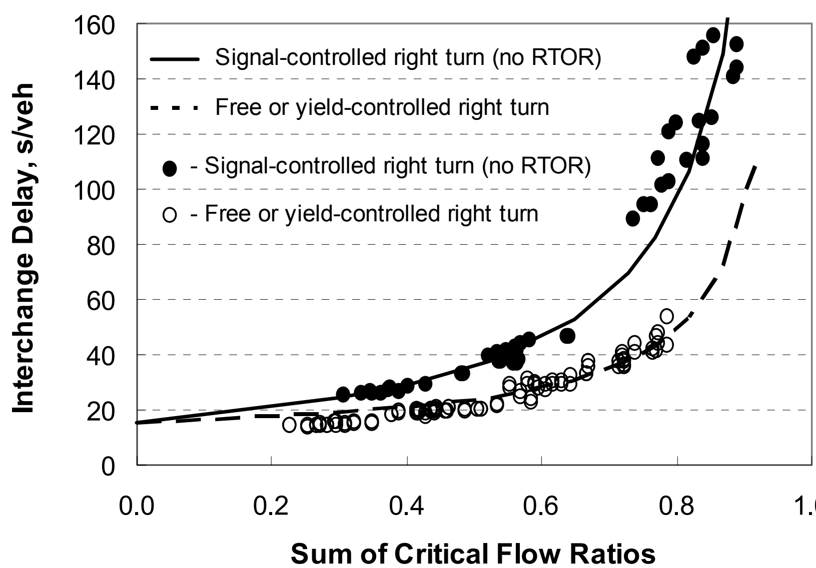


Figure 4-2. Relationship between Critical Flow Ratio and Signalized Interchange Delay.

The data in Figure 4-2 confirm that there is a strong relationship between the sum-of-critical-flow-ratios and interchange delay. The data also indicate that the interchanges with free right-turn lanes or with yield-controlled right-turn movements have lower delay than those interchanges where the right-turns are served by a signal. The difference in delay is significant because right-turn-on-red (RTOR) was not allowed at the SPUI (see discussion on pages 3-34 and 3-35). This difference was smaller at the other interchange types because RTOR was allowed at them.

The trend lines in Figure 4-2 represent curves of "best-fit" based on a regression analysis using Equation 24. However, it was modified to include an "indicator" variable to capture the effect of right-turn control mode (i.e., signal controlled, yield controlled, or uncontrolled with a free right-turn lane). A similar modification was made to Equation 25 for interchanges with two controllers. The modified form of Equations 24 and 25 are shown below as Equations 33 and 34, respectively:

$$d_I = b_0 + (b_1 + b_2 I_{y/f}) \frac{Y_c}{1 - Y_c} \tag{33}$$

$$d_I = b_0 + (b_1 + b_2 I_{y/f}) \frac{Y_{c,\max}}{1 - Y_{c,\max}} \tag{34}$$

where:

d_I = interchange delay, s/veh;
Y_c = sum of the critical flow ratios (single-controller interchange);
$Y_{c,max}$ = largest sum-of-critical-flow-ratios for two ramp terminals (two-controller interchange);
$I_{y/f}$ = indicator variable for right-turn control mode (= 0 for signal control, 1 otherwise); and
b_i = regression coefficients, i = 0, 1, ..., n.

As Equation 34 indicates, the second term in Equation 25 was eliminated from the final equation form. This second term was intended to represent the delays incurred at the ramp terminal that had the smaller sum-of-critical-flow-ratios. However, initial analyses using Equation 25 revealed that the second term did not contribute significantly to the model's predictive ability so it was eliminated.

The calibration coefficients obtained from the regression analysis are listed in Table 4-6. The corresponding trends for each interchange type are provided in the Appendix. Several trends can be seen in the calibration coefficients in this table. The intercept coefficient b_0 indicates the minimum delay under low-volume conditions. In general, the parclo A (2-quad) and parclo B (2-quad) interchange types have large values for this coefficient because of the geometric delay associated with their loop ramps. This coefficient is lower for the parclo A and parclo B because they eliminate one signalized traffic movement (which offsets the increased delay due to loop running time).

Table 4-6. Calibration Coefficients for Signalized Delay Model.

Interchange Type	Ramp Separation Distance, ft	Calibration Coefficients			Regression Statistics		
		b_0 s/veh	b_1	b_2	Obser-vations	Standard Deviation, s/veh	Coefficient of Determination (R^2)
SPUI	200	14.5	18.9	-11.4	150	7.9	0.95
	300	15.7	19.9	-11.6	150	8.7	0.94
TUDI	300	13.4	14.2	-1.4	150	4.2	0.93
Compressed dia.	700	19.2	9.4	-0.8	150	9.7	0.83
Conventional dia.	1100	17.1	5.0	-0.4	150	4.4	0.82
Parclo A	800	11.7	7.8	-1.2	180	2.1	0.91
Parclo A (2-quad)	800	19.1	8.3	-2.0	150	2.0	0.80
Parclo B	1200	9.3	3.5	-0.1	180	3.7	0.89
Parclo B (2-quad)	1200	26.2	3.9	-0.7	150	3.9	0.79

The "shape coefficient" b_1 tends to be influenced by ramp separation distance. For the SPUIs, it increases with an increase in ramp separation distance. This trend reflects the increase in delay associated with the longer all-red clearance intervals used at wider SPUIs. For the other interchange types listed in Table 4-6, there is a clear trend toward a decreasing value of b_1 (and the sum of b_1 and b_2) with increasing ramp separation distance. This trend reflects the increasing likelihood of queue interaction and spillback between ramp terminals associated with the narrower interchanges. This trend is shown in Figure 4-3.

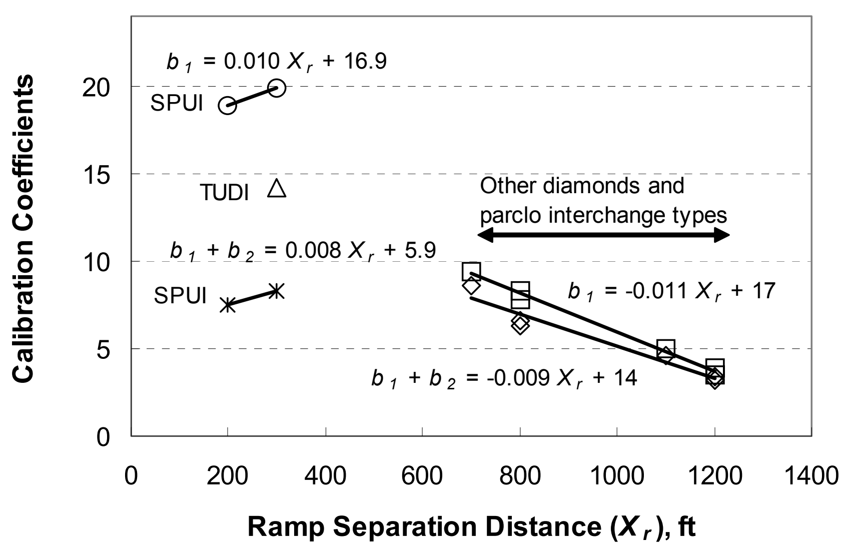


Figure 4-3. Effect of Distance on Calibration Coefficients for Signalized Interchanges.

The trend in Figure 4-3 indicates that the shape coefficient b_1 decreases linearly with ramp separation distance for all interchange types, except the SPUI. It would be incorrect to extrapolate this trend to the TUDI because its signalization is specifically designed to minimize queues on the crossroad, internal to the ramp terminals. However, spillback in the internal ramp terminal approaches is possible for the other diamond interchange types and the parclos. The interchange delay models described in the Appendix were modified to include this sensitivity to ramp separation distance.

Unsignalized Interchange Model Calibration

The model calibration activity for the unsignalized interchange consisted of computing the volume-to-capacity ratios for the crossroad and ramp left-turn movements for each of the simulation scenarios. Then, interchange delay was computed using Equation 20. The control delay contribution to interchange delay was obtained from the simulation output. The geometric delay associated with the loop ramps was computed using Equation 21. Regression analysis was used to calibrate the delay model. The delay data are shown in Figure 4-4 for the TUDI using only the larger of the two ramp left-turn volume-to-capacity ratios. Similar relationships were found for the other interchange types evaluated.

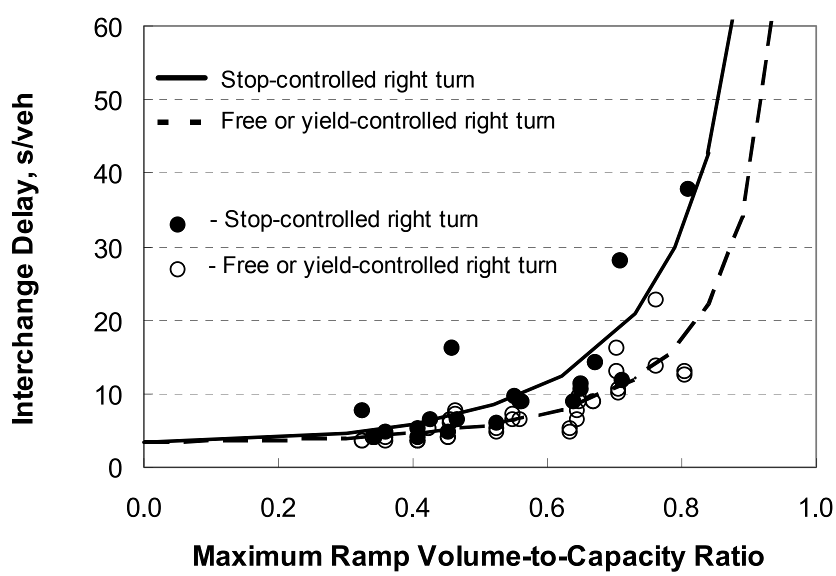


Figure 4-4. Relationship between Volume-to-Capacity Ratio and Unsignalized Interchange Delay.

The data in Figure 4-4 confirm that there is a strong relationship between the maximum ramp volume-to-capacity ratio and interchange delay. The data also indicate that the interchanges with free right-turn lanes or with yield-controlled right-turn movements had slightly lower delay than those interchanges where the right-turns were served at the ramp terminal. The difference in delay is slight because drivers turning right at the ramp terminal typically only had to stop momentarily (i.e., a safe gap was typically available in the outside lane of the conflicting traffic stream within a few seconds of the turning driver's arrival to the stop line).

The trend lines in Figure 4-4 represent curves of "best fit" based on a regression analysis using Equation 26. However, this equation was modified to include an "indicator" variable to capture the effect of right-turn control mode (i.e., stop controlled, yield controlled, or uncontrolled with a free right-turn lane). The modified form of Equation 26 is shown below as Equation 35:

$$d_I = b_0 + (b_1 + b_2 I_{y/f}) \frac{X_{r,\max}^2}{1 - X_{r,\max}} \qquad \textbf{(35)}$$

where:
$X_{r,\,max}$ = larger of the two exit-ramp volume-to-capacity ratios ($X_{r,\,left}$, $X_{r,\,right}$).

As Equation 35 indicates, several terms in Equation 26 were eliminated from the final equation form. These terms were intended to represent the delays incurred by the other left-turn movements. However, initial analyses using Equation 26 revealed that these terms did not contribute significantly to the model's predictive ability so they were eliminated.

One exception to the use of Equation 35 was at the parclo B interchange. This interchange does not have stop-controlled left-turn movements on the exit ramps. Thus, Equation 35 was changed to include the volume-to-capacity ratio of the crossroad left-turn movement. The modified form of this equation is:

$$d_I = b_0 + (b_1 + b_2 I_{y/f}) \frac{X_{c,\max}^2}{1 - X_{c,\max}} \qquad \textbf{(36)}$$

where:
$X_{c,\,max}$ = larger of the two crossroad volume-to-capacity ratios ($X_{c,\,left}$, $X_{c,\,right}$).

The calibration coefficients obtained from the regression analysis are listed in Table 4-7. The corresponding trends are presented graphically in the Appendix. Several trends can be seen in the calibration coefficients in this table. The intercept coefficient b_0 indicates the minimum delay under low-volume conditions. In general, the parclo A (2-quad) and parclo B (2-quad) interchange types have large values for this coefficient because of the geometric delay associated with their loop ramps. This coefficient is lower for the parclo A and parclo B because they eliminate one left-turn movement (which offsets the increased delay due to loop running time).

Table 4-7. Calibration Coefficients for Unsignalized Delay Model.

Interchange Type	Calibration Coefficients			Regression Statistics		
	b_0 s/veh	b_1	b_2	Observations	Standard Deviation, s/veh	Coefficient of Determination (R^2)
TUDI	3.4	8.9	-4.6	60	3.0	0.77
Compressed diamond	2.8	9.0	-2.9	60	3.9	0.73
Conventional diamond	1.8	14.8	-6.8	60	7.6	0.63
Parclo A	7.5	2.6	-0.1	90	7.9	0.98
Parclo A (2-quad)	11.2	13.9	-3.9	60	8.2	0.74
Parclo B	7.1	17.6	-2.9	83	1.2	0.70
Parclo B (2-quad)	12.4	32.9	-11.9	60	2.4	0.84

Similar to that found for the signalized interchanges, the "shape coefficient" b_1 for the diamond interchanges also appears to be influenced by ramp separation distance. Moreover, the intercept b_0 and the "right-turn control mode" coefficient b_2 also have some sensitivity to separation distance. These influences are illustrated in Figure 4-5.

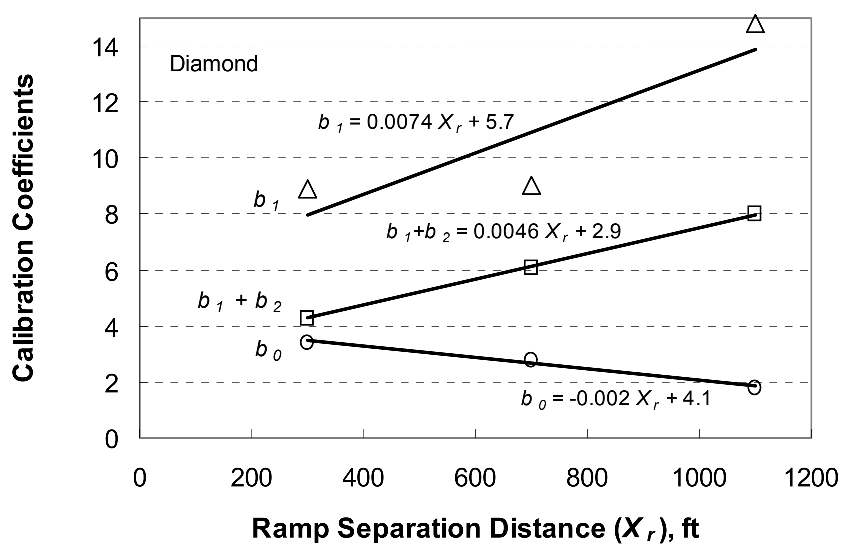


Figure 4-5. Effect of Distance on Calibration Coefficients for Unsignalized Diamonds.

The trends in Figure 4-5 indicates that the coefficient b_1 and the sum of coefficients b_1 and b_2 increase linearly with an increase in ramp separation distance. In contrast, the intercept coefficient b_0 decreases with an increase in distance. Further investigation revealed that these trends were due to differences in the crossroad driver's left-turn deceleration profile, as influenced by ramp separation distance. This deceleration was at a lower, more comfortable rate for the wider ramp separation distances. However, left-turn vehicles that decelerated at the lower rates effectively blocked the conflicting ramp left-turn movement for a much longer period of time than the through vehicles. The diamond interchange delay models described in the Appendix were modified to include this sensitivity to ramp separation distance.

PROCEDURE FOR COMPARING ALTERNATIVE INTERCHANGE TYPES

This section summarizes the procedure for comparing alternative interchange types and ramp configurations. This procedure compares alternatives on the basis of their impact on traffic operations. The procedure is composed of the models and equations described previously in this chapter. The actual procedure to be used for interchange evaluation is provided in the Appendix.

The procedure is based on the critical-movement-analysis approach that forms the basis for the signalized intersection analysis procedure in Chapter 16 of the *HCM* (*12*). The applicability of this approach to interchanges and the relationship between critical flow ratios and delay was demonstrated in the previous sections.

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 given specific volume levels, lane assignments, and ramp separation distances. The delay estimate can be useful for comparing alternative interchange types, ramp configurations, lane configurations, or ramp separation distances. If needed, a more precise delay estimate can be obtained through the direct use of simulation software using the techniques described in Chapter 3.

Assumptions

The procedure is based on three assumptions. First, it is assumed that *one* signal controller is used to control the SPUI, TUDI, and compressed diamond interchange. Two signal controllers are used to control the other interchange types.

Second, it is assumed that cycle time is allocated to the phases in proportion to the critical flow ratio, yielding an equal degree of saturation for all critical movements. This assumption is required for consistency with the critical-movement-analysis approach. It is a reasonable assumption when actuated signal control is used, provided that each phase has a reasonably short minimum green (e.g., 15 s or less) and large maximum green setting (e.g., 50 s or more). It is also a reasonable assumption when pretimed control is used and cycle time is explicitly allocated to the phases in proportion to the critical flow ratio.

Third, it is assumed that the left-turn movements are protected (no permissive operation) and that they are served independently of the adjacent through movement (i.e., that a dual-ring controller is used). This type of left-turn service yields equivalent movement delay for both leading and lagging left-turn phasing. Delay reductions resulting from all-red intervals slightly shorter than those identified in Table 3-11 are not explicitly considered and are reasoned to be negligible.

Steps

The procedure consists of three steps that are completed in sequence. Inputs to the procedure are the basic movement volumes, the movement saturation flow rates, the number of traffic lanes serving the basic movements, and the ramp separation distance. The steps are described as follows:

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. For signalized interchanges, additional information is needed that describes the saturation flow rate for each basic movement s and the number of lanes n allocated to these movements. 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 *HCM* (*12*).

Step 2. Determine the Controlling Volume Ratio

During this step, the movement volume, saturation flow rates, and lane allocations from Step 1 are used with the appropriate equation to compute the controlling "ratio." For signalized interchanges, this ratio is defined to be the sum-of-critical-flow-ratios. For unsignalized interchanges, this ratio is defined to be the maximum volume-to-capacity ratio of the left-turn movements.

Step 3. Determine Interchange Delay

For signalized interchanges, the sum-of-critical-flow-ratios from Step 2 is used with the appropriate equation to estimate interchange delay. Similarly, for unsignalized interchanges, the maximum volume-to-capacity ratio is used to estimate interchange delay. Table 4-8 is then checked to determine the corresponding level of service provided by the interchange.

Level of service is defined by the *HCM* (*12*) to reflect only control delay. These thresholds are intended to reflect traveler perception about the effect of time spent waiting in queue on trip quality. The inclusion of loop running time in interchange delay tends to add about 5.0 s/veh to the parclo interchange types. This amount of delay is relatively small and should not appreciably distort the intent of the level-of-service definitions.

Table 4-8. *Highway Capacity Manual* Level-of-Service Criteria. (*12*)

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

FORTHCOMING RESEARCH

The development of a procedure for comparing alternative interchange types and ramp configurations represents a relatively significant advancement in the set of "tools" available to the design engineer. However, additional tools are needed to more fully evaluate alternative interchange design options. These tools are anticipated for development in the forthcoming year of research for this project. They include:

- A procedure for estimating ramp queue length and associated ramp design guidelines.
- A procedure for estimating ramp crash frequency as a function of its design.

At the conclusion of this project, all procedures will be combined into a single, concise document that describes a set of tools for evaluating alternative interchange designs. These procedures will be presented at a level of detail that is consistent with the precision, and amount, of information available at the design concept planning and preliminary design stages.

Finally, the procedures will be used to develop design guidelines for interchanges in non-frontage-road settings. These guidelines will be described in a document titled *Recommended Ramp Design Procedures for Facilities without Frontage Roads*. This document will provide technical guidance for engineers who desire to design safe and efficient interchange ramps on facilities without frontage roads. It will also provide guidelines for: (1) selecting the most appropriate software product for modeling ramp traffic operations, and (2) using this product to evaluate alternative ramp designs.

CHAPTER 5. SUMMARY OF FINDINGS

OVERVIEW

This section summarizes the findings reported in the preceding chapters of this report. The focus is on the key findings that will be used to direct the remaining tasks of the research project. Initially, the findings from a review of ramp design practice are described. Then, the results from the operational analysis of alternative interchange types is presented.

REVIEW OF RAMP DESIGN PRACTICE

Based on a recent change in TxDOT policy, frontage roads are not to be included along controlled-access highways (i.e., freeways) unless a study indicates that the frontage road improves safety, improves operations, lowers overall facility costs, or provides essential access. The implications of this policy change on interchange ramp design and operations are listed in Table 5-1.

Table 5-1. Implications of TxDOT Policy on Ramp Design and Operations.

Change	Possible Implication
Exit-ramp gore area will always precede the interchange.	X-pattern ramp orientation will not be possible.
Ramp configuration is no longer limited to the slip, scissor, or buttonhook shapes.	1. A wider variety of ramp configurations can be considered. 2. May enable designer to use ramp configurations that better accommodate topography.
Frontage road cannot be used for vehicle storage at the exit-ramp terminal.	Exit-ramp design should provide for adequate storage to prevent queue spillback on the freeway.
Ramp terminal at frontage road eliminated.	Area of intense weaving activity on frontage road eliminated (along with associated design considerations).
Frontage road through movement eliminated at ramp terminal.	Easier to design ramp terminal to prevent wrong-way maneuvers (via channelization, leg offset, alignment deflection, etc.).
No longer have to keep ramp meter queues off of frontage road.	Distance between crossroad and end of entrance ramp is reduced because entire distance can be used for storage and acceleration.

As is indicated by the information in Table 5-1, the change in TxDOT policy has several significant implications on interchange ramp design and operations. These implications influence the horizontal geometry of the ramp and its cross section. They also influence the design of the ramp terminal in terms of its cross section, storage length, skew angle, and right-turn channelization.

Given the larger variety of ramp configurations that can be considered as a result of the new policy, guidelines to aid in the evaluation of alternative configurations are needed. They would be used during the design concept planning process to quantitatively compare road-user costs (in terms of operations and safety) associated with alternative configurations and could streamline the consideration of this key factor in the selection process. The guidelines should include a sensitivity to traffic volume, turn movement patterns, capacity, and the presence of frontage roads.

Ramps in non-frontage-road settings are more complicated to design than those in frontage-road settings for several reasons. First, they must be designed to safely and comfortably transition the exiting (and entering) vehicle 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 are present for the entrance ramp with ramp metering or high-occupancy-vehicle (HOV) bypass lanes.

In short, adequate ramp length, appropriate horizontal and vertical geometry, flaring to increase storage area at the ramp terminal, and other details might be used to design operationally superior ramps for non-frontage-road settings. A need exists to develop ramp design guidelines for TxDOT that address the aforementioned concerns and result in safe and efficient interchange ramps.

The review of the *Roadway Design Manual* (*6*) identified areas where additional guidance could be provided to facilitate ramp design for non-frontage-road facilities in Texas. This review focused on Chapter 3 of the *Roadway Design Manual*; however, there were some references to guidance provided in Chapter 2. The findings from this review are summarized in Table 5-2.

OPERATIONAL ANALYSIS OF ALTERNATIVE INTERCHANGE TYPES

The findings from the evaluation of interchange operation indicate that a sound, rational approach to interchange type selection and operational evaluation is feasible using the characteristic relationship between interchange delay and the sum-of-critical-flow-ratios (or volume-to-capacity ratio for unsignalized interchanges). The use of these "characteristic curves" can provide a simple solution to the challenging question of, "Which interchange type or ramp configuration is most efficient?" Previous research projects directed at answering this question have produced guideline statements that can be characterized as vague, subjectively based, or difficult to apply.

Table 5-2. Potential Ramp Design Content for the *Roadway Design Manual*.

Topic Area [1]	Type of NFR Content that Could be Added to the *Roadway Design Manual* [2]
Interchanges	1. Need a technique for comparing the operational performance of alternative interchange types. 2. Need guidance on the safety of alternative ramp configurations.
General Information	1. Need detail figures for one-quadrant, loop, and outer connection ramps.
Horizontal Geometrics	1. Need information about total length of a NFR ramp and its length components (deceleration to controlling curve, deceleration to stop, and storage). 2. Need to verify that the recommended deceleration lengths are adequate. 3. Need more guidance on application of ramp design speed to ramp elements.
Cross Section and Cross Slopes	1. Superelevation rates in Table 3-21 should be updated to be consistent with the current edition of the *Green Book*. 2. Need to provide guidelines regarding conditions amenable to two-lane ramps.
Metered Ramps	1. Need information about total length of a NFR entrance ramp with meter and its length components (decelerate to stop, storage, accelerate to freeway speed). 2. Need guidance on acceptable storage distances for metered entrance ramps.
Frontage Road Turnarounds and Intersection Approaches	1. Need more guidance regarding ramp terminal design speed. 2. Need a comprehensive set of criteria to determine the most appropriate ramp control mode. 3. Need to summarize steps and identify parameters for extending intersection capacity analysis techniques to interchange ramp terminal evaluation. 4. Need guidance for estimating storage length for ramp terminals. 5. Need guidance for selecting the most appropriate right-turn control and corresponding ramp terminal design. 6. Need quantitative guidance on minimum separation distance for access control in NFR settings.

Notes:
1 - Topic areas are based on section headings in Chapter 3 of the *Roadway Design Manual* (*6*, p. 3-93 to 3-114).
2 - NFR: non-frontage-road interchanges.

For signalized interchanges, the sum-of-critical-flow-ratios is a unique parameter that can combine an infinite number of interchange volume level, volume pattern, and geometry combinations into a single value. Furthermore, the analysis presented in Chapter 4 indicates that this parameter has a unique delay relationship based on interchange type and phase sequence. These attributes can be exploited to develop a family of characteristic curves for a range of ramp separation distances that collectively can be used to identify the most efficient interchange alternative. Similar statements can be made about the use of maximum volume-to-capacity ratio for unsignalized interchange evaluations.

The characteristic curves can be used during the design concept planning and preliminary design stages. At the concept planning stage, it should be sufficient to identify and sum the critical movement lane volumes and then divide this total by a representative saturation flow rate to obtain the sum-of-critical-flow-ratios. At the preliminary design stage, the critical movement flow ratios would be computed and summed using movement-specific saturation flow rates. Procedures for applying this analysis technique are described in the Appendix.

When applied to signalized interchanges, the only limitation of the critical movement approach is that it is based on the assumption that cycle time is allocated to the phases in proportion to the critical flow ratio (yielding an equal degree of saturation for all critical movements). However, it is a reasonable assumption when actuated signal control is used, provided that each phase has a reasonably short minimum green (e.g., 15 s or less) and large maximum green setting (e.g., 50 s or more). It is also a reasonable assumption when pretimed control is used and cycle time is explicitly allocated to the phases in proportion to the critical flow ratio.

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APPENDIX

PROCEDURE FOR COMPARING ALTERNATIVE INTERCHANGE TYPES AND RAMP CONFIGURATIONS

INTRODUCTION

This appendix describes a procedure for comparing alternative interchange types and ramp configurations. Specifically, the procedure compares these alternatives on the basis of their impact on traffic operations. The procedure is composed of the models and equations described previously in this report. Separate procedures are provided for each of the signalized interchange types and for each of the 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 for specified traffic conditions. The delay estimate can be useful for comparing alternative interchange types, ramp configurations, lane configurations, or ramp separation distances. The procedure consists of three steps. Inputs to the procedure are the basic movement volumes, the movement saturation flow rates, the number of traffic lanes serving these volumes, and the ramp separation distance. 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. For signalized interchanges, additional information is needed that describes the saturation flow rate for each basic movement s and the number of lanes n allocated to these movements. 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 *HCM* (*12*).

Step 2. Determine the Controlling Volume Ratio

During this step, the movement volume, saturation flow rates, and lane allocations from Step 1 are used with the appropriate equation to compute the controlling "ratio." For signalized interchanges, this ratio is defined to be the sum-of-critical-flow-ratios. For unsignalized interchanges, this ratio is defined to be the maximum volume-to-capacity ratio of the left-turn movements.

Step 3. Determine Interchange Delay

For signalized interchanges, the sum-of-critical-flow-ratios from Step 2 is used with the appropriate characteristic curve to estimate interchange delay. Similarly, for unsignalized interchanges, the maximum volume-to-capacity ratio is used to estimate interchange delay. Table A-1 is then checked to determine the corresponding level of service provided by the interchange.

Table A-1. *Highway Capacity Manual* Level-of-Service Criteria. (*12*)

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

Signalized Interchange Types

This section describes procedures for comparing 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 alternative interchange types consists of three steps. The information needed for each of these steps is identified in this section by interchange type.

Single-Point Urban Interchange

Movement Volumes and Lane Assignments. The basic movements at the SPUI are identified in Figure A-1.

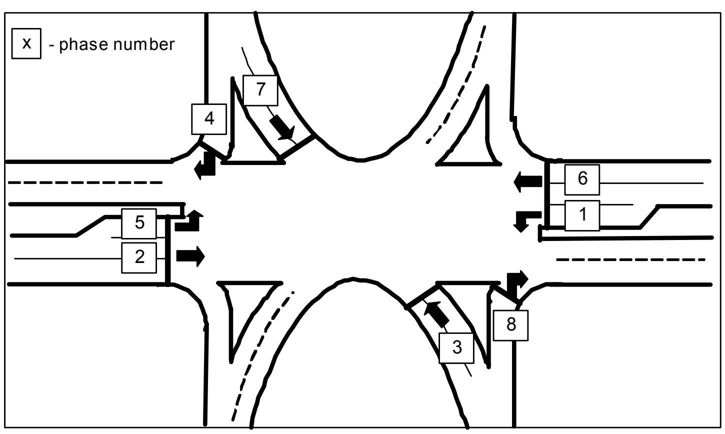


Figure A-1. Basic Movements for the SPUI.

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

$$Y_c = A + B \tag{A-1}$$

with,

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

$$B = Larger\ of\!: \left[\frac{v_3}{s_3 n_3} + \frac{v_4}{s_4 n_4} \ ; \ \frac{v_7}{s_7 n_7} + \frac{v_8}{s_8 n_8} \right] \tag{A-3}$$

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 exit-ramp movements.

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 provided 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 concept planning applications, the sum-of-critical-flow-ratios can be used with Figure A-2 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:

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

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

where:

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

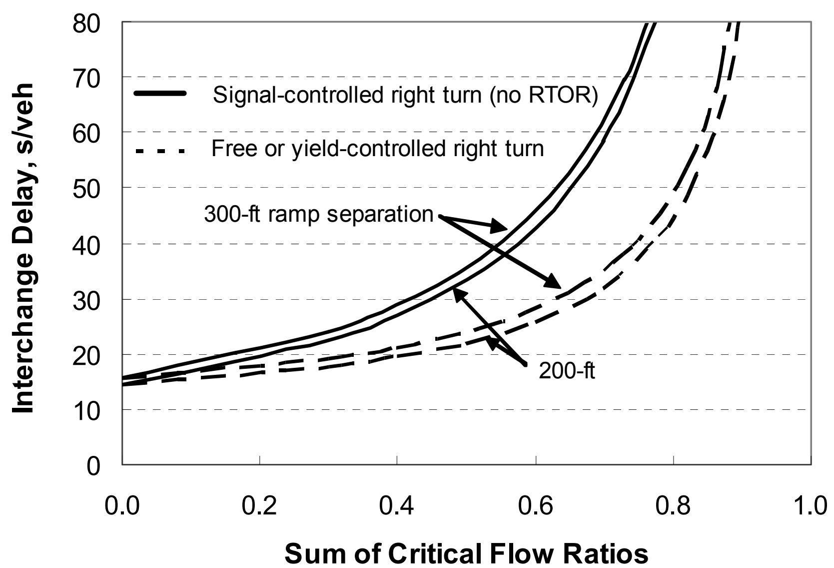


Figure A-2. SPUI Delay Relationship.

Equation A-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 A-4 and A-5. Specifically, this delay is estimated as a weighted average of the delay obtained from each equation where the weight assigned to Equation A-5 is "P_{RTOR}" and that assigned to Equation A-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}.

Movement Volumes and Lane Assignments. The basic movements at the TUDI are identified in Figure A-3.

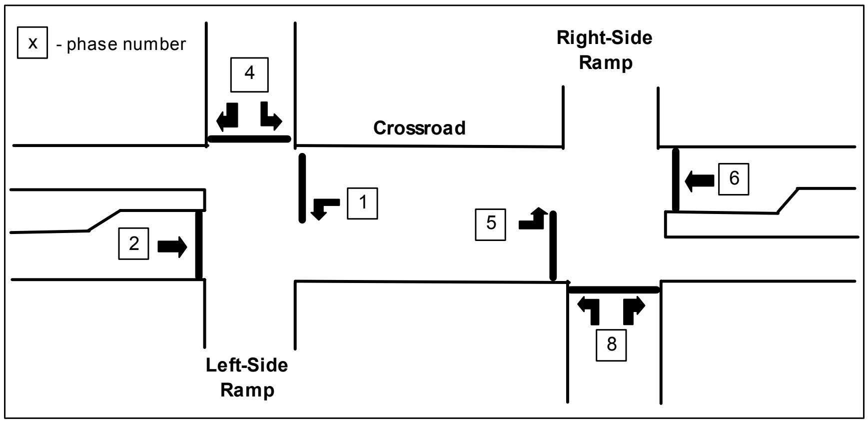


Figure A-3. Movement and Phase Numbering Scheme for the TUDI and Compressed Diamond.

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

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

with,

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

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

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

$$y_7 = \text{Smaller of: } \left[\frac{v_8}{s_8 n_8} \; ; \; y_t \right] \qquad \textbf{(A-10)}$$

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 A-7 and A-8 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 A-3), 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. For preliminary design applications, the following procedure can be used to compute Y_c using Figure A-4 to obtain a more refined estimate of y_t. First, compute the "unadjusted sum-of-critical-flow-ratios" using Equations A-6, A-7, and A-8 with the values of y_3 and y_7 set equal to zero. Then, use this "unadjusted" sum with Figure A-4 to obtain the effective flow ratio y_t. Next, use y_t in Equations A-9 and A-10 to obtain y_3 and y_7, respectively. Finally, use y_3 and y_7 in Equations A-6, A-7, and A-8 to compute Y_c.

Interchange Delay. For concept planning applications, the sum-of-critical-flow-ratios can be used with Figure A-5 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:

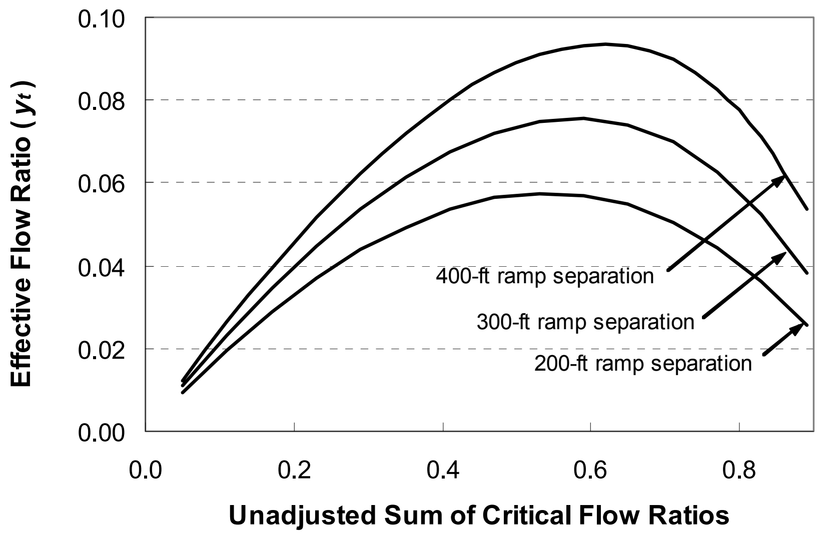


Figure A-4. Effective Flow Ratio.

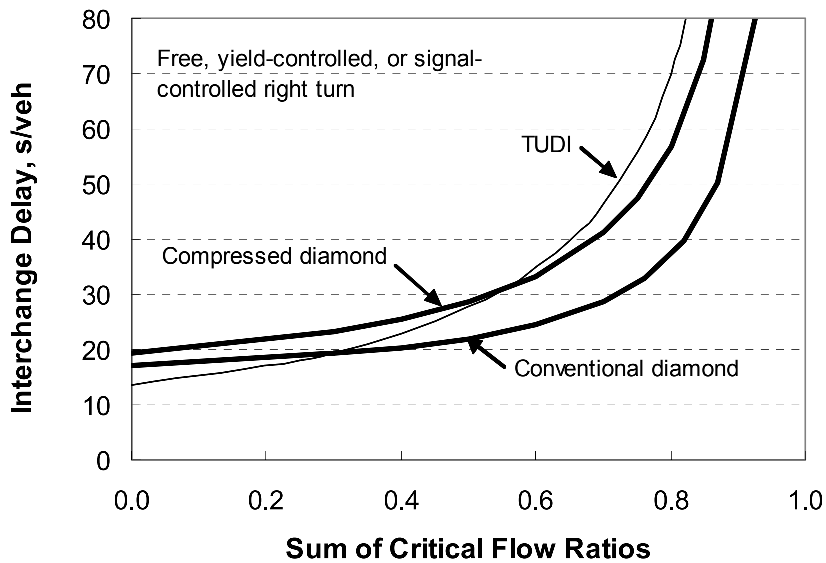


Figure A-5. Signalized Diamond Delay Relationship.

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

$$d_I = 13.4 + 12.8 \frac{Y_c}{1 - Y_c} \quad : \textit{free or yield-controlled right turn} \qquad \textbf{(A-12)}$$

Compressed Diamond Interchange

Movement Volumes and Lane Assignments. The basic movements at the compressed diamond are identified in Figure A-3.

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

$$Y_c = A + B \qquad \textbf{(A-13)}$$

with,

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

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

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

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

where:

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

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

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 A-5 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:

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

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

Conventional Diamond Interchange

Movement Volumes and Lane Assignments. The basic movements at the conventional diamond are identified in Figure A-6.

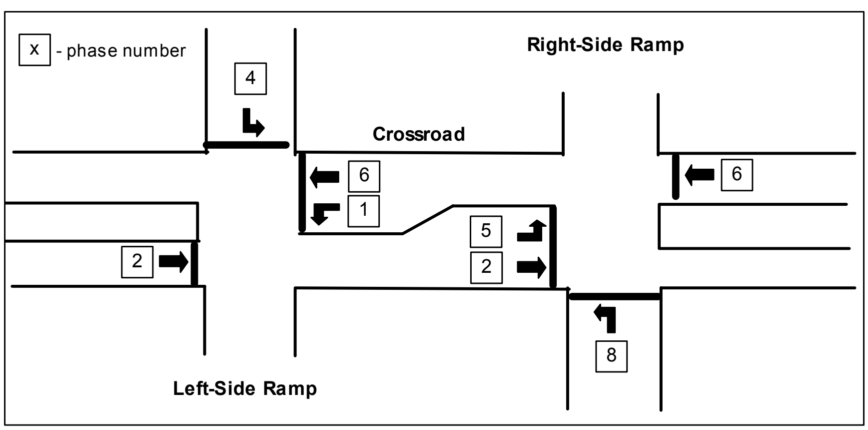


Figure A-6. Movement and Phase Numbering Scheme for the Conventional Diamond.

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:

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

with,

$$Y_c = A + B \tag{A-21}$$

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

$$B = Larger\ of\!: \left[\frac{v_4}{s_4 n_4} \ ; \ \frac{v_8}{s_8 n_8} \right] \tag{A-23}$$

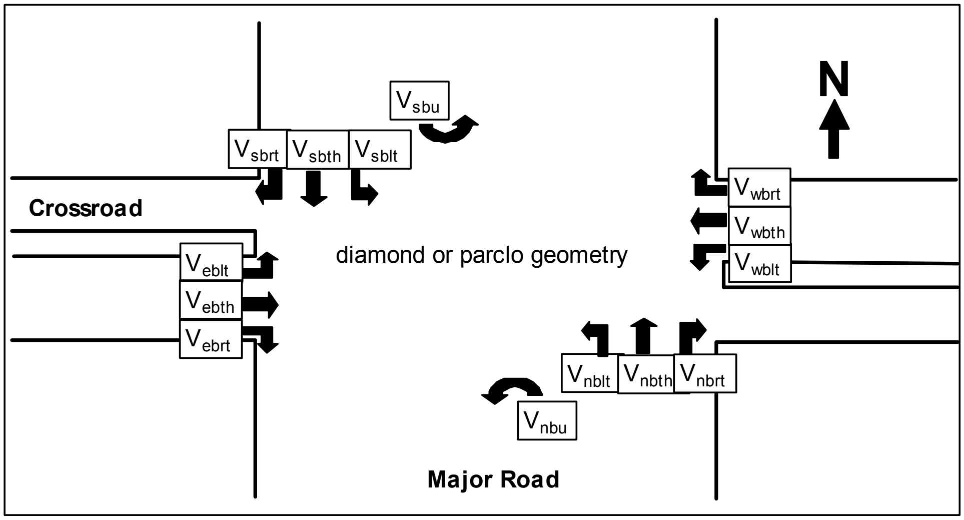
where:
$Y_{c,max}$ = largest sum-of-critical-flow-ratios for the two ramp terminals.

The basic traffic movements associated with volume variables v_i in Equations A-22 and A-23 are identified in Table A-2. 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*. Figure A-7 illustrates the basic movements at an interchange.

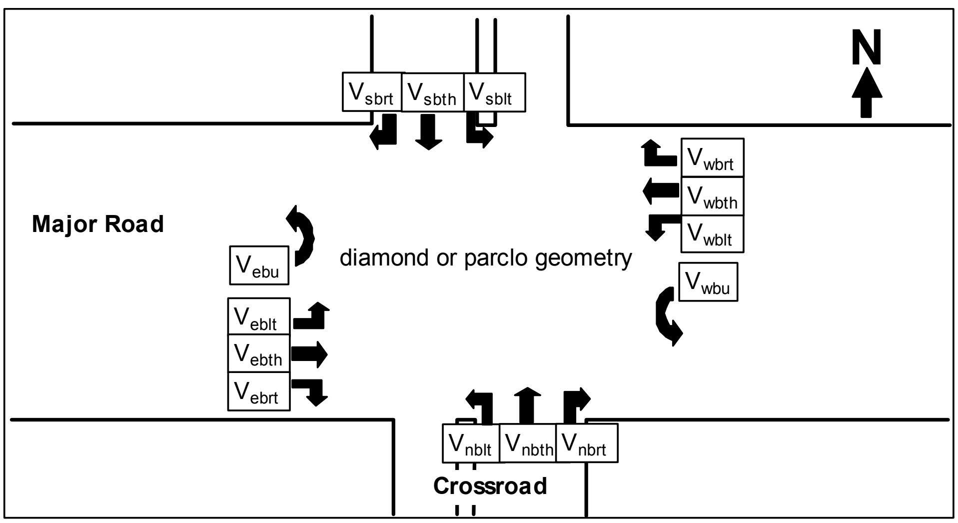
Table A-2. Basic Movement Volumes and Phase Numbers at the Conventional Diamond.

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 A-7, 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.



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



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

Figure A-7. Fourteen Basic Traffic Movements at an Interchange.

Equations A-21, A-22, and A-23 should be applied twice, once for each ramp terminal, to obtain the left-side and right-side sum-of-critical-flow-ratios. These values would then be used in Equation A-20 to obtain the maximum sum-of-critical-flow-ratios for the interchange.

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 A-5 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:

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

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

Parclo A Interchange

Movement Volumes and Lane Assignments. The basic movements at the parclo A are identified in Figure A-8.

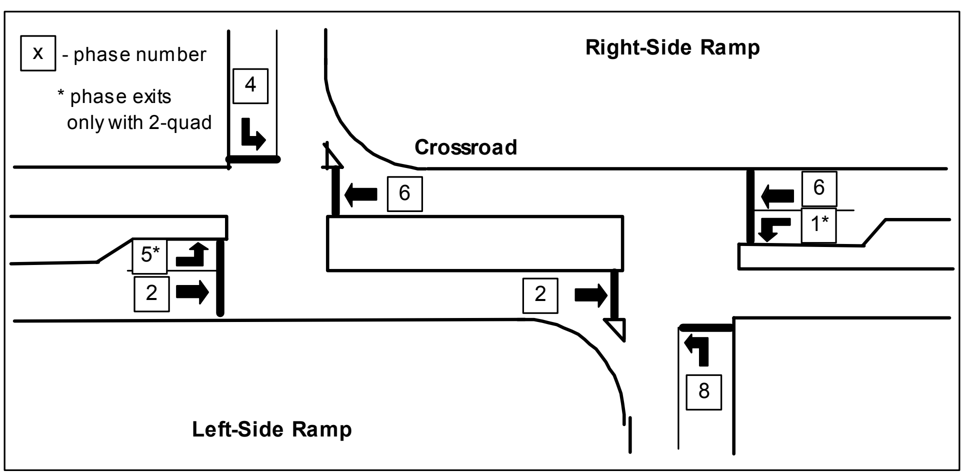


Figure A-8. Movement and Phase Numbering Scheme for the Parclo A and Parclo A (2-Quad).

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 A-22 and A-23 are identified in Table A-3. 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*. Figure A-7 illustrates the basic movements at an interchange.

Table A-3. 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 A-7, 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 A-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:

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

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

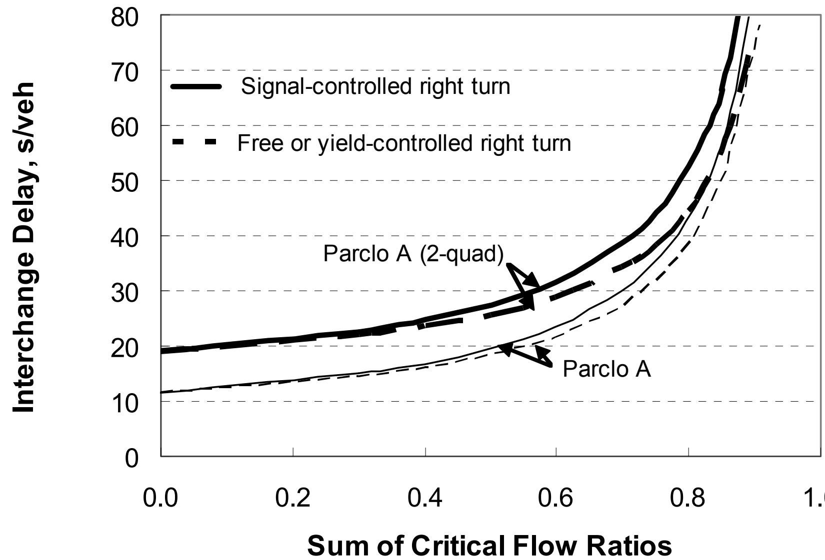


Figure A-9. Signalized Parclo A and Parclo A (2-Quad) Delay Relationship.

Parclo A (2-Quad) Interchange

Movement Volumes and Lane Assignments. The basic movements at the parclo A (2-quad) are identified in Figure A-8.

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 A-22 and A-23 are identified in Table A-3. 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*. Figure A-7 illustrates the basic movements at an interchange.

Interchange Delay. For concept planning applications, the maximum sum-of-critical-flow-ratios can be used with Figure A-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:

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

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

Parclo B Interchange

Movement Volumes and Lane Assignments. The basic movements at the parclo B are identified in Figure A-10.

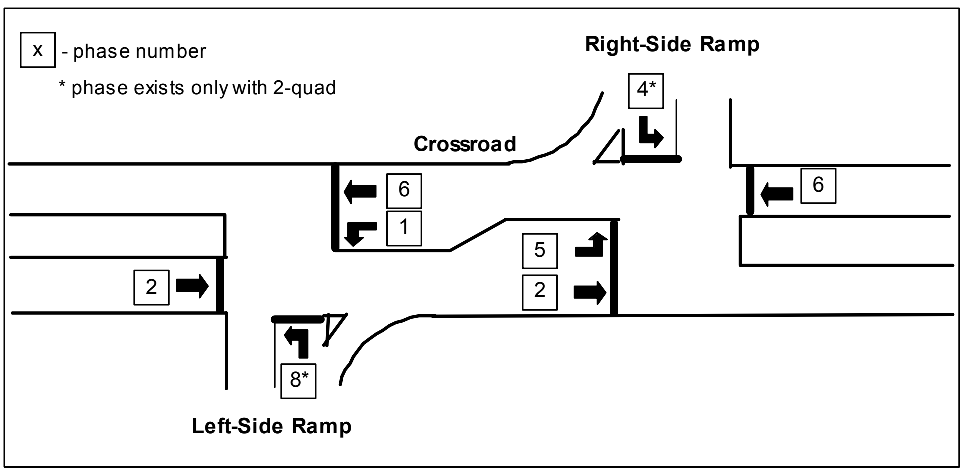


Figure A-10. Movement and Phase Numbering Scheme for the Parclo B and Parclo B (2-Quad).

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 A-22 and A-23 are identified in Table A-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*. Figure A-7 illustrates the basic movements at an interchange.

Table A-4. 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 A-7, 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 A-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:

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

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

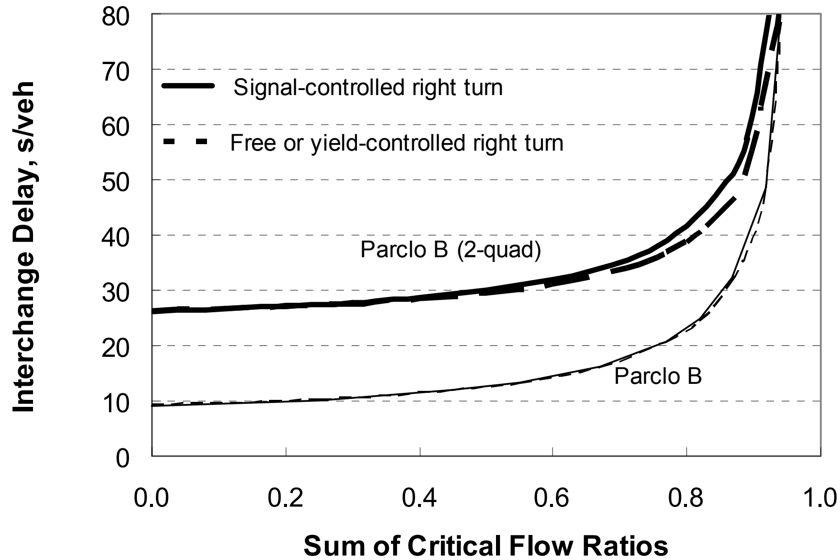


Figure A-11. Signalized Parclo B and Parclo B (2-Quad) Delay Relationship.

Parclo B (2-Quad) Interchange

Movement Volumes and Lane Assignments. The basic movements at the parclo B (2-quad) are identified in Figure A-10.

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 A-22 and A-23 are identified in Table A-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. Figure A-7 illustrates the basic movements at an interchange.

Interchange Delay. For concept planning applications, the maximum sum-of-critical-flow-ratios can be used with Figure A-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:

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

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

Unsignalized Interchange Characteristic Curves

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

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

As noted in the 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 initially because it applies to all of the interchange types listed above. Then, the 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 A-7. This figure should be consulted to identify the 10 basic movements at the non-frontage-road 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:

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

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

with,

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

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

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

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

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 A-5), veh/h;
$v_{r,\ right}$ = subject exit-ramp left-turn volume for right-side ramp terminal (see Table A-5), veh/h;
$v_{c,\ left}$ = subject crossroad left-turn volume for left-side ramp terminal (see Table A-5), veh/h;
$v_{c,\ right}$ = subject crossroad left-turn volume for right-side ramp terminal (see Table A-5), veh/h;
$v_{o,\ r,\ left}$ = volume opposing $v_{r,\ left}$ (see Table A-5), veh/h;
$v_{o,\ r,\ right}$ = volume opposing $v_{r,\ right}$ (see Table A-5), veh/h;
$v_{o,\ c,\ left}$ = volume opposing $v_{c,\ left}$ (see Table A-5), veh/h; and
$v_{o,\ c,\ right}$ = volume opposing $v_{c,\ right}$ (see Table A-5), veh/h.

The volume variables referenced in Equations A-36 through A-39 represent both the left-turn volumes at the interchange and the volumes that oppose, or conflict with, these left-turn movements. Table A-5 identifies the basic movement volumes (as identified in Figure A-7) 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 A-5. 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		Ramp Left-Turn Volumes	
			Subject [4] $v_{c,k}$	Opposing [4] $v_{o,c,k}$	Subject [4] $v_{r,k}$	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 A-7, 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 A-36 through A-39 is computed as a negative value, then the corresponding volume-to-capacity ratio should be set to 0.95. Moreover, if Equations A-38 or A-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 A-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:

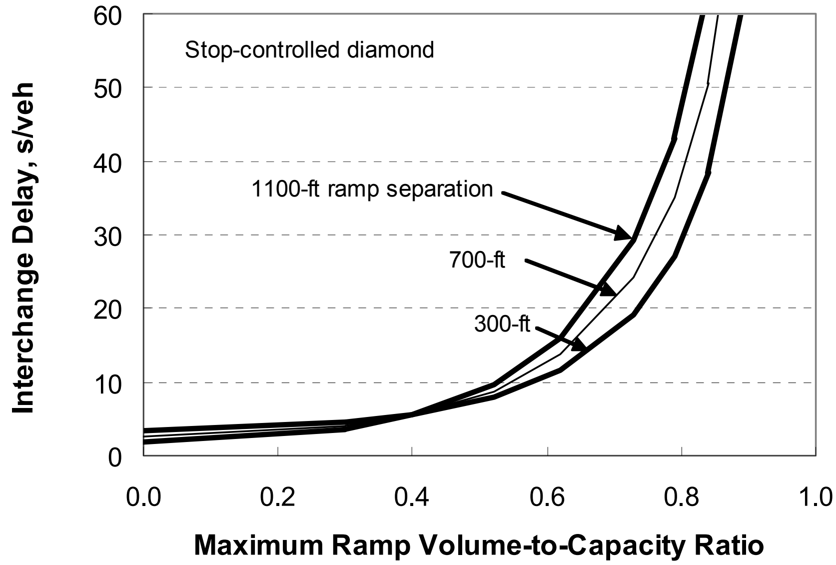
$$d_I = (4.1 - 0.002 D_r) + (5.7 + 0.0074 D_r) \frac{X_{max}^2}{1 - X_{max}} \quad : \textit{stop-controlled right turn} \qquad \textbf{(A-40)}$$

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

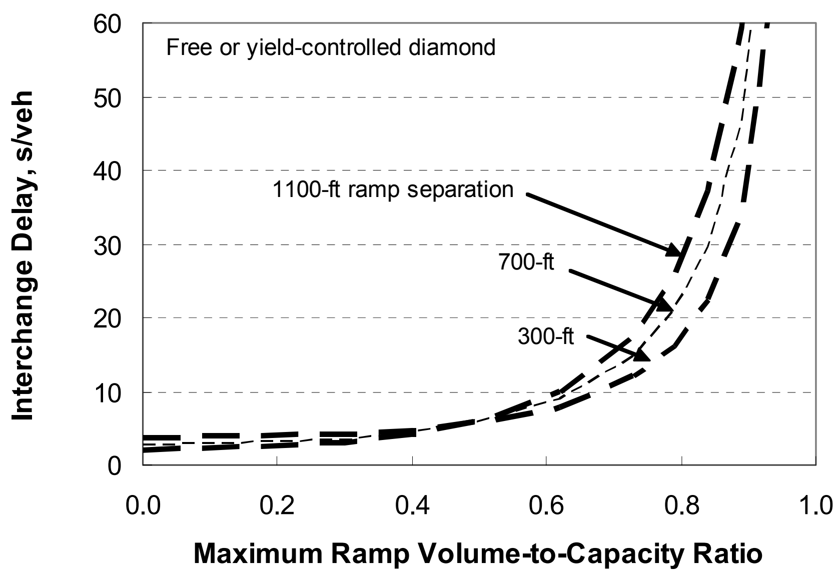
Parclo A Interchange. The maximum ramp volume-to-capacity ratio can be used with Figure A-13 to estimate the associated interchange delay for concept planning applications. This figure is applicable to ramp separation distances in the range of 700 to 1000 ft. Alternatively, for preliminary design analyses,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

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

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



a. Stop-Controlled Diamond.



b. Free or Yield-Controlled Diamond.

Figure A-12. Unsignalized Diamond Delay Relationship.

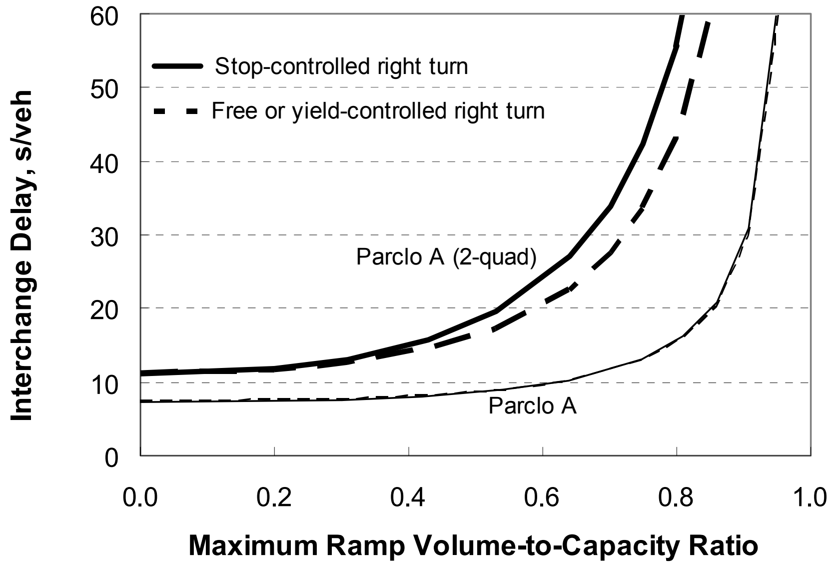


Figure A-13. Unsignalized Parclo A and Parclo A (2-Quad) Delay Relationship.

Parclo A (2-Quad) Interchange. The maximum ramp volume-to-capacity ratio can be used with Figure A-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:

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

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

Parclo B Interchange. The maximum *crossroad left-turn* volume-to-capacity ratio can be used with Figure A-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:

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

$$d_I = 7.1 + 14.7 \frac{X_{max}^2}{1 - X_{max}} \quad : free\ or\ yield\text{-}controlled\ right\ turn \qquad \textbf{(A-47)}$$

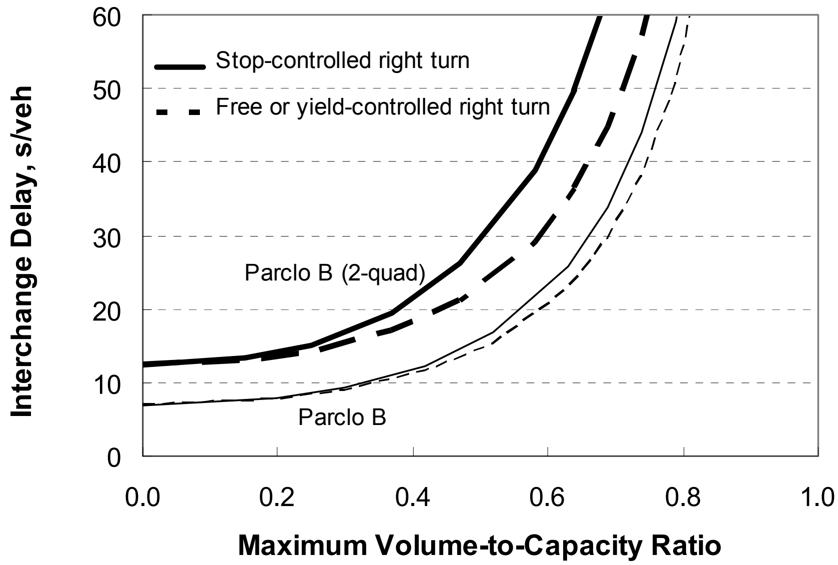


Figure A-14. Unsignalized Parclo B and Parclo B (2-Quad) Delay Relationship.

Parclo B (2-Quad) Interchange. The maximum ramp volume-to-capacity ratio can be used with Figure A-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:

$$d_I = 12.4 + 32.9 \frac{X_{max}^2}{1 - X_{max}} \quad : stop\text{-}controlled\ right\ turn \qquad \textbf{(A-48)}$$

$$d_I = 12.4 + 21.0 \frac{X_{max}^2}{1 - X_{max}} \quad : free\ or\ yield\text{-}controlled\ right\ turn \qquad \textbf{(A-49)}$$