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16. Abstract Texas is exploring the use of managed lanes in congested urban corridors. This report discusses the findings from an evaluation of managed lane ramp design issues. Most of the recent literature regarding ramp design has focused on ramp design speed and truck performance. To have an appreciation for current department of transportation practices, a search of each state's design manual was conducted via the Internet. Of the 23 states that had all or part of their design manuals online, 12 had some material available concerning the design of ramps. The potential managed lane system in Texas could contain elements of systems that are currently in use in other communities. As part of this research project, members of the research team visited the New Jersey Turnpike. Simulation was used to obtain an appreciation of the effects on corridor operations when several pairs of ramps are modeled. Speed was the primary measure of effectiveness used to evaluate the effects of different ramp spacings, volume levels, and weaving percentages. The research found that a direct connect ramp between a generator and the managed lane facility should be considered when 400 veh/hr is anticipated to access the managed lanes. If a more conservative approach to preserving freeway performance is desired, then a direct connect ramp should be considered at 275 veh/hr (which reflects the value when the lowest speeds on the simulated corridor for the scenarios examined were at 45 mph or less).					
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MANAGED LANE RAMP AND ROADWAY DESIGN ISSUES

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CHAPTER 1

INTRODUCTION

BACKGROUND AND PROBLEM STATEMENT

The Texas Department of Transportation (TxDOT) is investigating the managed lane concept for major freeways. Managed lanes are being considered in congested urban corridors where expansion possibilities are limited and forecasted conditions point to continuing congestion. The existing experience in both design and operations of managed lanes is limited, especially when considering the effects of varying operational strategies for vehicle type or payment rates. It is widely accepted that the design of a managed lane facility will have an impact on how well it performs. Narrow lanes and tight radius curves would result in speeds that are lower than desired for a limited access facility. The managed lane projects being considered involve retrofits of existing freeway sections with highly fixed right-of-way limits along with other limits such as operational configurations and established eligibility considerations. Therefore, trade-off decisions are frequently required to fit the facility within the limits of the project.

A potential source of information on how best to design and operate a managed lane is available from previous work on high-occupancy vehicle (HOV) lanes. Previous studies have examined criteria for HOVs, and the findings from those studies can be applied to managed lane facilities. A recent report, *Guidance for Planning, Operating, and Designing Managed Lane Facilities in Texas*, provides geometric guidance for the design of managed lane facilities (1). The report highlights the design and operation of managed lanes, and it includes additional information on planning, marketing, implementing, and enforcing managed lane facilities. It illustrates the critical design elements of managed lane facilities including geometric design criteria; the link between operations, design, and enforcement; and ingress/egress treatments.

A Managed Lane Symposium was held during the initial year of TxDOT Project 0-4160 (2). As part of the workshop, three separate groups discussed managed lane issues and determined priorities during interactive sessions. In these sessions, the moderator asked attendees what they thought the most important issues were associated with managed lanes. Participants identified their top five issues. Access design received many comments when the discussion focused on design-related issues.

STUDY OBJECTIVES

Based upon the availability of existing information on design criteria for HOV facilities and concerns expressed by participants of the Managed Lane Symposium, the primary direction set for Task 10 of TxDOT Project 0-4160 was to focus on ramp design issues. Specific objectives for the task included:

- summarizing available literature on ramp design and identifying recent materials available on the design of HOVs,
- identifying current state practices for ramp design,

- visiting a facility with elements relevant to the managed lane concept that has ramp design features of interest,
- conducting computer simulation to explore the effects of ramp spacing on freeway and managed lane speeds and to identify when a direct-connect ramp should be considered, and
- developing a draft chapter on geometric design to be used in the Managed Lane Manual.

ORGANIZATION OF REPORT

This report is organized into the following chapters:

Chapter 1: Introduction presents background information and the objectives of the research effort.

Chapter 2: Literature Review provides information on ramp design available in the literature.

Chapter 3: State Practices for Ramp Design presents the findings from a review of state manuals available on the Web.

Chapter 4: Case Studies discusses characteristics of the New Jersey Turnpike, especially with respect to the ramp design used in the dual-dual section of the Turnpike. This chapter also includes photographs of other facilities.

Chapter 5: Ramp Spacing Simulation presents the methodology and findings from a simulation of a corridor that examines the effects of varying ramp spacing, entering volume, and percent weaving on average and low freeway speed, weaving speed, and managed lane speed.

Chapter 6: Summary and Conclusions summarizes the findings from the research effort.

CHAPTER 2

RAMP DESIGN LITERATURE REVIEW

Researchers performed a comprehensive literature review on ramp design issues. The following summarizes the findings of this literature review in five primary categories: design elements, merging maneuvers, heavy vehicles, crashes, and freeway management.

DESIGN ELEMENTS

Hunter, Machemehl, and Tsyganov (3) observed and evaluated then-current freeway entry ramp design speed criteria in four Texas cities. They determined that observed ramp driver acceleration rates and American Association of State Highway Transportation Officials (AASHTO) values were comparable. Average ramp driver speeds on all observed entry ramps were consistently greater than 50 percent of the freeway design speed, leading them to recommend that the design criterion that allows an entrance ramp design speed of 50 percent of the freeway design speed should be deleted from AASHTO and TxDOT policy. Another important finding was the ability of entry ramp drivers to see traffic in the right lane of the freeway, into which merging is intended. This finding led to a recommendation that the AASHTO acceleration lane length measurement model for taper ramps be modified, such that the acceleration lengths should include only the lane portions from which ramp drivers can clearly view vehicles in the right-hand freeway lane. In other words, the acceleration lane should be considered to begin only when ramp drivers have an unobstructed view of freeway right-lane traffic.

A 1993 article by Keller (4) contains a summary of recent studies and a survey of state highway design agencies in three categories: factors that influence ramp alignment, superelevation, and horizontal alignment. He discusses four factors that directly affect ramp alignment and superelevation design: design consistency and simplicity, the roadway user, design speed, and sight distance. His findings are listed below.

- The motorist must receive simple and consistent feedback regarding the relationship between each element of the ramp geometry; when complex interchange designs are unavoidable, the designs should provide long sight distances, generous radii, and smooth transitions.
- Designing highways for “reasonably prudent” drivers leaves little margin for drivers whose capabilities are different, specifically older drivers and drivers of large trucks. Increased sight distance, simplified interchange layout, and more generous ramp radii will improve operations for these drivers.
- The ramp proper should be viewed as a transition area with a design speed equal to the speed of the higher-speed terminal wherever feasible. The terminals and the ramp proper should be evaluated as a system to ascertain the appropriate speed for design.
- Decision sight distance is desirable over stopping sight distance; however, stopping sight distance plus 25 percent is an acceptable alternative. Sight distances provided at controlled ramp terminals should be determined in the same way as conventional at-grade

intersections with consideration for the additional obstacles that are often present, such as bridge abutments and slopes.

In discussing superelevation, Keller states that AASHTO friction factors are considered inappropriate for large trucks because the limiting factor in truck operation is likely to be the rollover limit rather than skidding. He recommends using **formula (1)**, developed by the University of Michigan Transportation Research Institute, for determining maximum friction factors for large trucks.

$$f_{\max} = \frac{RT - SM}{1.15} - e_{pc} \quad (1)$$

where f_{\max} = maximum friction,

RT = rollover threshold value in terms of g (acceleration of gravity),

SM = safety margin = 0.10 g ,

e_{pc} = superelevation at the curve PC, and

1.15 = steering factor.

Keller also notes several key issues in the design of horizontal alignment on interchange ramps. Some basic alignment principles are as follows:

- The controlled ramp intersection angle should be as close to 90 degrees as possible, and the ramp should not intersect the crossroad and a sharp curve.
- The maximum degree of curvature should be used only in the most irresolvable situations, not as a means of expedient or least costly design.
- Large central (deflection) angles, greater than 45 degrees, are inherent in some ramp configurations. Small central angles should be absorbed by the longest curve practicable.
- Although the desirable curve lengths recommended by AASHTO for small central angles on open roads may not be achievable on ramps, curves must be long enough to provide proper superelevation.
- The gore neutral area can be used to facilitate elevation changes between the ramp and the main road, but it should always slope away from the mainline.
- Consistent alignment should be maintained and is facilitated by design that relates the horizontal elements to one another by design speed.
- Reverse curves should always be separated by tangents of a length adequate to provide proper superelevation runoff.
- The practical size of loop ramp radii should be 100 to 150 ft [30.5 to 45.8 m] for minor movements on highways with design speeds of 50 mph [80.5 km/h] or less, and 150 to 250 ft [45.8 to 76.3 m] for more important movements on highways with greater design speeds.
- Only 32 percent of states surveyed require spiral curves for interchange ramps, but they are particularly appropriate for providing superelevation and matching the driver's natural driving path.
- Lengths of simple curves should be long enough to provide the proper superelevation runoff plus a central portion at full superelevation at least 50 ft [15.3 m] long.

Keller concludes that a successful design balances the desirable features of high design speed, ample sight distance, and level, straight alignments with the constraints of right-of-way, costs, and social and political pressures. These constraints push designers to use minimum design values where above-minimum criteria may be more urgently needed. Keller further states that it is imperative that, prior to final design, the designer analyze the operation of each design element in a three-dimensional (3-D) manner to examine the appropriateness and feasibility of above-minimum values for those design elements.

Harwood and Mason (5) summarized the process for selecting the appropriate design speed for a ramp, based on the 1990 AASHTO guidelines (6). The discussion centered on the need to have a design speed that approaches that of the roadway into which the ramp is entering. The authors urged the selection of a design speed based on an anticipated operating speed, which could mean raising the design speed on a ramp or incorporating speed-control measures to keep motorists from exceeding the lower design speed. A summary of information contained in the then-current *A Policy on Geometric Design of Highways and Streets*, commonly called the *Green Book* (6) and referenced by the authors is listed in Table 2-1.

Table 2-1. Guide Values for Ramp Design Speed as Related to Highway Design Speed (5).

Highway Design Speed, mph [km/h]	30 [48.3]	40 [64.4]	50 [80.5]	60 [96.6]	65 [104.7]	70 [112.7]
Ramp Design Speed, mph [km/h]						
Upper Range	25 [40.3]	35 [56.4]	45 [72.5]	50 [80.5]	55 [88.6]	60 [104.7]
Middle Range	20 [32.2]	30 [48.3]	35 [56.4]	45 [72.5]	45 [72.5]	50 [80.5]
Lower Range	15 [24.2]	20 [32.2]	25 [40.3]	30 [48.3]	30 [48.3]	35 [56.4]
AASHTO guidelines from the 1990 <i>Green Book</i> (6) for selection of design speeds on specific types of ramps include:						
<ul style="list-style-type: none"> • For ramps that serve right-turn movements, upper-range design speeds are often attainable and the lower range is usually practicable. For diamond interchange ramps, a design speed in the middle range is usually practical. • For loop ramps that serve left-turn movements, the upper-range values are not attainable. High-speed loop ramps require large amounts of land and are not practical for most areas. • For semi-direct connection ramps, upper- and middle-range design speeds can generally be used. Design speeds less than 30 mph [48.3 km/h] should never be used. • For direct connection ramps, design speeds in the middle and upper ranges should also be used. Ramp design speeds should generally be 40 mph [64.4 km/h] or more and should never be less than 35 mph [56.4 km/h]. 						

The authors noted that ramps that connect highways with different design speeds should be designed to provide a smooth speed transition between the two highways. In general, the highway with the higher design speed should be the control in selecting the design speed for the ramp in Table 2-1. However, the transition to or from the highway with the lower design speed should also be considered. The controlling feature nearest to the lower-speed highway may be designed at that lower design speed, but the design speed of most of the ramp—and especially the controlling feature closest to the higher-speed highway—should be determined from the higher design speed in Table 2-1. The design speeds shown in Table 2-1 do not apply to ramp

terminals, which should be properly transitioned and provided with speed-change facilities adequate for the roadway speeds involved.

Harwood and Mason (5) concluded that AASHTO policies for selecting design speed for ramps were generally adequate and that most crashes occurred because the motorist was traveling at a speed that exceeded the design speed. The authors offer the following guidelines in selecting the design speed for an off-ramp:

- Consider physical and economic constraints in selecting a tentative design speed for the ramp. It is especially important to avoid the lower range of AASHTO suggested speeds on ramps that will carry substantial truck volumes.
- Identify the most critical curve on the ramp.
- Develop a forecast of operating speeds at the most critical curve on the ramp on the basis of actual speeds on existing ramps with similar design and operating speeds on the mainline and ramps.
- Raise the design speed if the projected ramp operating speed exceeds the design speed.

If the design speed could not be raised to a value higher than the anticipated operating speed, the following speed-control measures were suggested:

- Provide signing with an appropriate advisory speed for the ramp.
- Place the advisory speed signing so that drivers have sufficient length to slow down between the signing and the most critical curve.
- Increase the length of the deceleration lane or realign the ramp to increase the distance from the gore area to the most critical curve.
- Supplement the standard advisory speed signing to make the signing more conspicuous, to increase the distance from the signing to the most critical curve, and to draw the attention of truck drivers to the signing.
- Avoid designs in which the presence of a critical curve on a ramp is not obvious.
- Consider the use of collector-distributor roads in the interchange.

Where ramp designers find it necessary to use a reduced design speed for a ramp, an assessment should be made as to whether drivers are likely to slow down to the selected design speed. If operating speeds higher than the design speed are expected, the design speed should be increased whenever possible.

Sanchez (7) conducted a project to determine if a 3-D model would aid in providing better values for stopping sight distance for interchange connectors. A review of the study models from the driver's perspective revealed that line of sight was obstructed only by the barrier and not the roadway surface. This indicated that when cross slope is also considered, the horizontal alignment becomes the controlling geometric feature for all combinations of minimum horizontal and vertical curvatures.

In order to determine the conditions that produce a line of sight that is obstructed by the roadway surface, the study investigated less-than-minimum vertical curve lengths in which the line of sight intersected the point where the barrier met the roadway surface. [Table 2-2](#) shows the

values for a selection of 3-D vertical curve lengths. The author concluded that significant reductions in vertical curvature were possible if designers were designing a roadway simply on the basis of minimum sight distance. However, he added that more research would need to be performed to understand all of the ramifications of reducing the lengths of crest vertical curves on ramp connectors.

Table 2-2. Values of 3-D Curve Lengths (7).

Algebraic Diff. in Grade, %	Minimum Vertical Curve Length, ft [m]		% Decrease
	2-D	3-D	
14	1112.5 [339.11]	861.0 [262.44]	22.6
12	953.6 [290.66]	739.0 [225.25]	22.5
10	794.6 [242.20]	616.0 [187.76]	22.5
8	635.7 [193.77]	494.0 [150.57]	22.3

Values are for a lateral offset of 8 ft [2.44 m] and a cross slope of 0.06. For larger offsets and greater slopes, the percent decrease is greater.

Leisch (8) presented a practitioner’s checklist of essential criteria during planning and designing a new freeway facility or considering operational and design improvements to an existing facility. He notes that while the operational and design criteria discussed in his paper are present in various chapters of the 1990 *Green Book* (6), his intention is to clarify their application in freeway and interchange planning and design. Table 2-3 summarizes his criteria.

National Cooperative Highway Research Program (NCHRP) Synthesis 35 (9) identified the more successful design and operating practices then in use at freeway exit ramp terminals. The general conclusion was that the design of exit ramps should be related to both the freeway and the crossroad. Grades should be as flat as possible and, where possible, the entire ramp should be visible from the freeway exit. The ramp should have a relatively flat platform at the intersection with the crossroad. Adequate stopping sight distance must be provided throughout the length of the ramp, and enough sight distance is needed at the intersection to allow for safe turns.

Lin, Su, and Huang (10) conducted an exploratory study to begin to establish a set of level of service (LOS) criteria for the systemwide evaluation of freeway design and operation. In this study, they divided freeway travel lanes into three categories: inside lanes at sites away from ramps (Category 1), lanes upstream of off-ramps and shoulder lanes away from ramps (Category 2), and shoulder lanes downstream of on-ramps (Category 3). Because of the interaction with slower vehicles, traffic characteristics vary at the three categories of lanes. Table 2-4 shows some of the traffic flow characteristics for the three categories.

Based on data shown in Table 2-4, the authors chose 43.5 mph [70 km/h] as the boundary between congested and uncongested operations. Using these data and further calculations of flow rate and speed for various conditions, they then classified traffic operations into five descriptive LOSs with corresponding ranges of space-mean speed; the LOSs are shown in Table 2-5. The authors state that for operational analysis, the mean speed for a given combination of traffic and geometric design conditions should be estimated first to determine the LOS.

Table 2-3. Operational Design Criteria for Systems of Interchanges (8).

Concept Category	Criterion	Description/Summary
System Criteria	Basic number of lanes	The constant number of lanes assigned to a route, exclusive of auxiliary lanes.
	Lane balance and auxiliary lanes	At exits, the number of lanes approaching is equal to one lane less than the combined number departing. At entrances, the combined number of lanes after the merge should either be equal to or one lane less than the total number of lanes approaching the merge.
	Route continuity	The provision of a directional path along and throughout the length of a designated route.
Interchange Considerations	Appropriate interchange form	Consideration of the appropriate form may include classification of intersecting facilities, volume and pattern of existing and future traffic, physical constraints and right-of-way considerations, environmental requirements, local access and circulation considerations, construction and maintenance costs, and road-user costs.
	No weaving within interchange	Weaving within an interchange exhibits high accident experience and poor operational characteristics that usually affect not only entering and exiting traffic but mainline flow as well.
Operation Uniformity Criteria	Right exits and entrances	Satisfies drivers' expectancy and keeps slow-moving vehicles from left lanes and avoids weaving across all lanes of the freeway.
	Single exit per interchange in advance of crossroad	Simplifies the driver's task by providing only one decision point on the freeway and giving the driver a view of the exit ramp well in advance.
	Simplified signing	Can exist when exits are in advance of the crossroad and are on the right.
Related or Ancillary Guidelines	Decision sight distance	The distance at which a driver can perceive a decision point along the freeway.
	Freeway and ramp speed relationship	Refers to the distance required for the driver to decelerate the vehicle from the speed of the freeway to the speed of the controlling curve of the ramp.
	Ramp spacing or sequencing requirements	Provided in the <i>Green Book</i> , they are based on design requirements and capacity relationships.

Table 2-4. Traffic Flow Characteristics for Three Lane Categories (10).

	Capacity, passenger car/hour/lane	Space-Mean Speed at Capacity, mph [km/h]
Category 1	2300	43.5-46.6 [70-75]
Category 2	2000-2100	28.0-37.3 [45-60]
Category 3	2000	18.6 [30]

Table 2-5. Description of Level of Service for Freeway Lanes (10).

LOS Description	Space-Mean Speed, mph [km/h]	Comments
High	≥ 55.9 [90]	Free-flow; at or above speed limit
Moderate	49.7-55.9 [80-90]	Nearly free-flow
Marginal	43.5-49.7 [70-80]	Speed is sensitive to density or occupancy
Congestion I	31.1-43.5 [50-70]	Moderate congestion
Congestion II	< 31.1 [50]	Serious congestion

The values shown in Tables 2-4 and 2-5 indicate that the capacity for a Category 2 lane occurs in moderate congestion. The authors concluded that if an off-ramp is operating at capacity, over a period of time this will also affect the Category 1 lanes further upstream. The authors also say that if the speed-flow relationship under prevailing traffic and geometric design conditions can be identified, a planning and design effort may begin with the choice of a desired space-mean speed for use as a design control. The chosen speed should preferably be related to either a moderate or a high LOS. The flow rates that can be sustained at this speed in different lanes and at different sites can then be estimated. Based on these flow rates, the number of lanes needed for various freeway components (such as ramp junctions) to maintain the desired speed can be determined.

MERGING MANEUVERS

Koepke (11) summarized a survey of researchers and state agencies to determine current practices and research efforts on parallel versus taper design for entrance and exit ramps. The survey found that the use of parallel versus taper design was mixed across the country. Out of the 45 responses received, four agencies preferred the use of parallel design, 11 preferred a taper design, and 30 used both designs. Agencies using both designs used a tapered design for exit ramps, but a parallel design for entrance ramps. Most agencies used AASHTO policies as a basis for speed-change lane design and either complied with or exceeded AASHTO recommendations for deceleration lane lengths. AASHTO and all but four states surveyed preferred the taper design for exit ramps. Results for entrance ramps were not as defined as those for exit ramps. Most states surveyed prefer the parallel design for entrance ramps, but not as large a percentage as for tapered exit ramps.

Koepke stated that although many states used both tapered and parallel ramps depending on location and freeway conditions, nearly all states used deceleration lane lengths that equal or exceed AASHTO recommendations. The greatest design difference was not in the type of ramp, but in the length of the acceleration lane, which in some cases was less than AASHTO guidelines. Most researchers surveyed by Koepke indicated that there were very few operational problems with deceleration lanes, but the problem areas were in the gore of exit ramps and in gap acceptance on entrance ramps. Surveyed states attributed both conditions to the assumption that drivers simply do not fully understand the use of speed-change lanes on freeways.

Michaels and Fazio (12) developed a model based on an angular velocity criterion for a driver's decision to merge. Results from the model indicated that the length of the acceleration lane decreased with increasing ramp speed and with freeway volume. The required length was found to be independent of freeway volume over a range of 1200-2000 passenger car/hour/lane (pcphpl) and ramp design speeds from 30 to 45 mph [48.3 to 72.5 km/h]. At a freeway volume of 1200 pcphpl, a 50 percent increase in ramp design speed led only to a 20 percent reduction in required length. For design purposes, if a designer used the length defined by a volume of 1200 pcphpl, the length needed for most currently used ramp design speeds was between 650 and 800 ft [198.3 and 244 m]. Crash rates on acceleration lanes tended to reach a minimum at a length of 700 ft [213.5 m]. The nominal length to ensure 85 percent or more merge opportunities for ramp drivers was no more than 800 ft [244 m]. The authors also drew the conclusion from

the model that a tangent ramp connector with a small angle of convergence leads to a more effective merge process. This is because the ramp becomes a part of the acceleration lane on which drivers may accelerate prior to their search for a gap.

HEAVY VEHICLES

Bola (*13*) wrote a paper that summarized a project that evaluated a Truck-Activated Rollover Warning System (TARWS). This system was designed to warn drivers when the truck, based on ramp geometry and speed, was in danger of rollover if they did not reduce speed. Results of the study indicated that there were no crashes on the ramp during the two-year study period, compared to six in the 6.3 previous years. An additional factor was that the average truck average daily traffic (ADT) increased from 7759 to 9154 from the before period to the after period. The authors performed a benefit-cost analysis that estimated the system would pay for itself if it prevented one fatal or four injury accidents over its 10-year projected life.

Harwood, Glauz, and Elefteriadou (*14*) examined the limitations imposed by existing roadway geometrics on the ability of the roadway system to accommodate potential larger and heavier trucks at some future time. Horizontal curves on freeway on- and off-ramps were identified as being in need of improvement in any scenario in which the swept-path width for the truck in question, while traveling at near zero speed, exceeds 15 ft [4.6 m]. This is the typical minimum width required by AASHTO geometric design policy for one-lane ramps that have no provision for passing a stalled vehicle. In addition, AASHTO geometric design policy requires that ramps on sharper curves typical of lower roadway design speeds be widened to accommodate truck offtracking. For example, on a ramp curve with a radius of 273 ft [83 m], which is the minimum radius permitted by AASHTO policy for a horizontal curve with a 30-mph [48 km/h] design speed, a lane width of 16 ft [4.9 m] is recommended.

Tom and Fong (*15*) wrote a report summarizing the activities of an evaluation of truck merging operations at a selection of four California freeway on-ramp locations with significant truck volumes. The 50th percentile merge location for truck combinations was approximately 1072 ft [327 m], compared to 498 ft [151.9 m] for cars, 489 ft [149.1m] for recreational vehicles, and 565 ft [172.3 m] for all vehicles. Researchers found that as more length was provided to accelerate and merge, more length was used by drivers. At each site, more than half of the merging maneuvers took place in the latter half of the length provided. In addition, the longer the acceleration lane, the higher the speeds of the merging vehicles, approaching the speeds of through vehicles on the freeway. The fatal and injury crash rates for the rural sites were higher than the expected rates, but only a site with no auxiliary lane had a statistically significant increase.

Knoblauch and Nitzburg (*16*) conducted a study of ramp signing to reduce rollover of high-speed and/or top-heavy loaded trucks. The research addressed methods for treating interchange ramps that are prone to cause high center of gravity vehicles to lose control and overturn. In a survey of professional truck drivers, the elements determined to be the most effective were a rear silhouette of a tipping truck, a diagrammatic arrow, and an advisory speed value. Researchers conducted a field test at three ramps fitted with experimental devices. Results indicated that there was a temporary reduction in speeds for trucks on the ramps with experimental signs, but speeds soon

returned to levels comparable to those before improvements. There were other decreases in approach speeds and ramp speeds, particularly among the 90th and 95th percentiles; however, reductions were not statistically significant. Researchers concluded that truckers had a relatively high level of understanding regarding the rollover problem and the meaning of the truck tipping sign. However, the field test results showed no operational effect (e.g., speed changes) to support this conclusion.

A research study by Perera, Ross, and Humes (17) focused on the task of determining the critical speed of a ramp for heavy vehicles, then translating that into a safe operating speed. Through the use of computer simulation models and input of geometric design elements from an example ramp, researchers determined critical speed and safe operating speed for a baseline vehicle and a vehicle with a high center of gravity. The critical speed was the lowest speed at which the wheels of the subject vehicle would either begin to lift off of the ramp surface (rollover) or run off the ramp (offtracking). The safe operating speed was determined by dividing the critical speed by a factor of safety, assumed to be 2.0 for this test. For both vehicles (baseline and high center of gravity), the safe operating speed was determined to be 13 to 18 mph [20.9 to 30.4 km/h] lower than the corresponding AASHTO design speed, based on the equations developed through the model. The authors also noted that the simulation was performed for dry road conditions; wet conditions would have an added element to consider. The authors stressed the importance of considering large trucks in ramp design and in determining safe operating speeds.

Glines (18) wrote an article to summarize a research project to determine the impact of specific geometric features on truck operations and safety at expressway interchanges. While the direct cause of most rollovers was attributed to speeding, the study found numerous instances that ramp design contributed to the excessive speed. It also found that professional truck drivers are under the impression that the rear axle of their trailer tracks inward while traveling around the radius of a ramp when, in fact, it tracks outward. As a result, many rollovers occurred, especially when a curb was built into the ramp. The study said drivers forced to slow down suddenly because of short-radius curves were often involved in jackknife crashes due to over-braking and rollovers due to excessive speed. It also found outer curbs a “special hazard” for tractor-trailers moving through high-speed curves. The article summarized several other conclusions; the ones related to design practices are listed below:

- Jackknife accidents are found ahead of curves that appear to pose a threat of rollover to vehicles traveling near or above the advisory speed. Truck drivers apply excessive braking in an attempt to reduce speed before entering the curve, suffering wheel lock-up conditions causing a jackknife before the curve is reached.
- AASHTO’s policy for geometric curve design provides for virtually no margin of safety against rollover for certain trucks.
- Deceleration lanes that realistically reflect the braking constraints of trucks should be 30-50 percent longer than AASHTO guidelines suggest.
- The mismatch between the provided lengths of acceleration lanes and the acceleration length demands of loaded trucks may be prompting truck drivers to speed in the later portions of many interchange ramps to mitigate the inevitable conflicts associated with merging.

- AASHTO's policy of accepting ramp downgrades as high as 8 percent may be ill-advised at sites on which a relatively sharp curve remains to be negotiated toward the bottom of the grade.
- Curve warning signs were observed to be improperly selected or placed an insufficient distance ahead of the curve.
- State transportation departments should review ramps that have a history of accidents involving heavy-duty trucks. The use of improved warning and advisory speed signs, or removing curbs, may offer effective short-term countermeasures.
- Assurance of adequate pavement friction levels for safe operation of trucks calls for new research in truck tire traction.

Ervin (19) summarized a research study to establish how particular expressway ramps cause drivers of tractor-semitrailers to lose control of their vehicles. The most basic conclusion of the study was that highway design in the United States does not sufficiently account for the special maneuvering limitations of heavy trucks. The most fundamental of those are low resistance to rollover and poor braking capability. Additionally, peculiar response characteristics pose problems when curbs are installed along the outside of curved lanes and when the surface texture on wet pavement is rather smooth. An aggravating condition is ramp geometry that encourages truck drivers to accelerate while still on the ramp to lessen the conflict of merging with faster-moving traffic.

Ervin's study (19) recommended that all AASHTO policies relating to the geometric design of highway ramps and other curved roadways be examined from the viewpoint of the maneuvering requirements of heavy trucks. State highway agencies were encouraged to survey interchange ramps within their jurisdiction in light of the study's findings, especially where the site had experienced frequent loss-of-control truck accidents. The author recommended that advisory speed signs be reviewed to assure that all ramps with virtually no margin of safety for heavily loaded tractor-semitrailers have adequate warning signs. Also recommended was that control limits of trucks be improved, so that the vehicles become more compatible with the existing road system. The study report also suggested that truck drivers be better informed about the hazards associated with expressway ramps.

Ervin, MacAdam, and Barnes (20) described a research study in which accidents by semitrailer trucks on expressway ramps were found to depend largely on the interaction between highway geometrics and vehicle dynamic behavior. It was inferred from accident reports that a substantial number of truck drivers tended to take ramps too fast, perhaps because of the desire to keep up speed in anticipation of merging or simply because of a lack of appreciation for the small tolerance that some ramp designs afford for trucks exceeding the advisory speeds. When considering the margins of safety that existing truck ramps provide for truck operations, the authors concluded that the considerations that underlie AASHTO ramp design recommendations make little or no allowance for the special requirements of trucks. Through the use of a simulation model, five different cases illustrated the more significant aspects of ramp design. Table 2-6 describes these cases.

Ramps that were designed according to AASHTO recommendations and guidelines in effect at the time were used as examples. Those guidelines, however, were not sufficient to address the

truck-related issues presented in [Table 2-6](#). The authors conceded that implementation of more truck-friendly design elements would involve major reconstruction of ramps and high associated costs; however, their conclusion was that it would be rational that highways be designed so that truckers obeying the posted speeds can be assured of nominally safe travel. The authors also recommended that highway agencies examine ramp sites with high rates of truck accidents to determine if any of the study cases applied to them and could be remedied using the discussion provided here.

Table 2-6. Study Cases for Design Element Conflicts with Heavy Trucks (20).

Study Case	Description/Summary
Side friction factor is excessive given the roll stability limits of many trucks.	Truck characteristics, such as heavy loading and higher center of gravity, yield different thresholds of side friction factor and superelevation rate than passenger vehicles. This leads to a higher propensity for rollover crashes.
Truckers assume that the ramp advisory speed does not apply to all curves on the ramp.	Many ramps have multiple curved segments. It appears that truck drivers sometimes assume that they have passed the curve or curves that warranted the advisory speed, so they begin to speed up in preparation for merging, only to find that the remaining curve also warrants the low advisory speed. This increases the likelihood of rollover or jackknife crashes.
Deceleration lane lengths are deficient for trucks, resulting in excessive speeds at the entrance of sharply curved ramps.	Many ramps incorporate a sharp curve right at the end of the deceleration lane such that the low value of advisory ramp speed must be achieved very quickly after departure from the main through lanes. This challenges the braking capabilities of the truck.
Lightly loaded truck tires are sensitive to pavement texture in avoiding hydroplaning on high-speed ramps.	Light tire loads associated with empty truck combinations have a footprint of tire-roadway contact that is unusually incapable of expelling water. Accordingly, very lightly loaded truck tires are vulnerable to a pronounced traction deficiency on ramps covered with water. The friction factors used in the ramp design are too high for these conditions.
Curbs placed on the outer side of curved ramps pose a peculiar obstacle that may trip and overturn articulated truck combinations.	Low-speed offtracking is common for trucks making low-speed, low-radius turns. However, trailers in articulated truck combinations could actually have a path that swings outward of the tractor if the lateral acceleration levels are sufficiently high. A truck that turns in this manner could have its rearmost axles strike a curb that is situated along the outer side of the curve.

CRASHES

As an alternative to using accident history data to determine the focus of safety improvement programs, Isaacs, Saito, and McKnight (21) developed a method based instead on expert opinion, similar to the Road Safety Audit method. The results of a survey of experts were synthesized into numerical ratings that describe the relative level of hazard of specific characteristics of highway ramps with regard to truck rollovers. Based on survey responses, authors developed relative hazard ratings for all of the ramp and deceleration lane characteristics rated by the experts. The ratings are relative to each other, as a synthesis of expert opinion on the relative likelihood that a specific ramp characteristic will contribute to a truck rollover. [Table 2-7](#) lists the hazard ratings. The hazard ratings are used to produce a notice rating, which provides a relative level of hazard for an entire ramp.

The authors then developed a six-step process to apply the hazard ratings to existing ramps; the steps are listed as follows:

1. Inventory of Characteristics of Ramps in Jurisdiction
2. Classification of Characteristics and Assignment of Hazard Ratings
3. Assignment of Hazard Ratings for each Characteristic of the Ramp (radius, deceleration lane length, etc.)
4. Calculation of Notice Rating (sum of hazard ratings for all characteristics)
5. Listing of Corrective Measures
6. Application of Cost-Effectiveness Procedures

The authors applied this process to four ramps in a jurisdiction that is evaluating safety improvements. The result was a table containing a priority listing of projects for the four ramps, along with their relative cost-effectiveness.

Table 2-7. Relative Hazard Ratings for Ramp and Deceleration Lane Characteristics (21).

Characteristic	Relative Hazard Rating				
	100%	80%	60%	40%	20%
Radius Adequacy, mph [km/h]					
$V \leq 20$ [32.2]	6	199	290	453	609
$20 < V \leq 40$ [32.2 < $V \leq 64.4$]	8	165	263	448	610
$V \geq 40$ [64.4]	2	182	356	514	620
Deceleration Lane Length Adequacy					
$V \leq 40$ [64.4]	0	5	11	15	31
$40 < V \leq 60$ [64.4 < $V \leq 96.6$]	0	5	17	23	31
$V \geq 60$ [96.6]	0	8	20	28	32
Characteristic	Rating	Characteristic	Rating	Characteristic	Rating
Deceleration Lane Downgrade		Lane Width		Deceleration Lane Pavement	
0%	0	≥ 13 ft [4 m]	18	Dry	0
1-2%	7	12 ft [3.7 m]	109	Wet	18
3-4%	17	11 ft [3.4 m]	214	Snow	23
5-6%	31	10 ft [3.1 m]	327	Ice	32
Type of Transition Curve		9 ft [2.7 m]	431	Type of Compound Curve	
Spiral	17	≤ 8 ft [2.4 m]	435	Sharp \rightarrow flat	236
Compound	173	Cross-Slope Difference		Flat \rightarrow sharp	261
on tangent and curve	148	6%	116	Sharp \rightarrow flat \rightarrow sharp	403
		8%	218		
All on tangent	184	10%	293	Flat \rightarrow sharp \rightarrow flat	322
Ramp Downgrade		12%	396		
0%	22				
1-2%	95	Presence of outside curb	496	Drop > 4 in [10.2 cm] at pavement	398
3-4%	217				
5-6%	363				
> 6%	495				

Note: If ramp is part of interchange, use factor of 1.4.

Twomey, Heckman, Hayward, and Zuk (22) conducted a study to summarize previous safety research that associated crashes and safety with interchange features. In general, the research

indicated that interchange ramps should be designed with flat horizontal curves and designers should avoid the maximum degree of curvature. Sharp curves at the ends of ramps and sudden changes from straight alignment to sharp curves were also to be avoided. The design of cloverleaf ramps, scissor ramps, and left-side ramps should be avoided where possible. The authors encouraged collector-distributor roads in high-volume interchange designs and especially designs in which loop and cloverleaf ramps were used. The relative safety of entrance and exit terminals was enhanced by designs that provided at least 800 ft [244 m] of length on acceleration or auxiliary lanes.

FREEWAY MANAGEMENT

Butorac and Dudek (23) determined the geometric and operational differences of multiple-lane ramp meters compared to traditional single-lane ramp meter systems and identified the geometric and operational issues associated with this freeway management technique. Using information gathered from a variety of states and agencies, they developed a proposed set of application guidelines for ramp meters:

- Multiple-lane ramp meters provide the ability to meter a wide range of on-ramp demand volumes, ranging from approximately 200 to 1800 vehicles per hour, compared with the traditional single-lane configuration, which can only effectively accommodate up to approximately 900 vehicles per hour.
- The additional queue storage (reservoir) area provided under the multiple-lane design strategy nearly doubles that available under the single-lane design strategy and provides the ability to handle increased on-ramp demand volumes and platoon arrivals from upstream traffic signals more effectively.
- Multiple-lane ramp meter configurations allow agencies to provide preferential lanes to carpools, vanpools, and transit vehicles.
- Multiple-lane ramps tend to provide a better self-enforcement environment when compared to single-lane configurations because of the presence of motorists next to one another.
- Metered on-ramps should maintain sufficient acceleration and merge distances between the ramp meter stop bar and the end of the on-ramp on auxiliary lane, based on the freeway operating speeds.
- Proposed ramp metering locations should provide an adequate queue storage reservoir for existing and future on-ramp conditions (i.e., demand volumes and arrival patterns). Ramps that do not maintain adequate queue storage reservoirs should maintain excessive queue management traffic control capabilities.
- Single-lane ramp meters should not be used in locations where the demand volumes are expected to exceed approximately 900 vehicles per hour even after traffic is redistributed between on-ramps within a specific section of freeway that will be metered. In addition, single-lane ramp meters should not be used on locations that maintain on-ramp demand volumes in excess of 750 vehicles per hour and highly variable arrival patterns.
- Multiple-lane ramp meters should provide the proper pavement widths and lateral clearances to accommodate two lanes of traffic within the queue storage reservoir. In addition, preferential lanes should provide proper access to avoid queue spillbacks (i.e., mixed-traffic queues should not block access to the preferential lane).

- Grades in the vicinity of the ramp meter stop bar should be minimized to avoid vehicles losing traction within both the queue storage reservoir and acceleration areas of the on-ramp in the event of inclement weather. This geometric design detail should be addressed through the placement of the ramp meter signals, since the re-grading of freeway on-ramps is typically not geometrically or economically feasible.
- To maintain effective freeway management during poor weather conditions, metered on-ramps should be plowed and sanded and receive a high priority in snow removal scheduling.
- On-ramps should be monitored during potential icy/snowy conditions by closed-circuit television, pavement sensors, or other methods to determine when ramp metering may need to be discontinued.
- On-ramps should not be metered when the demand volumes do not exceed 250 vehicles per hour or mainline freeway conditions cannot be provided with any measurable benefit from the dispersion of traffic (e.g., freeway mainline occupancy levels lower than 12 percent or greater than 35 percent).

Witkowski, Summers, Mouasher, and Marum (24) described the procedures and results of a study to evaluate the impacts of a freeway management system on freeway traffic operations in a metropolitan area. A before-and-after study evaluated several measures of effectiveness for a freeway corridor outfitted with ramp meters, variable message signs, traffic loop detectors, and closed-circuit television cameras. Results indicated that travel time improved between 2 percent and 6 percent on the 4.35-mi [7 km] section of the freeway where the ramp meters were in operation; significant positive response by drivers was measured as result of the variable message signs, and vehicle emissions on the freeway were estimated to have gone down in the after period. No significant change was measured overall in incident response time or incident duration even though these values did decline in the after period. Freeway accident rates increased in the after period, but the authors did not wish to attribute this solely to the freeway management system, due to other factors that may have affected this result.

Ullman (25) conducted a study to highlight how freeway traffic management components can be better accommodated in the design or redesign/reconstruction phase of freeways. One of these components was ramp metering operations. From a ramp metering perspective, entrances located a considerable distance downstream of the frontage road intersection are preferred. One way to increase the distance between an upstream intersection and an entrance ramp is to employ an X-shaped ramp design at cross-street arterial interchanges. *Procedures to Determine Frontage Road Level of Service and Ramp Spacing* by Fitzpatrick, Nowlin, and Parham contains a methodology for determining the desirable spacing for a ramp meter (26).

The cross-section design of the ramp subbase and base is another area where early decisions can result in greater flexibility during a reconstruction (25). Ramp metering systems can include the provision of HOV bypass lanes, and some systems include dual ramp lanes to allow two vehicles at a time to enter the freeway. Some agencies factor a dual-lane ramp into their calculations for earthwork. They may initially install pavement for a single ramp lane onto the base constructed for two lanes. This allows for easier expansion to the ramp if needed in the future. Provisions must be made to allow vehicles at a stop on the ramp to accelerate up to merging speeds prior to joining the main lanes of the freeway. Longer ramps offer greater flexibility for future

implementation of metering strategies. This implies that shallow merge angles will be preferable since this increases the effective ramp length.

CHAPTER 3

STATE PRACTICES FOR RAMP DESIGN

To have an appreciation for current DOT practices across the country, researchers conducted a search of each state's design manual via the Internet. Of the 23 states that had all or part of their design manuals online, 12 had some material available concerning the design of ramps. Those states and their respective manuals are listed below:

- California (Highway Design Manual; Dated July 1, 1995, revised November 1, 2001) (27)
- Connecticut (Highway Design Manual; Dated January 1999, revised December 2000) (28)
- Iowa (Design Manual-English; Dated September 1, 1995, revised January 2002) (29)
- Michigan (Road Design Manual; Dated June 5, 1996, revised March 8, 2002) (30)
- Minnesota (Traffic Engineering Manual; Dated July 1, 2000) (31)
- New Jersey (Design Manual-Roadway, Metric Units; Dated 2001) (32)
- New York (Highway Design Manual; Dated July 1972, last revised August 9, 2001) (33)
- North Carolina (Roadway Design Manual; Dated January 2, 2002) (34)
- Ohio (Roadway Design Manual; Dated December 1992, revised August 19, 2002) (35)
- South Dakota (Road Design Manual; Dated March 14, 2002) (36)
- Texas (Roadway Design Manual; Revised April 2002) (37)
- Washington (Design Manual-English Version; Dated June 1999, revised February 2002) (38)

In addition, the relevant material from the AASHTO *Green Book* (Fourth Edition, 2001) (39) was reviewed and compared, as well as the California Ramp Meter Design Manual (January 2000) (40), the HOV Facilities chapter of the Washington Design Manual (38), and the AASHTO *Guide for the Design of High Occupancy Vehicle Facilities* (1992) (41). Following is a summary of the information.

CRITICAL DESIGN ELEMENTS

There are elements in the design of a ramp that are critical to the success of the design. In addition to AASHTO criteria and the Texas design manual, two states in this review, Connecticut and Minnesota, define "critical design elements." Table 3-1 lists the various critical design elements identified by the various agencies.

In addition, Washington's manual (38) lists requirements for planning the design of an HOV facility. The travel demand and capacity must first be established, suitable corridors identified, the facility's length and location evaluated, and HOV demand estimated. A viable HOV facility will satisfy the following criteria:

- be part of an overall transportation plan,
- have the support of the community and public,
- be in response to demonstrated congestion or near-term anticipated congestion,
- bypass a local bottleneck or sufficient length to provide a travel time savings of at least 5 minutes during the peak period,
- have a sufficient number of HOV users for a cost-effective facility and be able to avoid the perception of underutilization, and
- have a design that provides for safe, efficient, and enforceable operation.

Table 3-1. Critical Elements of Ramp Design.

	AASHTO (39)	Texas (37)	Connecticut (28)	Minnesota (31)
Design speed	X	X		X
Portion of ramp to which design speed is applicable	X			
Ramps for right turns	X			
Connection type (loop, direct, semidirect)	X			
At-grade terminals	X			
Horizontal alignment (curvature)	X	X		X
Vertical alignment (grade and profile)	X	X		X
Sight distance	X	X		X
Superelevation and cross slope	X	X		X
Cross section (lane width)				X
Gores	X			
Distance between successive ramps		X		
Minimum length of acceleration for entrance ramp			X	X
Parallel portion of acceleration lane for entrance ramp			X	
Minimum length of deceleration for exit ramp			X	X
Deflection (taper) angle for taper exit ramp			X	
Horizontal clearance				X
Vertical clearance				X
Bridge width				X
Bridge structural capacity				X

ACCESS TO AND FROM HOVS

Direct Access to HOV Facilities

According to the Washington High Occupancy Vehicles Facilities design chapter (38), exclusive HOV access ramps for an inside HOV lane are recommended to maximize the efficiency of the HOV system. Direct access eliminates the HOV user crossing the general-purpose lanes since most of the mainline entrances are from the right side. Providing the HOV user access to the inside HOV lane without mixing with the general-purpose traffic saves the user travel time and aids in safety and enforcement, incident handling, and overall operation of the HOV facility. Priority should be given to locations that serve the greatest number of transit vehicles and other HOVs. Transit agencies may provide funding for the construction of a direct access ramp that serves transit vehicles. Direct access of any type is usually very expensive due to the structural and right-of-way requirements. However, if direct access is not provided in the initial project, provisions should be made so that they can be added later or at least the design should not preclude their addition at a later date. Figure 3-1 shows designs for a typical HOV flyover direct-connect on-ramp.

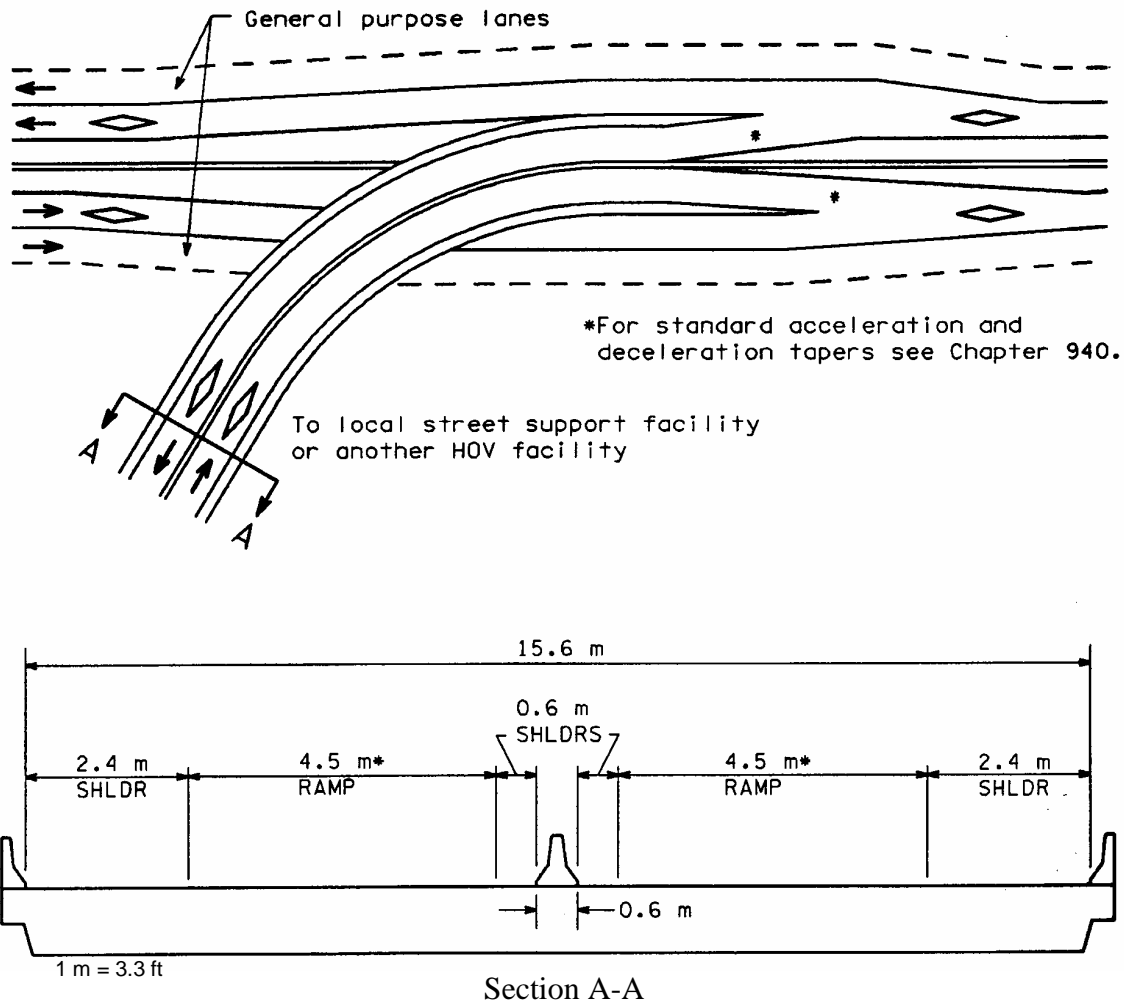
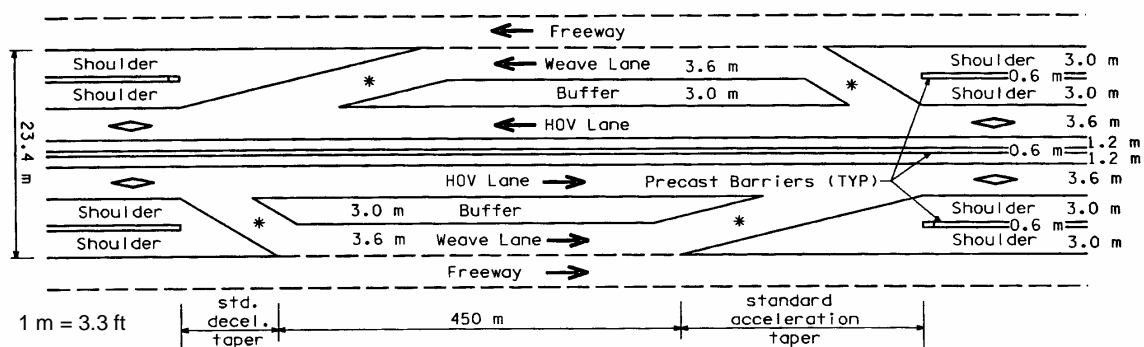


Figure 3-1. Typical HOV Flyover – Washington Design Manual Figure 1050-5a (38).

The AASHTO HOV design guide (41) states that terminal connections to HOV facilities from the adjacent freeway mainline should be made with flyover ramps at both terminal end connections if feasible. This allows buses and other vehicles using the HOV facilities to exit and enter the freeway mainline on the right instead of the inner high-speed lanes. This eliminates the high-speed lane merge, which is inherently more difficult to execute, especially for HOV traffic such as buses and vans. Depending on the interchange spacing, it could also eliminate the need for the HOVs to make several rapid lane changes in order to access the HOV lane or exit the freeway. If traffic patterns warrant, separated HOV facilities should tie to the existing street system within the central business district. These direct ramps are preferable to merging HOV traffic with other freeway traffic in advance of the central business district, provided conditions permit. From outer areas, connections may be provided into freeway frontage roads for either collection or distribution of HOVs through the use of flyover ramps.

Slip Ramps to HOV Facilities

The Washington HOV Design Manual (38) mentions that a less expensive alternative to a flyover ramp is a slip ramp, shown in Figure 3-2. Slip ramps provide access to and from the barrier-separated facility from the inside lane of the mainline. However, as a result of the operational problems associated with a left-hand slip ramp, a thorough operational analysis should be conducted and adequate signing should be provided.



For standard acceleration and deceleration tapers, see Washington Design Manual Chapter 940.

Figure 3-2. Typical HOV Slip Ramp – Washington Design Manual Figure 1050-6 (38).

The AASHTO HOV design guide (41) advises that where limited right-of-way and/or high costs prohibit the use of elevated flyover ramps, at-grade slip ramps can be used. At-grade slip ramps are also appropriate where the HOV facility is reversible; however, proper signing and/or barriers will be required to eliminate wrong-way entry and exit.

HOV Lane Termination

According to the Washington HOV Design Manual (38), the beginning and end of an HOV facility should be at logical points and should typically avoid existing freeway ramps. There should be adequate sight distance at the terminals, and adequate signing and pavement markings must be provided. For the termination of an HOV lane, the principles that apply to merge or

diverge maneuvers should be used. When the HOV lane is on the inside of the freeway, the desirable or higher values should be used since the interface is with the “fast” lane. The preferred method is to provide a straight-through move into a mixed-flow lane and drop a general-purpose lane. However, volumes for both the HOV lanes and general-purpose lanes and the geometric conditions should be analyzed so that the operational performance of the general-purpose lanes is not compromised.

FREEWAY RAMPS

Freeway-Freeway Ramps

According to AASHTO (39), the minimum design speed for direct connections should preferably be 40 mph [64.4 km/h]. They should have a paved shoulder width of 8 to 10 ft [2.4 to 3.1 m] on the right and 1 to 6 ft [0.3 to 1.8 m] on the left. The applicable superelevation rate should be similar to that used in open-road conditions.

Details in the Texas design manual (37) do not elaborate specifically on freeway-to-freeway connectors. Directional connections are used for important turning movements instead of loops to reduce travel distance, increase speed and capacity, reduce weaving, and avoid loss of direction in traversing a loop. Figure 3-32 of the Texas design manual, reproduced in Figure 3-3, contains design details for one- and two-lane ramps and direct connectors.

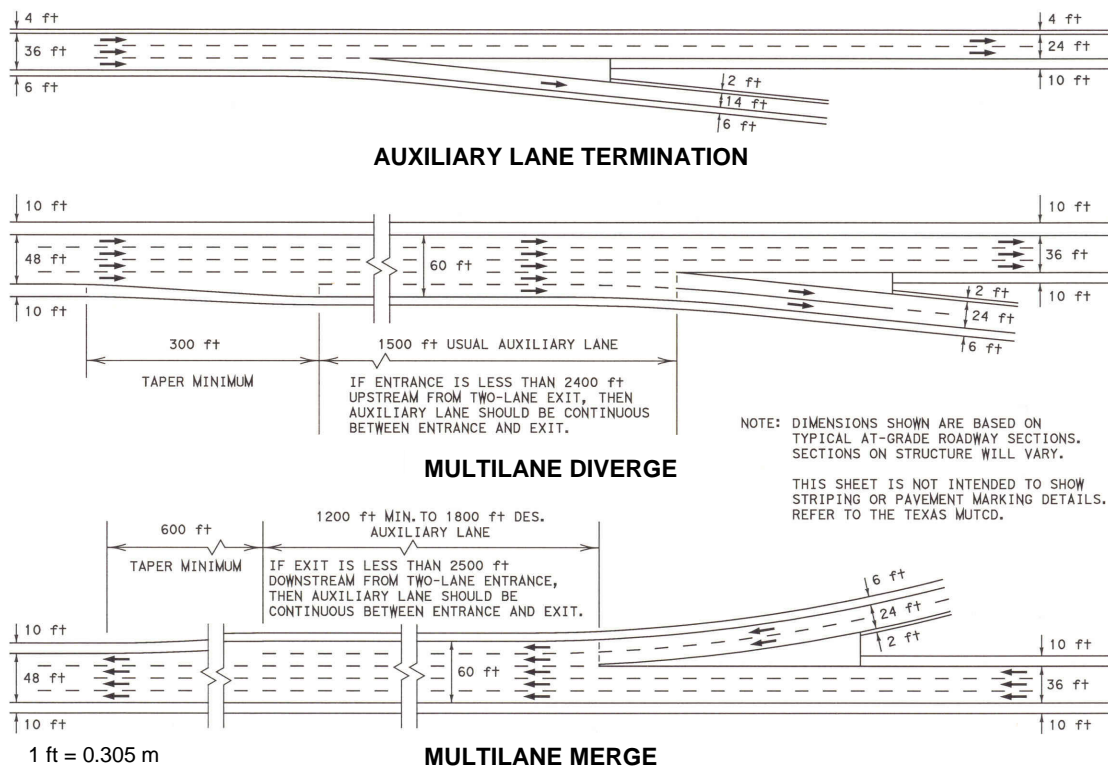


Figure 3-3. Design Details for One- and Two-Lane Ramps or Direct Connectors – Texas Roadway Design Manual Figure 3-32 (37).

California's design manual (27) includes additional detail on ramps that connect two freeways. The design speed should be a minimum of 50 mph [80 km/h], or adequate vertical sight distance should be provided for small-radius curves. The maximum profile grade should not exceed 6 percent. Shoulder width should be 5 ft [1.5 m] on the left and 10 ft [3.1 m] on the right for one- and two-lane ramps. Single-lane connectors in excess of 1000 ft [305 m] in length should be widened to two lanes to provide for passing maneuvers. Where design year volume is 900 to 1500 pcph, initial construction should provide a single-lane connection with the capability of adding an additional lane. A multilane connection should be provided when the design year volume exceeds 1500 pcph.

Freeway-to-freeway connectors may also be metered when warranted. According to the California Ramp Meter Design Manual (40), the installation of ramp meters on connector ramps shall be limited to those facilities that meet or exceed the following geometric design criteria: standard lane and shoulder widths; and "tail light" sight distance, measured from 6.7 in [17 cm] eye height to a 23.6 in [60 cm] object height, is provided for a minimum design speed of 50 mph [80 km/h]. All lane drop transitions on connectors shall be accomplished with a taper of 50:1 minimum.

Loop Ramps

AASHTO (39) states that the practical size of loops resolves into approximate radii of 100 to 170 ft [30.5 to 51.9 m] for minor movements on highways with design speeds of 50 mph [80 km/h] or less and 170 to 250 ft [51.9 to 76.3 m] for more important movements on highways with higher design speeds. A continuous additional lane is needed for deceleration, acceleration, and weaving between the on- and off-ramps.

The Texas manual (37) mentions loop ramps only in the context of cloverleaf interchanges, which it discourages for new construction. The manual states that cloverleaves should not be used where left-turn volumes are high (greater than 1200 pcph) since loop ramps are limited to one lane of operation and have restricted operating speed. The capacity restrictions and short weaving lengths between loops are cited as disadvantages of cloverleaf interchanges. The manual states that when cloverleaves are used, the design should include collector-distributor roads to provide more satisfactory operations.

Other states specify loop ramps only for low-speed connections. Loop ramps, compared to directional or semidirectional ramps, have smaller radii and lower design speeds, and loop on-ramps are often followed closely by a loop off-ramp. Curb and gutter may also be used on loop ramps. Consideration should be made for adequate approach and departure tangents for loop ramps with significant truck volumes.

DESIGN CONTROLS

Design Speed

The *Green Book* (39) states that desirable ramp design speeds should approximate the low-volume running speed on the intersecting highways. However, this design speed is not always

practical and lower design speeds may be selected, but they should not be less than the low range presented in [Table 3-2](#). Only those values for highway design speeds of at least 50 mph [80 km/h] apply to freeway and expressway exits.

Table 3-2. Guide Values for Ramp Design Speed – *Green Book* Exhibit 10-56 (39).

Ramp Design Range	Highway Design Speed									
U.S. Customary (mph)	30	35	40	45	50	55	60	65	70	75
Upper Range (85%)	25	30	35	40	45	48	50	55	60	65
Middle Range (70%)	20	25	30	33	35	40	45	45	50	55
Lower Range (50%)	15	18	20	23	25	28	30	30	35	40
Metric (km/h)	50	60	70	80	90	100	110	120		
Upper Range (85%)	40	50	60	70	80	90	100	110		
Middle Range (70%)	30	40	50	60	60	70	80	90		
Lower Range (50%)	20	30	40	40	50	50	60	70		

The Texas design manual (37) has a table similar to [Table 3-2](#), in metric units and in U.S. customary units. All ramps and connections should be designed to enable vehicles to leave and enter the traveled way of the freeway at no less than 50 percent (70 percent usual, 85 percent desirable) of the freeway’s design speed, which correspond to the upper, middle, and lower ranges in the various tables. The design speed for a ramp should not be less than the design speed on the intersecting frontage roads. The designer should refer to the *Green Book* for additional guidance.

Each state reviewed has a table for determining the appropriate ramp design speed for the corresponding mainline design speed, existing radius, and, perhaps, grade. Most states permit or encourage the use of variable design speeds, where the design speed at the freeway connection is higher than the local street connection. The minimum design speeds for freeway ramps range from 40 to 60 mph [64.4 to 96.6 km/h] for the freeway connection (or high range) and 25 to 35 mph [40.3 to 56.4 km/h] for the local street connection (or low range). Most states’ tables are similar, if not identical, to [Table 3-2](#).

The AASHTO HOV design guide (41) says that all terminal and intermediate access connections should have high design standards. Tapers on entrance and exit ramps should be designed the same as for other freeway ramps except that designers should give special consideration to the acceleration and deceleration characteristics of loaded buses. This is especially critical where ramp grades are significant. Ramps that connect to adjacent facilities or to cross streets should be designed to the same standards as comparable facilities that connect freeways to crossroads. The *Green Book* describes the HOV designs for these types of connections.

Ramp Radii/Horizontal Alignment

AASHTO (39) states that the minimum radius is a limiting value of curvature for a given design speed. It is determined from the maximum rate of superelevation and the maximum side friction factor selected for design.

The Texas manual (37) refers to radii on ramps only in the context of superelevation rates and design speed. For a maximum superelevation rate of 6 percent, the minimum radius for a design speed of 37.3 mph [60 km/h] is 442.6 ft [135 m]. The superelevation tables included in the Texas design manual have the complete set of values for minimum radii.

Although the actual radius of a ramp is based on the design speed, a few of the states specifically discuss minimum radii. The minimum radii for loop ramps range from 150 to 250 ft [45.6 to 76.2 m], although in Connecticut (28), a loop ramp can be considered as a turning roadway with a minimum radius of 410 ft [125 m] for a design speed of 37.5 mph [60 km/h]. For all other ramps, Connecticut uses criteria for rural highways, which incorporate the *Green Book* table for 6 percent maximum superelevation for values of minimum radii.

Grades/Vertical Alignment

According to AASHTO (39), ramp grades should be as flat as practical to minimize the driving effort needed in maneuvering from one road to another. For any one ramp, the gradient to be used is dependent on a number of factors unique to that site and quadrant. The flatter the gradient on a ramp, the longer it will be, but the effect of gradient on ramp length is not substantial. In general, adequate sight distance is more important than a specific gradient control and should be favored in design. The gradient for a ramp with a high design speed should be flatter than for one with a low design speed. As general criteria, it is desirable that upgrades on ramps with a design speed of 45 to 50 mph [72.3 to 80.5 km/h] be limited to 3 to 5 percent. Where appropriate for topographic conditions, grades steeper than desirable may be used. One-way downgrades on ramps should be held to the same general maximums, but in special cases they may be 2 percent greater.

Texas (37) defines minimum lengths of crest and sag vertical curves on ramps and direct connectors to be the same as those on highways, based on the headlight distance for various design speeds and algebraic differences in grade. Texas encourages values greater than minimum. The tangent or controlling grade on ramps and direct connectors should be as flat as possible, and preferably should be limited to 4 percent or less. The designer may refer to the *Green Book* for additional discussion.

Most states base their maximum grades on the design speed of the ramp, with other considerations for downgrade and high levels of truck traffic. For high-speed ramps, the consensus on the suggested range of maximum grade is 3 to 5 percent, with allowances for an additional 2 percent on downgrades. North Carolina has a different approach, however; their design manual (34) specifies a detailed procedure for determining grade that involves superelevation rates and elevations at specific regular points throughout the length of the ramp.

Sight Distance

AASHTO (39) states that sight distance along a ramp should be at least as great as the design stopping sight distance. There should be a clear view of the entire exit terminal, including the exit nose and a section of the ramp roadway beyond the gore. The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum stopping sight distance for the through traffic design speed, desirably by 25 percent or more. Decision sight distance is desired where feasible. There should be a clear view of the entire exit terminal, including the exit nose. Chapter 3 of the *Green Book* contains descriptions and ranges of design values for decision sight distance and stopping sight distance on horizontal and vertical curves for turning roadways and open road conditions.

Texas policy (37) for sight distance on ramps is to provide the stopping sight distance. Design stopping distances for highway speeds range from 524.6 to 934.4 ft [160 to 285 m], depending on design speed. The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum stopping sight distance for the freeway design's speed, preferably by 25 percent or more; decision sight distance is a desirable goal.

Each state reviewed uses the AASHTO *Green Book* guidelines as the basis for their appropriate sight distance values. Some states emphasize decision sight distance for drivers approaching ramps while others emphasize stopping sight distance. Most states also have allowances for variations based on curvature, grade, or type of ramp.

Superelevation

The *Green Book* (39) provides a list of guidelines to be used for cross-slope design on ramps:

- Superelevation rates, as related to curvature and design speed on ramps, are given in the *Green Book* in Exhibits 3-21 through 3-25. Where drainage impacts to adjacent property or the frequency of slow-moving vehicles are important considerations, the superelevation rates and corresponding radii in *Green Book* Exhibit 3-40 can be used.
- The cross slope on portions of ramps on tangents normally should be sloped one way at a practical rate ranging from 1.5 to 2 percent for high-type pavements.
- In general, the rate of change in cross slope in the superelevation runoff section should be based on the maximum relative gradients listed in *Green Book* Exhibit 3-27.
- The maximum algebraic difference in cross slope between the auxiliary lane and the adjacent through lane is shown in *Green Book* Exhibit 9-49. For a design speed of 35 mph [56.4 km/h] or more, the maximum algebraic difference is 4 to 5 percent.
- Designers should study the exit terminal, the ramp proper, and the entrance terminal in combination to ascertain the appropriate design speed and superelevation rates.

Superelevation rates and radii for diamond ramps should reflect a decreasing sequence of design speeds for the exit terminal, ramp proper, and entrance terminal. Curvature of a loop ramp is determined by the design speed and superelevation rate used; the superelevation should be gradually developed into and out of the curves for the ramp proper. The design speed and superelevation rates for direct and semi-direct ramps are comparable to open-road conditions.

In the Texas design manual (37), the superelevation rates, as related to curvature and design speed of the ramp, are given in the *Roadway Design Manual* Table 3-21. For design speeds greater than 43.4 mph [70 km/h], the appropriate superelevation rates are the same as those used in open-road conditions. The superelevation rate used should be as high as possible, preferably in the upper half or third of the indicated range, particularly in descending grades.

The other states vary on their maximum superelevation rates, largely due to the potential effects of weather. For states that are more likely to see effects from ice and snow, the rates are lower; New Jersey (32) and Connecticut (28) have a 6 percent maximum superelevation rate. For states less likely to see winter weather, the rates are higher; California (27) allows a 12 percent maximum rate, except in areas where snow and ice prevail. The states use tables to determine the specific rate applicable to the specific conditions of the ramp; Table 3-3 is a reproduction of the relevant table from the New Jersey design manual.

Table 3-3. Interchange Ramp Superelevation – New Jersey Design Manual Table 7-2 (32).

U.S. Customary									
Design Speed (mph)	Radius (ft)								
	150	230	310	430	600	1000	1500	2000	3000
25	4-6	3-6	3-6	3-5	2-4	2-3	2	2	2
30		6	5-6	4-6	3-5	3-4	2-3	2	2
35				6	5-6	4-5	3-4	2-3	2
40						5-6	4-5	3-4	2-3
Metric									
Design Speed (km/h)	Radius (m)								
	45	70	100	150	200	300	500	700	1000
40	4-6	3-6	3-6	3-5	2-4	2-3	2	2	2
50		6	5-6	4-6	3-5	3-4	2-3	2	2
60				6	5-6	4-5	3-4	2-3	2
70						5-6	4-5	3-4	2-3

Acceleration/Deceleration Lanes and Transitions

The *Green Book* (39) provides for acceleration lane lengths as long as 1790 ft [546 m] and deceleration lane lengths up to 660 ft [201.3 m]. There are also adjustment factors for upgrades and downgrades. AASHTO recognizes that the tapered transition lane more closely resembles a driver's natural path and the parallel lane provides more space for acceleration and merging; however, the *Green Book* makes no recommendations as to whether the taper design or the parallel design is preferred.

The Texas design manual (37) provides an illustration, shown in Figure 3-4, of typical entrance and exit ramps, which indicates a parallel-type entrance ramp of 442.6 ft [135 m] in length and a taper-type exit ramp with a variable length. The variable length ranges from 147.5 to 655.7 ft [45 to 200 m] for deceleration lanes and 164 to 1705 ft [50 to 520 m] for acceleration lanes, dependent on highway design speed.

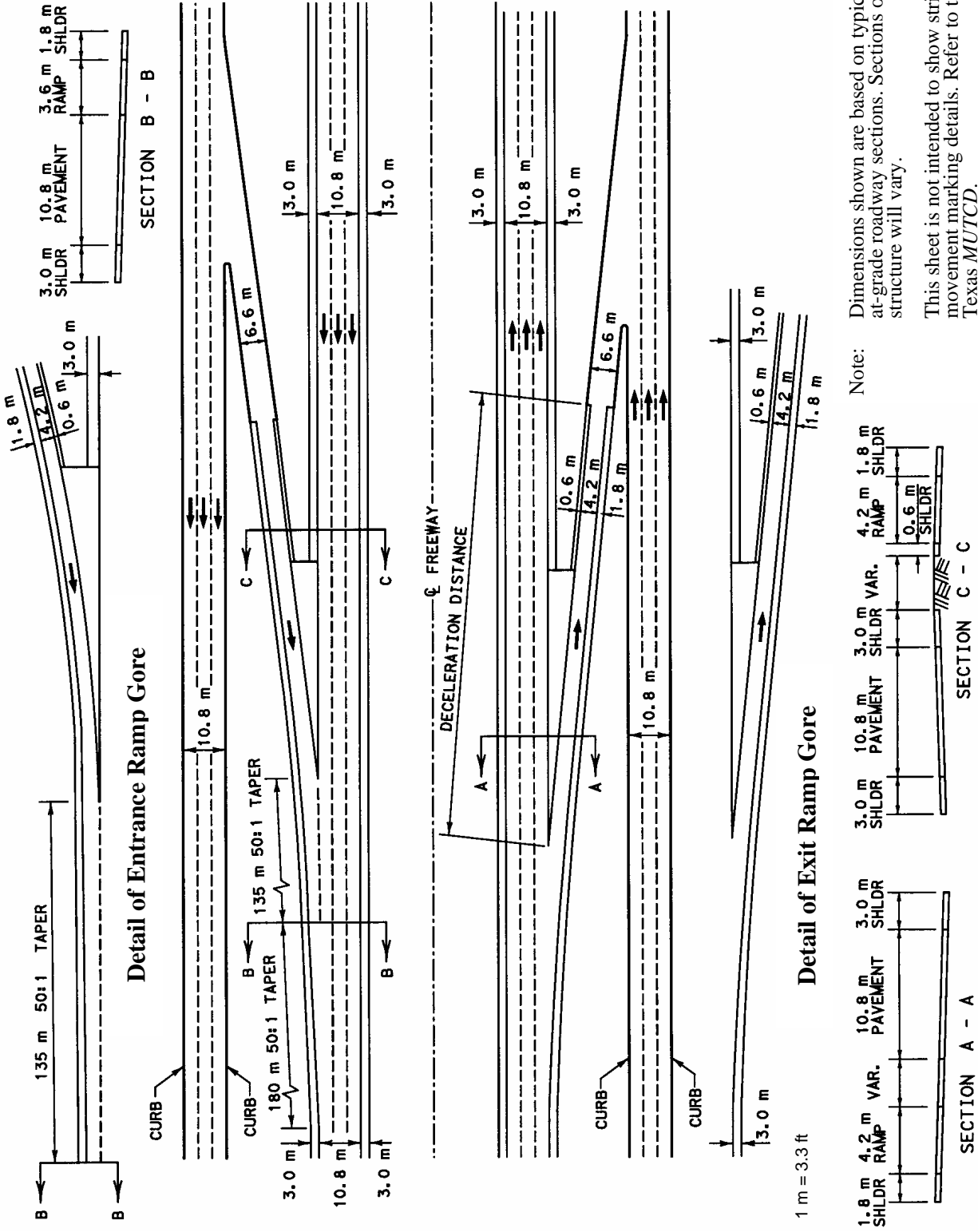


Figure 3-4. Entrance/Exit Ramps – Texas Roadway Design Manual Figure 3-26 (37).

In general, lengths for both acceleration and deceleration lanes in other states ranged between 150 and 600 ft [45.8 and 183 m], depending on the speed of the mainlanes; however, both Connecticut (28) and Washington (38) allowed for acceleration lanes longer than 1400 ft [427 m]. Acceleration lanes were longer, on average, than deceleration lanes. Most states that indicated a preference or a standard policy favored taper designs for deceleration lanes and parallel designs for acceleration lanes.

On metered ramps in California (40), where truck volumes are 5 percent or greater on ascending entrance ramps to freeways with sustained upgrades, a minimum 983.6 ft [300 m] length of auxiliary lane should be provided beyond the ramp convergence point. This auxiliary lane should also be provided when ramp volumes exceed 1500 vph.

Cross Section/Ramp Width

The *Green Book* (39) states that ramp traveled-way widths are governed by the type of operation, curvature, and volume and type of traffic. The roadway width for a turning roadway includes the traveled-way width plus the shoulder width or equivalent clearance outside the edges of the traveled way. Pavement widths for a one-way, one-lane ramp with significant single unit (SU) vehicles to govern design range from 18 to 26 ft [5.5 to 7.9 m], depending on the radius of the inner edge of pavement. A complete table is found in *Green Book* Exhibit 10-67 (39). The sum of the shoulder widths should not exceed 10 to 12 ft [3.1 to 3.7 m], with an acceptable paved shoulder width on the left equal to 2 to 4 ft [0.6 to 1.2 m]. The *Green Book* specifies modifications for additional lanes, fully directional lanes, and overpasses and underpasses.

In Texas (37), the cross section of a ramp or direct connector is a function of the following variables: number of lanes determined by traffic volume; minimum lane and shoulder width; lane balance; and, where two lanes are required by volume, the provision of parallel merging two lanes onto the mainlanes that must be provided at the terminal. Shoulder widths range from 2 to 4 ft [0.6 to 1.2 m] on the inside of the curve and 8 ft [2.4 m] on the outside of the curve. Lane widths are 14 ft [4.2 m] for single-lane ramps and 12 ft [3.6 m] per lane for multilane ramps.

Minimum ramp lane widths in other states vary between 12 and 18 ft [3.7 and 5.5 m] for a one-way, one-lane ramp. The width of the right shoulder is 6 to 8 ft [1.8 to 2.4 m] for a minimum value; the left shoulder width is generally 4 ft [1.2 m], although it ranges from 2 ft [0.6 m] in Washington (38) to 7 ft [2.1 m] in New Jersey (32). Some states also specify dimensions for multilane ramps, with widths of the additional lanes specified at 10 to 14 ft [3.1 to 4.3 m].

According to California's Ramp Meter Design Manual (40), pavement widths on one- or two-lane metered ramps should be 12 ft [3.6 m] for each lane, 4 ft [1.2 m] for the left (inside) shoulder, and 8 ft [2.4 m] for the right (outside) shoulder. For three-lane ramps, shoulders are reduced to 2 ft [0.6 m] on each side. On local street entrance ramps, the multilane segment should transition to a single lane width between the ramp meter limit line and the 6.6 ft [2 m] separation point from the mainline edge of traveled way.

Curbs

AASHTO (39) states that curbs should be considered only to facilitate particularly difficult drainage situations, such as in urban areas where restrictive right-of-way favors enclosed drainage. Curbs may also be used on low-speed ramps and at T-intersections for turning movements and the protection of pedestrians.

Texas (37) allows curbs on ramps in a few situations. Where a curb is present on a new ramp lane, it is to be mountable and limited to 3.9 in [100 mm] or less in height. The width of the curbed ramp lane is 21.6 ft [6.6 m] and is measured face to face of curb. Existing curb ramp lanes widths of 18.7 ft [5.7 m] may be retained.

Other states' design manuals prohibit curbs on ramps in the vast majority of conditions, with specific exceptions for controlling drainage or connecting with a local street (to protect pedestrians, improve channelization, or provide continuity with the local street).

Ramp Terminals

The *Green Book* (39) defines the terminal of a ramp as that portion adjacent to the through traveled way, including speed-change lanes, tapers, and islands. Ramp terminals may be the at-grade type, as at crossroad terminals, or the free-flow type, where ramp traffic merges with or diverges from high-speed traffic at flat angles. Design elements for the at-grade type are provided in Chapter 9 of the *Green Book* with intersection design. Details for free-flow ramps are provided in Chapter 10 of the *Green Book* and are divided by single-lane entrance terminals, single-lane exit terminals, and multilane terminals.

The Texas manual (37) states that all ramps and direct connections should be designed for one-lane operation with provision for emergency parking; however, if the anticipated volume exceeds the capacity of one freeway lane, two-lane operation may be provided with consideration given to merges and additional entry lanes downstream. Chapter 3 of the Texas manual provides diagrams of various types of ramps and connecting roadway arrangements.

Instructions on ramp terminals in other states consist mainly of the proper procedures for determining superelevation rates, providing ample recovery area, and discouraging wrong-way entry. The manuals also contain diagrams of typical ramp terminal sections. Figure 3-5 is a typical urban freeway ramp terminal design from the Ohio design manual (35).

Gore Areas

AASHTO (39) defines a "gore" as an area downstream from the shoulder intersection points. The physical nose is a point upstream from the gore, having some dimensional width that separates the roadways. The painted nose is a point, having no dimensional width, occurring at the separation of the roadways. As a general rule, the width at the gore nose is typically between 20 and 30 ft [6.1 and 9.2 m], including paved shoulders, measured between the traveled way of the mainline and that of the ramp. The entire triangular area between the painted nose and the gore nose should be striped to delineate the proper paths on each side and to assist the driver in

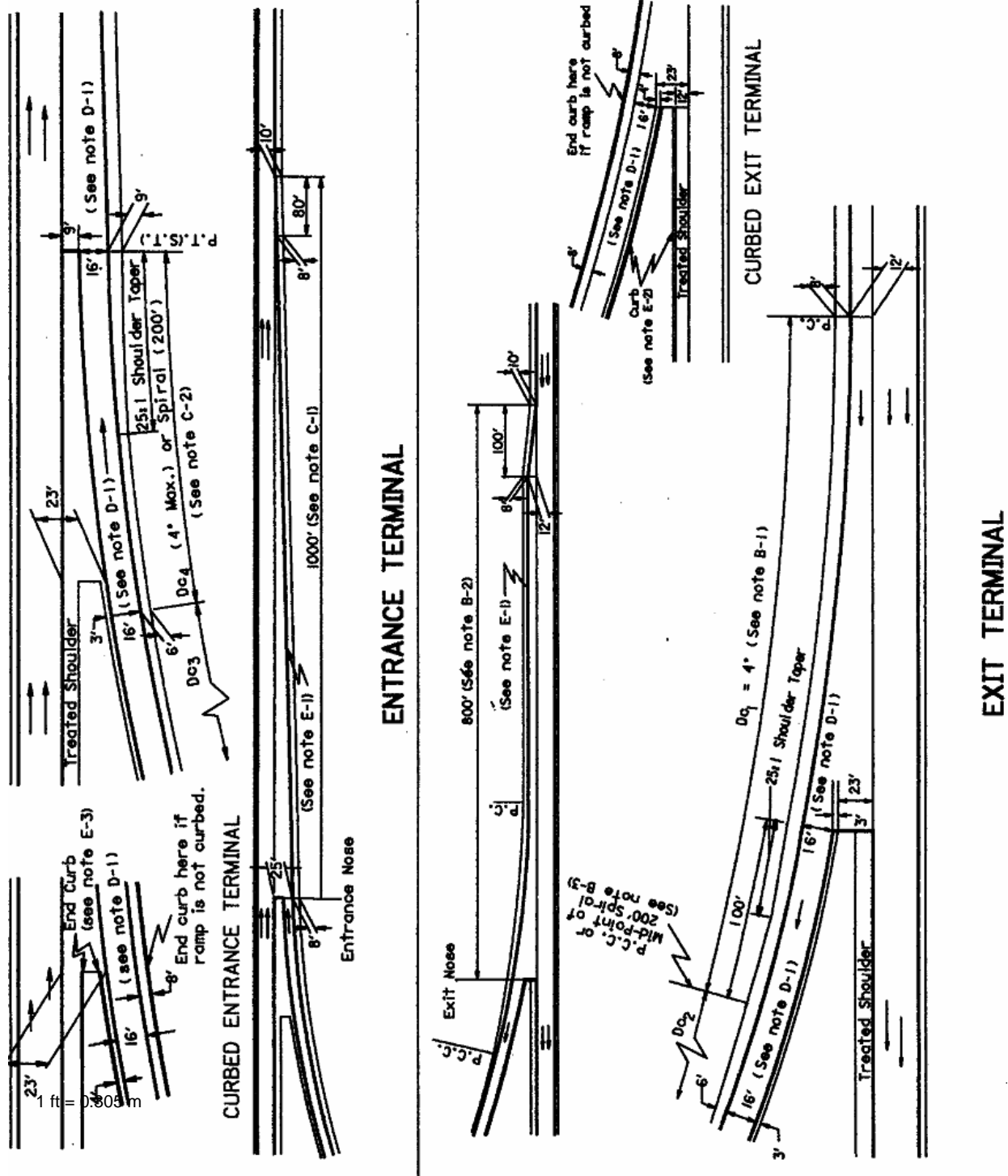


Figure 3-5. Class II Entrance and Exit Terminals – Ohio Design Manual Figure 404-3 (35).

identifying the gore area. The gore area should be kept as free of obstructions as practical to provide a clear recovery area. The unpaved area beyond the gore nose should be graded as nearly level with the roadways as practical. If fixed objects must be located within the gore area, they must be shielded with guardrails, cushions, or energy-dissipating devices.

The Texas design manual (37) provides a variety of figures illustrating the dimensions of gore areas for different configurations of entrance and exit ramps. The diagrams are similar to Figure 3-6.

Four states specifically mentioned gore areas in their discussions of ramps or interchanges, and each provided essentially the same information to designers. The area beyond the physical nose of the gore should be free from obstructions; however, when it is necessary to place a sign or other obstruction in that area, it should be shielded by a barrier, crash cushion, or guardrail. Signs placed in that area should be breakaway or frangible. The grade in the gore area should be as flat as practical, nearly level with the roadways as possible, and safely traversable. Figure 3-6 is a drawing of a typical gore area, from the Washington design manual (38).

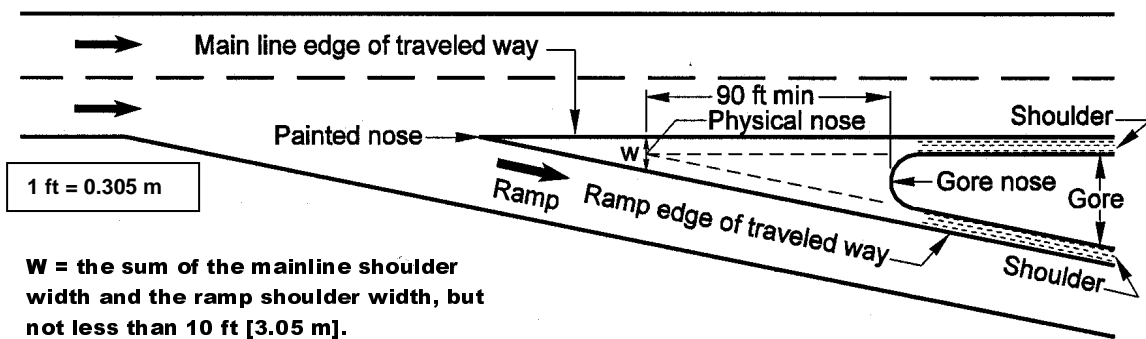


Figure 3-6. Gore Area Characteristics – Washington Design Manual Figure 940-11 (38).

RAMP INTERSECTIONS

According to the *Green Book* (39), at-grade ramp intersections can be designed according to the guidelines for other at-grade intersections. These intersections should be located an adequate distance from the separation structure to provide adequate sight distance for all approaches. Desirably, high-speed entrance ramp terminals should be located on descending grades to aid truck acceleration. The *Green Book* mentions two specific kinds of special ramp intersections: diamond interchanges and single-point urban interchanges (SPUIs). With diamond interchanges, the intersection on the crossroad formed by the terminals functions as any other T-intersection at grade and should be designed as outlined in Chapter 9 of the *Green Book*. SPUIs are typically characterized by narrow right-of-way, high construction costs, and greater capacity than conventional tight diamond interchanges. They are primarily suited for urban areas where right-of-way is restricted but may also be applicable to rural settings where it is undesirable to utilize adjacent right-of-way due to environmental, geographical, or other constraints. Details for SPUIs are provided in Chapter 10 of the *Green Book*.

In general, ramp approaches to intersections are treated as other approaches, but there are some variations. Texas policy (37) states that turnaround lanes are to be provided at all interchanges with major arterials in urban and suburban areas where the freeway lanes are flanked with one-way frontage roads, but not with two-way frontage roads. When the cross street overpasses the freeway, the resulting turnarounds will be on bridge structures, in which case designers should carefully evaluate the sight lines and distances for bridge-related sight obstructions.

The design of a ramp/local street intersection in other states is generally treated the same as any other local street intersection with a few special considerations. There must be sufficient storage capacity for the ramp's approach to the intersection to avoid problems with traffic backing up onto the freeway. There must be proper consideration of adequate sight distance, so that bridge piers do not obstruct the drivers' line of sight. There must also be proper signing to minimize the potential for wrong-way movements. The special one-way nature of most ramps and the fact that much of the traffic at ramp intersections consists of turning movements must also be considered.

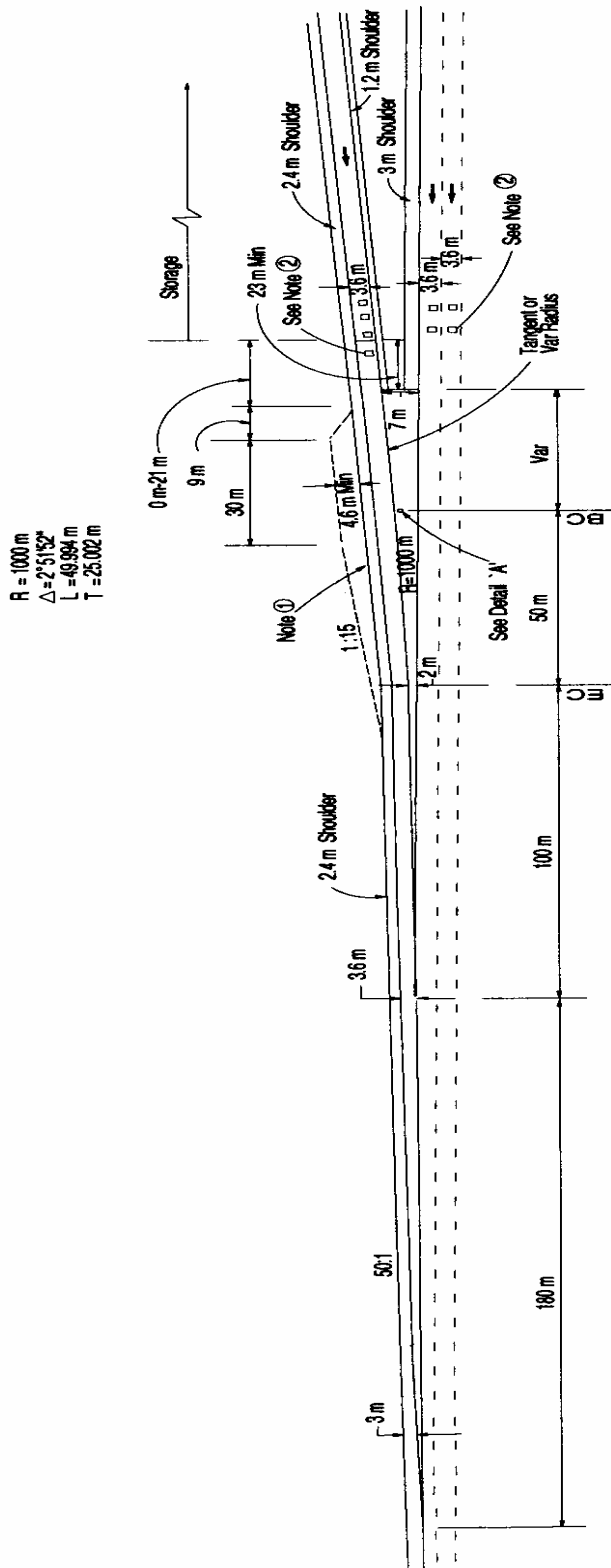
RAMP METERING

The *Green Book* (39) contains only a brief discussion of ramp metering, stating that it may be limited to only one ramp or integrated into a series of entrance ramps. It discusses the general procedure involved in ramp metering, and then refers the reader to the *Highway Capacity Manual* and the *AASHTO Guide for the Design of High-Occupancy Vehicle Facilities*.

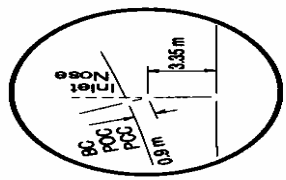
The Texas design manual (37) states that where ramps are initially, or subsequently, expected to accommodate metering, the geometric design features shown in Design Criteria for Ramp Metering (future reference to be provided in the manual) may be considered. Ramp metering, when properly designed and installed, has been shown to have potential benefits for the operation of the mainlanes. However, since ramp meters are installed to control the number of vehicles that are allowed to enter the mainlanes, an analysis of the entire roadway network area should be done to determine any adverse operational impacts to other roadways. It is suggested that the analysis specifically include both frontage road and adjacent cross street operations of through traffic, turning movements, and queue lengths.

California's Ramp Meter Design Manual (40) states that the decisions for new ramps should be based on projected peak-hour traffic volumes 20 years after completion of construction while improvements to existing ramps should be based on current peak-hour traffic volumes. Single-lane metered ramps are sufficient for volumes up to 900 vehicles per hour; multilane ramps are necessary for higher volumes. Each lane on a metered ramp is to be 12 ft [3.6 m] wide. HOV preferential lanes are to be provided at all metered ramps. A pullout area for enforcement and maintenance should be provided as well as sufficient graded area to allow for future ramp widening. Figure 3-7 is a typical single-lane freeway entrance ramp with a ramp meter.

Washington's HOV design chapter (38) states that HOV bypass lanes are typically located on the left side of the ramp. The design of the ramp meter should be determined by the existing conditions at each location; however, Figure 3-8 shows a typical design.



R = 1000 m
 $\Delta = 2^\circ 51' 52''$
L = 49.984 m
T = 25.002 m



DETAIL 'A'
INLET NOSE
No Scale

NOTES:

- ① Enforcement area to be graded for future widening.
- ② The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Loop Detector Details for typical installations.

1 m = 3.3 ft

Figure 3-7. Typical Freeway Entrance with One-Lane Ramp Meter – California Ramp Meter Design Manual Figure 1 (40).

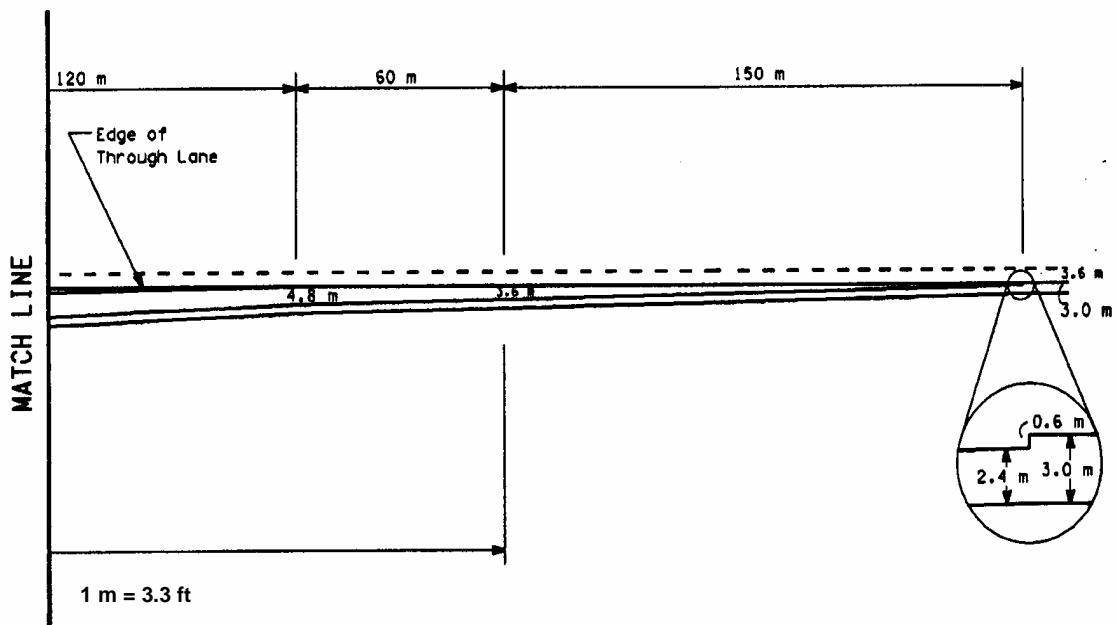
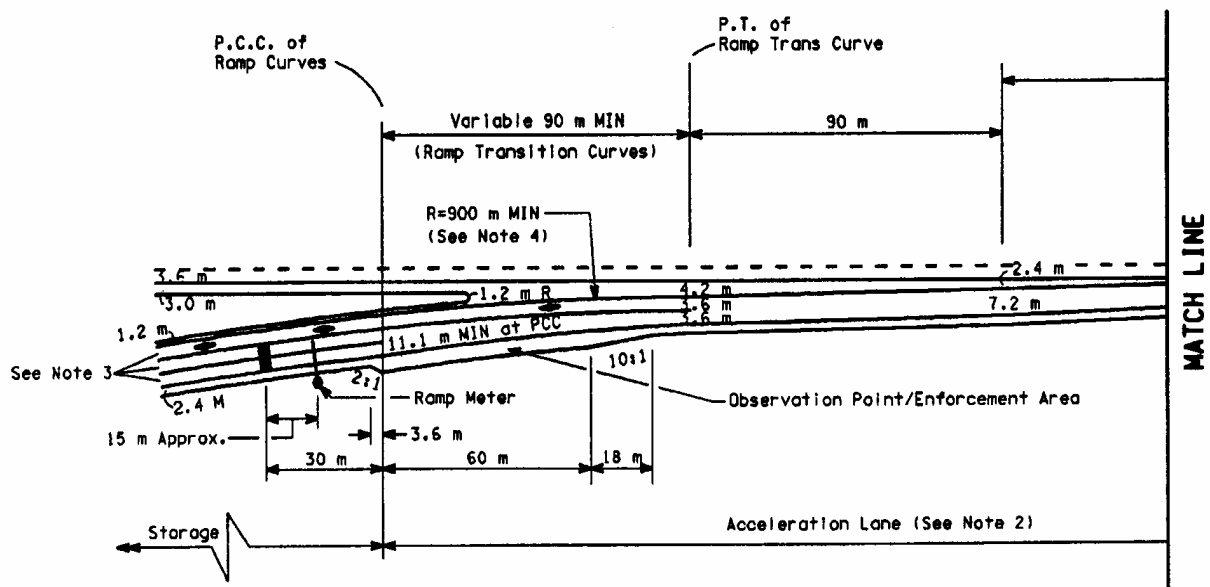


Figure 3-8. Typical Two-Lane Ramp Meter with HOV Bypass – Washington Design Manual Figure 1050-4b (38).

The AASHTO HOV design guide (41) states that bypass lanes at metered ramps provide an opportunity to give priority treatment to and encourage the use of HOVs. These bypass lanes can be restricted to buses only or can be made available to all HOVs; the decision on eligibility should depend on the goals of the community, the traffic patterns on the ramp, and the geometric conditions at the site. The design of the ramp meter bypass should be determined by the conditions at each location. Bypasses should be 12 ft [3.7 m] wide with full ramp shoulders where possible and should extend 300 ft [91.5 m] beyond the metering signal to permit HOVs to merge with normal ramp traffic. The ramp bypass traffic should merge first with regular ramp traffic and then with freeway traffic. Normal merging design standards should be used for both.

SPACING

Successive Ramps - Spacing and Location

AASHTO (39) specifies 1000 ft [305 m] between a pair of entrances or a pair of exits, 500 ft [152.5 m] for an exit followed by an entrance, and 2000 ft [610 m] for a weaving section on a system-to-service interchange. An added guideline is that there should be an auxiliary lane for entrance-exit pairs (weaving sections) with a distance of less than 1500 ft [457.5 m]. AASHTO also states that left-hand entrances and exits are contrary to the concept of driver expectancy when intermixed with right-hand entrances and exits. Therefore, extreme care should be exercised to avoid left-hand entrances and exits in the design of interchanges.

For an entrance ramp followed by an exit ramp, the Texas manual (37) calls for 1475 ft [450 m] with an auxiliary lane or 1967.2 ft [600 m] without an auxiliary lane. For a pair of exit ramps, the minimum distance is 983.6 ft [300 m]. Right-side ramps are superior to left-side ramps in operational characteristics and safety and are encouraged for all new construction.

Each state with specific discussions on successive ramps calls for either 900 ft or 300 m spacing. In addition, New Jersey (32) and South Dakota (36) specify a minimum of 1967.2 ft [600 m] for weaving sections and 491.8 ft [150 m] for an exit followed by an entrance. Each state also specified a preference or a requirement that all ramps be placed on the right side of the traveled way to minimize drivers' confusion and the need for weaving.

Interchange Spacing

The *Green Book* (39) states that a general rule of thumb for minimum interchange spacing is 1 mi [1.6 km] in urban areas and 2 mi [3.2 km] in rural areas. In urban areas, spacing of less than 1 mi [1.6 km] may be developed by grade-separated ramps or by adding collector-distributor roads.

Texas (37) does not define a minimum distance between interchanges. References are only to distances between successive ramps, which can be found in the relevant section listed previously in this report.

Three states mention interchange spacing as an important consideration. In New Jersey (32), the minimum spacing of interchanges for proper signing on the main road should be at least 1 mi

[1.6 km] between urban crossroads. Spacing of less than 1 mi [1.6 km] may be developed by using grade-separated ramps or by adding collector-distributor roads. Ohio (35) has a similar requirement, with an additional preference that the average spacing between urban interchanges be at least 2 mi [3.2 km]. The Washington design manual (38) also contains a similar policy of 2 mi [3.2 km] for interchange spacing.

Access Control

AASHTO (39) states that access control should be an integral part of the design of any highway whose primary function is mobility. The control of access improves safety and makes operations more efficient. Adequate access control minimizes spillback on the ramp and crossroad approaches to the ramp terminal, provides adequate distances for crossroad weaving, provides space for merging maneuvers, and provides space for storage of turning vehicles at access connections on the crossroad. Elements to be considered in determining access separation and access-control distances include the distances needed to enter and weave across the through-traffic lanes, move into the left-turn lane, store left turns with a low likelihood of failure, and extend from the stop line to the centerline of the intersecting road or driveway. Where only right-turn access is involved and there are no left turns or median breaks, the weaving distance governs.

Texas policy (37) recommends at least 246 ft [75 m] between the intersection of the ramp with the local street and the nearest driveway or side street. Desirable spacing is at least 459 ft [140 m] and increases with the number of weaving lanes and volume on the ramp or side street.

Control of access for 300 ft [91.5 m] from the intersection of the ramp with the local street was a common theme in other states, although California (27) reduced that number to 100 ft [30.5 m] in urban areas. North Carolina (34) prefers a minimum of 1000 ft [305 m] of access control along Y lines at interchanges, but 350 ft [106.8 m] is acceptable.

CHAPTER 4

CASE STUDIES

The potential Texas managed lane system could contain elements of systems that are currently in use in other communities. Information on how those elements are operating can help in the selection of components best suited for Texas. Examples include how special-use lanes are signed or marked, their typical dimensions for lane and shoulder widths, and how they are accessed.

As part of this research project, members of the research team visited the New Jersey Turnpike facility. The visit concentrated on the portion of the turnpike where two sets of lanes operate in each direction. The visit included driving the majority of the turnpike, observing the operations of the HOV lanes and the entrances and exits to both the inner and outer lanes, and meeting with the Turnpike Authority to obtain additional information on the turnpike's performance. This chapter contains information from the New Jersey Turnpike visit.

As opportunity presented itself, members of the research team visited other facilities to obtain on-site appreciation for the design and operations of the special-use lanes. When possible, photographs were taken of the lane(s), the existing signing and marking, and any other feature of interest. This chapter contains photographs of special-use lanes visited in Seattle, Washington, D.C., and Chicago.

DUAL-DUAL ROADWAY OF THE NEW JERSEY TURNPIKE

Construction on the New Jersey Turnpike began in January 1950 and was completed in 1952. It is currently a 148-mi [238.3 km], limited-access toll road that connects New York City to Philadelphia (see [Figure 4-1](#)). The Turnpike has 12-ft [3.7 m] lanes and 10-ft [3.1 m] shoulders with opposing directions separated by a median strip and a 42-inch [106.7 cm] high concrete barrier. The acceleration and deceleration lanes are 1200 ft [366 m] long at the 28 interchanges and 12 service areas.

A 32-mi [52 km] segment of the Turnpike was expanded to two separate roadways in each direction of travel beginning in the 1970s. [Figure 4-2](#) is a photograph of the dual-dual roadway that exists between Interchanges 8A and 14 (see [Figure 4-1](#)). The lane configuration between Interchange 8A and 9 is 2-3-3-2, between Interchange 9 and 11 it is 3-3-3-3, and between Interchange 11 and 14 is 4-3-3-4. Similar geometric design criteria were used on each section of roadway, called a "barrel." This allows trucks to be on either barrel, if needed, during an incident or maintenance. The objective of the dual-dual roadway was to improve operations and safety by separating heavy vehicles from light vehicles and to increase capacity in the most heavily traveled section of the Turnpike. It was also intended to provide greater flexibility for using the roadway during periods of heavy congestion such as a major incident, since dynamic message sign (DMS) technology could be applied to warn approaching drivers and divert them to the less-congested roadway (see [Figure 4-3](#)). Note that in certain jurisdictions, the dynamic message signs are called variable message signs or changeable message signs. [Figures 4-4](#) and [4-5](#) show

examples of the signs used to separate the vehicle types for the different barrels from a toll plaza or service area. Figures 4-6 to 4-9 show the split of the turnpike into the dual-dual position.

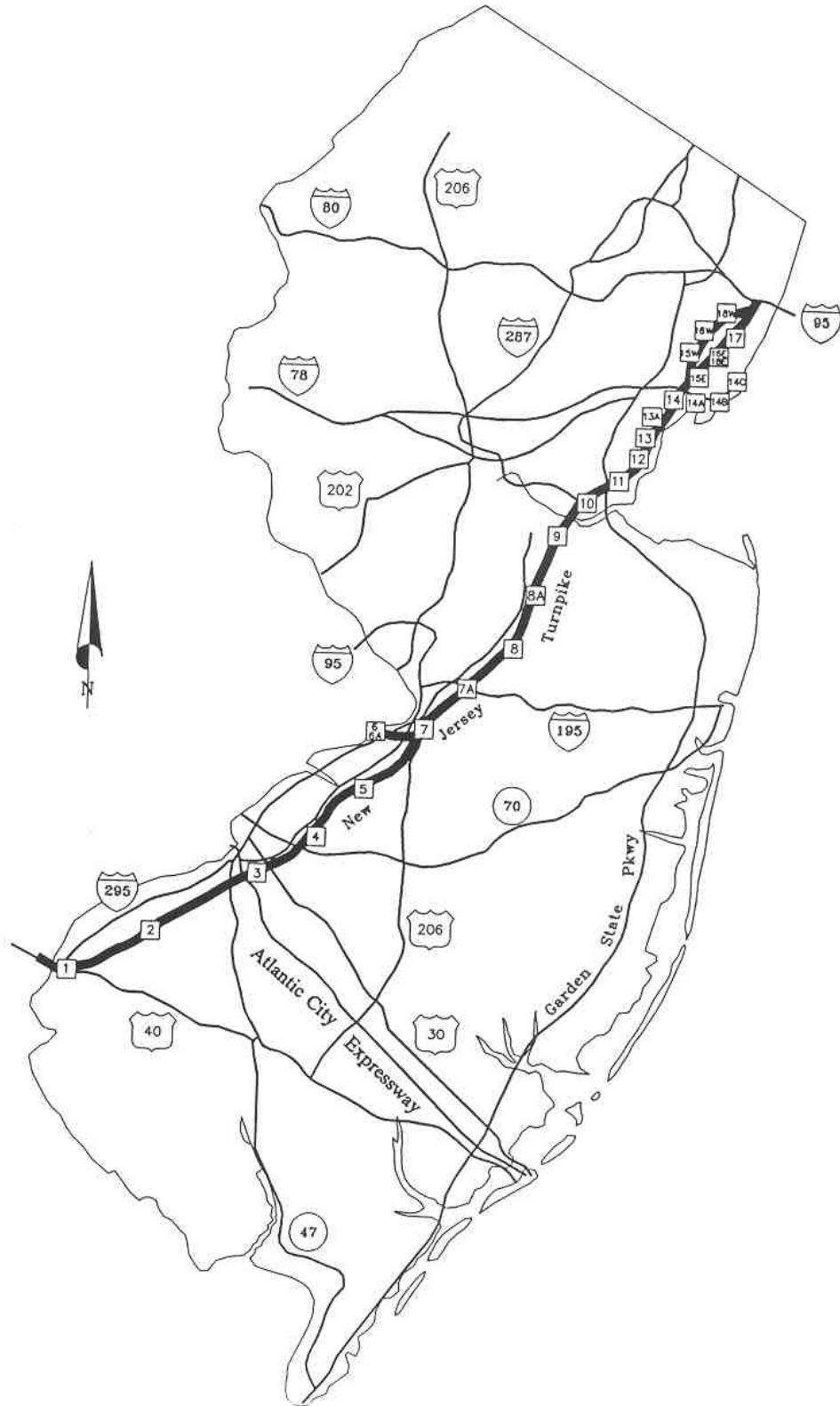


Figure 4-1. New Jersey Roadway System (42).



Figure 4-2. Example of Dual-Dual Portion of New Jersey Turnpike.



Figure 4-3. Example of a Dynamic Message Sign Diverting Traffic.



Figure 4-4. Example of Signs Used to Separate Traffic between Barrels.



Figure 4-5. Another Example of Signs Used to Separate Traffic between Barrels.



Figure 4-6. Example of the Split of New Jersey Turnpike into Dual-Dual Portion.

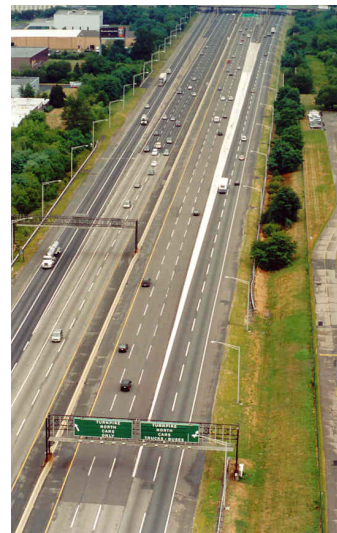


Figure 4-7. Overhead Example of the Split of the Turnpike into Dual-Dual Portion.



Figure 4-8. Example of Signs for the Turnpike Divide.



Figure 4-9. Example of Signs Used to Separate Traffic for the Turnpike Divide.

The inside lanes of the dual-dual roadway are for automobiles only while the outer lanes accommodate all vehicles types. [Figure 4-10](#) shows an example of trucks in the outer lanes and passenger cars only in the inner lanes. Between Interchanges 11 and 14, the left-most lane of the outer roadway is designated as an HOV lane between the hours of 6 a.m. and 9 a.m. in the northbound direction and between 4 p.m. and 7 p.m. in the southbound direction. The HOV lanes are reserved for cars and vans carrying three or more persons and to all buses and motorcycles. [Figure 4-11](#) is a picture of the HOV lane and the signing for the lane.



Figure 4-10. Example of Trucks Only on Outer Roadway.



Figure 4-11. HOV Sign on New Jersey Turnpike.

The inner lanes are separated by a concrete median barrier with gaps provided between interchanges. The advance signing and an example of a U-turn opening that exists in the median barrier between the opposing traffic are shown in [Figures 4-12](#) and [4-13](#), respectively. Within the dual-dual roadway portion, the two barrels are separated by metal guard rails with Z-turn openings. Advance signing, an example of a Z-turn opening, and the sign used at the Z-turn opening are shown in [Figures 4-14](#), [4-15](#), and [4-16](#), respectively. The U-turn and Z-turn openings are provided for use by emergency vehicles. In case of a serious incident, the Z-turn openings could also be used to move traffic from one barrel to another, although with difficulties for the longer vehicles.

The preferred method of closing a section of a barrel is to divert traffic at the upstream interchange and service plaza. Figures 4-17 and 4-18 are examples of the signs used at an entrance ramp when one of the barrels can be closed while Figures 4-19 and 4-20 show close-up views of the gate arm that can be used to close the entrance to one of the barrels.

Dynamic message speed limit and hazard warning sign combinations are located throughout the system. One combination was installed over each roadway, between each interchange, or at a maximum spacing of every 3 mi [4.8 km]. When a hazard exists on a roadway, the warning signs display the message “Reduce Speed Ahead – (Accident, Ice, Snow, Fog, Construction, Congestions).” Each warning sign is accompanied by a dynamic matrix speed limit sign, which displays the speed limit in 5 mph [8.1 km/h] increments from 30 to 60 mph [48.3 to 96.7 km/h]. Figure 4-21 shows an example of the combination



Figure 4-12. Advance Sign for a U-Turn.



Figure 4-13. U-Turn Opening.



Figure 4-14. Advance Sign for Z-Turn.

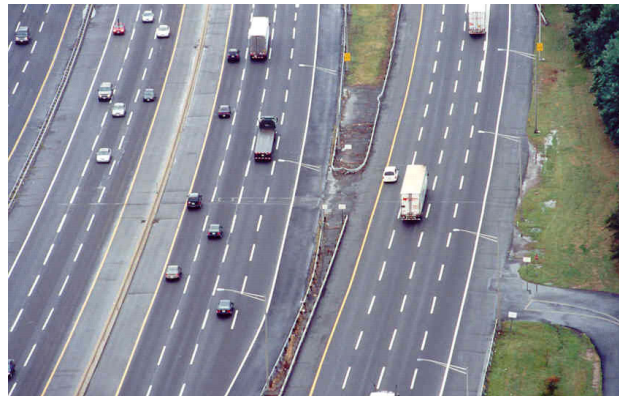


Figure 4-15. Z-Turn Opening.



Figure 4-16. Sign at Z-Turn Opening.



Figure 4-17. Example of Signs Used at Entrance Ramp of Closed Barrel.



Figure 4-18. Another Example of Signs Used at Entrance Ramp of Closed Barrel.



Figure 4-19. Close-Up of Gate at Ramp from Service Station.



Figure 4-20. A Close-Up of Another Gate at Ramp from Service Station.



Figure 4-21. A Dynamic Sign and Hazard Warning Sign Combination.

Figures 4-22 and 4-23 are aerial photos of interchanges on the New Jersey Turnpike. Note that each barrel has its own exit or entrance ramp. The inner roadway traffic does not need to weave across the outer roadway traffic to reach an exit. The traffic from each same-direction barrel merges prior to the toll plaza. The ramp designs used at the interchanges result in having all traffic moving through one toll plaza for each interchange (see Figure 4-24). This allows for consolidation of personnel and equipment (and resulting cost savings) in the collection of tolls. Both trumpet and slip-ramp designs are employed. Figure 4-25 shows the style of markings used at a gore.

To provide access to both directions, maintenance ramps are provided at specific locations. Figures 4-26 and 4-27 are examples of a maintenance ramp over an overpass. Note that the ramp uses different geometric criteria for shoulder widths and horizontal curvatures than what would be used for general-purpose traffic.



Figure 4-22. Interchange Design.



Figure 4-23. Aerial View of Interchange Design.



Figure 4-24. Example of Toll Plaza.



Figure 4-25. Markings at a Ramp Gore.



Figure 4-26. Overpass Bridge and Maintenance Ramp.



Figure 4-27. Maintenance Ramp over an Overpass.

Two-way daily traffic volumes on the dual-dual roadways range from 125,000 to 225,000 on weekdays. Trucks constitute a significant proportion of the total traffic: total truck volumes (two-plus axles) range from 26,000 to 40,000 (17 to 21 percent of the total volume) while the volume of heavy trucks (three-plus axles) ranges from 22,000 to 30,000 (13 to 18 percent of the total volume) (43).

Douglas (43) presents crash information to support the theory that the dual-dual roadway system enhances safety. During the five years before completion of the dual-dual roadway (1965-69), the average annual accident rate was 94.1 accidents per million vehicle miles; in the succeeding five years the rate was 79.2 accidents per million vehicle miles, a reduction of over 18 percent. Figure 4-28 shows the total accident rates for the five-year period from 1994 to 1998. In each of the five years, the crash rate on each of the dual-dual roadways (outer and inner) is 26 to 61 percent less than on the segments of the turnpike without separate roadways. He cautions that more detailed evaluation of the accident records and corresponding roadway conditions is needed to determine how much of the difference is attributable to the separation of vehicles and how much is attributable to other factors such as fewer lanes and higher levels of congestion on the non-separated portions. He closes with “the data clearly indicate that accident rates are lower in the areas with the dual-dual roadways (43).”

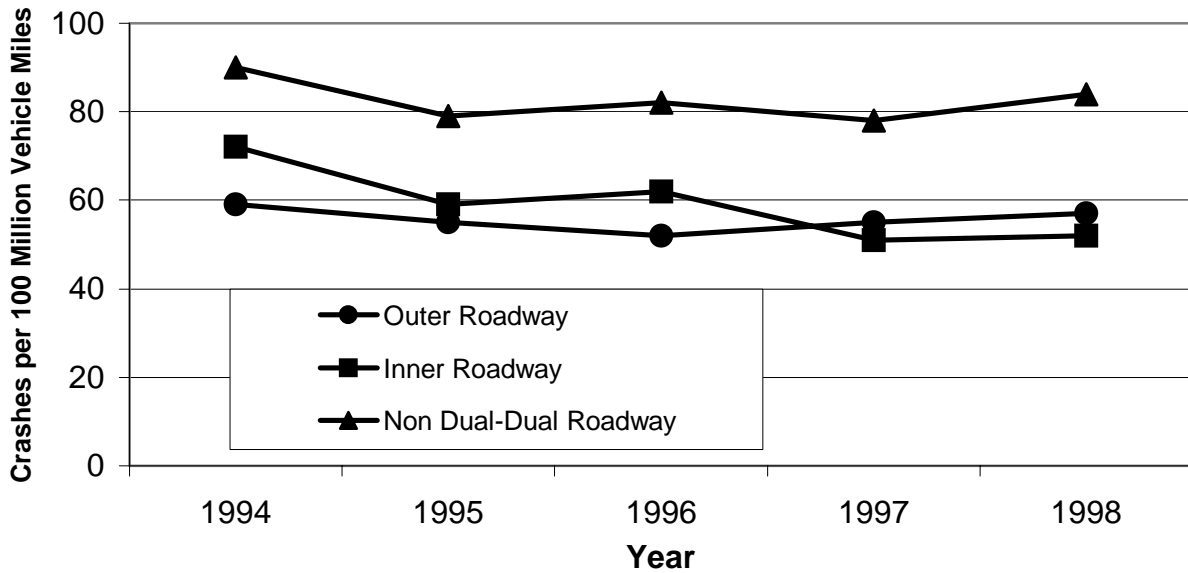


Figure 4-28. Crash Rates on New Jersey Turnpike (43).

The dual-dual portion of the New Jersey Turnpike clearly demonstrates the operational and safety benefits of separating vehicle modes. Having the entrance to a HOV or passenger-car exclusive facility located in the center of a corridor without a dedicated ramp requires vehicles to weave across each of the general-purpose lanes. The direct access to each barrel provided on the New Jersey Turnpike eliminates this weaving maneuver (which promotes a safer and more operationally efficient system). Maintaining similar geometric criteria for both barrels also provides greater flexibility in moving traffic between the barrels as needed for incidents and maintenance. Douglas' (43) finding that the dual-dual portion has a lower crash rate supports separating trucks and passenger cars.

SEATTLE

The core of the Seattle HOV lane system is composed of approximately 50 mi [80.5 km] of interlocking HOV lanes spanning several major roads including I-5, I-405, I-90, SR 67, and SR 520. The first successful HOV project—known as the Blue Streak—began in the 1960s and involved the operation of eight different transit routes using reversible express median lanes along I-5 in conjunction with downtown ramps. Later, HOV expansion projects brought HOV lanes to parts of I-90; I-5 North reversible lanes were extended; concurrent-flow HOV treatments were used on I-5, I-405, and SR 67; and the SR 520 HOV operation grew as well, expanding its hours and eligibility requirements to meet increased demand.

These systems enjoyed much success in safety ratings, efficiency, and public use, and they continued to rise in popularity and size throughout the 1980s. In 1992, a 7.5-mi [12.1 km] segment of I-90, which had been experiencing lane balance problems, was reconstructed with a combination of lane conversion and new lane construction to create new eastbound and westbound HOV lanes.

The system as it is today utilizes advance dynamic signing and varied occupancy signs. It includes an express lane flyover connection between I-405 and I-90 and both concurrent and barrier-separated lanes. Figures 4-29 to 4-39 provide examples of HOV overhead and posted signs and the cross sections used on the facilities.



Figure 4-29. Ramps to Express Lanes on I-5 Northbound (NB), Seattle.



Figure 4-30. I-5 Express Lanes.



Figure 4-31. Signing for Concurrent Lane on I-5 NB.



Figure 4-32. I-405 NB.



Figure 4-33. I-405 HOV Flyover.



Figure 4-34. I-90 Westbound (WB).



Figure 4-35. Close-Up of Sign on I-90 WB.



Figure 4-36. I-90 WB at Mercer Island.



Figure 4-37. I-90 Westbound at Mercer Island – Barrier Separated.



Figure 4-38. HOV Sign on SR 520 Southbound (SB).



Figure 4-39. Sign on Highway 520 at Montlake Interchange.

WASHINGTON, D.C.

Researchers visited several HOV systems in the Washington, D.C. area. The Shirley Highway (I-95/I-395) is the main road link between Washington, D.C., and Richmond, Virginia, and is a major access route for thousands of northern Virginia commuters. The highway contains 11 mi [17.7 km] of a two-lane, reversible HOV facility in the middle of the freeway, separated from general purpose traffic by concrete barriers (see [Figure 4-40](#)). Park-and-ride lots, direct-access ramps, and slip ramps are provided at strategic points along the corridor for HOV lane users. Dynamic message signs are used to indicate when the restricted lanes are open or closed (see [Figures 4-41](#) and [4-42](#)).



Figure 4-40. I-395 HOV Lane in Virginia.



Figure 4-41. Example of DMS Indicating Restricted Lanes are Closed on I-395.



Figure 4-42. Restricted Lanes Open Sign on I-395.

HOV lanes are also present on I-66. [Figure 4-43](#) shows a static sign on the local street system. [Figure 4-44](#) shows an advance DMS for the entrance. Buses and authorized vehicles are allowed on a short portion of I-66. [Figure 4-45](#) is an example of the markings and speed limit sign for this situation.



Figure 4-43. HOV Static Sign to East I-66.



Figure 4-44. HOV DMS to East I-66.



Figure 4-45. "Buses on Shoulder 25 MPH" Sign on East I-66.

Researchers also visited the HOV facility in the I-270 corridor. I-270, called the “Technology Corridor,” is the location of the first HOV lanes built in Maryland, and it is a critical link between the Washington, D.C. metropolitan area, western and central Maryland, and beyond. The HOV facility on I-270 was opened in 1993 along the northbound East Spur and was expanded as construction continued until 1996. It is a concurrent-flow buffer-separated system designated for motorcyclists and vehicles carrying two or more people. Figures 4-46 to 4-50 provide examples of signs and pavement markings used on I-270. It includes a flyover ramp network that allows HOV lanes to merge with the Capital Beltway from the left. Figures 4-51 to 4-53 show an example of the crossover barrier openings used on I-270.



Figure 4-46. Example of Express Lane and HOV Signs on I-270.



Figure 4-47. Example of Overhead Signing for HOV and Freeway Split on I-270.



Figure 4-48. Example of Roadside Sign on I-270.



Figure 4-49. Example of Overhead HOV Sign on I-270.



Figure 4-50. Example of HOV Pavement Markings and Overhead Sign on I-270.



Figure 4-51. Barrier Openings on I-270.



Figure 4-52. Close-Up of Barrier Openings on I-270.



Figure 4-53. Close-Up of Sign at Barrier Opening on I-270.

KENNEDY EXPRESSWAY IN CHICAGO

The express lanes on the Kennedy Expressway (I-90/I-94) in Chicago have a reversible two-lane cross section. [Figure 4-54](#) is an example of the cross section for the facility. Photos showing the cross section of the entrance ramps are shown in [Figures 4-55](#) and [4-56](#). [Figures 4-57](#) and [4-58](#) are examples of the overhead signs located prior to the lanes.

Safety features used include several types of advance signing, swing gates which rotate out of concrete barrier walls to direct traffic away from reversible lane entry ramps (see examples in [Figures 4-58](#) to [4-60](#)), and surveillance cameras which view the dynamic elements of the system (i.e., DMS, swing gates, etc.) and confirm that they are appropriately configured prior to switching traffic direction. Also, several wire mesh restraining assemblies have been placed at the entry ramps of the reversible lanes. These were designed to prevent head-on collisions by safely stopping errant vehicles from entering the reversible lane in the wrong direction.



Figure 4-54. Typical Cross Section for Two-Lane Express Lane Facility.



Figure 4-55. Typical Cross Section for Ramp.



Figure 4-56. Typical Cross Section for Ramp.



Figure 4-57. Example of Overhead Sign.



Figure 4-58. Overhead Sign and Gates Indicating that the Express Lanes are Closed.



Figure 4-59. Example of Gate Placement When Entrance Ramp is Closed.



Figure 4-60. Example of Gate Placement When Entrance Ramp is Open.

CHAPTER 5

SIMULATION OF RAMP SPACING AND PERCENT WEAVING

Researchers used simulation to obtain an appreciation of the effects of ramp spacing on freeway operations. A previous effort (Task 5) within TxDOT Project 0-4160 (44) focused on the impact of managed lane access and egress weaving behavior for a single pair of ramps. Simulation of several ramp pairs is needed to identify the impact on a corridor of vehicles consistently weaving across free lanes to access or egress a managed lane facility.

MODEL SELECTION

The VISSIM microscopic traffic simulation model (45) was selected for the ramp spacing and weaving simulation conducted for Task 10 of TxDOT Project 0-4160. Given the previous Task 5 simulation that used the model, VISSIM was the logical choice for the simulation effort performed in Task 10. The ability to easily create appropriate vehicle mix within the modeled traffic stream and the flexibility afforded by the model in generating vehicle routing behavior and origin-destination pairs to support vehicle routing were of particular use to researchers. The fact that the VISSIM input file is an ASCII text file was also beneficial in that researchers were able to use a rapidly created simple text editor and make changes to the more than 100 input files used in the Task 10 simulation effort.

The version of VISSIM used in the Task 10 simulation study, version 3.60, is an update to the version used in Task 5, which was version 3.50. Though the user interface and data file input details are virtually identical between the two versions, some details of the simulation software were changed with respect to vehicle behavior within a modeled traffic stream. To ensure that version 3.60 was calibrated for the Task 10 simulations, a procedure identical to that used in Task 5 (44) was employed. Both maximum throughput volume at capacity and example weaving problems from the Highway Capacity Manual (46) (HCM) were studied. As measure of performance differences between VISSIM and the HCM were less than 5 percent, the model was validated for the Task 10 study. It should be noted that the updates to VISSIM between versions 3.50 and 3.60 led to a more robust modeling of intra-model vehicle interaction. Accordingly, calibration of the updated model was a less time-consuming/iterative process.

EXPERIMENT DESIGN

The simulation performed as part of TxDOT Project 0-4160 Task 10 had the following goals:

- Quantify the effects of ramp spacing on freeway operations. Consider both the spacing of the ramps to the free lanes and the ramps between free lanes and managed lanes.
- Continue the investigation of when to consider a direct ramp between the managed lanes and a generator or surface street system.

The goals of the Task 10 modeling were to quantify the effects of ramp spacing on freeway operations and to investigate when to consider a direct ramp between managed lanes and the

surface street system. Speed was the primary measure of effectiveness used to evaluate the effect of the different ramp spacing, volume levels, and weaving percentages found within the simulation scenarios.

Geometric Layout

Common to all of the simulations performed under Task 10 were the basic geometric components of the freeway and the traffic stream routing details related to background (i.e., non-managed lane weaving) traffic. Researchers used a single-direction freeway cross section with four freeway mainlanes and two medially located managed lanes. Ramps between the surface street network and the freeway lanes were called the “freeway ramps.” Ramps between the freeway lanes and the managed lanes were called the “managed lane ramps.” The freeway entrance and exit ramps (i.e., between the surface streets and the freeway lanes) were located to the right of the freeway traffic flow. The managed lane ramps (i.e., those ramps that moved traffic from the freeway lanes to the managed lanes or that provided access from the managed lanes to the free lanes) were located to the left of the freeway traffic flow and to the right of the managed lane traffic flow. [Figure 5-1](#) shows the basic geometric outline used for the simulation.

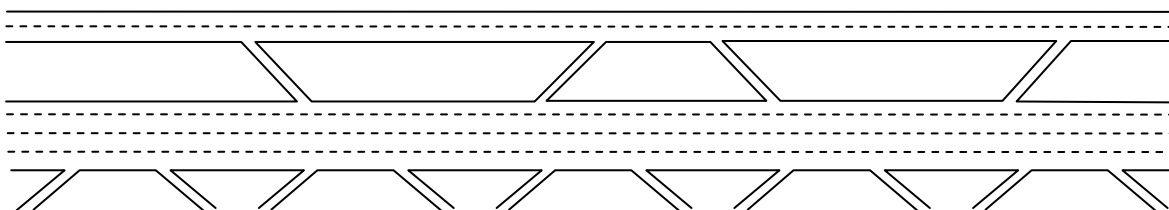


Figure 5-1. Basic Geometry for Freeway and Managed Lanes and Ramps.

Managed lane access and/or egress ramps were located at a spacing of two times the spacing of the freeway entrance/exit ramp pair. Weaving from a managed lane exit ramp was restricted so that 50 percent of the traffic was destined for the next downstream freeway exit ramp and 50 percent was destined for the second downstream freeway exit ramp. Based on guidelines in *A Policy on Geometric Design of Highways and Streets* (39), a freeway auxiliary lane between an entrance ramp and the downstream exit ramp was provided when the ramps were within 1500 ft [457.5 m] of one another.

The total length of the simulation was variable, as it was necessary to vary the intra-ramp spacing as an experimental variable, and it was desirable to have the same number of ramps simulated in each of the 64 simulation scenarios. As there were five entrance ramps with downstream exit ramp pairs, a total of 10 freeway mainlane ramps were simulated. And, since the spacing of managed lane ramps was twice the spacing of mainlane ramps, there were four managed lane ramps (as two pairs of entrance ramp with downstream exit ramp). To clarify the geometric configuration of the simulation, refer to [Figure 5-1](#).

Variables

The key experimental variables for the simulation scenarios for Task 10 were:

- ramp spacing,
- initial freeway per lane volume, and
- percent of freeway entrance ramp traffic weaving to managed lane facility.

Ramp spacing of 1000, 2500, 4000, and 5500 ft [305, 762.5, 1220, and 1677.5 m] was used. Freeway initial volumes of 1250, 1500, 1750, and 2000 vehicles per hour per lane (veh/hr/ln) were also used. Finally, the percentage of freeway entrance ramp traffic that desired to maneuver to the next managed lane access point was varied between 0, 10, 20, and 30 percent of the traffic on the (source) freeway entrance ramp. The 0 percent weaving scenario provided a baseline condition of how the freeway would operate without the managed lane facility. With three unique experimental variables at four levels each, a total of 64 unique simulation scenarios were created. Input files with combinations of the key experimental variables were developed for each scenario. Each simulation scenario was modeled three times so that final results would be an average of multiple simulation runs with random variation. A total of 192 unique simulation input files were developed for use with the VISSIM program.

Background Traffic Conditions

In addition to the basic geometric elements and the variation of the primary variables in the Task 10 VISSIM simulations, a variety of “background” geometric and traffic volume and routing details were extant within the model. A heavy vehicle percentage of 10 percent was used for the traffic streams within the model.

In terms of the managed lanes, researchers restricted volume to 75 percent of the volume per lane found on the adjacent freeway mainlanes. This restriction was used to ensure that speed performance of the managed lanes was always near 50 mph [80.5 km/h] (a goal set forth by the Advisory Committee of the project sponsor, the Texas Department of Transportation). In some cases, the performance on the managed lanes would drop due to congestion on the freeway, preventing vehicles from exiting the managed lanes. The queue that formed on the managed lane exit ramp backed onto the managed lane facility.

Volume on the freeway entrance ramps was 70 percent of the entering initial volume of a freeway mainlane, divided by the number of entrance ramps per mile. In the 5500-ft [1677.5-m] ramp spacing situation, one entrance ramp would exist per mile, and it would have 1400 veh/hr when the initial freeway volume was 2000 veh/hr. For the 2500-ft ramp [762.5 m] spacing, two entrance ramps would occur within a mile, and each ramp would have a volume of 700 veh/hr for the 2000 veh/hr simulation runs. Structuring the simulation scenarios in this way, there is a constant freeway corridor volume for each level of initial freeway volume, regardless of the ramp spacing. This experimental design approach was selected so that it would be possible to compare operation results for different ramp spacing at a uniform corridor volume level.

Volume on the normal freeway exit ramps is 60 percent of the initial freeway per hour per lane volume, again divided by the number of exit ramps per mile to create a constant corridor volume.

Note that since freeway mainlane volumes are at fixed levels by experiment design and more traffic enters the freeway at each entrance ramp than exits at each downstream exit ramp, the freeway volume increases as you proceed further downstream. This condition was created to simulate a peak period condition in the direction of higher flow.

The managed lane entrance ramps had volumes composed of only the percentage of freeway entrance ramp traffic desiring to access the managed lanes. Stated in another manner, 0, 10, 20, or 30 percent of the freeway entrance ramp traffic was the managed lane entrance ramp volume, depending on which scenario was under review. Managed lane exit ramp volume was set to equal the managed lane entrance volume so that operations within the managed lanes were stable and of high quality (i.e., nominally, 50 mph [80.5 km/h] operation or better). Vehicles exiting the managed lanes were routed such that 50 percent of the volume weaved to the nearest downstream freeway exit ramp and 50 percent of the traffic weaved to the following downstream freeway exit ramp.

VISSIM OUTPUT

Users of the VISSIM simulation model have the ability to specify the type, quantity, and aggregation of simulation output desired. For the Task 10 simulation effort, speed was the primary measure of performance within the system. Accordingly, travel time measurement data collection markers were configured within VISSIM so that speed data could be collected for the freeway mainlanes, the managed lanes, and the vehicles making entrance and exit maneuvers to access or egress the managed lanes. So that performance information included variation in speed along the freeway and managed lanes, space mean speed was captured between each entrance/exit ramp and the next downstream entrance/exit ramp.

To capture the performance data for weaving vehicles wishing to access or exit the managed lanes, data collection points were added where each freeway or managed lane entrance or exit ramp joined with the freeway mainlanes. [Figure 5-2](#) provides a reference diagram for speed data collection points within the model.

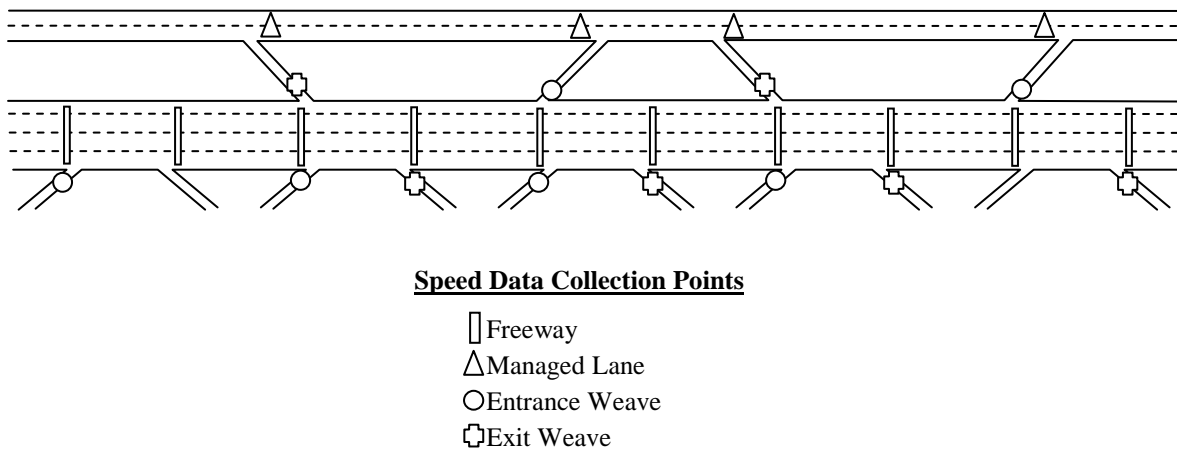


Figure 5-2. Freeway and Managed Lane Speed Data Collection Points.

In addition to the speed data gathered during each simulation, VISSIM automatically created an error log file that contained information about the unserved demand within the network and indicated where that unserved demand was located. For instance, higher volume simulations had the potential to exhibit freeway congestion, which could in turn cause standing queues on the freeway mainlanes and/or the freeway entrance ramps. The VISSIM error file could, in this sense, document congestion in that it indicated how many vehicles were unable to enter the network from the mainlanes or ramps during the simulation hour.

FINDINGS

Freeway Speeds

Figure 5-3 shows the average freeway speeds by the different ramp spacing levels when no vehicles are weaving to and from the managed lanes (also called the 0 percent weaving scenario). It provides an appreciation of the freeway performance without any effects from vehicles weaving across all the freeway lanes to enter or exit the managed lanes. When the initial freeway volumes are 1250 and 1500 veh/hr/ln, the average freeway lane speeds are about 57 mph regardless of the ramp spacing. At an initial freeway volume of 1750 veh/hr/ln, more variability is shown in the average freeway speeds (ranging between 52 and 56 mph [83.7 and 90.2 km/h]); however, speeds are still high (above 45 mph [72.5 km/h]). A 2000 veh/hr/ln initial freeway volume results in a 44 mph [70.8 km/h] average freeway speed when ramp spacing is at 1000 ft. In the simulation, ramp spacings of 2500 ft [762.5 m] and above had 50 mph [80.5 km/h] or higher speeds. Ramp spacing only affected average freeway speeds when the initial freeway volumes were very high (2000 veh/hr/ln) and ramp spacing was at the lowest value used in the simulation (1000 ft [305 m]).

Researchers discovered similar findings to the 0 percent weaving scenario when 10 percent of the entering volume weaved across the freeway lanes to enter the managed lanes. Speeds were high for the 1250 and 1500 veh/hr/ln initial freeway volume situations, became more variable for the 1750 veh/hr/ln situation, and experienced a less than desirable value (i.e., less than 45 mph [72.5 km/h] operations) for low ramp spacing (1000 ft [305 m]) and high volumes (2000 veh/hr/ln).

Freeway speeds for the 20 percent weaving scenario had similar findings as the 10 percent weaving scenario; however, the point when speeds fall below the undesirable level occurs earlier. In the previous scenarios, the managed lane speeds were approximately 58 mph [93.4 km/h]. When 20 percent of the entering volume is weaving into the managed lane entrance ramps (and the related exiting volume is leaving the managed lanes to weave to a freeway exit), there are situations when the weaving vehicles influence the managed lane performance.

Ramp spacing and entering volume levels are directly related. When ramp spacing increases, entering volume per ramp also increases so that the number of vehicles attempting to enter the corridor remains the same. An objective of the simulation was to maintain constant corridor volumes with the assumption that even though ramps have a greater spacing, the same number of vehicles want to use the facility. A side effect to the approach is in respect to bottlenecks

forming at the higher volume, but greater spaced ramps. Varying where the managed lane exit ramp was located with respect to the freeway exit ramp was beyond the scope of this simulation effort; however, it has been demonstrated that the ramp's relative location can have a pronounced impact on the operations of both the managed lanes and the freeway. With a 1750 veh/hr/ln initial freeway volume, 20 percent weaving, a ramp spacing of 5500 ft [1677.5 m], and a freeway entrance ramp volume of 2552 veh/hr; the **average** freeway speed on the corridor is an acceptable 51 mph [82.1 m]. The **lowest** freeway speed recorded at a point in that corridor, however, was only 19 mph [30.6 km/h]. The managed lane operations were also impacted for this scenario. The average speed was 53 mph [85.3 km/h] with a low speed of 41 mph [66 km/h] recorded. Figure 5-4 shows an illustration of the bottleneck that occurred at a managed lane exit as vehicles are attempting to weave from the managed lane to the freeway ramp exit.

The results for a 2000 veh/hr/ln initial freeway volume were similar to the 1750 veh/hr/ln findings, although they had an even lower managed lane speed recorded.

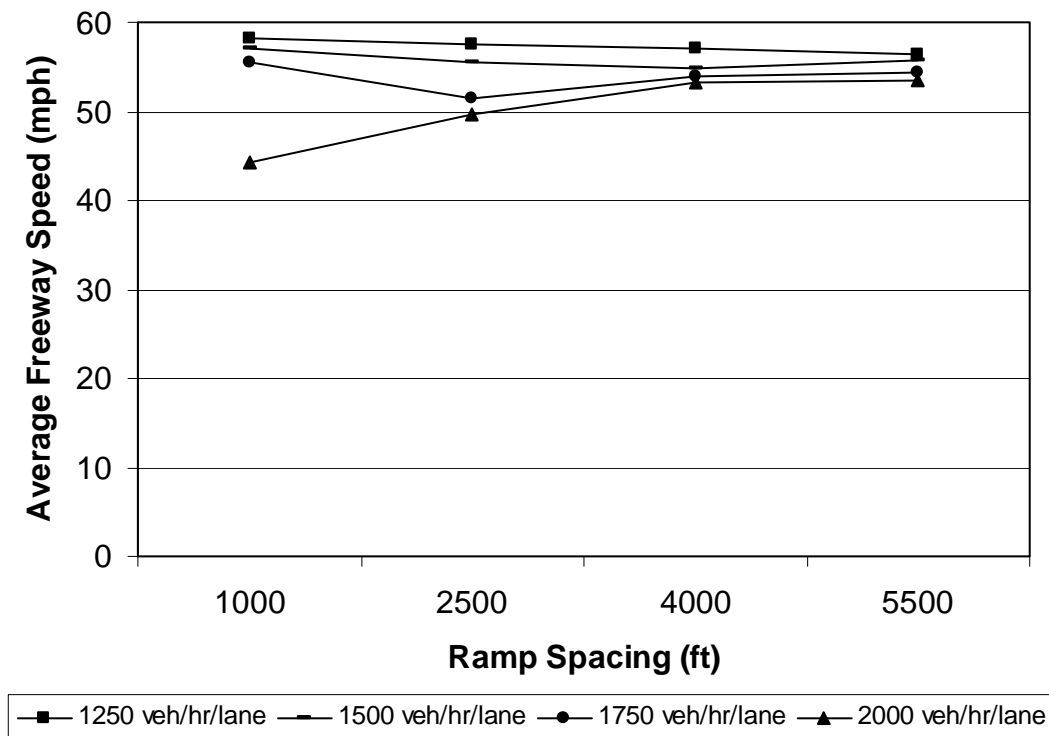


Figure 5-3. Average Freeway Speed for Four Entering Freeway Volumes and 0 Percent Weaving.

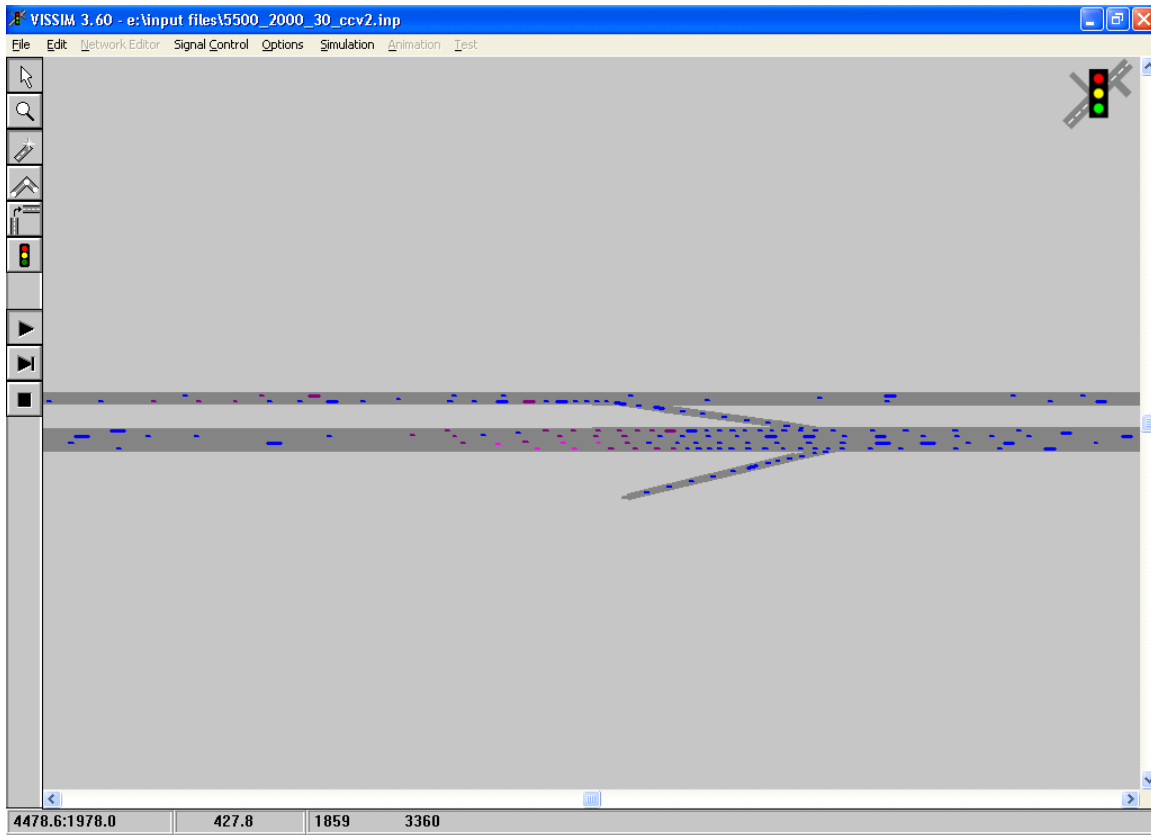


Figure 5-4. Screen Capture Showing Congestion at Managed Lane Exit Ramp and Freeway Entrance Ramp.

Figure 5-5 shows the findings at the 30 percent weaving scenarios. Only for the 1250 and 1500 veh/hr/ln initial freeway volumes (and 938 and 1125 veh/hr/ln initial managed lane volumes) are average freeway speeds and average managed lane speeds consistently at or above 48 mph [77.3 kmh]. For all other combinations, the speeds on both the freeway and the managed lanes show poorer performance than when lower percentages of vehicles are weaving across the freeway lanes. For the 1750 veh/hr/ln initial freeway volume scenario, average freeway speeds are below 45 mph except for the 5500-ft [1677.5 km/h] ramp spacing scenario (see Figure 5-5). Given the much higher volumes entering at the ramp spacing, a lower speed was anticipated. A review of the simulation runs showed that a high percentage of vehicles were not being serviced for this scenario.

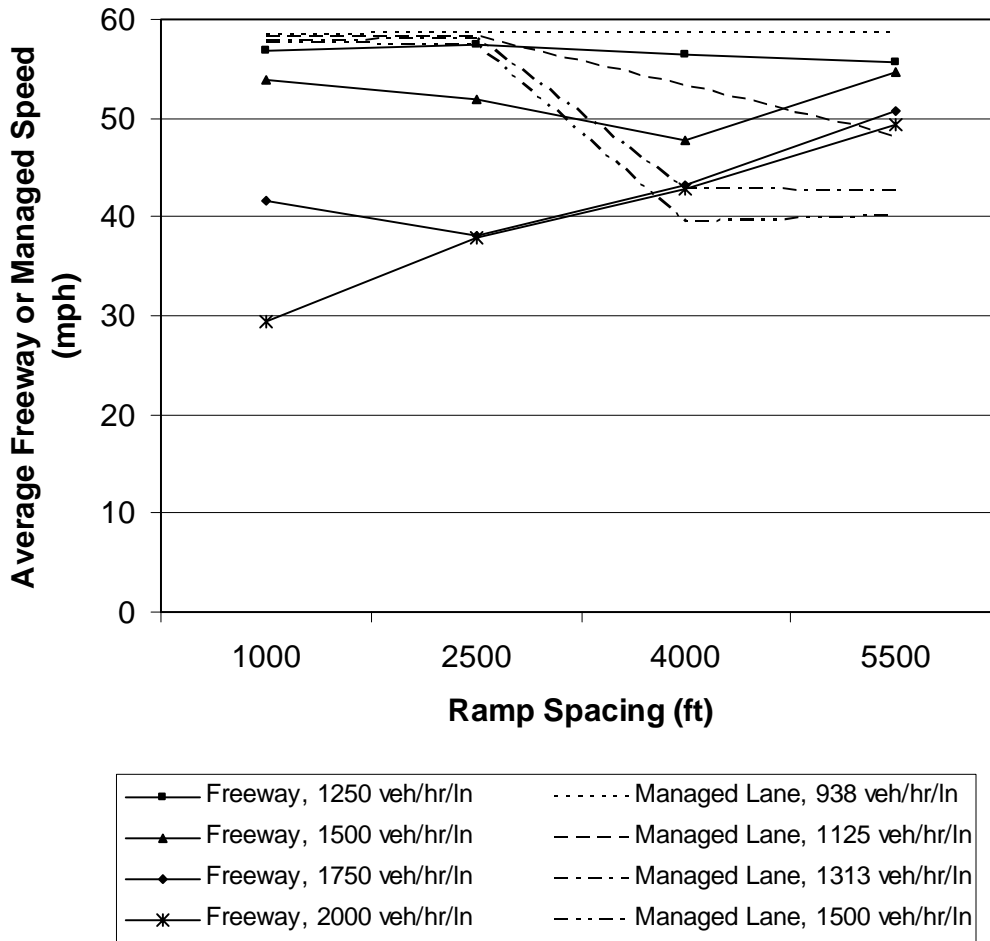


Figure 5-5. Average Freeway and Managed Lane Speeds at 30 Percent Weaving.

Unserved Demand

VISSIM reports any unserved demand on input links in the simulation network for each model run in that run's error file. Within the error file, the input link is referenced along with an indication that the flow rate input on the link is greater than the flow rate that the link could process (either because upstream congestion limited input flow rate or because the specified input rate was unrealistic given prevailing vehicle and traffic flow fundamentals inherent within the VISSIM model). The error file indicates the number of vehicles that could not be served during the simulation time period. In the case of the current simulation effort, this situation translated into (peak) hourly unserved volume for each link within each simulation run that could be compared against the desired input flow on that link (freeway beginning/entry point or freeway entrance ramp). From these values, it was possible to calculate a "percent unserved" statistic to indicate what relative quantity of desired, specified input entrance ramp flow (demand) was not able to enter the simulated network due to congestion.

The significance of the “percent unserved” statistic becomes apparent when the overall scope and intent of the modeling effort is considered. As it was an objective to relate network performance (i.e., speed) to (variable) ramp spacing, volume levels, and weaving levels, it was necessary to know where the desired input volume could not enter the network and, in following, not be able to impact network performance. Essentially, an awareness of when and where congestion was restricting entrance ramp volume input flow prevented erroneous conclusions being drawn under higher volume conditions, where specified input ramp flow could not be directly linked to freeway performance impacts (i.e., the desired input volume could not enter the network in the first place).

Figure 5-6 documents percent unserved volume for this simulation effort. When interpreting Figure 5-6, note that the simulation was designed around maintaining a constant traffic volume within the corridor for each initial freeway volume level (note that four different initial freeway volume **levels** were used in the experiment). A direct result of the constant corridor constraint was that ramp volume increased with ramp spacing within each overall level of freeway volume. A noteworthy detail concerning the relationship between weaving percent and percent unserved is that weaving level is directly related to percent unserved volume, which can (logically) be interpreted to mean that congestion level increases with increasing weaving percentage.

Figures 5-7 to 5-10 document performance trends resulting from the freeway performance of percent unserved traffic. Note that in each case, increases in entering volume are related to a decrease in performance until an inflection point is reached. The significance of the inflection point is related to the unserved demand since the discontinuity in the performance trend is caused by congestion, as is the presence of unserved demand. As congestion “meters” incoming flow, downstream speed performance increases on the freeway, creating the (erroneous) impression that speed improves with increasing entering volume. These results are shown so that the reader is able to identify the onset of congestion and assess the significance of its impact.

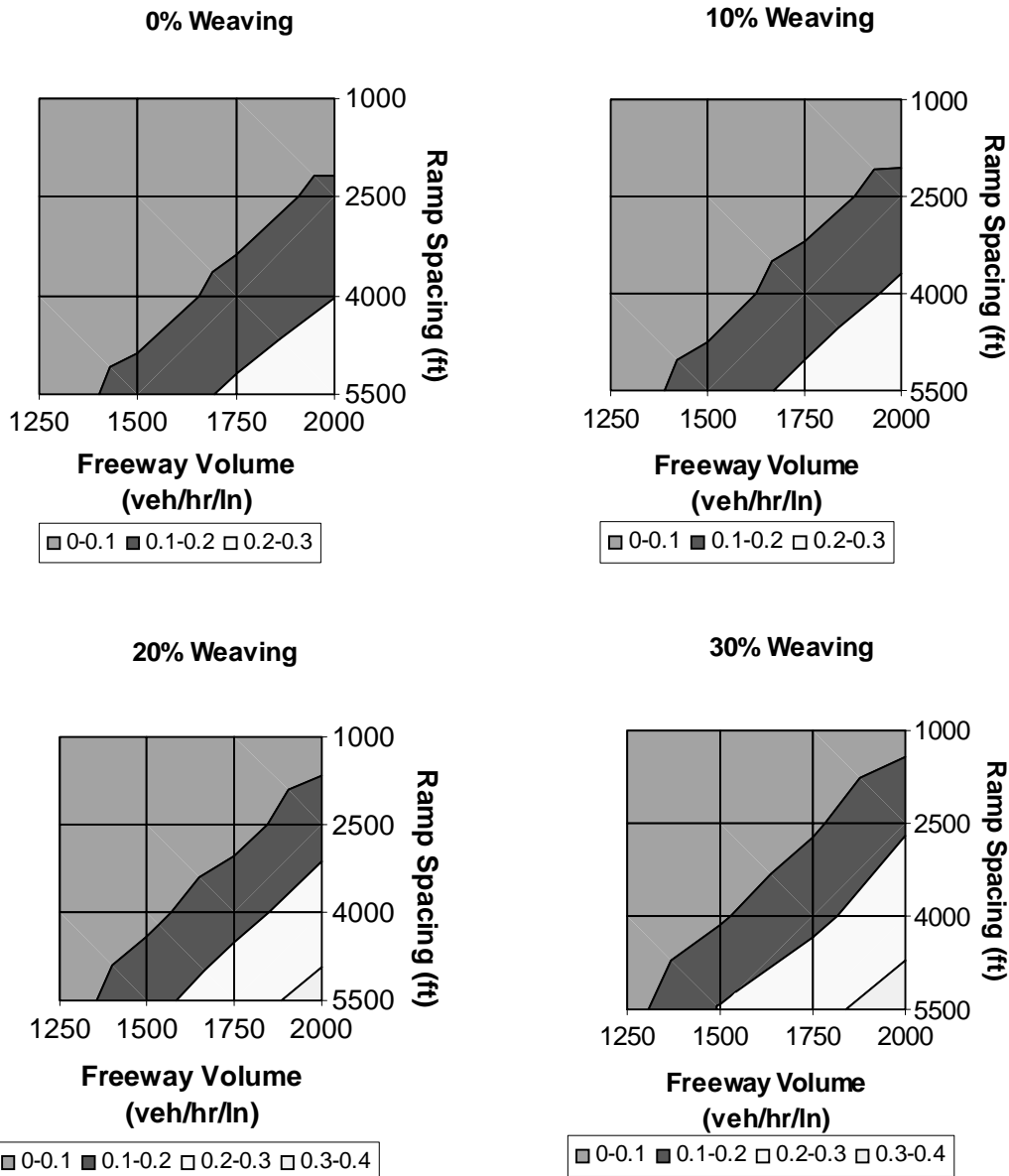


Figure 5-6. Unserved Volume by Freeway Volume and Ramp Spacing (Expressed in Decimals).

Speeds Influenced by Entrance Ramp Vehicles

Figure 5-7 shows the average freeway speed by entrance ramp volume. In each weaving level comparison, the average freeway speed dropped faster for the shorter ramp spacing (look especially at the almost vertical line representing the data for the 1000-ft [305 m] spacing scenario as compared to the other spacing scenarios). The slope of the curves in Figure 5-7 shows that operations are more sensitive to small increases in traffic volumes when ramp spacing is shorter. Figure 5-7 also shows that the speeds become lower as weaving percentages increase.

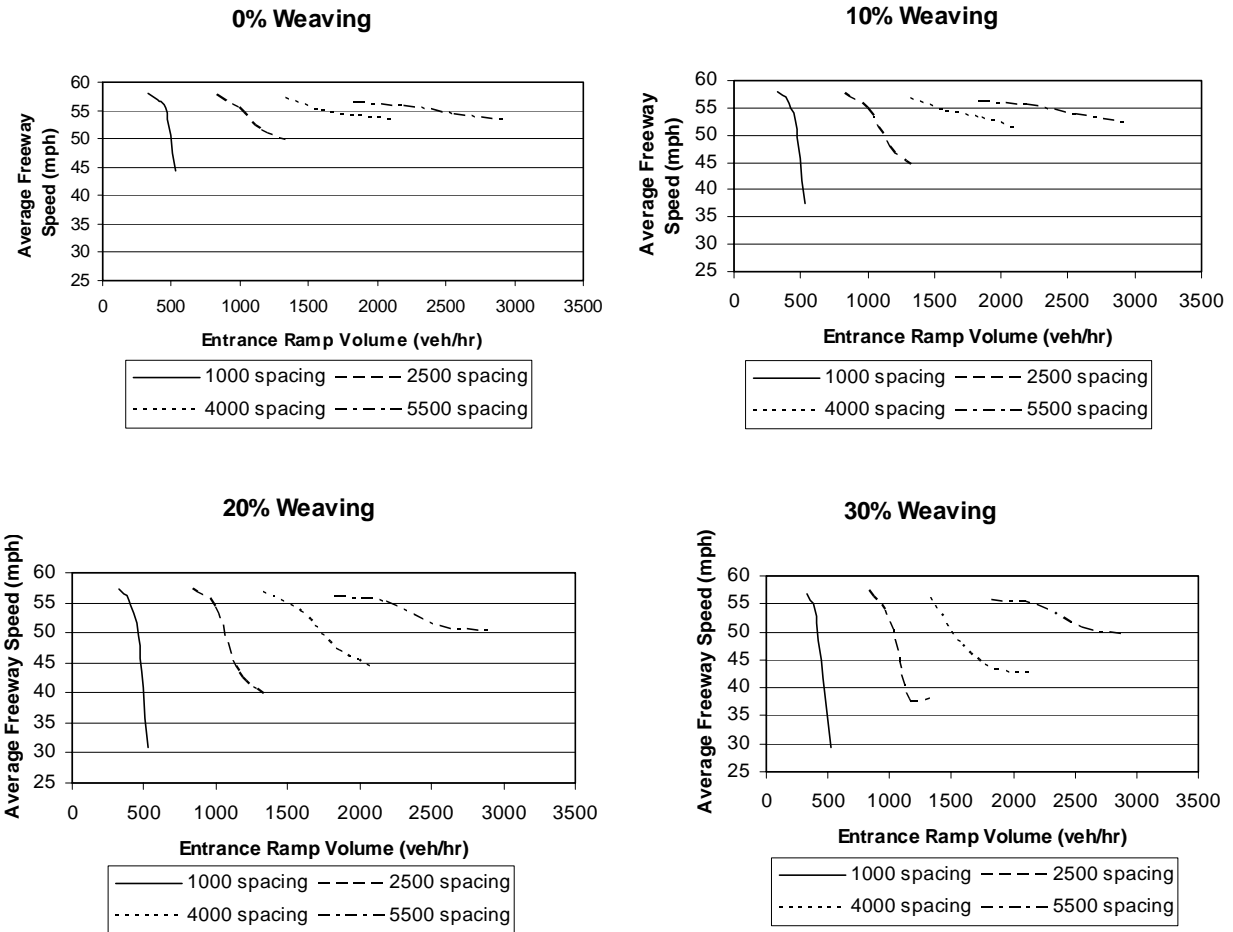
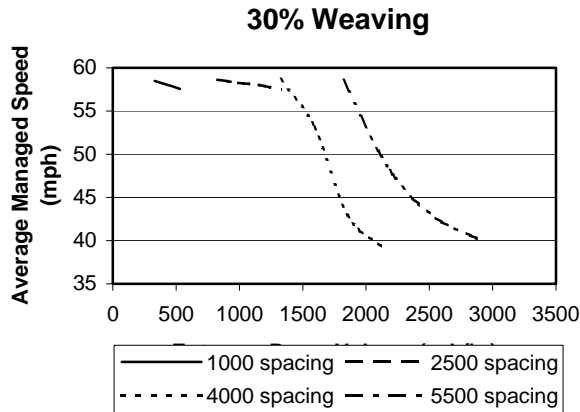
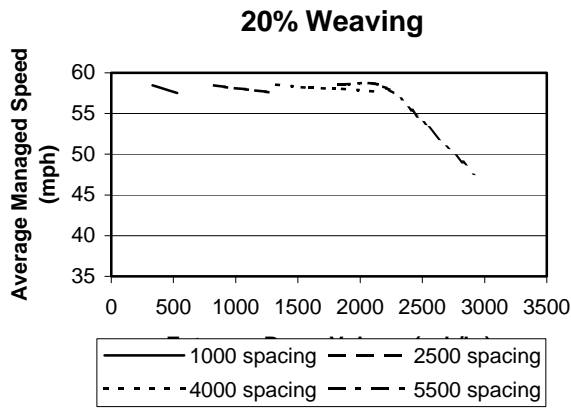
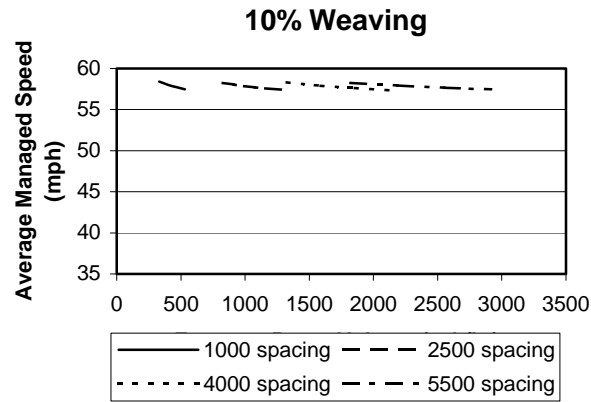


Figure 5-7. Average Freeway Speed vs. Entrance Ramp Volume.

Figure 5-8 shows similar findings when the speeds for the managed lanes are plotted against the volume on the freeway entrance ramp (of which a percentage is traveling across the freeway lanes and then onto the managed lane entrance ramp). At entrance ramp volumes of about 3000 with a 5500-ft [1677.5 m] ramp spacing and 20 percent weaving, the managed lanes operation drops to near 45 mph [72.5 km/h]. The speeds are below 45 mph [72.5 km/h] when 30 percent of the entrance ramp is weaving to the managed lanes and entrance ramp volumes are about 1850 veh/hr.

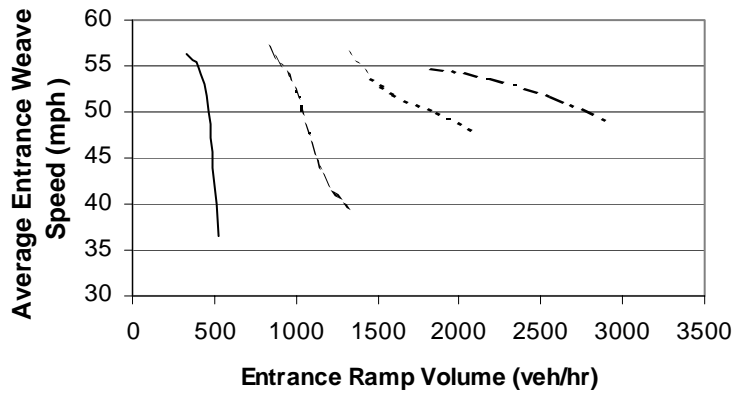
When the speed of the weaving vehicle is examined, the performance of the system is not as good (see Figure 5-9). Weaving speeds quickly drop below 45 mph [72.5 km/h] as entrance ramp volumes increase, and the rate of the drop is higher with increase in percent weaving. For a ramp spacing of 1000 ft [3305 km/h], the average entrance weave speed is at 45 mph [72.5 km/h] or less when entrance ramp volumes are about 450 veh/hr. For a 2500-ft [762.5 m] spacing, the ramp volumes of approximately 1100 are associated with average entrance weave speeds of 45 mph [72.5 km/h] or less.



1 mi = 1.61 km

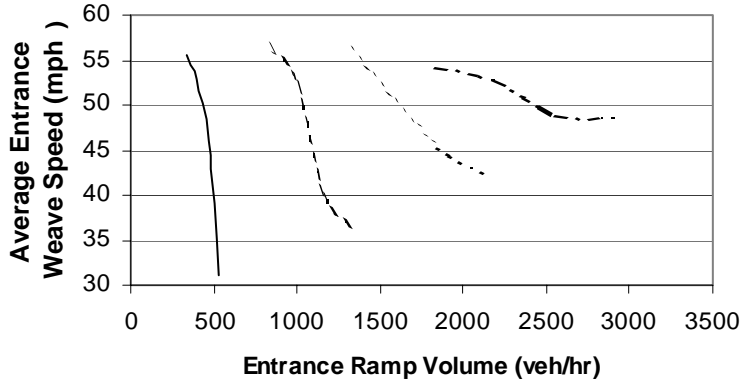
Figure 5-8. Average Managed Speed vs. Volume by Spacing.

10% Weaving



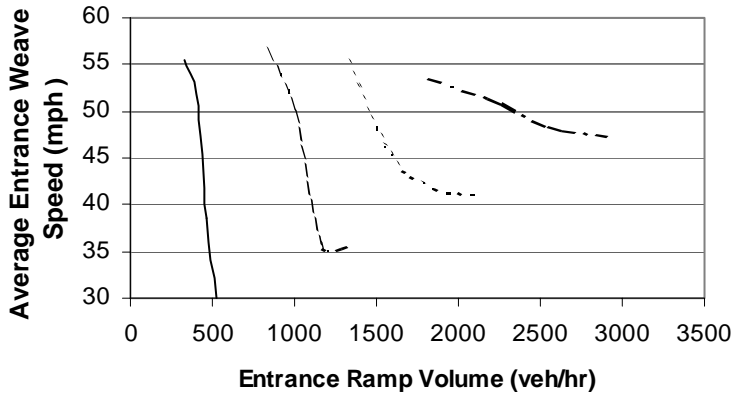
— 1000 spacing - - - - 2500 spacing ······ 4000 spacing - · - · - 5500 spacing

20% Weaving



— 1000 spacing - - - - 2500 spacing ······ 4000 spacing - · - · - 5500 spacing

30% Weaving



— 1000 spacing - - - - 2500 spacing ······ 4000 spacing - · - · - 5500 spacing

1 mi = 1.61 km

Figure 5-9. Entrance Weave Speed vs. Volume by Spacing.

As the percent unserved showed, the performance at the locations where different traffic streams merge can have a pronounced impact on the operations of the freeway. To obtain an appreciation for the effects of those locations on performance, the “merging volume” was determined as the combination of initial freeway vehicles per hour per lane volume plus the entrance ramp volume. The “merging volume” would reflect the number of vehicles in an hour attempting to merge at an entrance gore. [Figure 5-10](#) shows the results for the average freeway speeds. The findings from the merging volumes mirror the findings from the entrance ramp volumes. Performance is better for higher ramp spacing and at lower percent weaving volumes (see [Figure 5-10](#)). [Table 5-1](#) lists the minimum and average speeds for the different percent weaving levels used in the simulation.

Similar patterns exist for the plots of low freeway speed and low entrance weave speed, however, with speeds that are much lower than the average freeway speeds. [Figure 5-11](#) shows that the speeds on the freeway were measured as low as 13 mph [20.9 km/h] in some scenarios. Even just a small percent of traffic weaving across the freeway can create notable reductions in minimum speeds on the facility.

Table 5-1. Minimum and Average Freeway Speed for Different Percent Weaving Levels.

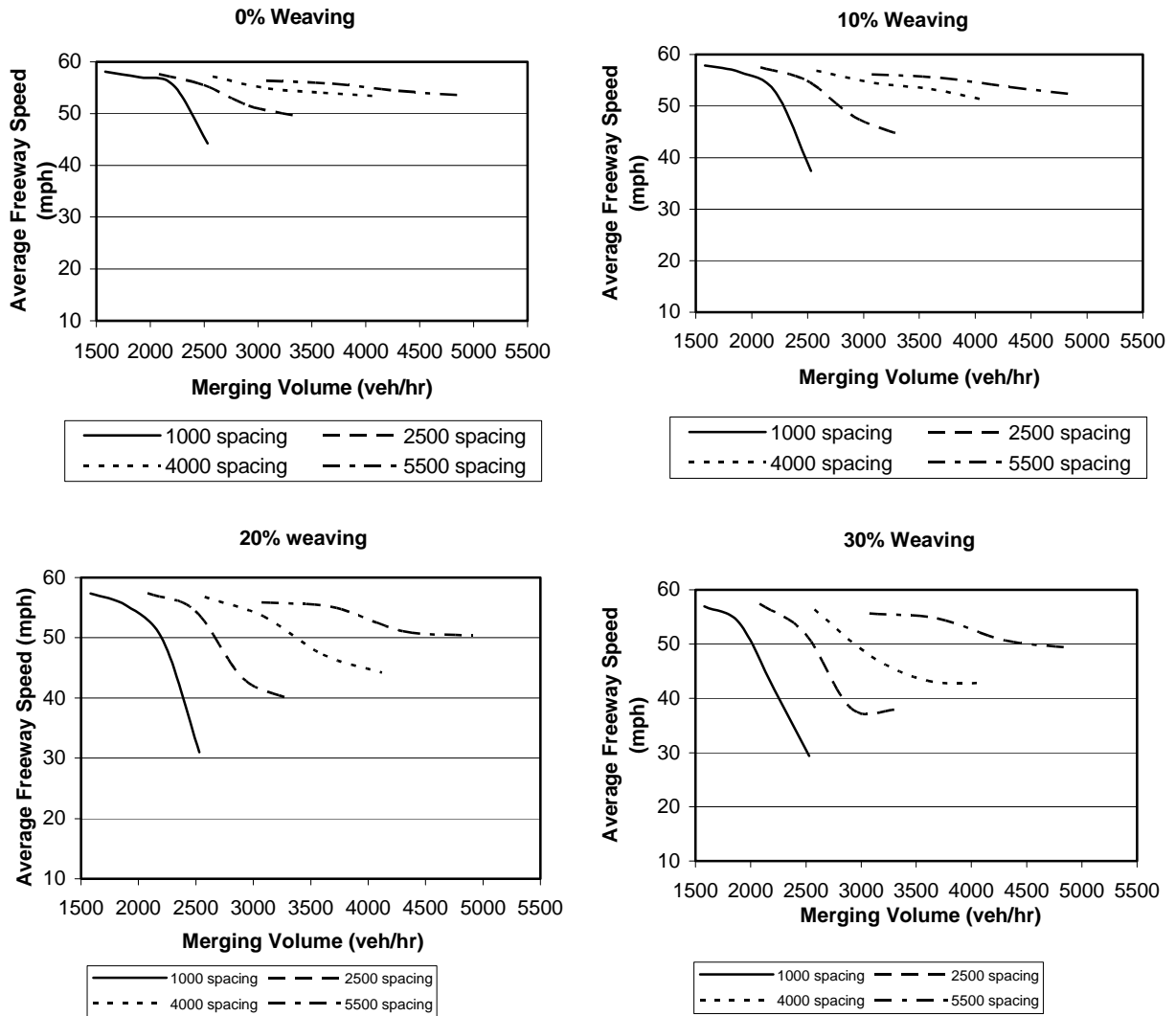
% Weaving	Minimum Speed mph [km/h]	Average Speed mph [km/h]
0	34 [54.7]	44 [70.8]
10	22 [35.4]	37 [59.6]
20	16 [25.8]	21 [33.8]
30	13 [20.9]	20 [32.2]

Weaving Vehicle Speeds

The number of vehicles attempting to weave across the four freeway lanes to enter the managed lanes can have a pronounced impact on the operations of the freeway. The number of vehicles weaving from one side of the freeway to the other side from each entrance ramp was determined as the percent weave multiplied by the entrance ramp volume. [Figure 5-12](#) shows the average and low freeway speeds for the different weaving volumes. The plots include all combinations of weaving percents, initial entering freeway volume levels, and ramp spacing. To provide an appreciation of those conditions when congestion is metering the amount of vehicles able to enter the freeway, the symbols on the plots are smaller when the unserved demand is in excess of 10 percent.

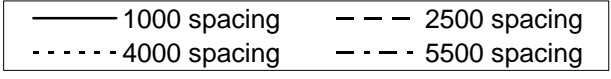
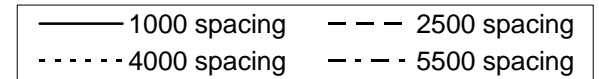
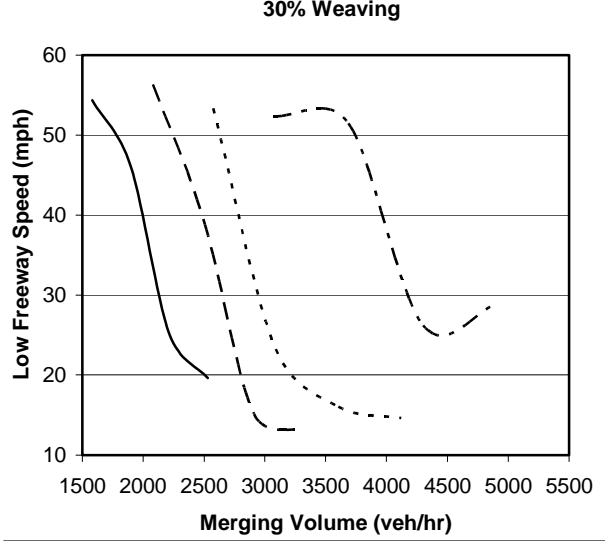
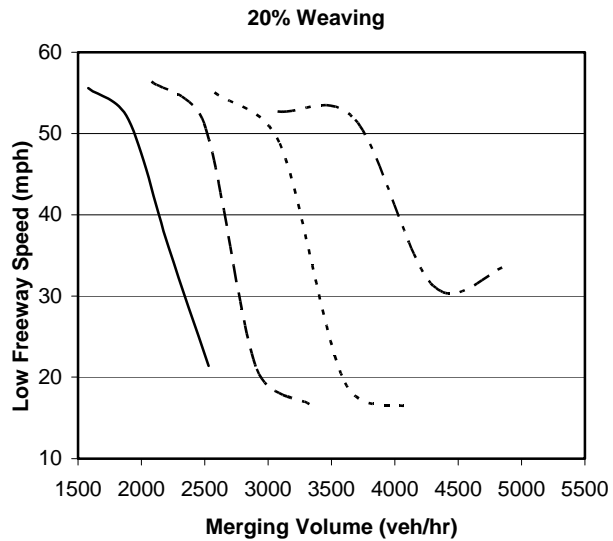
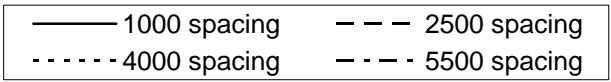
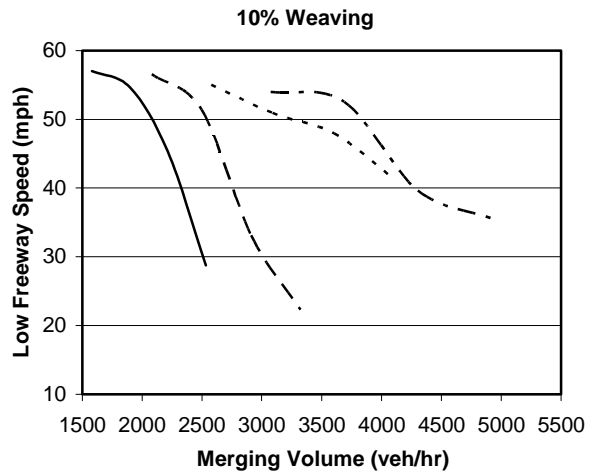
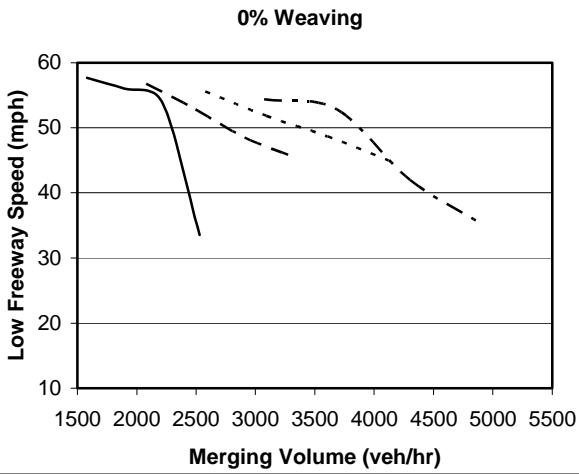
With the exception of short spacing in combination with high initial freeway volumes, the average freeway speeds recorded from the simulation runs are generally above 45 mph until approximately 500 vehicles per hour are attempting to weave across the freeway and enter the managed lanes. When the plot of the lowest freeway speed recorded is reviewed (see [Figure 5-12](#)), the point when less than desirable operations occur is at approximately 250 veh/hr. [Figure 5-13](#) shows the average and low entrance ramp speeds for the weaving volumes. Average entrance weave speeds below 45 mph [72.5 km/h] occur when approximately 300 veh/hr are

attempting to weave. Low entrance weave speeds below 45 mph [72.5 km/h] occur at lower volumes, at about 250 veh/hr.



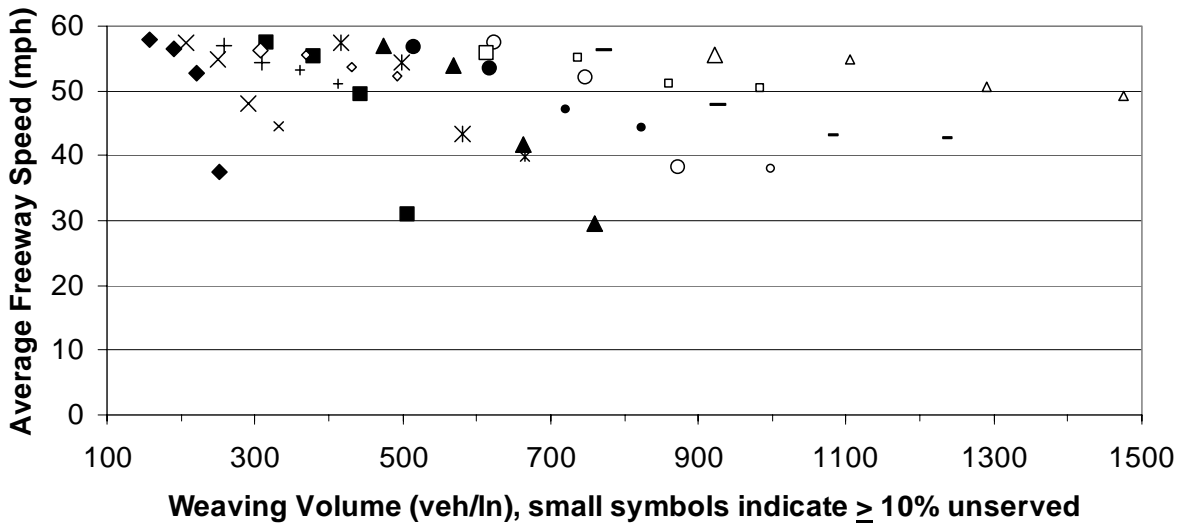
1 mi = 1.61 km

Figure 5-10. Average Freeway Speeds vs. Entering Volume.

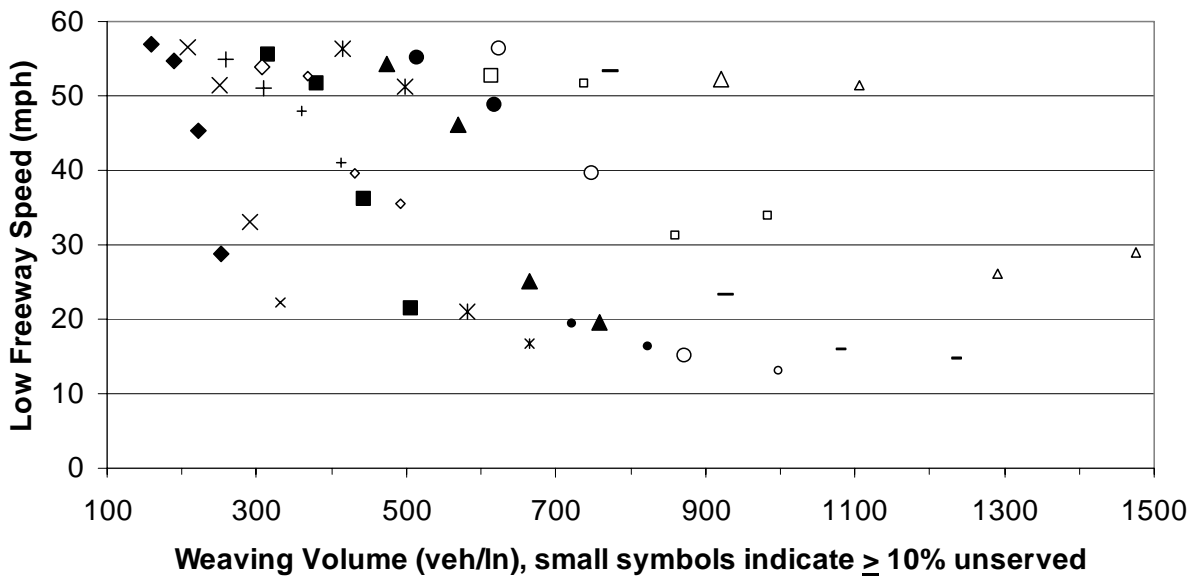


1 mi = 1.61 km

Figure 5-11. Low Freeway Speeds vs. Entering Volume.



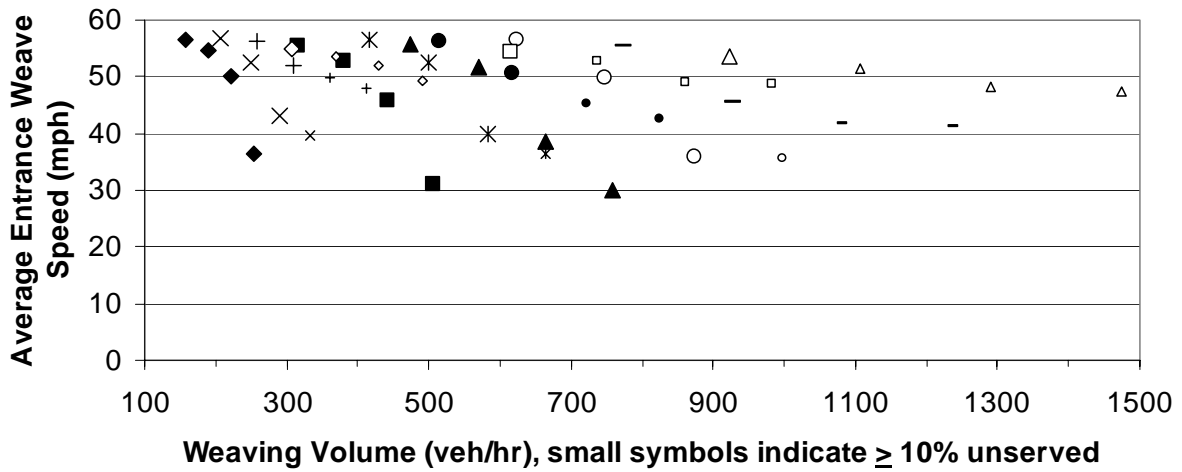
◆ 1000 spacing, 10% weaving	■ 1000 spacing, 20% weaving	▲ 1000 spacing, 30% weaving
× 2500 spacing, 10% weaving	✖ 2500 spacing, 20% weaving	○ 2500 spacing, 30% weaving
+ 4000 spacing, 10% weaving	● 4000 spacing, 20% weaving	— 4000 spacing, 30% weaving
◇ 5500 spacing, 10% weaving	□ 5500 spacing, 20% weaving	△ 5500 spacing, 30% weaving



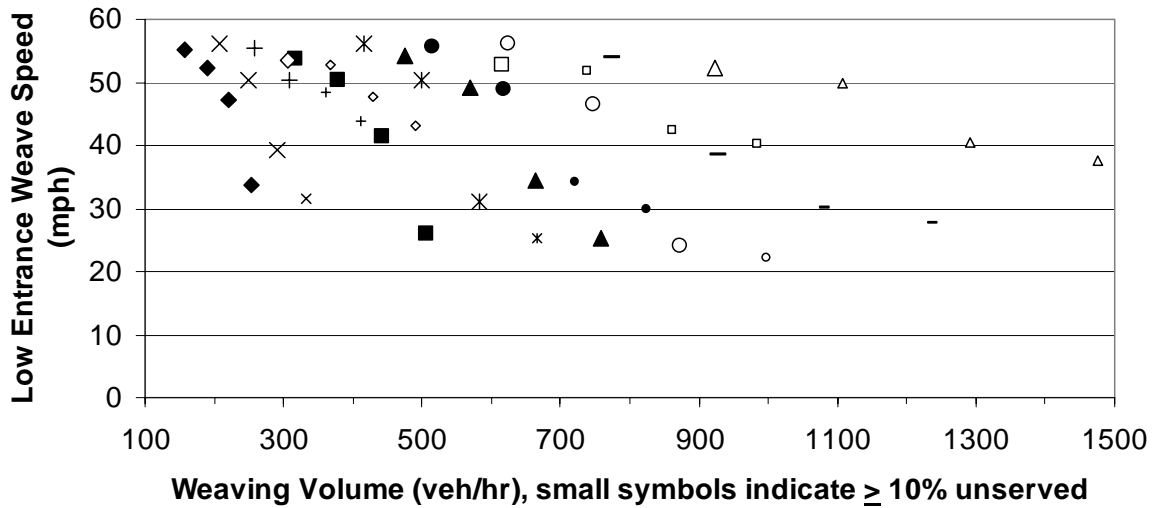
◆ 1000 spacing, 10% weaving	■ 1000 spacing, 20% weaving	▲ 1000 spacing, 30% weaving
× 2500 spacing, 10% weaving	✖ 2500 spacing, 20% weaving	○ 2500 spacing, 30% weaving
+ 4000 spacing, 10% weaving	● 4000 spacing, 20% weaving	— 4000 spacing, 30% weaving
◇ 5500 spacing, 10% weaving	□ 5500 spacing, 20% weaving	△ 5500 spacing, 30% weaving

1 mi = 1.61 km

Figure 5-12. Freeway Speed vs. Weaving Volume.



◆ 1000 spacing, 10% weaving	■ 1000 spacing, 20% weaving	▲ 1000 spacing, 30% weaving
× 2500 spacing, 10% weaving	✱ 2500 spacing, 20% weaving	○ 2500 spacing, 30% weaving
+ 4000 spacing, 10% weaving	● 4000 spacing, 20% weaving	— 4000 spacing, 30% weaving
◇ 5500 spacing, 10% weaving	□ 5500 spacing, 20% weaving	△ 5500 spacing, 30% weaving



◆ 1000 spacing, 10% weaving	■ 1000 spacing, 20% weaving	▲ 1000 spacing, 30% weaving
× 2500 spacing, 10% weaving	✱ 2500 spacing, 20% weaving	○ 2500 spacing, 30% weaving
+ 4000 spacing, 10% weaving	● 4000 spacing, 20% weaving	— 4000 spacing, 30% weaving
◇ 5500 spacing, 10% weaving	□ 5500 spacing, 20% weaving	△ 5500 spacing, 30% weaving

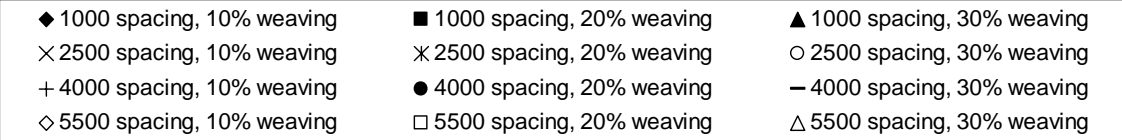
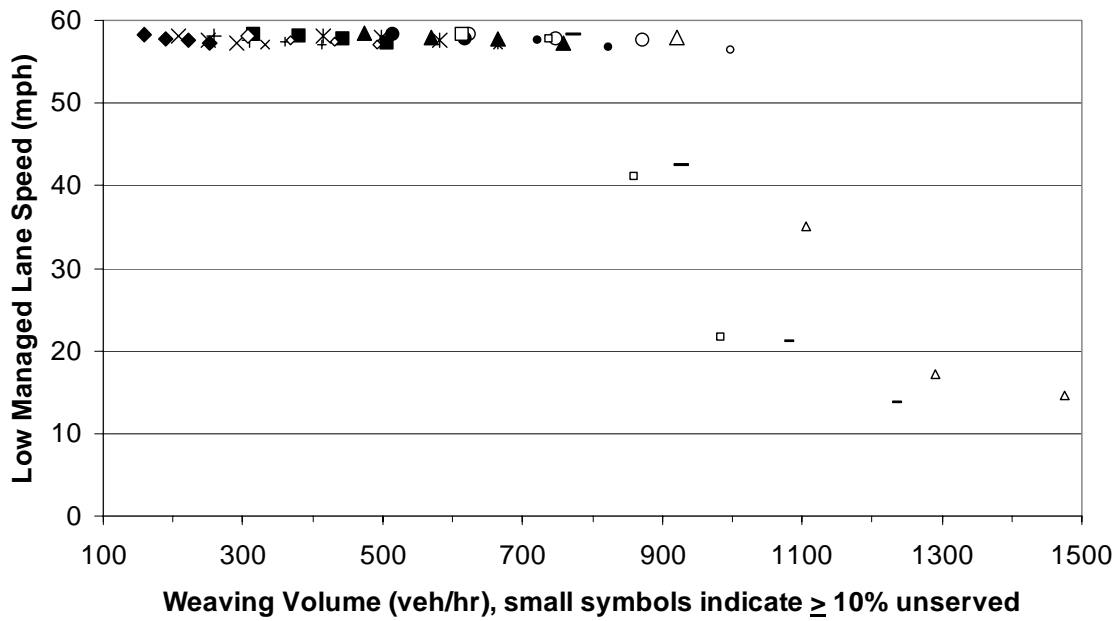
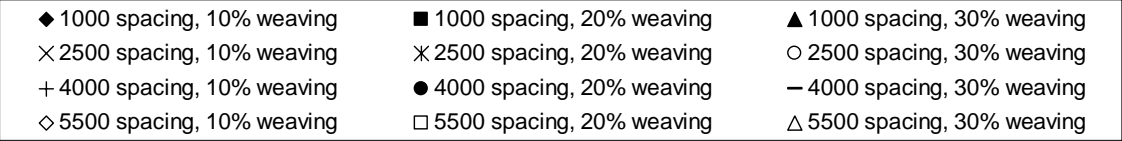
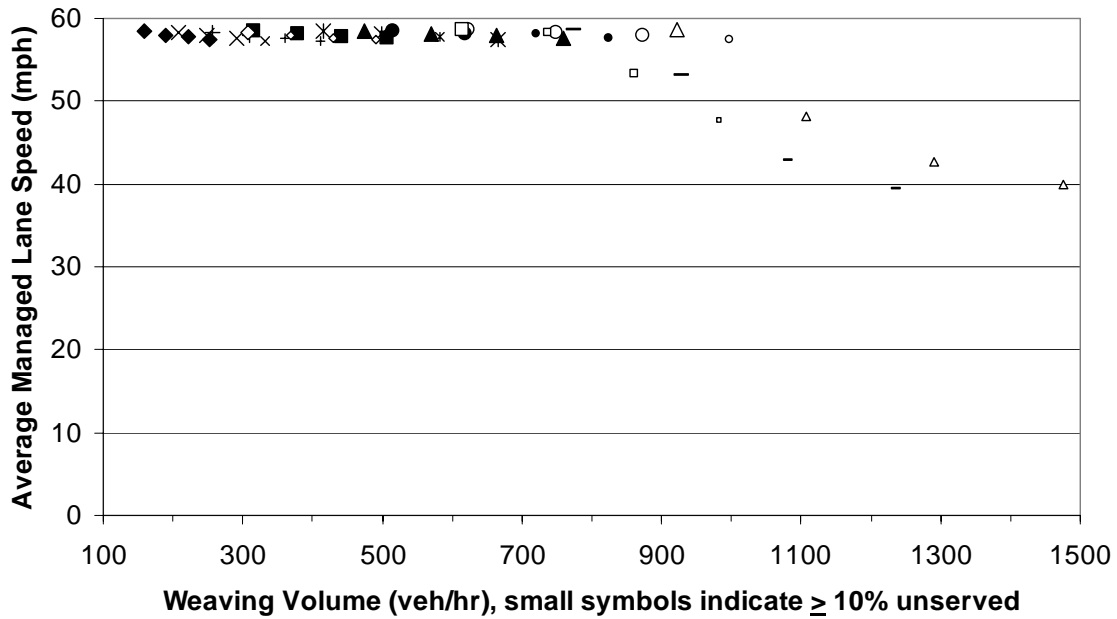
1 mi = 1.61 km

Figure 5-13. Entrance Weave Speed vs. Weaving Volume.

WHEN TO USE A DIRECT CONNECT RAMP

The *High-Occupancy Vehicle Facilities: A Planning, Design, and Operations Manual* (47) indicates that a direct connect ramp should be considered when ramp volume is 400 veh/hr. The findings from this simulation support that number. When considering average speeds, the number is about 500 veh/hr for the freeway traffic and about 300 veh/hr for the entrance weaving traffic. Using this simulation, a value of 400 veh/hr could be a reflection of a rounded value that gives consideration for both average freeway speeds and average entrance vehicle speeds. If the preference is to consider lowest speeds observed (a more conservative situation), then a direct connect ramp should be considered at 275 veh/hr.

When the average managed lane speeds are compared to the number of vehicles that are weaving into the managed lanes, the findings are more comparable to the situation when two merging streams experience turbulence. The managed lanes began to experience lower speeds at an entrance ramp volume of about 850 veh/hr, and speeds less than 45 mph [72.5 km/h] were observed at about 1050 veh/hr (see [Figure 5-14](#)). As seen in previous evaluations, the lower managed lane speeds are heavily influenced by the congestion at the managed lane exits.



1 mi = 1.61 km

Figure 5-14. Managed Lane Speed vs. Weaving Volume.

CHAPTER 6

SUMMARY AND CONCLUSIONS

SUMMARY

Designers are considering managed lanes in congested urban corridors where expansion possibilities are limited and forecasted conditions point to continuing congestion. The existing experience in both design and operations of managed lanes is limited, especially when considering the effects of varying operational strategies for vehicle type or payment rates. A potential source of information on how to design and operate a managed lane is available from work on HOV lanes. Criteria for HOV lanes have been examined in previous studies, and the findings from those studies can be applied to managed lane facilities. A recent report, *Guidance for Planning, Operating, and Designing Managed Lane Facilities in Texas*, provides geometric guidance for the design of managed lane facilities (1).

A Managed Lane Symposium was held during the initial year of TxDOT Project 0-4160 (2). As part of the workshop, three separate groups discussed managed lane issues and determined priorities during interactive sessions. Participants discussed several design-related issues, and access design received many comments. Based upon the availability of existing information on design criteria for HOV facilities and concerns expressed by participants of the Managed Lane Symposium, the primary direction set for Task 10 was to focus on ramp design issues. Following is a summary of the findings from TxDOT Project 0-4160 Task 10.

Available Literature on Ramp Design

Most of the recent literature regarding ramp design has focused on ramp design speed and truck performance. Key findings from the literature review include the following:

- Ramps should provide a smooth speed transition between the two roadways they connect. The roadway with the higher design speed should be the control for selecting the design speed of the ramp, but the controlling feature nearest to the lower-speed roadway may be designed at that lower speed to aid in the transition between the two roadways.
- Taper designs are preferred for exit ramps, but results are mixed between taper and parallel designs for entrance ramps.
- Drivers may not fully understand the proper use of speed-change lanes on freeways. There are few operational problems on the lanes themselves, but changing from the freeway lane to the speed-change lane (or vice-versa) is still an operational issue, particularly around the gore areas on exit ramps and gap acceptance behavior on entrance ramps.
- In general, the use of minimum AASHTO design values for ramp design provides little to no margin for error for large and/or heavily loaded trucks. A large number of ramps do not contain the additional AASHTO recommendations for wider lanes or larger radii to accommodate volumes of truck traffic.

- It is unclear whether drivers of large trucks fully understand the effects of the different characteristics of their trucks compared to passenger vehicles. The process of slowing down to exit, negotiating a curve, and then speeding up to merge is complex, and drivers may misunderstand that a ramp advisory speed applies to the entire ramp, or they may misjudge the maneuvering abilities of their vehicles.
- A few advanced warning systems have been tested for use in preventing rollover crashes and excessive truck speed on ramps. These systems, based on benefit-cost analysis on initial tests, can be cost-effective ways of improving truck operations on ramps.

State Practices for Ramp Design

To have an appreciation for current DOT practices, a search of each state's design manual was conducted via the Internet. Of the 23 states that had all or part of their design manuals online, 12 had some material available concerning the design of ramps. Key findings from the review include the following:

- Two state manuals define critical elements of ramp design to include items such as minimum length of acceleration for entrance ramp and minimum length of deceleration for exit ramp.
- The HOV Design Guide (41) and the Washington HOV Facilities chapter in the Washington Design Manual (38) discuss the benefits of exclusive HOV access ramps and encourage their inclusion; however, specific guidelines on when to use them are not presented.
- According to AASHTO (39), the minimum design speed for direct connection ramps should preferably be 40 mph [64.4 km/h]. The Texas (37) and California (27) roadway manuals discuss direct connectors with California stating that the design speed should be a minimum of 50 mph [80 km/h].
- Several elements associated with ramp design are discussed in the manuals including: design speed, ramp radii, grades, sight distance, superelevation, acceleration/deceleration lanes and transitions, cross section/ramp width, curbs, ramp terminals, and gore areas.
- Each state with specific discussions on the spacing between successive ramps used approximately 900 to 1000 ft [300 m] spacing.
- AASHTO (39) specifies a 2000-ft [610 m] weaving section for a system-to-service interchange.
- Each state and AASHTO (39) specified a preference that all ramps be placed on the right side of the traveled way to minimize drivers' confusion and the need for weaving.
- Three states mention interchange spacing as an important consideration with the recommendation being either 1 or 2 mi [1.6 or 3.2 km] of spacing.

Case Study

The potential Texas Managed Lane system could contain elements of systems that are currently in use in other communities. Information on how those elements are operating can help in the selection of components best suited for Texas. Examples include how special-use lanes are signed or marked, their typical dimensions for lane and shoulder widths, and how the special-use

lanes are accessed. As part of this research project, members of the research team visited the New Jersey Turnpike facility. Following are key observations from the visit:

- A 32-mi [52 km] segment of the turnpike was expanded to two separate roadways in each direction of travel (see [Figure 4-2](#)) with each same direction roadway called a barrel. The objective of the “dual-dual” roadway was to improve operations and safety by separating heavy vehicles from light vehicles and to increase capacity (heavy vehicles are restricted to the outer lanes). It was also intended to provide greater flexibility for using the roadway during periods of heavy congestion such as a major incident since changeable message signs technology could be applied to warn approaching drivers and divert them to the less-congested barrel (see [Figure 4-3](#)).
- Each barrel has its own exit and entrance ramps. The inner roadway traffic does not weave across the outer roadway traffic to reach an exit (see [Figures 4-22](#) and [4-23](#)). The traffic from barrels in the same direction merges prior to the toll plaza (see [Figure 4-24](#)). The ramp designs used at the interchanges result in having all traffic moving through one toll plaza for each interchange. This allows for consolidation of personnel and equipment (and resulting in cost savings) in the collection of tolls. New Jersey employs both trumpet and slip-ramp designs.
- Available crash information supports the theory that the dual-dual roadway system enhances safety. During the five years before completion of the dual-dual roadway (1965-69), the average annual accident rate was 94.1 accidents per million vehicle miles; in the succeeding five years, the rate was 79.2 accidents per million vehicle miles, a reduction of over 18 percent. For the five-year period from 1994 to 1998, the crash rate on each of the dual-dual roadways (outer and inner) was 26 to 61 percent less than on the segments of the turnpike without separate roadways. The author of the study cautioned that more detailed evaluation of the accident records and corresponding roadway conditions is needed to determine how much of the difference is attributable to the separation of vehicles and how much is attributable to other factors such as fewer lanes and higher levels of congestion on the non-separated portions. The data, however, clearly indicate that accident rates are lower in the areas with the dual-dual roadways.

The dual-dual portion of the New Jersey Turnpike clearly demonstrates the operational and safety benefits of separating vehicle modes. Having the entrance to a HOV or passenger-car exclusive facility that is located in the center of a freeway corridor without a dedicated ramp requires vehicles to weave across each of the general-purpose lanes. The direct access to each barrel provided on the New Jersey Turnpike eliminates this weaving maneuver (which promotes a safer and more operationally efficient system). Maintaining similar geometric criteria for both barrels also provides greater flexibility in moving traffic between the barrels as needed for incidents and maintenance. In addition, the finding that the dual-dual portion has lower crash rates supports separating trucks and passenger cars.

Computer Simulation

Researchers used simulation to obtain an appreciation of the effects of ramp spacing on freeway operations. A previous effort (Task 5) within TxDOT Project 0-4160 ([44](#)) focused on the impact of managed lane access and egress weaving behavior for a single pair of ramps. Simulation of

several ramp pairs is needed to identify the impact on the corridor of vehicles from different entrance ramps consistently weaving across free lanes to access a managed lane facility. The simulation performed as part of TxDOT Project 0-4160 Task 10 had the following goals:

- Quantify the effects of ramp spacing on freeway operations.
- Continue the investigation of when to consider a direct ramp between the managed lanes and a generator or surface street system.

Speed was the primary measure of effectiveness used to evaluate the effects of the different ramp spacing, volume levels, and weaving percentages. Ramp spacing of 1000, 2500, 4000, and 5500 ft [305, 762.5, 1220, and 1677.5 m] was used. Freeway initial volumes of 1250, 1500, 1750, and 2000 vehicles per hour per lane (veh/hr/ln) were also used. Finally, the percentage of freeway entrance ramp traffic that desired to maneuver to the next managed lane access point was varied between 0, 10, 20, and 30 percent of the traffic on the (source) freeway entrance ramp. The 0 percent weaving scenario provided a baseline condition of how the freeway would operate without the managed lane facility.

Key findings from the simulation include the following:

- In the simulation, ramp spacing only affected average freeway speeds when the initial freeway volumes were very high (2000 veh/hr/ln) and ramp spacing was at the lowest value used in the simulation (1000 ft [305 m]).
- In each weaving level comparison, the average freeway speed dropped faster for the shorter ramp spacing (look especially in [Figure 5-7](#) at the almost vertical line representing the data for the 1000-ft [305 m] spacing scenario as compared to the other spacing scenarios). This shows that operations are more sensitive to small increases in traffic volumes when ramp spacing is shorter.
- The number of vehicles attempting to weave across the four freeway lanes to enter the managed lanes can have a pronounced impact on the operations of the freeway. With the exception of short spacing in combination with high initial freeway volumes, the average freeway speeds recorded from the simulation runs are generally above 45 mph [72.5 km/h] until approximately 500 vehicles per hour are attempting to weave across the freeway and enter the managed lanes. When the plot of the lowest freeway speed recorded is reviewed, the point when less than desirable operations occur is at approximately 250 veh/hr.
- Average entrance weave speeds below 45 mph [72.5 km/h] occur when approximately 300 veh/hr are attempting to weave. Low entrance weave speeds below 45 mph [72.5 km/h] occur at lower volumes, at about 250 veh/hr.
- The *High-Occupancy Vehicle Facilities: A Planning, Design, and Operations Manual (47)* indicates that a direct connect ramp should be considered when ramp volume is 400 veh/hr. The findings from this simulation support that number. When considering average speeds, the number is about 500 veh/hr for the freeway traffic and about 300 veh/hr for the entrance weaving traffic. Using this simulation, a value of 400 veh/hr could be a reflection of a rounded value that gives consideration for both average freeway speeds and average entrance vehicle speeds. If the preference is to consider

lowest speeds observed (a more conservative situation), then a direct connect ramp should be considered at 275 veh/hr.

Draft Material for Use in the Managed Lane Manual

Using information contained in several recent publications, especially the *Guidance for Planning, Operating, and Designing Managed Lane Facilities in Texas* report, draft material for use in the Managed Lane Manual will be developed.

CONCLUSIONS

Information on geometric design features for ramps is available in a number of sources including the AASHTO *Green Book* and the Texas *Roadway Design Manual*. A review of state design manuals demonstrated that the Texas manual includes more discussion and examples on ramp design than most other state manuals. An issue not well discussed in any document is where to place the ramp with respect to other entrance and exit ramps. General guidelines are provided (900 to 1000 ft [300 m]); however, these guidelines are not sensitive to the expected ramp volume, the anticipated destination of the ramp vehicles (e.g., the next exit ramp or a downstream entrance to a managed lane facility), or the number of lanes on the freeway. Work completed as part of TxDOT Project 0-4160 Task 5 provided recommendations for spacing needs for cross-freeway weaving (e.g., between a right-side entrance ramp and a downstream left-side exit ramp to a managed lane facility) (44).

Research conducted as part of TxDOT Project 0-4160 Task 10 found that a direct connect ramp between a generator and the managed lane facility should be considered when 400 veh/hr are anticipated to access the managed lanes. If a more conservative approach to preserving freeway performance is desired, then a direct connect ramp should be considered at 275 veh/hr (which reflects the value when the lowest speeds on the simulated corridor for the scenarios examined were at 45 mph [72.5 km/h] or less).

The New Jersey Turnpike has two separate roadways in each direction of travel with each roadway having its own exit and entrance ramps. The “dual-dual” roadway improves operations and safety by separating heavy vehicles from light vehicles and increases capacity (heavy vehicles are only permitted on the outer roadway). It also increases flexibility for managing incidents as drivers can be directed to the roadway without the incident through the use of changeable message signs. Available crash information showed lower crash rates for the dual-dual portion as compared to segments of the turnpike without separate roadways (between 26 and 61 percent for 1994 to 1998). The dual-dual design used on a portion of the New Jersey Turnpike has significant operational and safety benefits. These benefits need to be quantified and a benefit-cost evaluation needs to be performed to determine if this approach is feasible within Texas. If the approach is feasible, research should determine under what conditions and when the design should be considered.

Recent literature on ramp design has focused on ramp design speed and truck performance. The current process allows for as much as a 50 percent reduction in design speed from a freeway to a ramp. Research has shown that the use of these minimum values provides little to no margin for

error for large and/or heavily loaded trucks. The use of such large reduction can also impact operating speeds as a vehicle moves from one facility to another. To maintain high performance for the managed lane facilities, the design speed selected for the ramps must consider the anticipated speeds of the vehicles entering the ramp, the desired speed of the vehicles on the ramp, and the speeds of the vehicles the ramp vehicles will encounter when they are attempting to merge. A design speed less than the anticipated or desired operating speed will affect the performance of the managed lane. If trucks are a primary vehicle type for the facility, they need to be explicitly considered during the selection of the design features for both the ramp and the managed lane as well as the signing to be used.

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