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Distance Requirements for Frontage-Road Ramps to Cross Streets: Urban Freeway Design

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Heavy ramp volumes on urban freeway frontage roads can cause operating problems at adjacent diamond interchanges if the spacing between the cross street and the ramps is not adequate. Spacing requirements from diamond interchanges were studied for both exit ramps and entrance ramps. Exit ramps should provide sufficient storage for the vehicles queued at the cross street as well as an adequate distance for the weaving and braking maneuvers performed by exiting vehicles. Entrance ramps should provide sufficient spacing to prevent queueing through the cross-street intersection. Studies were made on several Texas freeways to provide data to be used to determine spacing requirements. Volume and queue counts were conducted to determine realistic volumes. These studies were used to determine design criteria and to further justify the analytic exit- and entrance-ramp models presented.

The modern urban freeway is conceived and constructed to move large numbers of persons and goods safely and efficiently over considerable distances. The basic design objective is to provide a high level of service in an economical manner. One of the consequences of this design objective is that the spacings of ramps and interchanges are relatively long because this minimizes the effects of weaving on the freeway flow. Apparently, little attention has been given to the resulting negative effects on the connecting ramps and frontage roads. Neither AASHO (1) nor Leisch (2) discuss this question. The traffic engineer currently has a minimum of design criteria or procedures available to use in selecting ramp spacings. Some designers have used rule-of-thumb procedures—e.g., 152 m (500 ft) for exit ramps and 229 m (750 ft) for entrance ramps.

Significant operating problems have been observed on urban freeway ramps and frontage roads near diamond interchanges, especially those where ramp-metering systems are in operation. In most cases, these problems on connecting exit and entrance ramps are directly related to insufficient ramp spacings. These problems are of three different types:

1. Signal queues at interchanges that block merge areas of exit ramps and the frontage road (see Figure 1),
2. Signal queues at interchanges that back into freeway main lanes, and
3. Ramp-metering queues that back into cross-street intersections (see Figure 2).

Freeway exit ramps should be long enough to store enough vehicles to prevent the spillback of the vehicles onto the freeway. The dangerous condition of spillback should not be tolerated as a recurring event and should occur only as a result of unusual circumstances. Freeway entrance ramps should be long enough to minimize queue spillback into the adjacent cross-street intersection because of ramp metering. The installation of ramp metering is a frequently used solution to the problem of congestion on urban freeways, even newly constructed ones.

The objective of this study was the investigation of the locations of entrance and exit ramps with respect

to effects on the operation along the frontage-road approaches to an interchange and the development of criteria for the determination of the appropriate distances between ramps and interchanges. This paper will identify the different components that determine ramp-distance requirements and incorporate them into analytic models for exit and entrance ramps. The paper also provides information on various field studies conducted on freeways throughout Texas. Finally, design criteria and recommendations based on the data evaluated will be presented.

Figure 1. Signal queue at interchange that blocks merge area of exit ramp.

Figure 2. Ramp-metering queue that backs into cross-street intersection.

EXIT-RAMP SPACING

Exit-Ramp Model

The approach used in the determination of the storage length needed to prevent blockage of the ramp merge area considers three storage-length components. The geometric configuration shown in Figure 3 gives the three traffic-operating components used to compute the exit ramp-to-interchange spacing. These three components—weaving, braking, and queueing distances—are discussed below; perhaps it would be best to develop the length from the exit ramp to the signalized intersection of the interchange.

The distance needed to perform the weaving maneuver is the first distance to be provided. The basic weaving model presented in the Highway Capacity Manual (3) is used. Table 1 presents the required weaving distances for three levels of weaving quality of flow based on urban and suburban arterial-operating criteria. Total weaving volumes must be estimated from exit-ramp and frontage-road volumes and their respective turning movements at the interchange. No braking should be required during the weaving movement; the motorist should not be required to perform more than one basic driving task at a time. Therefore, the weaving movement should be completed before the motorist begins to brake to a stop.

The next distance required is the safe stopping distance. This length can be readily found by using Equation 1:

$$SSD = 2.78V \times T + (V^2/253f) \quad (1)$$

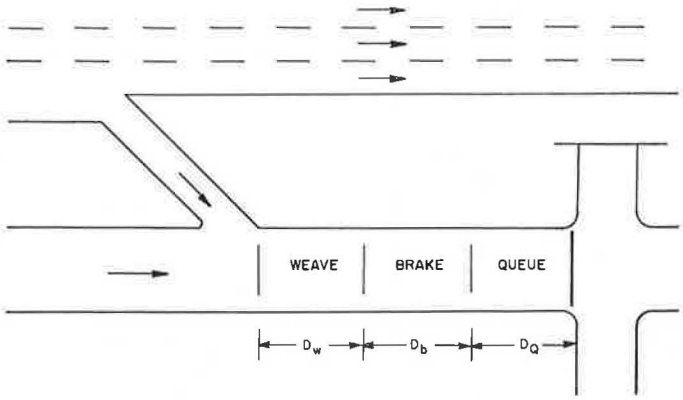
where

SSD = safe stopping distance (m),
V = frontage-road speed (km/h),
T = perception-reaction time (s), and
f = coefficient of friction.

The driver must safely stop before he or she reaches the end of the queue of vehicles stopped at the interchange traffic signal. Solutions to the SSD equation are shown in Figure 4 for perception-reaction times of 1.0 and 2.5 s. Deceleration rates and the resulting coefficients of friction vary with approach speed. Values used in this paper are those given by AASHO (1). A 1.0-s perception-reaction time provides only a minimum-condition reaction time; a 2.5-s time is more desirable.

The queue length at the interchange is the final component in the exit-ramp model. The design queue length can be obtained from Figure 5 (4).

Figure 3. Exit-ramp spacing required.



Exit-Ramp Studies

To develop and test the model, several types of studies were conducted on several different freeway locations. US-75 (North Central Expressway) in Dallas, US-59 (Southwest Freeway) and I-10 (Katy Freeway) in Houston, and some studies in Corpus Christi were chosen. These freeways vary with respect to geometrics and volumes experienced.

Three types of studies on exit ramps—volume counts, queue counts, and spacings between ramp and interchange—were conducted. Exit-ramp volumes were taken on I-10 in Houston and US-75 in Dallas. The cumulative exit-ramp volumes used to classify the volume levels are shown in Figure 6. Eighteen exit ramps are included in the morning and afternoon I-10 studies. Twenty-two exit ramps are included in the US-75 counts. A particularly troublesome high-volume exit ramp in Corpus Christi has a peak-hour volume of 1025 vehicles/h. These volumes provide a base for calculating weaving distances.

Figure 7 presents the measured exit-ramp distances versus the cumulative percentage of ramps for 20 ramps on US-75 in Dallas and 10 ramps on US-59 in Houston.

Table 1. Weaving lengths for variable weaving volumes and design levels.

Total Weaving Volume[a] (equivalent passenger automobiles/h)	Quality of Flow (level of service)		
	3-4 (A-B)[b]	4 (C-D)[b]	5 (E)[b]
100	15	15	15
200	15	15	15
300	15	15	15
400	30	15	15
500	30	15	15
600	30	15	15
700	60	30	15
800	75	30	15
900	90	45	15
1000	110	60	15
1100	120	60	15
1200	140	75	15
1300	150	90	15
1400	170	90	15

[a]Total weaving volume is assumed to be 63 percent of total frontage-road approach volume.
[b]Level of service is based on urban and suburban arterial criteria (3, pp. 173-5) and assumes that number of lanes is adequate for weaving.

Figure 4. Graphical solution of Equation 1 at perception-reaction times of 1.0 and 2.5 s.

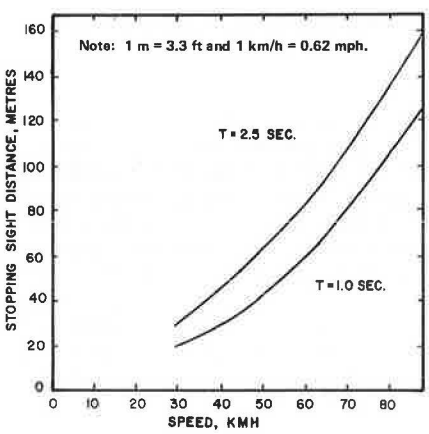


Figure 5. Storage length versus turning volume (modified Poisson distribution).

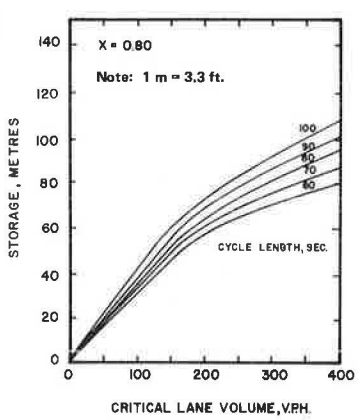


Figure 6. Cumulative percentages of exit-ramp volumes: US-75 in Dallas and I-10 in Houston.

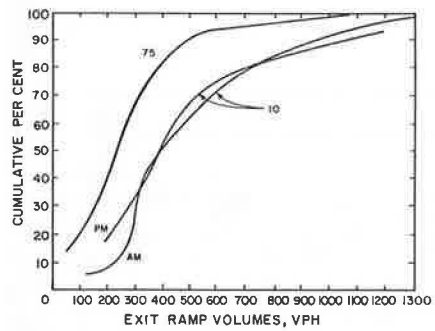


Figure 7. Cumulative percentages of exit-ramp distances: US-75 in Dallas and US-59 in Houston.

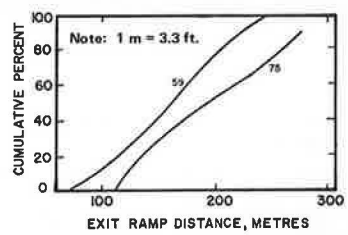


Figure 8. Heavy-volume traffic conditions.

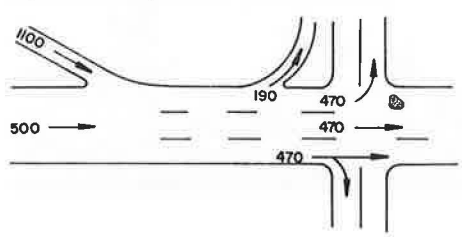


Table 2. Exit-ramp design criteria for three design levels.

Design Criterion	Design Level 1: Desirable	2: Usual Minimum	3: Absolute Minimum
Operating speed, km/h	50-58	45-50	30
Weaving quality	3-4	4	5
Weaving volume, %	63	63	63
Perception-reaction time, s	2.50	1.75	1.00
Stopping distance, m	25.91	16.76	7.01
Cycle length, s	90	80	70
Signal saturation, X	0.80	0.80	0.80
Maximum lane volume[a]	$F \times V_T$	$F \times V_T$	$F \times V_T$

Note: 1 km/h = 0.6 mph and 1 m = 3.28 ft.
[a]$F = 0.4 + 0.6 \exp(-0.13\, V_T C/3600)$ where V_T = total frontage-road approach volume.

The median (50th percentile) distances are about 152.4 m on US-59 and 182.9 m on US-75. Distances were measured from the physical nose to the stop line of the intersection. These studies can be used to compare the results of the model with existing ramp spacings.

Exit-Ramp Design Criteria

The exit-ramp spacing model has been formulated in terms of the three component lengths discussed above—weaving distance, braking distance, and queue length. The distance required for weaving is primarily related to the exit-ramp volumes and the total weaving volume. The exit-ramp-volume data indicate that the 95th percentile exit-ramp volumes of the two study sites shown in Figure 6 are approximately 690 and 1100 vehicles/h. These are termed moderate and high-volume conditions and are the two basic design-volume conditions defined in this paper. The assumed volume distributions for the 1100 vehicle/h exit-ramp flow is shown in Figure 8; frontage road, U-turn, and lane distributions were selected as being representative of the high-volume conditions. Any other exit-ramp volume is assumed to have the same percentage distribution of traffic movements as does the 1100-vehicle volume level; other volumes can be scaled to lower or higher levels than that shown in Figure 8 depending on how they compare to 1100. These volumes can then be used to determine total weaving volumes and resulting required weaving distances.

To determine the trade-off options between freeway level of service and frontage-road operating conditions, exit-ramp design levels of performance were defined as (a) desirable, (b) usual minimum, and (c) absolute minimum (corresponding to qualities of flow of 3-4, 4, and 5 respectively). Although these design levels are not defined specifically in terms of equivalent levels of service, they represent approximately levels of service C, D, and E respectively. Design criteria selected for the model variables are given in Table 2. These variables include quality of weaving, safe approach speed for stopping, perception-reaction time, and signalized-intersection cycle length. The values selected define reasonable and desirable conditions for operations but are certainly not ideal conditions. Design level 3 is an absolute minimum or capacity level and its use is not recommended.

Exit-Ramp Spacings

The exit-ramp spacings calculated by the model for total frontage-road volumes ranging from 200 to 2000 vehicles/h are given in Table 3 for the design levels defined above. Distances are from the exit-ramp centerline point of merge with the frontage road to the stop line at the signalized intersection. This distance may be 30-61 m (100-200 ft) less than the actual distance from the physical nose of the exit ramp to the intersection because of the exit ramp entry angle (4°) and any pedestrian crosswalks at the intersection.

Exit-Ramp Summary

Experience has shown that exit ramps may experience operating blockages at their point of merge with frontage roads because of queue spillback from the adjacent signalized intersection. It is highly probable that many exit ramps have this type of problem, and these ramps should be redesigned to provide greater frontage-road spacing. A desirable level of design should be used where possible. Trade-off analyses should be made between the freeway and frontage-road operations if providing a more desirable exit-ramp spacing would also lower the level of service on the freeway.

Careful consideration should be given before designing the spacing of exit ramps to less than that required for a frontage-road volume level of 600 vehicles/h. Planning data and projected volumes are based on numerous assumptions and estimations of future events and, consequently, exit-ramp volume projections may be in considerable error. Likewise, exit ramps that are expected to feed adjacent major arterials or traffic generators probably should not be designed for a volume level of less than 1600 vehicles/h.

ENTRANCE-RAMP SPACING

Entrance-Ramp Model

Ramp-metering systems are becoming an accepted practice on urban freeways. This has caused concern about the spacing provided between diamond interchanges and the point of merge from the entrance ramp to the freeway main lanes. Queues form at these metered ramps and sometimes back into the cross-street intersections, as shown in Figure 2. The number of vehicles stored behind the ramp signal over a period of time depends on the ramp demand volume and the operating capacity of the ramp-metering signal.

By assuming Poisson arrivals to the ramp and Poisson departures from the ramp-metering signal, Morse (5) has shown that the probability that a ramp of a known queue storage (N) will overflow is given by

$$\text{Probability of overflow} = (\text{volume/capacity})^{N+1} \quad (2)$$

Results of this model are shown in Figure 9 for volume-to-capacity ratios (V:C) of the ramp-metering signal of 0.80, 0.90, and 0.95. The higher the V:C ratio, the longer the ramp storage required for a given probability of queue overflow. For a given ramp demand-volume level, the required ramp storage increases as the freeway level of service decreases.

From a theoretical viewpoint, the queue-length distribution over time and space can be determined if the demand volume and metering capacity of the ramp are known. Few studies have been published relating ramp-metering capacity to freeway-lane-one (outside-lane) volumes. Brewer and others (6), however, have developed a theoretical model of merging-control operations that was later validated in Houston. An approximation of this model is

$$C_r = 1620 - 0.81V_1 \quad (3)$$

where C_r = capacity of metered entrance ramp (vehicles/h) and V_1 = volume of lane one (outside) of freeway (vehicles/h). The normal ramp capacity theoretically cannot be less than the minimum acceptable cycle length if 1 automobile/cycle is metered onto the freeway. Cycle lengths should not be less than 4.0 to 4.5 s (900 to 800 vehicles/h). Ramp volumes greater than 800 vehicles/h usually have high violation rates and multiple vehicle entries during the green.

The volume on the outside lane of the freeway (lane one) varies with the total freeway flow. An estimate of the volume in lane one can be determined from Figure 10. By using level-of-service criteria given in the Texas design manual (7), a range of level-of-service D lane-one volumes and the resulting ramp-metering capacities were developed. From these results, required ramp vehicle storages were calculated at a 5 percent probability of overflow by using Morse's equation; these results are given in Table 4. Level of ser-

Table 3. Distance requirements for separation between exit ramp and cross streets for different design levels.

Total Frontage-Road Volume[a] (vehicles/h)	Approximate Exit-Ramp Volume[b] (vehicles/h)	Design Level		
		1: Desirable	2: Usual Minimum	3: Absolute Minimum
200	140	152	116	79
400	275	170	140	110
600	410	192	152	122
800	550	210	165	131
1000	690	232	180	137
1200	830	265	195	146
1400	960	296	210	152
1600	1100	326	235	162
1800	1240	360	262	168
2000	1380	396	296	177

[a]Exit-ramp volume plus existing frontage-road volume.
[b]Exit-ramp volume assumed to be 69 percent of total volume.

Figure 9. Relationship between volume:capacity ratio of ramp metering and probability of ramp storage overflow.

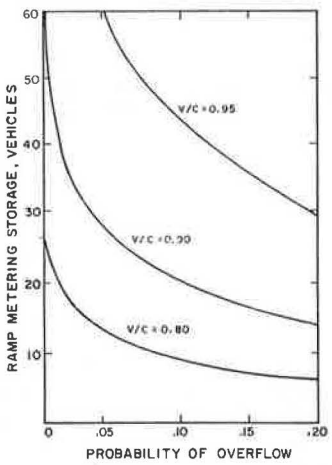
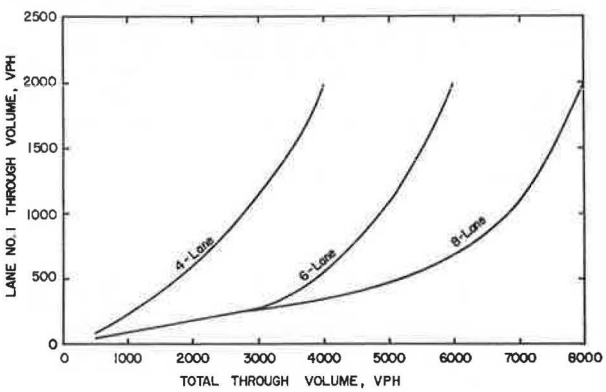


Figure 10. Approximate volume of through traffic in lane no. 1 in vicinity of ramp gores.



vice D was selected as the base because peak-hour metering frequently operates within this high-volume stable-flow region.

Entrance-Ramp Studies

Several studies of geometric and operating characteristics were conducted to establish the model parameters, to confirm the realism of the model results, and to provide field data with which to objectively evaluate existing freeway geometrics.

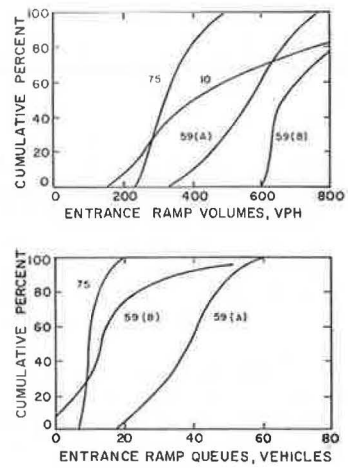
Entrance-ramp volume data were collected at 21

Table 4. Required ramp vehicle storage for given ramp volumes.

Volume (automobiles/h)	Freeway Level of Service		
	Near C	Mid-D	D Near E
Lane no. 1	1000	1200	1400
Ramp metering capacity	810	648	486
Ramp demand			
300	2	3	6
400	4	6	15
500	5	11	-[a]
600	7	38	-[a]
700	11	-[a]	-[a]
800	25	-[a]	-[a]

[a]Large queue: cannot be calculated by theory.

Figure 11. Relationship between entrance-ramp volumes and entrance-ramp queues.



metered entrance ramps on US-75 (North Central Expressway) in Dallas during the peak hour for 2 d during April 1976. A cumulative frequency plot (converted to percent) is given in the top illustration of Figure 11 as curve 75. Most metered ramp volumes ranged from 250 to 400 vehicles/h; the maximum was 510. No connecting roadways from interchanges to the freeway were included in this sample. All of these ramps were on continuous frontage-road sections.

Metered entrance-ramp volume studies were also conducted on US-59 (Southwest Freeway) in Houston during the spring of 1976. Four high-volume ramps—Bellaire, Westpark, Chimney Rock, and Enloe—were observed to study high volume and delay conditions. These ramps are in an area of southwest Houston beyond loop I-610 and near a large shopping center complex, and most do not have frontage roads or other convenient alternative freeway routes. Cumulative percentage plots of ramp-volume data taken during the peak hour are given in the top illustration of Figure 11 as curve 59(A). Most ramp volumes were between 450 and 650 vehicles/h; two-thirds were greater than 500 vehicles/h.

Another set of entrance ramps on US-59, these inside of loop I-610, were also studied. Four ramps near the Summit arena—inbound and outbound Buffalo Speedway and outbound Shepherd and Kirby drives—were counted. These ramps are on a continous frontage-road section of an eight-lane freeway section of US-59. Curve 59(B) in Figure 11 shows that high volumes occur on these ramps; most of the queues observed on these ramps ranged between 10 and 25 vehicles.

A fourth set of ramp-volume data was obtained from a study of I-10 (Katy Freeway) in Houston between the outer loop (I-610) and West Belt Drive. A total of 16 entrance ramps were counted during the peak hour in the inbound or outbound direction. These average

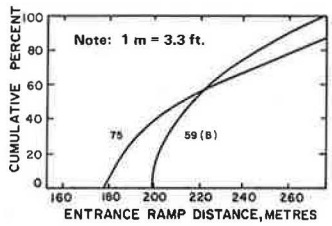


Figure 12. Relationship between recommended design spacings and existing ramp spacings.

volume results are shown in Figure 11 as curve 10. This curve is much flatter and has a wider range of volumes; about 20 percent exceed 800 vehicles/h. These ramps are not metered. The capacity of a metered ramp seldom exceeds 800 vehicles/h. Metered ramp volumes tend to balance the traffic demand over a section of freeway, thereby reducing hot spots or local pockets of high-density flow.

Queue-count studies at metered entrance ramps were also conducted on US-75 (North Central Expressway) in Dallas and US-59 (Southwest Freeway) in Houston. These peak-hour results are shown in the lower illustration of Figure 11. Most queues observed on US-75 in Dallas ranged from 5 to 15 vehicles, although none of the 21 ramps in the continuous frontage-road sections studied operate at what might be considered high volumes. Some of the lower volume ramp-queue counts were not included in this data set; the ramp metering combined with the continuous frontage roads and progressive operations tended to balance out the ramp loadings. Motorists were frequently observed to divert from joining a ramp queue if it exceeded 8 to 10 vehicles. By referring to the table above and Figure 11, one can see that ramp volumes in the 300-400 vehicle/h range would be expected to generate queue lengths in the 5-15 vehicle range during the peak hour (as did the US-75 ramps shown in Figure 11). Ramp volumes in the 400-600 range normally would be expected to generate moderately long queues (6-38 vehicles at midlevel of service D). This rather wide queueing range is illustrated by the higher volume results of curve 75 in Figure 11 and the lower volume queues of curve 59(A). Ramp volumes greater than 500 vehicles/h may generate large ramp queues, as did US-59 outside of I-610, where some queues frequently exceeded 50 vehicles when the freeway was operating near or at level of service E.

On the other hand, ramp volumes in the 600-800 range may generate only moderate queues, as did US-59 inside of US-10, if the freeway is operating at a level of service near C at the metered ramp.

Based on the results of this study, it appears that the number of vehicles in a ramp queue depends primarily on the operating level of service on the freeway, the ramp demand volume, and whether continuous frontage roads (and possibly frontage-road progression) are available.

Entrance-Ramp Design Criteria

It is recommended that the design of entrance ramps in urban areas provide adequate spacing between the diamond interchange and the point of entrance-ramp merge to the freeway such that ramp metering can be installed and operated and not generate queues that overflow into the adjacent interchange.

There are basically only two parts required in determining spacing requirements—the metering section and the queue storage. The first part of the ramp design must provide an adequate distance between the ramp signal and the point of merge to allow an automobile to accelerate to a reasonable merge speed and select a gap if available. Everall (8), indicates that 61-76 m (200-250 ft) is required to provide adequate time to merge. Ramp-metering studies in Dallas and Houston support these guidelines. However, 61 m should be considered the minimum distance to the merge point and 76 m the desirable.

Recent research in Texas has shown that vehicles store at about 7.6-m (25-ft) intervals behind traffic signals (4). Thus, the queue storage needs discussed above must be multiplied by 7.6 m/automobile and this added to the 61-76 m required from the ramp signal to the merge point to determine the required ramp spacings.

Entrance-Ramp Spacings

The recommended entrance-ramp-spacing design requirements (measured from the curb line of the adjacent diamond interchange to the point of merge of the entrance ramp and the freeway) are given below (1 m = 3.28 ft).

Ramp Demand Volume (vehicles/h)	Spacing (m)	
	Desirable	Minimum
<300	229	137
400	305	175
500	381	213
600	457	251
700	533	290
>800	610	328

Desirable and minimum design spacings were selected based on the previous study results and the considered judgment of the researchers.

A comparison can be made between the recommended design spacings given above and the existing entrance ramp spacings determined for two of the freeways described above. These spacings for US-75 in Dallas and US-59 near the Summit arena are shown in Figure 12. In general, the minimum ramp spacings are being provided by the two current freeway designs although some ramps may be deficient. An analysis of the individual ramp volumes and spacings on US-75 showed that 55 percent of the entrance ramps did not meet the desirable spacing criteria. All the US-59 ramps inside I-610 studied fail to meet the desirable spacing criteria because of the high volume levels experienced. In the US-75 data set, there was no correlation between ramp volumes and the ramp-spacing provided. That is, a low-volume ramp was just as likely to have a long spacing as a higher volume ramp.

Entrance-Ramp Summary

The design of entrance ramps in urban areas should consider the possibility that ramp metering will be installed. Adequate ramp spacings are required between the adjacent diamond interchanges and the point of merge of the ramp and the freeway to ensure smooth ramp-metering operations and little queue overflow into the interchange. Minimum and desirable spacing design spacings have been presented that should be considered in future design work. An investigation should be conducted of the current adequacy of all entrance ramps in urban centers in Texas to evaluate the potential need to redesign those ramps that have deficient spacings.

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Abridgment

Stimulus-Response Lane-Changing Model at Freeway Lane Drops

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Lane-changing behavior brought about by a freeway lane drop should differ considerably from the other types of lane-changing behavior considered in past studies because drivers in the dropped lane must merge into the adjacent through lane by or just downstream of the end of the lane-drop taper. Every lane-drop site has warning signs or pavement markings or both to alert drivers of the impending drop.

The stimulus-response lane-changing model was developed as a part of the freeway-lane-drop study (1) to relate driver lane-changing responses to various types of stimuli that forewarn of the lane drop. The model seeks to bridge the gap between macroscopic empirical observations of lane-changing behavior at freeway lane drops and microscopic human-behavior models that attempt to represent individual decision-making processes.

The lane-drop study collected approximately 2.5-3 h of traffic data at 18 mainline lane-drop sites by using a time-lapse photographic technique. Each site was divided into five 122-m (400-ft) subsections for data-reduction purposes; the boundary between the fourth and fifth subsections was always located at the end of the lane-drop taper. Traffic measures were reduced on a subsection or an entire-site basis. Lane-changing data reduced in this manner formed the basis for calibration of the model.

MODEL STRUCTURE

The basic behavioral assumption of the stimulus-response lane-changing model is that each sign or pavement marking or the view of the lane drop itself can be considered as a stimulus that will induce a certain proportion of drivers to change lanes. Thus, the model is most effective in representing the behavior of drivers who rely on warning signs and the visibility of the lane drop in their lane-changing decision-making processes. Drivers, such as commuters, who frequently traverse the lane-drop section probably respond more to their preexisting knowledge. The discussion below applies only to those drivers who are unfamiliar with the lane-drop site.

In the simplest form, suppose that each stimulus causes a certain proportion (p) of drivers to change lanes. Then, the probability (P_i) that a randomly chosen driver entering the lane-drop area within the dropped lane will change into the adjacent through lane in response to the ith stimulus obeys a geometric distribution with parameter p.

However, the geometric distribution is only an approximation, because it requires an infinite number of stimuli to cause all drivers to leave the drop lane. In actuality, only a finite number of stimuli, say n, are present. An improved representation can be obtained by assuming that the beginning of the lane-drop taper constitutes a final (n+ 1st) stimulus that will cause all