Design Guidance for Freeway Mainline Ramp Terminals

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By B. Ray Derr
Staff Officer
Transportation Research Board

This report presents design guidance for freeway mainline ramp terminals based on current driver and vehicle behavior. The report will be useful to freeway designers and those responsible for developing design standards for freeway ramps. Special attention was given to the behavior of tractor-trailers and the report includes several speed-distance curves for tractor-trailers on grades up to 9% that could be useful in other applications (e.g., climbing lanes).

The design values for freeway ramps in the AASHTO *Policy on Geometric Design of Highways and Streets (Green Book)* rely heavily on research conducted in the late 1930s and early 1940s, predating the development of the Interstate system. The studies relied entirely on passenger cars for acceleration and deceleration rates, without consideration of trucks and buses; this was based on the assumption that the acceleration distances for heavy vehicles would be "entirely out of reason." Vehicle characteristics have changed since the original research. For example, the weight-to-horsepower ratio for trucks in 1965 was 400 lb/hp compared with the currently used ratio of 200 lb/hp.

According to the *Green Book*, the mainline terminal of a ramp is that portion adjacent to the through traveled way, including speed-change lanes and tapers. There are two basic designs for freeway ramp terminals: tapered and parallel. Transportation agencies tend to adopt one of these designs as a standard (often different for entrances and exits), but there is little objective information available for designers on their relative strengths.

In NCHRP Project 15-31, CH2M Hill, MRIGlobal, and the Texas Transportation Institute reviewed the relevant literature and developed conceptual models for freeway entrance and exit maneuvers that represent the driver's thought processes and decision points. Critical factors and issues for ramp terminal design (including design vehicles) were described as well as interrelationships between them. The research team then prepared an updated work plan for collecting data, analyzing data, and developing design guidance for freeway mainline ramp terminals.

Due to staffing changes, CH2M Hill withdrew from the research team and MRIGlobal and TTI continued the work in NCHRP Project 15-31A. Crash analysis was used to identify typical types of crashes at ramp terminals; observational field studies and driver behavioral studies were used to update basic assumptions in the equations. These results were then used to assess typical ramp terminal designs and develop design guidance for parallel and tapered freeway mainline ramp terminals.

The following appendices are available on the TRB website (http://www.trb.org/Main/Blurbs/167516.aspx):

- Appendix A: Aerial View of Study Locations,
- Appendix B: Histograms of Observed Acceleration Rates,
- Appendix C: Verbal Instructions for Behavioral Study, and
- Appendix D: Potential Changes Proposed for Consideration in the Next Edition of the *Green Book*.

CONTENTS

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

SUMMARY

Design Guidance for Freeway Mainline Ramp Terminals

The design of ramp-freeway junctions is a critical component in the overall safety performance of a controlled access facility. Interchanges present the greatest safety and operational challenges for drivers on the freeway system, as most freeway crashes occur in the vicinity of interchanges. Entry and exiting maneuvers near interchanges place increased workload on drivers associated with navigational decision making, speed-changing, and tracking demands.

This report focuses on the design of freeway mainline ramp terminals (i.e., acceleration and deceleration lanes). Design values for freeway mainline ramp terminals in the 2004 *Policy on Geometric Design of Highways and Streets* (commonly known as the *Green Book*) rely on somewhat outdated research. The objective of this research is to develop improved design guidance for freeway mainline ramp terminals based on modern driver behavior and vehicle performance capabilities. This research addresses the following questions:

- Is the fundamental AASHTO model or set of operational assumptions behind freeway mainline ramp terminal design sufficient to describe the full range of design parameters?
- Are acceleration and deceleration rates inherent in the AASHTO speed-change lane (SCL) models appropriate for today's driver population and vehicle fleet?
- Should trucks be used as the design vehicle for freeway mainline ramp terminal design?
- Are there differences in the operational performance between parallel and tapered SCLs?
- Does driver behavior differ for low-speed ramps compared with higher speed ramps?
- What are the implications of any design changes on current roadway practice and existing designs?

The primary steps to address these key questions included a crash analysis and field studies. The crash analysis was conducted to investigate the vehicle types involved in crashes near freeway mainline ramp terminals. If trucks were disproportionately represented in crashes near freeway mainline ramp terminals, an argument could be made that a truck, rather than a passenger car, should be the design vehicle for an entrance or exit ramp. Field studies were conducted to examine vehicle performance and driver behavior near freeway mainline ramp terminals. An observational field study was performed to collect speed, acceleration/deceleration, and distance information on a large number of vehicles at several freeway entrance and exit ramps. A driver behavior study was also performed in which subjects drove an instrumented vehicle along a selected course using designated entrance and exit ramps. In addition to behavioral data, speed, acceleration/deceleration, and location information were also gathered during the behavioral study. Data from both studies were analyzed to address the central questions related to this research.

Several key findings from the crash analysis are as follows:

- In general, there was no difference between truck crash rates and crash rates for all vehicles at either entrance or exit ramps. The exit ramp configurations where truck crash rates exceeded the overall crash rates by the greatest amount included parclo loop, free-flow loop, and "other" ramps.
- Trucks are involved in approximately the same proportion of crashes in the vicinity of freeway mainline ramp terminals as they are along the mainline of urban freeways (about 10 percent).
- Where the ramp truck average daily traffic (RTADT) exceeds 1,000 trucks/day, it appears that truck crashes occur more frequently compared to terminals with lower RTADT.

Several key findings from the observational and behavioral studies related to entrance ramp terminals (i.e., acceleration lanes) are as follows:

- Vehicles tend to merge later under constrained-merge conditions than under free- or forced-merge conditions.
- The distribution of merge locations for trucks is very similar to that for passenger cars.
- Vehicles merge later at tapered SCLs than at parallel SCLs.
- Vehicles merge at speeds closer to freeway speeds under constrained- and forced-merge conditions than under free-merge conditions.
- Late mergers enter the mainline freeway lanes at speeds closer to freeway speeds than early mergers.
- The speed differential between merging vehicles and mainline freeway vehicles is nearly the same for both straight and loop ramps.
- Vehicles merge closer to freeway speeds at tapered acceleration lanes than at parallel acceleration lanes.
- The median acceleration rates for free-flow passenger cars under free-merge conditions are greater than the assumed acceleration rates in the 2004 *Green Book.*
- Entrance-ramp vehicles depart the controlling features of entrance ramps at speeds higher than assumed in the 2004 *Green Book.*
- Acceleration rates for free-flow trucks are lower than for free-flow passenger cars.
- In uncongested or lightly congested conditions, drivers tend to glance into their mirror or over their shoulder about three times before merging onto the freeway. A typical glance lasts about 2.5 to 3.0 s, and drivers increase speeds by approximately 2.5 mi/h during each glance.

Several key findings from the observational and behavioral studies related to exit ramp terminals (i.e., deceleration lanes) are as follows:

- Under free-diverge conditions the distribution of diverge locations for trucks is very similar to that for passenger cars.
- At the point where vehicles maneuver from the freeway to the deceleration lane, speeds of diverging vehicles are typically 4 to 7 mi/h below average freeway speeds.
- Diverge speeds of trucks are lower than for passenger cars.
- Diverging vehicles decelerate at rates lower than those assumed in the 2004 *Green Book.*
- There is little difference between deceleration rates of passenger cars and trucks.
- Deceleration rates of exiting vehicles are greater for vehicles that diverge closer to the painted nose than for vehicles that diverge further upstream from the painted nose.
- Deceleration rates are greater on loop ramps than straight ramps.

- Deceleration rates are substantially higher on parallel SCLs than on tapered SCLs.
- The conceptual approach used in AASHTO policy for the design of exit ramps, which assumes that drivers decelerate in gear (i.e., coast) for 3 s, is consistent with current driver behavior, assuming the definition of coasting includes the time used to remove the driver's foot from the throttle. A substantial portion of this deceleration time occurs within the mainline freeway lanes.

Several general conclusions related to the design of freeway mainline ramp terminals based upon the findings of this research are as follows:

- Passenger cars should remain the principal design vehicle for freeway mainline ramp terminals, except where truck volumes on ramps are substantial, in which case further consideration should be given to more fully accommodating trucks within the design.
- Freeway mainline ramp terminals should be designed based upon free-flow conditions.

Several conclusions specific to entrance ramps based upon the findings of this research are as follows:

- In free-merge conditions, many vehicles choose to enter the freeway at speeds much lower than the speed of freeway traffic. Drivers simply choose not to use the full length of the ramp and SCL when gaps are abundant and merging is not difficult.
- Heavy vehicles do not perform as well as passenger cars at entrance ramps. Their acceleration rates are lower, and they merge onto the freeway at lower speeds. However, at ramps with a small proportion of truck traffic, their merging behavior does not appear to negatively impact the overall operation of the ramp terminals.
- Vehicles are more likely to use the full length of a tapered SCL to accelerate to near freeway speeds before merging, in contrast to parallel SCLs where vehicles may merge earlier along the ramp and at lower speeds.
- There is no substantive difference in operational performance between low-speed (loop) and high-speed (straight) entrance ramps under free-merge conditions.
- The conceptual approach used in AASHTO policy for the design of entrance ramps, which assumes constant acceleration, is a reasonable approach for determining minimum acceleration lane lengths for design. In addition, the current values provided in *Green Book* Exhibit 10-70 are conservative estimates for minimum acceleration lane lengths, given the current vehicle fleet and driver population, and do not need to be modified. In particular, they provide sufficient length for vehicles to merge onto the freeway under a range of freeway operating conditions. In the design of entrance ramps where free-merge conditions are expected for the foreseeable future and constraints make it difficult to provide the recommended minimum acceleration lengths, the minimum acceleration lane lengths presented in the *Green Book* can be reduced by 15 percent without creating operational problems.

Several conclusions specific to exit ramps based upon the findings of this research are as follows:

- Most diverge maneuvers begin before or within the taper area or within the first or middle thirds of the SCL. Few diverge maneuvers take place in the final third of the SCL or beyond the painted nose.
- Vehicles that diverge earlier along the deceleration lane diverge closer to freeway speeds than vehicles that diverge later along the deceleration lane (i.e., closer to the painted nose).

- Where the deceleration lane length is longer than the recommended minimum length presented in the *Green Book,* most vehicles decelerate at rates lower than those assumed by the *Green Book.* This is in varying degrees due to the additional length provided and to vehicles decelerating in the freeway lane prior to initiating the diverge maneuver.
- Deceleration rates of exiting vehicles are greater for vehicles that diverge closer to the painted nose than for vehicles that diverge further upstream from the painted nose.
- When exiting the freeway, trucks decelerate at rates very comparable to those of passenger cars. In addition, trucks typically diverge from the freeway at lower speeds than passenger cars.
- Crash rates for trucks are higher at parclo, free-flow, and “other” ramp configurations than at diamond; outer connection; direct or semi-direct connection; and button hook, scissor, and slip ramp configurations.
- Drivers exiting on loop ramps tend to reduce their speed in the freeway lane more, and decelerate along the SCL at a greater rate, than drivers exiting on straight ramps. This may be due the visual perceptions of drivers as they approach the horizontal curvature of a loop ramp.
- The geometry of parallel deceleration lanes generally leads to substantially higher deceleration rates than on tapered deceleration lanes. This may be the result of vehicles diverging slightly closer to freeway speeds along parallel deceleration lanes than along tapered deceleration lanes. The disparity between deceleration rates is most apparent on straight ramps.
- The conceptual approach used in AASHTO policy for the design of exit ramps, which assumes a two-step process of deceleration, is a reasonable approach for determining minimum deceleration lane lengths for design. In addition, the current values provided in *Green Book* Exhibit 10-73 are conservative estimates for minimum deceleration lane lengths, given the current vehicle fleet and driver population, and do not need to be modified. No critical or unusual diverge maneuvers were observed at the study sites that met and exceeded the current design criteria. In addition, at these study locations, vehicles decelerated at rates well within the capabilities of the vehicle fleet and driver preferences. This was in part due to some deceleration by diverging vehicles in the freeway mainline prior to the diverge maneuver. Given this last point, it is beneficial to have a conservative design process that does not assume that vehicles begin decelerating in the freeway mainline.

Several potential changes for consideration in the next edition of the *Green Book* are proposed based on the findings and conclusions of this research as follows:

- Include a statement in the *Green Book* text that tapered SCLs are preferred over parallel SCLs at entrance ramps because vehicles tend to merge closer to freeway speeds at tapered SCLs, and, if parallel SCLs are used, they are most appropriate at ramps expected to experience constrained- or forced-merge conditions because they provide greater flexibility in selecting a merge location along the SCL.
- Include a statement in the text accompanying *Green Book* Exhibit 10-70 that design values in the exhibit are conservative, and where free-merge conditions are expected for the foreseeable future and where constraints make it difficult to provide the recommended minimum acceleration lengths presented in the *Green Book,* minimum acceleration lane lengths may be reduced by 15 percent.
- Include additional exhibits that provide speed-distance curves for trucks in a range from 140 to 200 lb/hp. This will provide the designer with more flexibility to select an appropriate heavy vehicle for design, and in some cases find a better compromise between designing for passenger cars and trucks, especially at entrance terminals with substantial truck

volumes. These exhibits will ideally be provided in Chapter 3 of the *Green Book* because they could be used for more general design purposes, rather than just for acceleration lanes. A reference to the new exhibits will ideally be included in Chapter 10 of the *Green Book.*

- Emphasize in the *Green Book* text that values presented for minimum deceleration lane length on exit ramps are conservative and do not account for deceleration in the mainline freeway lanes. While some drivers do accomplish a considerable portion of their deceleration on the freeway, it is prudent for the designer not to assume deceleration in the mainline freeway lanes in the design of an exit ramp.
- Mention within the text that providing deceleration lanes longer than the minimum values in *Green Book* Exhibit 10-73 may promote casual deceleration by exiting drivers, particularly under uncongested or lightly congested conditions. This is not necessarily a negative result, but it does change the operational characteristics of the ramp.

SECTION 1

Introduction

1.1 Background

The design of the ramp-freeway junction is a critical component in the overall safety performance of a controlled access facility. Interchanges present the greatest safety and operational problems for drivers, as most freeway crashes occur in the vicinity of interchanges (Lunenfeld, 1993). Interchanges are inherent points of conflict involving entering and exiting traffic. Both entry and exit maneuvers place increased demands and workload on drivers associated with navigational decision making, speed changing, and tracking. The combination of these demands results in an increased likelihood of driver error.

Freeway ramps consist of the following three elements:

- The freeway mainline ramp terminal,
- The ramp proper, and
- The crossroad terminal.

This report primarily focuses on the freeway mainline ramp terminal element of freeway ramps (i.e., acceleration and deceleration lanes), but also evaluates operations of vehicles along the ramp proper. Chapter 10 of the American Association of State Highway and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets* (AASHTO, 2004) (hereinafter referred to as the *Green Book*) describes recommended dimensional guidance for freeway mainline ramp terminals, including both entrance and exit terminals. Such guidance focuses on the speed relationship of the ramp proper to each ramp terminal and the linear requirements to affect appropriate speed-change behavior.

Design values for freeway mainline ramp terminals in the *Green Book* rely on outdated research, much of which was conducted prior to the development of the Interstate system. The underlying design guidance is, therefore, based on assumptions and vehicle performance characteristics that may no longer be applicable. It is also clear that driver performance has changed over the years, with particular focus on the increasing proportion of older drivers and the increasing number of trucks in the traffic stream.

1.2 Objective and Scope

The objective of this research is to develop improved design guidance for freeway mainline ramp terminals based on modern driver behavior and vehicle performance capabilities. This research addresses the following questions:

- Is the fundamental AASHTO model or set of operational assumptions behind freeway mainline ramp terminal design sufficient to describe the full range of design parameters?
- Are acceleration and deceleration rates inherent in the AASHTO speed-change lane (SCL) models appropriate for today's driver population and vehicle fleet?
- Should trucks be used as the design vehicle for freeway mainline ramp terminal design for all ramps, or for ramps under certain design conditions?
- Are there important differences in the substantive operational performance of parallel and tapered SCLs?
- How does driver behavior differ for low-speed ramps compared with higher speed ramps?
- What are the implications of any design changes on current roadway practice and existing designs?

The research methodology was formulated to address these questions central to the design of mainline freeway ramp terminals.

The scope of this research includes only independent freeway mainline ramp terminals (i.e., outside the influence of upstream or downstream operations of other freeway mainline ramp terminals). This research does not investigate the influence of ramp or interchange spacing. In addition, this research focuses on single-lane freeway mainline ramp terminals and does not specifically address freeway mainline ramp terminals with two or more lanes. Finally, this research does not address weaving areas where a continuous auxiliary lane is provided between entrance and exit terminals, nor does it address issues specifically related to ramp metering.

Based upon the findings and conclusions from this research, proposed changes to the next edition of the *Green Book* are provided.

1.3 Overview of Research Methodology

The research conducted during this study was divided into two phases. In Phase I, the literature was summarized to gain knowledge concerning prior research on the safety performance of freeway mainline ramp terminals, vehicle performance characteristics of passenger cars and trucks, human factor considerations in the design and operation of freeway mainline ramp terminals, and the operational performance of freeway mainline ramp terminals. Next, the research team evaluated conceptual models to explain both the merging and diverging processes. The research team also reviewed truck-related crashes near freeway ramps to assess an appropriate design vehicle for freeway mainline ramp terminals. Following these three tasks, a research plan was formulated for execution in Phase II.

The primary Phase II efforts consisted of two types of field studies to examine both vehicle performance and driver behavior near freeway mainline ramp terminals. An observational field study was performed in which laser guns, cameras, and traffic classifiers were used to collect speed, acceleration/deceleration, and distance information on a large number of vehicles at several freeway entrance and exit ramps. A driver behavior study was also performed in which subjects drove an instrumented vehicle along a selected course, entering and exiting the freeway using designated entrance and exit ramps. Speed, acceleration/deceleration, and location information were gathered in this study, as well. Data from the driver behavior study were used to supplement the data collected during the observational study. Data from both studies were reduced and analyzed to develop proposed changes to the next edition of the *Green Book,* to improve design guidance for freeway mainline ramp terminals.

1.4 Outline of Report

This final report documents the entire research effort. The remainder of this report is organized as follows. Section 2 summarizes the literature related to freeway mainline ramp terminals. Section 3 summarizes the conceptual models utilized by AASHTO in determining minimum acceleration and deceleration lane lengths for entrance and exit terminals, respectively. Section 4 presents an analysis of truck-related crashes near freeway mainline ramp terminals. Section 5 describes an observational study conducted to gather speed and distance information for a large number of merging and diverging vehicles covering a range of ramp types and merge/diverge types. Section 6 presents a behavioral study designed to collect detailed information on a limited number of drivers to investigate driver behaviors while performing merge and diverge maneuvers onto and off of freeways. Section 7 presents the conclusions from this research, including future research needs. Section 8 presents the references cited in this report. The appendices to this report, which are available on the TRB website at http://www.trb.org/Main/Blurbs/167516.aspx include the following. Appendix A presents an aerial view of the study locations included in the observational study. Appendix B provides histograms of acceleration rates from the observational study. Appendix C presents the verbal instructions given to participants in the behavioral study. Appendix D presents proposed changes to the next edition of the *Green Book,* based upon the findings and conclusions of this research.

SECTION 2

Literature Review

This section first summarizes the current AASHTO design policy for freeway mainline ramp terminals and then summarizes existing literature on geometric design and safety performance, vehicle performance characteristics, human factors considerations, and operational performance issues related to freeway mainline ramp terminals.

2.1 AASHTO Design Policies for Speed-Change Lanes and Freeway Mainline Ramp Terminals

AASHTO defines an SCL as an auxiliary lane, including tapered areas, primarily for the acceleration or deceleration of vehicles entering or leaving the through-traffic lanes. The terms SCL, acceleration lane, and deceleration lane apply broadly to the added pavement joining the traveled way of the highway or street with that of the turning roadway and do not necessarily imply a definite lane of uniform width. An SCL should have sufficient length to enable a driver to perform an appropriate change in speed between the highway and the turning roadway in a safe and comfortable manner. *Green Book* design values for freeway ramps rely heavily on research conducted in the late 1930s and early 1940s. The 1965 American Association of State Highway Officials (AASHO) *Blue Book* provides more information on the design guidelines of SCLs than the later versions of AASHTO design policies (AASHO, 1965; AASHTO, 1990; AASHTO, 1994; AASHTO, 2001; AASHTO, 2004). Basic components that make up the SCLs are described below.

Acceleration Lane: An acceleration lane begins where the driver transitions from the ramp curvature to the flatter geometry of the SCL. The lane allows for acceleration and checking for gaps in the freeway traffic. The length of the acceleration lane is determined on the basis that merging vehicles should enter the through lane at a speed approximately equal to the running speed of the freeway. The acceleration lane ends at the beginning of the taper. The taper section begins where the width of the SCL becomes less than 12 ft and ends at the point where the SCL has fully merged with the freeway through lane. The taper section is not included in the length of the SCL.

The 1965 AASHO *Blue Book* describes the factors for the calculation of the length of an acceleration lane: (a) the speed at which drivers merge with through traffic, (b) the speed at which drivers enter the acceleration lane, and (c) the manner of acceleration. The acceleration rates are determined from the studies done in the late 1930s. AASHO notes that the acceleration rates are based on passenger vehicle operation, and that trucks and buses require much longer distances but that providing such lengths would be out of reason. It assumes that slower entry is unavoidable and is generally accepted by the public. AASHO also mentions that if there are a substantial number of heavy vehicles entering, then lengths may be increased or, if feasible, the entries may be located on downgrades. The 1965 *Blue Book* provides the most detailed information on how the current design values in the 2004 *Green Book* are determined, but it should be noted that for some combinations, the 2004 *Green Book* acceleration lane lengths differ slightly from the 1965 *Blue Book* values. Also, AASHTO's current geometric design guidance is based on a single set of operational assumptions that can be characterized as representing free or unconstrained merging behavior.

Deceleration Lane: A tapered section of roadway, where the pavement transitions from the freeway edge of pavement to a point where the width of the SCL is 12 ft, precedes a deceleration lane. From the end of the taper to where the driver transitions onto the ramp curvature, the deceleration lane provides a means for vehicles to decelerate from the freeway speed to a speed commensurate with the controlling feature of the ramp.

Similar to the acceleration lane, deceleration lane length is based on three factors: (a) the speed at which drivers maneuver onto the auxiliary lane, (b) the speed at the end of the deceleration lane, and (c) the manner of deceleration. The length of the deceleration lane is determined by the speed differential, which is the difference between the average running speed on the mainline and the speed on the sharp or controlling terminal curve on the turning roadway. The average running speed is used instead of the design speed, based on the assumption that the exiting drivers travel at the average running speed when highway volumes are low.

All design lengths are based on passenger car operations. Even though the 1965 AASHO guide recognizes that trucks require longer distances to decelerate for the same difference in speed, AASHO indicates longer lanes are not justified because average speeds of trucks are lower than passenger car speeds.

Two general forms of SCLs are the taper and parallel types. Below is the description of entrance and exit terminals for both types of SCLs.

2.1.1 Single-Lane Free-Flow Terminals, Entrances

2.1.1.1 Taper-Type Entrance

A taper-type entrance as shown in Figure 1A merges into the freeway with a long, uniform taper with a desirable rate of taper of about 50:1 to 70:1 (longitudinal to lateral) between the outer edge of the acceleration lane and the edge of the through-traffic lane. The geometrics should be designed in a way that drivers attain a speed that is within 5 mi/h of the operating speed of the freeway by the time they reach the point where the left edge of the SCL joins the traveled way of the freeway. AASHTO sets this location as the point where the right edge of the SCL and traveled way are 12 ft apart. The length required for the vehicle to

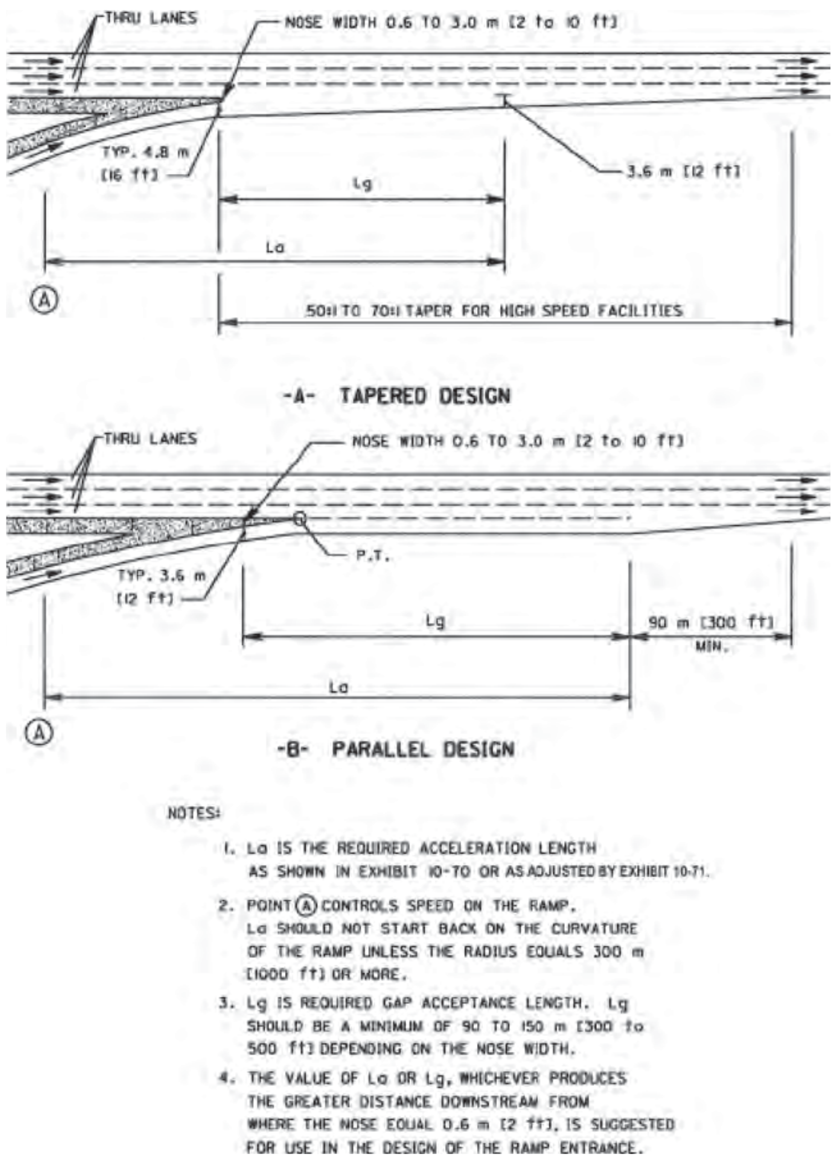


Figure 1. Tapered and parallel designs for entrance ramps (AASHTO, 2004).

achieve this speed is referred to as the acceleration length and is measured from the end of the governing curve on the ramp proper to where the right edge of the SCL and through lane are 12 ft apart. Adjustment factors are used to increase the recommended acceleration lane lengths for ramps with positive grades and decrease acceleration lane lengths for ramps with negative grades.

2.1.1.2 Parallel-Type Entrance

A parallel-type entrance as shown in Figure 1B provides an added lane of sufficient length to enable a vehicle to accelerate to near freeway speed prior to merging. A taper is provided at the end of the added lane. AASHTO recommends a taper length of approximately 300 ft for design speeds up to 70 mi/h.

The distance needed for acceleration in advance of the merge point is governed by the speed differential between the operating speed on the entrance curve of the ramp and the operating speed of the freeway. *Green Book* Exhibit 10-70 recommends minimum acceleration lane lengths for entrance terminals, applicable to both taper- and parallel-type entrances. *Green Book* Exhibit 10-69 provides minimum lengths for gap acceptance. The larger value between the acceleration lane length and the gap acceptance length is used in the design of freeway mainline ramp terminals.

In places where it is anticipated that the ramp and freeway will frequently carry traffic volumes approximately equal to the design capacity of the merging area, a parallel acceleration length of at least 1,200 ft plus taper is desirable.

Where grades are present on ramps, minimum recommended acceleration lanes ideally will be adjusted in accordance to *Green Book* Exhibit 10-71.

2.1.2 Single-Lane Free-Flow Terminals, Exits

2.1.2.1 Taper-Type Exit

A taper-type exit, as shown in Figure 2, provides a direct path to the ramp proper. The divergence angle is usually between 2 and 5 degrees. The length of the deceleration lane is measured from the point where the lane is a 12-ft width on the ramp to the first horizontal curve on the exit ramp.

2.1.2.2 Parallel-Type Exit

A parallel-type exit terminal, as shown in Figure 2, generally begins with a taper, followed by an added lane that is parallel to the traveled way. The length of a parallel-type deceleration lane is measured from where the added lane attains a 12-ft width to the point where the alignment of the ramp roadway departs from the alignment of the freeway. Transition may be provided at the end of the deceleration lane if the ramp proper is curved. The taper portion of a parallel-type deceleration lane should have a taper of approximately 15:1 to 25:1.

Green Book Exhibit 10-73 provides recommended minimum deceleration lane lengths for various combinations of design conditions for both taper- and parallel-type exit terminals. Where grades are present on ramps, minimum recommended deceleration lane lengths ideally will be adjusted in accordance to *Green Book* Exhibit 10-71.

Key elements of the AASHTO *Green Book* model are described below.

2.1.3 Design Speed

Design speed is the selected speed used to determine the various geometric design features of a roadway and should be consistent with the topography, anticipated operating speed, adjacent land use, and functional classification of the roadway. In some situations, AASHTO policy uses a speed other than design speed to determine the various geometric design features. In the case of freeway mainline ramp terminals, minimum acceleration and deceleration lane lengths are based upon operating speeds along the freeway and the entrance/exit curve of the ramp. In the case of acceleration lanes, AASHTO policy assumes that a vehicle merges onto the freeway at a speed of 5 mi/h below the operating speed of the freeway.

For acceleration lanes, AASHTO policy notes that part of the ramp proper may be considered in the acceleration lane provided that the curve approaching the acceleration lane has a radius of approximately 1,000 ft or more. Similarly, with deceleration lanes, a portion of the ramp may be considered as part of the deceleration lane when the initial curve has a radius of 1,000 ft or more. Thus, AASHTO policy assumes that the operating speeds of vehicles on the ramp are affected by the horizontal alignment of the ramp when the curve radius is less than 1,000 ft.

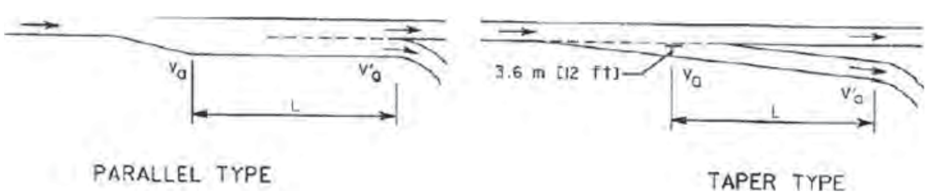


Figure 2. Parallel and tapered designs for exit ramps (AASHTO, 2004).

2.1.4 Sight Distance

Sight distance along a ramp should be at least as great as the design stopping sight distance. Sight distance for passing is not needed. 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.

2.1.5 Grade and Profile Design

A ramp typically consists of a ramp proper with an appreciable grade, while the ramp terminals (at both the crossroad and the freeway) generally have flatter grades. AASHTO guidelines indicate that it is desirable to limit upgrades based on the design speed of the ramp as follows:

- Design speed of 45 to 50 mi/h: 3 to 5 percent maximum upgrade,
- Design speed of 40 mi/h: 4 to 6 percent maximum upgrade,
- Design speed of 25 to 30 mi/h: 5 to 7 percent maximum upgrade, and
- Design speed of 15 to 25 mi/h: 6 to 8 percent maximum upgrade.

Ramp grades should be as flat as practical, but where topographic conditions dictate, 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.

2.1.6 Vertical Curves

Ramp profiles assume the shape of the letter "S" with a sag vertical curve at the lower end and a crest vertical curve at the upper end. If a crest or sag vertical curve extends onto the ramp terminal, the length of the curve should be determined by using a design speed between those on the ramp and highway.

2.1.7 Superelevation and Cross Slope

AASHTO has established limiting values of superelevations and friction factors for different design speeds on open roadways. It recommends the cross slope on portions of ramps on tangent normally to be sloped one way at a rate ranging from 1.5 to 2 percent for high-type pavements.

2.1.8 Gore Areas

At an exit terminal, the ramp "gore" is the area downstream from where the shoulder of the ramp and the shoulder of the freeway intersect. The physical nose is a point upstream from the gore, having some dimensional width, occurring at the separation of the roadways. The width of the gore nose is typically between 20 and 30 ft, including paved shoulders, measured between the traveled way of the main line and that of the ramp. In a series of interchanges along a freeway, the gores should be uniform and have the same appearance to drivers, and the entire triangular area should be striped to delineate the proper paths on each side. The gore area and the unpaved area beyond should be kept free of obstructions when possible to provide a clear recovery area. At an entrance terminal, the gore points downstream and is less of a decision area for drivers since the traffic streams are in separate lanes.

2.1.9 Ramp Traveled-Way Widths

Traveled-way widths are governed by the type of operation, curvature, and volume and type of traffic. Design widths of ramp traveled ways are classified into three general design traffic conditions:

- Traffic Condition A—mainly passenger vehicles, but some consideration to single-unit (SU) trucks.
- Traffic Condition B—SU vehicles to govern design and some consideration to semitrailer vehicles.
- Traffic Condition C—sufficient buses and combination trucks to govern design.

2.1.10 Left-Hand Entrances and Exits

Left-side ramp terminals break up the uniformity of interchange patterns and are also contrary to the concept of driver expectancy. Care should be exercised to avoid left-hand entrances and exits in the design of interchanges.

2.1.11 Ramp Terminal Profile

Ramp terminal profiles should be designed in association with horizontal curves to avoid sight restrictions that will adversely affect operations. It is desirable to design a platform, at least 200 ft in length, on the ramp side of the approach nose or merging end.

2.2 Geometric Design and Safety

Several studies investigated the relationship of geometric design elements and the safety performance of freeway mainline ramp terminals. Relevant findings are as follows:

- Exit ramps have higher crash rates than entrance ramps (Twomey et al., 1993; Khaoshadi, 1998).
- Ramps show increasing crash rates with increasing degrees of curvature (Twomey et al., 1993).

- One study found the safety of entrance terminals is enhanced when 800-ft or longer acceleration lanes are provided (Twomey et al., 1993). Other studies indicate that a minimum SCL length of 1,230 ft, including the taper, is desirable for comfortable merging and anything longer than 1,400 ft does not improve merging behavior (Hassan et al., 2006; Ahammed et al., 2006).
- Drivers tend not to begin the acceleration process until they have a clear view of freeway right-lane traffic (Hunter et al., 2001). Even if a ramp is designed to allow vehicles to begin acceleration prior to reaching the painted nose, drivers typically do not begin the acceleration phase until they have a clear view of the mainline traffic (Hunter and Machemehl, 1999).
- Entrance ramps with vertical profiles that limit ramp driver sight distance and had marginal SCL lengths exhibited significant ramp driver speed changes (Hunter et al., 2001).
- Speeds in the right lane of the freeway were affected where there was inadequate sight distance and acceleration distance was short. This was particularly noticeable at high freeway and ramp volumes (Hunter et al., 2001).
- Poor ramp geometry led to a more aggressive ramp merge beyond the painted nose (Hunter et al., 2001).
- One study (Eustace and Indupuru, 2008) suggested a constant acceleration rate of 2.5 ft/s^2 should be used in the design of acceleration lanes. Another study (Hunter and Machemehl, 1999) indicated that drivers accelerate from 0 to 2.9 ft/s^2 on ramps with adequate length compared to undulating acceleration rates from ±5.9 ft/s^2 on poorly designed ramps. Yet, another study (Hassan et al., 2006) concluded the 85th percentile maximum comfortable acceleration rate was 6.5 ft/s^2.
- Free-merge vehicles merge onto a freeway at arbitrary locations (Yi and Mulinazzi, 2007).
- Platooned merge vehicles follow a natural smooth path and force themselves into the mainline traffic within a certain area of a merge lane (Yi and Mulinazzi, 2007).
- For entrance ramps, a taper design causes vehicles to use a greater portion of the ramp than a parallel SCL of the same length (Fukutome and Moskowitz, 1960).
- More ramp length is used to accelerate at low volumes than at high volumes (Fukutome and Moskowitz, 1960).
- For ramps with adequate sight distance and SCL lengths, drivers typically have a smooth transition into the through-vehicle traffic. Conversely, ramps with vertical profiles limiting the driver's sight distance often cause abrupt speed changes and inhibit the flow of traffic (Hunter and Machemehl, 1999).
- At locations with adequate ramp design and acceleration lane length, drivers tend to traverse the entire length of the SCL before entering the through traffic, but at sites where inadequate design is provided, drivers tend to merge more aggressively and do not use the provided space for fear of being trapped at the end of the dropped lane (Hunter and Machemehl, 1999).
- Deceleration lanes of 900 ft or more reduce traffic friction on the through lanes and account for reduced crash rates (Twomey et al., 1993).
- Taper designs allow the driver to clearly view the start of the deceleration lane (Garcia and Romero, 2006).
- Even with long deceleration lanes, drivers start to decelerate before exiting the mainline. A reduction of 10.5 mi/h on the mainline was observed (Garcia and Romero, 2006).
- Deceleration lane length is indirectly proportional to early exiting and directly proportional to overtaking. As the deceleration lane length decreases, overtaking reduces and early exits increase (Garcia and Romero, 2006).
- No relation was found between lane length and deceleration rate (Garcia and Romero, 2006).
- Longer deceleration lanes are more likely to reduce injury severity (Wang et al., 2009).
- When comparing four types of exit ramps (Type 1: parallel from a tangent single-lane exit ramp; Type 2: single-lane exit ramp without a taper; Type 3: two-lane exit ramp with an optional lane; and Type 4: two-lane exit ramp without an optional lane), the Type 1 exit ramp had the best safety performance in terms of lowest crash frequency and crash rate (Lu et al., 2010).

Key geometric variables identified to contribute to the safety performance of interchange ramps and SCLs include the following (Bauer and Harwood, 1998; Khaoshadi, 1998; Yi and Mulinazzi, 2007):

- Freeway volume,
- Ramp volume,
- Speed of right freeway lane,
- Area type (urban or rural),
- Ramp type (entrance or exit ramp),
- Ramp configuration,
- Length of ramp, and
- Length of SCL.

Several studies evaluated the adequacy of current policies for freeway-ramp design. Harwood and Mason (1993) concluded that current AASHTO policies for freeway-ramp design are adequate so long as drivers adjust their speeds to levels that are less than or equal to the design speed. Where problems are anticipated, geometric design changes may be appropriate in order to increase the ramp design speed or deceleration distance available to the driver. Koepke (1993) reviewed both taper and parallel entrance and exit design and concluded that current design practices are acceptable for today's driving conditions. Lomax and Fuhs (1993) reviewed the design practices of freeway entrance ramp meters and HOV bypass lanes and noted that the designs performed well operationally. Hunter et al. (2001) concluded that the AASHTO acceleration rate model used to estimate acceleration lane lengths should not be changed. Fitzpatrick and Zimmerman (2007) examined potential changes to the 2004 *Green Book's* adjustment factors for entrance and exit terminals. They recommended potential adjustment factors to acceleration and deceleration lengths for different grades. In a similar study, Fitzpatrick and Zimmerman (2007A) assessed that the acceleration lane lengths contained in the 2004 *Green Book* should potentially be increased.

2.3 Vehicle Performance

Vehicle performance characteristics affect the geometric design of roads. Truck performance characteristics, rather than those of passenger cars, often govern the design of a facility except when truck volume is extremely limited or it is cost prohibitive to design the facility based upon truck performance capabilities. Harwood et al. (2003) critiqued the design criteria for acceleration lane lengths from the 2001 AASHTO *Green Book.* In their critique, Harwood et al. (2003) assessed truck performance characteristics implied by the design criteria using a truck speed profile model (TSPM). Harwood et al. (2003) used the TSPM to calculate the minimum acceleration lengths required to enable a 180-lb/hp vehicle to reach the given conditions as specified in the current design criteria. These minimum acceleration lengths were, on average, about 1.8 times greater than the minimum acceleration lengths given in the 2004 *Green Book.* After analyzing speed/acceleration data of trucks near commercial vehicle weigh stations, Gattis et al. (2008) suggested that acceleration lane lengths of the same order of magnitude as those provided by Harwood et al. (2003) should be considered at locations where significant volumes of trucks enter a freeway.

In an unpublished NCHRP Project 3-35 (Reilly et al., 1989) report, a methodology was developed for determining acceleration lane lengths based on human factors, traffic flow characteristics, and vehicle dynamics. The recommended minimum design lengths for acceleration lanes from this research, which are based on truck operations, are significantly longer than the AASHTO design criteria.

In their critique of the AASHTO 2001 design criteria for deceleration lane lengths, Harwood et al. (2003) note that as vehicles exit the freeway mainline, speed changes are normally made with controlled deceleration rates of which trucks are clearly capable, and there is no indication that driver choice of faster operating speeds is the result of short deceleration lanes or is correctable by using longer deceleration lanes. However, Ervin et al. (1986) recommended that trucks require deceleration lane lengths on the order of 30 to 50 percent longer than those in the *Green Book.* Firestine et al. (1989) also recommended that minimum deceleration lane lengths required to accommodate trucks are approximately 15 to 50 percent longer than those required for passenger cars.

2.4 Human Factors

Several studies investigated human factor issues related to the design and/or operation of freeway mainline ramp terminals. Michaels and Fazio (1989) proposed that the merge process is a four-step sequential process: initial steering control, acceleration, gap search, and merge steering; and indicated that these steps work in an iterative fashion until a successful merge is completed. Reilly et al. (1989) proposed the merging process consists of five sequential decision components: (1) steering control (SC) zone, which involves the steering and positioning of the vehicle along a path by steering from the controlling ramp curvature onto the SCL; (2) initial acceleration (IA) zone, in which the driver accelerates to reduce the speed differential between the ramp and freeway vehicles to an acceptable level for completing the merge process; (3) gap search and acceptance (GSA) zone, during which the driver searches, evaluates, and accepts or rejects the available lags or gaps in the traffic stream; (4) merge steering control (MSC) zone, during which the driver enters the freeway and positions the vehicle in Lane 1, followed by (5) visual clear (VC) zone, which provides a buffer between the driver and the end of acceleration lane. Figure 3 shows the five components described in the merging model suggested by Reilly et al. (1989).

In other research, Staplin et al. (1998) note, "age-diminished capabilities that contribute most to older drivers' difficulties at freeway interchanges include losses in vision and information-processing ability, and decreased flexibility in the neck and upper body." Lunenfeld (1993) indicated that the key human factors considerations in the design of interchanges and associated design features include sensory-motor attributes, visual capability of the driver, importance of sight distance, and meeting driver expectancy.

The role of congestion on the freeway lanes in drivers' merging behavior is not clear. Because speeds are lower in congested conditions, one may argue that the merging task is simpler; however, merging into congested situations may require more cognitive effort as the driver adjusts his/her own speed and repeatedly checks for gaps. In addition, in a congested situation, drivers must make more social judgments regarding the intentions of both lag vehicles and of mainline traffic. Sivak (2002) pointed out that driver gap acceptance behavior is not always rational. A driver may set a criterion for gap size that is safe and reasonable, but as the amount of time the driver waits to find that gap increases, the acceptable gap may decrease. Michaels and Fazio (1989) point out another important aspect of the merging process—that it is a collaborative effort between merging and mainline traffic. Sarvi et al. (2002) note that if no gap is

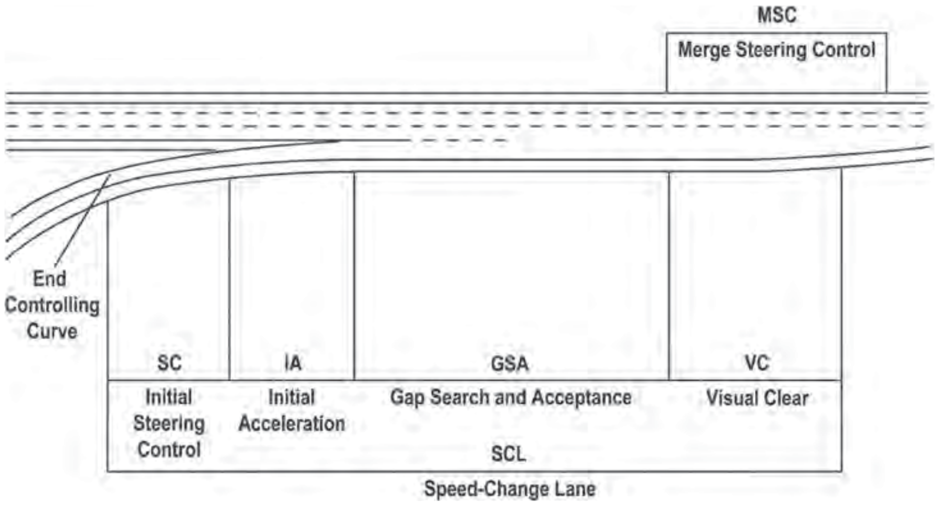


Figure 3. NCHRP Project 3-35 entrance model (Reilly et al., 1989).

available, drivers may accelerate to create a gap or decelerate to wait for a later gap. Choudhury et al. (2009) note that a driver who decides to "force-in," for instance, is likely to accept smaller gaps and accelerate to facilitate the merge and developed a merge model based upon merging plan choice, gap acceptance, target gap selection, and acceleration decisions of drivers. Kondyli and Elefteriadou (2009) indicate that a driver's decision to initiate a forced merge depends on traffic-related factors such as freeway speed, congestion, and gap availability, and in congested conditions, driver behavior displays less variability.

2.5 Operational Issues at Freeway Mainline Ramp Terminals

The 2000 *Highway Capacity Manual* (HCM) (TRB, 2000) provides a methodology to assess the level-of-service (LOS) at ramp-freeway junctions. Three major components of the methodology for uncontrolled ramp-freeway junctions include:

- Vehicle flow in the two rightmost lanes of the freeway immediately downstream/upstream of the merge/diverge area.
- Likelihood of congestion based upon capacity values and demand flows (either existing or predicted). These capacity values include maximum total flow approaching a major diverge area on the freeway, maximum total flow departing from a merge or diverge area on the freeway, maximum total flow entering the ramp influence area, and maximum flow on a ramp.
- Determine LOS based on the density of flow in the ramp influence area and estimate average vehicle speed within the influence area.

Models of this methodology address the design of entrance and exit ramps with various lane configurations for the mainline and ramp junctions on both left- and right-hand sides of the freeway. The ramp influence area is defined as the area 1,500-ft downstream of the painted nose of an entrance ramp and 1,500-ft upstream of the painted nose of an exit ramp and includes the acceleration or deceleration lane and Lanes 1 and 2 of the freeway.

The capacity of merging areas depends primarily on the capacity of the downstream mainline segment. The HCM indicates that neither the traffic turbulence created in the merge area nor the number of acceleration lanes affects this capacity, but that it is based solely upon the sum of the upstream mainline and entrance ramp traffic volumes. When predicting flows for Lanes 1 and 2, the HCM states that longer acceleration lanes encourage less turbulence as ramp vehicles enter the freeway traffic stream and therefore lead to lower densities in the influence area and higher flows in Lanes 1 and 2. When an entrance ramp has a higher free-flow speed, vehicles tend to enter the freeway at higher speeds, and approaching freeway vehicles tend to move farther left to avoid the possibility of high-speed turbulence.

When determining the LOS of the ramp-freeway terminal in a merge area, acceleration lane length is an input variable used by the model to determine the density of the influence area. Increasing the acceleration lane length decreases the density of the merge influence area. This calculated density is then used to establish a LOS value. The length of the acceleration lane is also used in the model to determine the average speeds in the vicinity of freeway-ramp terminals.

For exit ramps, failure at a diverge area is often related to the capacity of one of the exit legs, most often the ramp. When determining the LOS for a diverge area, deceleration lane length is an input variable for the model to determine the density of the influence area, and increasing the deceleration lane length decreases the density of the diverge influence area.

In other research, Polus et al. (1985) conclude that SCLs are used as much for gap location recognition as they are for vehicle acceleration. Lighter vehicles merge much sooner than heavy trucks, and average speed differences between the mainline traffic and entering traffic at the end of the acceleration lane range from 6.5 to 10.2 mi/h. Abella et al. (1976) found a difference of 5 mi/h between the speed of the mainline traffic and merging vehicles.

2.6 Summary

Some general findings from the research highlight the gaps in knowledge but also provide direction. It is clear that under certain circumstances, current design policy does not sufficiently provide for the needs of all drivers and all vehicle types. The merging and diverging tasks are complex and are difficult to accommodate within design, especially given changing driver populations and vehicle fleets. On the other hand, there is only limited evidence that current design policy values produce substantive safety concerns for the normal range of conditions.

Vehicle performance characteristics form the basis for the geometric design of roadways. Although research indicates the potential need to increase acceleration lane length to accommodate heavy trucks, no research was found to indicate that trucks had difficulty with acceleration lanes designed by the current criteria.

Attempts to model the complex driver/vehicle actions associated with the driving task (and in this case, the entry or exit maneuver) invariably require a series of simplifying assumptions or assumed performance characteristics. Many models describe the task as a sequential series of events. When all such models are assembled, they may result in findings that appear logical, but produce design dimensions that appear unreasonable, doubtful, or overly conservative.

AASHTO's current geometric design guidance is based on a single set of operational assumptions that can be characterized as representing free or unconstrained merging behavior. Driver behavior may vary, depending on whether the mainline is congested or not.

SECTION 3

AASHTO Models for Freeway Entrance and Exit Terminals

This section presents background information on the conceptual models utilized by AASHTO in determining minimum recommended acceleration and deceleration lane lengths for entrance and exit terminals, respectively. The focus of this research is on evaluating the appropriateness of the existing AASHTO models given current driver behavior and vehicle performance characteristics.

3.1 Freeway Mainline Entrance Terminals

At least since the 1965 *Blue Book,* and still in the 2004 *Green Book,* AASHO/AASHTO policies have, in principle, used a simple kinematic equation (i.e., Equation 1) with constant acceleration to calculate minimum acceleration lane lengths for freeway mainline entrance terminals. The minimum acceleration lane length is based on a combination of the following:

A. The speeds at which drivers enter the acceleration lane (i.e., the speed at the end of the ramp's controlling feature).
B. The manner of acceleration.
C. The speeds at which drivers merge with through traffic (i.e., the speed at the end of the acceleration lane).

These three components are used in the following equation to determine the recommended length of the acceleration lane:

$$L_{Acc} = \frac{(1.47V_m)^2 - (1.47V_r)^2}{2a} \quad (1)$$

where: L_{Acc} = Acceleration lane length, ft
V_m = Merge speed, mi/h
V_r = Initial speed on ramp (or speed after exiting controlling feature), mi/h
a = Acceleration rate, ft/s^2

The 1965 *Blue Book* provides the most detailed information on how the current design values in the 2004 *Green Book* are determined, but it should be noted that for some combinations, the 2004 *Green Book* acceleration lane lengths differ slightly from the 1965 *Blue Book* values.

From the 1965 *Blue Book,* the following points are noted concerning the three primary inputs:

- For Point A, the 1965 *Blue Book* assumes that drivers will exit the curve at an "average running speed" that is less than the curve's design speed. Because a longer acceleration length is needed to bring the vehicle's speed up to the merging speed, this is a conservative assumption.
- For Point B, the 1965 *Blue Book* provides graphs that show acceleration rates for different conditions. The curve used to generate the 1965 *Blue Book* acceleration rates was for "normal acceleration" for passenger vehicles on level grade as determined in a 1937 Bureau of Public Roads study. Per the 1965 *Blue Book,* the data were converted to show "distance traveled while accelerating from one to another." The resulting curves could be used to determine the acceleration distance from initial speed values to speed reached values, although the 1965 *Blue Book* also included these values.
- For Point C, the 1965 *Blue Book* states that the speed of the entering vehicles should approximate that of the through traffic which would be equal to the average running speed of traffic on the highway. Later, the 1965 *Blue Book* states "it is satisfactory and does not unduly inconvenience through traffic for vehicles from the acceleration lane to enter the through pavement at a speed approximately 5 mi/h less."

Table 1 presents the minimum acceleration lane lengths for entrance terminals from *Green Book* Exhibit 10-70, for flat grades of 2 percent or less. Table 1 also presents the accelera-

Table 1. 2004 *Green Book* minimum acceleration lane length values and corresponding acceleration rates (adapted from AASHTO, 2004).

Design speed (mi/h)	Speed reached (mi/h)	Acceleration length, L (ft) for entrance curve design speed (mi/h)								
		Stop	15	20	25	30	35	40	45	50
		Initial speed (mi/h)								
		0	14	18	22	26	30	36	40	44
2004 *Green Book*										
30	23	180	140	–	–	–	–	–	–	–
35	27	280	220	160	–	–	–	–	–	–
40	31	360	300	270	210	120	–	–	–	–
45	35	560	490	440	380	280	160	–	–	–
50	39	720	660	610	550	450	350	130	–	–
55	43	960	900	810	780	670	550	320	150	–
60	47	1,200	1,140	1,100	1,020	910	800	550	420	180
65	50	1,410	1,350	1,310	1,220	1,120	1000	770	600	370
70	53	1,620	1,560	1,520	1,420	1,350	1230	1,000	820	580
75	55	1,790	1,730	1,630	1,580	1,510	1,420	1,160	1,040	780
Acceleration Rates (ft/s^2) Used to Reproduce Acceleration Lengths										
30	23	3.18	2.57	–	–	–	–	–	–	–
35	27	2.81	2.62	2.73	–	–	–	–	–	–
40	31	2.88	2.76	2.55	2.45	2.57	–	–	–	–
45	35	2.36	2.27	2.21	2.11	2.12	2.19	–	–	–
50	39	2.28	2.17	2.12	2.04	2.03	1.92	1.87	–	–
55	43	2.08	1.98	2.03	1.89	1.89	1.86	1.87	1.79	–
60	47	1.99	1.91	1.85	1.83	1.82	1.77	1.79	1.57	1.64
65	50	1.92	1.84	1.79	1.79	1.76	1.73	1.69	1.62	1.65
70	53	1.87	1.81	1.77	1.77	1.71	1.68	1.63	1.59	1.63
75	55	1.83	1.77	1.79	1.74	1.68	1.62	1.61	1.48	1.51

tion rates used to generate those values, based on Equation 1. Overall, the pattern is a decreasing acceleration rate as the initial speed increases, but there are situations when this overall pattern is not followed.

Exhibit 10-71 in the 2004 *Green Book* includes adjustment factors for when the acceleration lane occurs on a grade of 3 to 4 percent or 5 to 6 percent (see Table 2). Similar values were included in the 1954 AASHO *Policies on Geometric Highway Design* (an earlier version of the AASHO *Blue Book*). Given the similarities between the values in the 2004 and 1954 editions, it appears that the source of the adjustment factors in the 2004 *Green Book* could be the values in the 1954 *Blue Book*. The source of the adjustment factors per the 1954 *Blue Book* was to apply principles of mechanics to rates of speed change for level grades. The direct quote from the 1954 *Blue Book* follows:

> Deceleration distances are longer on downgrades and shorter on upgrades, while acceleration distances are longer on upgrades and shorter on downgrades. Data on driver behavior while decelerating or accelerating on grades are not available, but they may be approximated by applying principles of mechanics to rates of speed change for level grades, recognizing that drivers accelerating on upgrades open throttles more than the equivalent for normal acceleration on level grades. Calculations result in lengths of acceleration and deceleration lanes on grades as compared with those on the level . . . The ratio . . . multiplied by the length (on level) . . . gives the length of speed-change lane on grade.

3.2 Freeway Mainline Exit Terminals

At least since the 1965 *Blue Book*, and still in the 2004 *Green Book*, AASHO/AASHTO policies have used a basic two-step process for establishing design criteria for exit ramps. Deceleration is accomplished first as the driver removes his or her foot from the accelerator pedal and the vehicle slows in gear for a period of time (assumed to be 3 s) without the use of brakes, and then as the driver applies the brakes and decelerates at a comfortable rate. Equations 2 and 3 are used to represent this dual process. The minimum deceleration lane length is based on the combination of the following inputs:

A. The speed at which drivers maneuver onto the auxiliary lane.
B. The speed at which drivers turn after traversing the deceleration lane.
C. The manner of deceleration.

$$L_{Decel} = 1.47V_h t_n - 0.5 d_n \left(t_n\right)^2 + \frac{\left(1.47V_r\right)^2 - \left(1.47V_a\right)^2}{2d_{wb}} \quad (2)$$

$$V_a = \frac{1.47V_h + d_n t_n}{1.47} \quad (3)$$

where: L_{Decel} = Deceleration lane length, ft
V_h = Highway speed, mi/h

Table 2. SCL adjustment factors as a function of grade (AASHTO, 2004).

Deceleration Lanes		
Design speed of highway (mi/h)	**Ratio of length on grade to length on level for design speed of turning curve (mi/h)***	
	3 to 4% Upgrade	**3 to 4% Downgrade**
All speeds	0.9	1.2
	5 to 6% Upgrade	**5 to 6% Downgrade**
All speeds	0.8	1.35

Acceleration Lanes					
Design speed of highway (mi/h)	**Ratio of length on grade to length on level for design speed of turning curve (mi/h)***				
	20	**30**	**40**	**50**	**All speeds**
	3 to 4% Upgrade				**3 to 4% Downgrade**
40	1.3	1.3	–	–	0.7
45	1.3	1.35	–	–	0.675
50	1.3	1.4	1.4	–	0.65
55	1.35	1.45	1.45	–	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
	5 to 6% Upgrade				**5 to 6% Downgrade**
40	1.5	1.5	–	–	0.6
45	1.5	1.6	–	–	0.575
50	1.5	1.7	1.9	–	0.55
55	1.6	1.8	2.05	–	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5

*Ratio in this table multiplied by length of acceleration/deceleration distances gives length of acceleration/deceleration distance on grade.

V_a = Speed after t_n s of deceleration without brakes, mi/h

V_r = Entering speed for controlling exit ramp curve, mi/h

t_n = Deceleration time without brakes (assumed to be 3 s), s

d_n = Deceleration rate without brakes, ft/s^2

d_{wb} = Deceleration rate with brakes, ft/s^2

The 1965 *Blue Book* provides the most detailed information on how the current design values in the 2004 *Green Book* are determined, but it should be noted that the 2004 *Green Book* deceleration lane lengths differ slightly from the 1965 *Blue Book* values. From the 1965 *Blue Book,* the following points are noted concerning the three primary inputs:

- For Point A, the 1965 *Blue Book* states that "most drivers travel at a speed not greater than the average running speed of the highway." For example, on a freeway with a 70 mi/h design speed, the assumption is that a driver will enter the auxiliary lane at 58 mi/h.
- For Point B, the 1965 *Blue Book* provides assumed average running speeds for the ramp, based on the design speed of the limiting curve on the ramp. For example, a ramp with a 40 mi/h curve design speed, the speed reached at the end of the deceleration length is assumed to be 36 mi/h.
- The values for points A and B are clearly provided in the 1965 *Blue Book.* However, for Point C—the manner of decelerating or the deceleration factors—the values are not as clear. The 1965 *Blue Book* states that deceleration is a two-step process: first, the accelerator pedal is released (assumed for 3 s) and the vehicle slows in gear without the use of brakes and second, the brakes are applied. Two graphs are included in the 1965 *Blue Book* (*Blue Book* Figure VII-15) to provide these distances. The graphs were based on data from studies conducted in the 1930s.

The 1965 *Blue Book* also states:

> "A comfortable overall rate of deceleration while braking from 70 to a complete stop has been found to be about 6.2 mi/h per second (9 ft per second) . . . In applying this rate at approaches to intersections, it is logical to assume that it decreases as the approach speed is lowered in a manner similar to that found in approaching a stop sign. Accordingly, the overall deceleration rate is assumed to vary from 6.2 mi/h per second (f = 0.28) for initial speed of 70 to 4 mi/h per second (f = 0.18) for initial speed of 30 mi/h."

Table 3 provides the deceleration rates for the recommended minimum deceleration lane lengths as provided in *Green Book*

Table 3. 2004 *Green Book* minimum deceleration lane length values and corresponding deceleration rates (adapted From AASHTO, 2004).

Highway design speed (mi/h)	Speed reached (mi/h)	Deceleration length, L (ft) for design speed of exit curve (mi/h)								
		Stop	15	20	25	30	35	40	45	50
		For average running speed on exit curve (mi/h)								
		0	14	18	22	26	30	36	40	44
2004 *Green Book*										
30	28	235	200	170	140	–	–	–	–	–
35	32	280	250	210	185	150	–	–	–	–
40	36	320	295	265	235	185	155	–	–	–
45	40	385	350	325	295	250	220	–	–	–
50	44	435	405	385	355	315	285	225	175	–
55	48	480	455	440	410	380	350	285	235	–
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390
1st Deceleration Rates (ft/s^2) While Coasting in Gear Used to Reproduce Deceleration Lane Lengths										
30	28	–1.04	–1.04	–1.04	–1.04	–	–	–	–	–
35	32	–1.53	–1.53	–1.53	–1.53	–1.53	–	-	–	–
40	36	–1.52	–1.52	–1.52	–1.52	–1.52	–1.52	–	–	–
45	40	–2.01	–2.01	–2.01	–2.01	–2.01	–2.01	–2.51	–	–
50	44	–2.51	–2.51	–2.51	–2.51	–2.51	–2.51	–2.01	–2.51	–
55	48	–2.01	–2.01	–2.01	–2.01	–2.01	–2.01	–2.98	–2.01	–
60	52	–2.98	–2.98	–2.98	–2.98	–2.98	–2.98	–2.50	–2.98	–2.98
65	55	–2.50	–2.50	–2.50	–2.50	–2.50	–2.50	–2.50	–2.50	–2.50
70	58	–2.50	–2.50	–2.50	–2.50	–2.50	–2.50	–2.99	–2.50	–2.50
75	61	–2.99	–2.99	–2.99	–2.99	–2.99	–2.99		–2.99	–2.99
2nd Deceleration Rates (ft/s^2) While Braking Used to Reproduce Deceleration Lane Lengths										
30	28	–5.75	–6.42	–5.49	–8.97	–	–	–	–	–
35	32	–5.83	–5.75	–5.38	–6.68	–10.29	–	–	–	–
40	36	–5.66	–5.22	–5.11	–5.66	–5.95	–4.42	–	–	–
45	40	–6.38	–6.05	–5.68	–6.91	–6.83	–6.18	–	–	–
50	44	–6.74	–6.57	–6.28	–7.20	–7.86	–6.55	–6.23	N/A	–
55	48	–7.10	–7.08	–6.86	–7.55	–7.86	–7.46	–7.49	–7.86	–
60	52	–7.07	–7.19	–6.99	–7.45	–7.70	–7.34	–6.98	–7.43	–16.84
65	55	–7.55	–7.43	–7.27	–8.03	–7.76	–7.49	–7.51	–7.66	–9.60
70	58	–7.40	–7.41	–7.27	–7.77	–7.53	–7.60	–7.45	–7.54	–8.17
75	61	–7.76	–8.02	–7.90	–8.06	–7.85	–7.65	–7.70	–7.62	–8.49

Exhibit 10-73, based upon the assumptions described above. The overall pattern shows higher rates of deceleration for higher speeds. (Note: working down several of the columns in Table 3, some of the initial deceleration rates during braking are inconsistent with other rates in the column. The inconsistencies are a result of scaling distance and speed information from graphs in the 1965 *Blue Book*.)

Similar to the design of acceleration lanes, the 2004 *Green Book* includes adjustment factors for when the deceleration lane occurs on a grade of 3 to 4 percent or 5 to 6 percent (see Table 2). The adjustment factors are based upon principles of mechanics to rates of speed change for level grades.

For comparison purposes, Table 4 provides deceleration rates for the recommended minimum deceleration lane lengths as provided in *Green Book* Exhibit 10-73, assuming a constant rate of deceleration from the average running speed on the freeway to the average running speed entering the controlling exit ramp curve. The deceleration rates in Table 4 are less than the deceleration rates while braking provided in Table 3.

3.3 Operational Assumptions of AASHTO Models

AASHTO policy for the geometric design guidance of freeway mainline ramp terminals is based on a single set of operational assumptions that can be characterized as representing free or unconstrained merging behavior. Acceleration lane lengths are based on vehicle capabilities, driver comfort, and simplifying assumptions regarding initial speed and freeway operating speed of the merge. One could characterize this operational model as being reflective of rural, high-speed, low-density conditions.

Perhaps the greatest apparent need is to explicitly consider design under a range of freeway density, speed, and gap conditions. It is now common practice for agencies to reconstruct urban freeways, or construct new interchanges with design-year traffic and conditions no better than LOS E. As such, the designer intends that traffic during the design hour will merge into a traffic stream with limited gaps. Such gaps may occur

Table 4. Corresponding deceleration rates for 2004 *Green Book* minimum deceleration lane length values assuming constant deceleration.

Highway design speed (mi/h)	Speed reached (mi/h)	Deceleration (ft/s^2) for design speed of exit curve (mi/h)								
		Stop	15	20	25	30	35	40	45	50
		For average running speed on exit curve (mi/h)								
		0	14	18	22	26	30	36	40	44
2004 *Green Book*										
30	28	–3.59	–3.16	–2.91	–2.30	–	–	–	–	–
35	32	–3.93	–3.56	–3.59	–3.14	–2.50	–	–	–	–
40	36	–4.36	–4.01	–3.95	–3.72	–3.60	–2.75	–	–	–
45	40	–4.47	–4.31	–4.22	–4.07	–3.98	–3.42	–	–	–
50	44	–4.79	–4.62	–4.50	–4.40	–4.30	–3.91	–3.06	–2.07	–
55	48	–5.16	–4.98	–4.84	–4.77	–4.61	–4.31	–3.80	–3.22	–
60	52	–5.49	–5.39	–5.33	–5.19	–5.07	–4.79	–4.33	–3.96	–3.44
65	55	–5.71	–5.63	–5.59	–5.47	–5.38	–5.19	–4.77	–4.51	–4.18
70	58	–5.88	–5.78	–5.74	–5.63	–5.56	–5.41	–5.06	–4.86	–4.52
75	61	–6.06	–5.97	–5.89	–5.80	–5.70	–5.67	–5.32	–5.18	–4.92

randomly, and may not occur when the entering driver is prepared to merge. As a result, drivers routinely are faced with judging gaps, adjusting their speeds, and often aborting their merge or producing extreme acceleration and/or speed behavior. The current AASHTO model does not provide for such behavior, but it is less clear whether the dimensions the model produces can accommodate extreme acceleration or speeds. There may be circumstances where it does, but also some where it does not.

Ideally, a robust ramp merging model will reflect, within reason, the above design expectation. The following conditions represent potential traffic operational scenarios, any one of which may control the geometric design of a given entrance ramp:

Condition I—Free Merge at LOS A to C. This traffic condition is consistent with current AASHTO design assumptions and approaches. At high LOS, gaps are frequent and well in excess of the minimum acceptable gap. Merging speeds may vary to reflect the range in AASHTO mainline operating speeds (50 to 70 mi/h dependent on the context). Traffic operations entering the freeway are based on vehicle performance capabilities and driver comfort. The "task" of navigation and merge is relatively simple. The driver workload may be so simple as to allow driving tasks to be shared or overlapped as long as the three-dimensional geometry provides sufficient sight lines. In other words, drivers can search for a gap and comfortably accelerate at the same time.

Condition II—Constrained Merge at LOS D to E. This traffic condition is common in urban areas with high-volume freeways. Freeway operating speeds may still be relatively high (e.g., 40 to 50 mi/h), but long gaps are non-existent, median gaps are shorter, and acceptable gaps may be less frequent than merging vehicle arrivals. The merging task is thus inherently more complex. Drivers may vary their acceleration behavior, slowing down to wait for a gap or speeding up to catch a gap before upstream traffic reaches the painted nose. Gap acceptance behavior would appear most significant in designing for this condition.

Condition III—Forced Merge at LOS E to F. When mainline traffic is dense in Lane 1 and overall operating speeds are below 40 mi/h, the entire merging maneuver is significantly different from the previous two conditions. Drivers can easily adjust to the mainline speed because mainline traffic is typically moving slowly, but they may need considerable length to find or "negotiate" a gap for merging.

Consideration of a conceptual merging model in terms of these three operating conditions would acknowledge the need for a ramp to serve a range of conditions varying throughout a day. Correspondingly, it may be desirable to evaluate a conceptual diverging model in terms of these three operating conditions, although the diverge maneuver should be less critical under constrained and forced conditions, since vehicles will be entering the deceleration lane at reduced speeds.

SECTION 4

Analysis of Truck-Related Crashes Near Freeway Mainline Ramp Terminals

Passenger cars are currently the primary design vehicle for freeway mainline ramp terminals. However, many freeway mainline ramp terminals serve a considerable amount of truck traffic and, given the difference in performance capabilities between passenger cars and trucks, a primary question to be answered is whether freeway mainline ramp terminal design should consider the performance characteristics of heavy vehicles. For example, several studies (Reilly et al., 1989; Ervin et al., 1986; Firestine et al., 1989; and Gattis et al., 2008) recommend longer acceleration/deceleration lane lengths to better accommodate heavy vehicles.

No safety study has been conducted to indicate whether trucks have difficulty using freeway ramps designed according to the current criteria. Therefore, a crash analysis was conducted to determine if there are specific truck-related safety problems near freeway mainline ramp terminals. If specific truck-related safety problems were identified near freeway mainline ramp terminals, then an argument could be made that a truck rather than a passenger car should be the design vehicle for an entrance or exit ramp (at least in some instances). On the other hand, if crash data were to indicate that truck crashes are not over-represented near freeway mainline ramp terminals, then continuing to design freeway mainline ramp terminals based upon the performance characteristics of passenger cars may be justified.

4.1 Overview of Crash Analysis

An exploratory crash analysis was conducted to investigate the vehicle types involved in crashes near freeway mainline ramp terminals and other related issues. The crash analysis was conducted using the Highway Safety Information System (HSIS) database for the state of Washington. Supplemental interchange characteristic data were also obtained by reviewing interchange diagrams. The Washington database was used because it has a relatively complete set of ramp inventory data, including information on acceleration and deceleration lanes. In addition, information was available in the database to assign the location of a crash occurring near a freeway mainline ramp terminal to a specific area, such as the acceleration lane, deceleration lane, ramp proper, or freeway mainline adjacent to the acceleration/deceleration lane. Other databases were also considered but lacked the level of detail necessary for such an analysis.

The scope of the investigation was limited to interchange areas including the ramp proper, acceleration lanes, deceleration lanes, and the adjoining mainline roadway segments. Because the initial inventory data contained all ramps for interchanges occurring on state routes, study sites were screened to exclude ramps connecting to auxiliary lanes (e.g., collector/distributor roads and parallel frontage roads), ramps with left exits/entrances, and ramps that become additional lanes on the freeway. Ramps containing two-way sections and weaving areas were also excluded.

Three years (2002 to 2004) of traffic volume and crash history data were obtained. The geometric and traffic volume variables of interest for each interchange area included:

- Annual average daily traffic (AADT),
- Annual truck volume percent,
- Length,
- Interchange configuration, and
- Type of ramp (entrance/exit).

Criteria were developed to assign crashes to the study locations. Crashes that occurred within the termini locations of the sites that were identified as "non-intersection and non-intersection related" were included in the analysis. Additionally, truck crashes were identified if Vehicle 1 or Vehicle 2 was classified as one of the following:

- Flatbed truck,
- Truck over 10K,
- Truck tractor,
- Truck tractor and semitrailer, and
- Other truck combination.

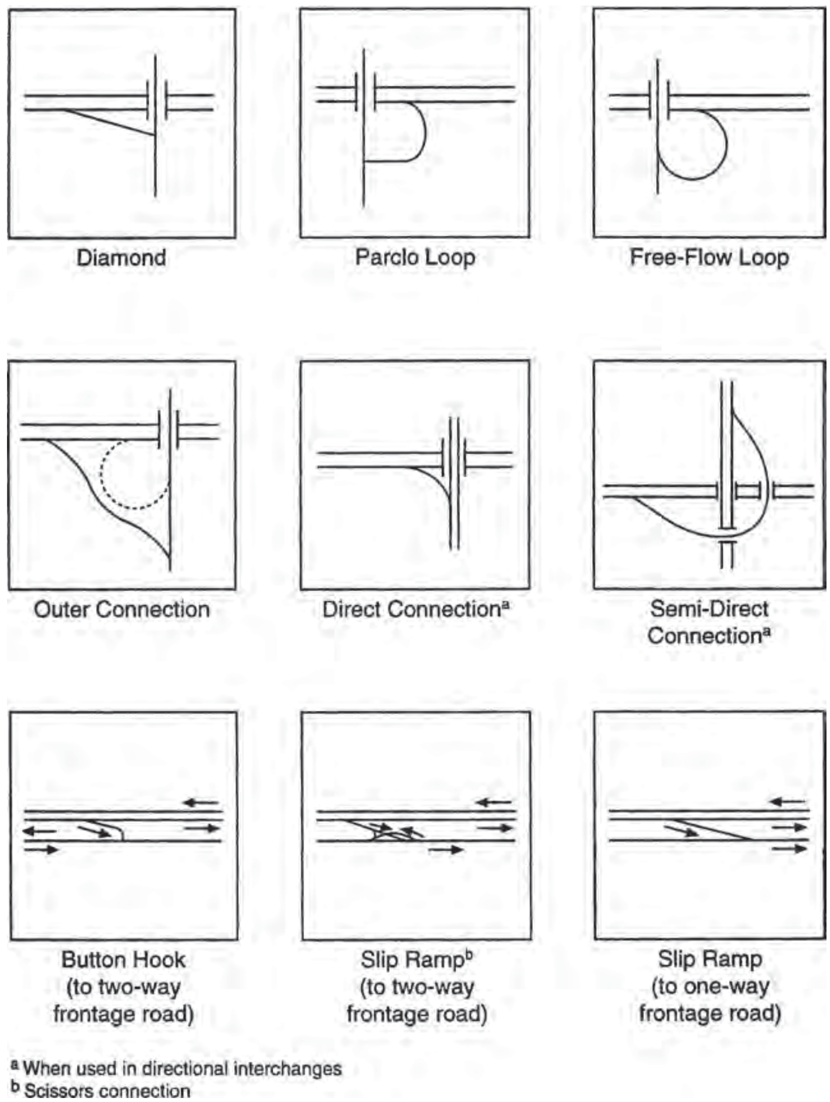


Figure 4. Typical ramp configurations (Bauer and Harwood, 1998).

In the analysis, ramps were grouped into seven configurations:

- Diamond,
- Parclo loop,
- Free-flow loop,
- Outer connection,
- Direct or semi-direct connection,
- Button hook, scissor, and slip ramps, and
- Others.

Figure 4 illustrates the typical ramp configurations included in the seven groupings used in this analysis. Table 5 presents the number of ramps included in the analysis by configuration type and by entrance or exit ramp. Data for 525 entrance ramps and 550 exit ramps were included in the database. The majority of

Table 5. Number of ramps by configuration.

	Diamond	Parclo loop	Free-flow loop	Outer connection	Direct or semi-direct connection	Button hook, scissor, and slip ramps	Other	Total
Entrance ramps	316	24	21	29	34	20	81	525
Exit ramps	329	18	17	47	41	14	84	550

Table 6. Ramp mileage by ramp configuration.

	Diamond	Parclo loop	Free-flow loop	Outer connection	Direct or semi-direct connection	Button hook, scissor, and slip ramps	Other	Total
Entrance ramps	**Total length (mi)**							
Acceleration lane	76.07	5.25	3.12	6.65	5.74	3.88	14.86	115.57
Freeway mainline	76.07	5.25	3.12	6.65	5.57	3.88	14.86	115.40
Ramp proper	55.61	1.93	3.85	5.38	3.11	2.21	13.16	85.25
Total	207.75	12.43	10.09	18.68	14.42	9.97	42.88	316.22
Exit ramps	**Total length (mi)**							
Deceleration lane	34.26	1.90	1.43	4.65	4.20	1.48	9.87	57.79
Freeway mainline	34.26	1.90	1.43	4.65	4.25	1.48	9.87	57.84
Ramp proper	60.10	1.99	3.14	9.76	7.73	2.22	12.84	97.78
Total	128.62	5.79	6.00	19.06	16.18	5.18	32.58	213.41

ramps were of the diamond type. Table 6 shows the overall mileage of the ramps and roadway segments included the analysis. The mileage is divided by freeway mainline ramp terminal location (i.e., acceleration/deceleration lane, ramp proper, and freeway mainline adjacent to acceleration/deceleration lane). Figure 5 illustrates the three locations that accidents were assigned near the freeway mainline ramp terminals.

4.2 Analysis Results

The crash analysis results focus on comparing crash frequencies/rates for all vehicle types combined (i.e., total crashes) to crash frequencies/rates for heavy vehicles (i.e., trucks). Table 7 shows total crash frequencies and truck-related crash frequencies by ramp configuration and crash location (i.e., acceleration or deceleration lane, freeway mainline adjacent to acceleration/deceleration lane, or ramp proper). Table 7 shows that 38 percent of the total crashes (4,049) occurred at or near diamond ramps and 35 percent (3,907) occurred near "other" ramps. Similar percentages hold for truck-related crashes; 38 percent of truck-related crashes (407) occurred near diamond ramps, while 35 percent (379) occurred near "other" ramps. These frequencies and percentages could be somewhat misleading because 645 of the 1,075 ramps included in the analysis were diamond ramps and 165 ramps were classified as "other" (see Table 5). The next most common ramp type was outer connections, with 75 included in the database, so it is expected that the greatest number of total and truck-related crashes occurred on the two most common type of ramp configurations. Exposure levels are not considered in Table 6. It can also be reasoned that by definition the "other" ramp type is not a typical ramp configuration or design so these ramps could go against driver expectation.

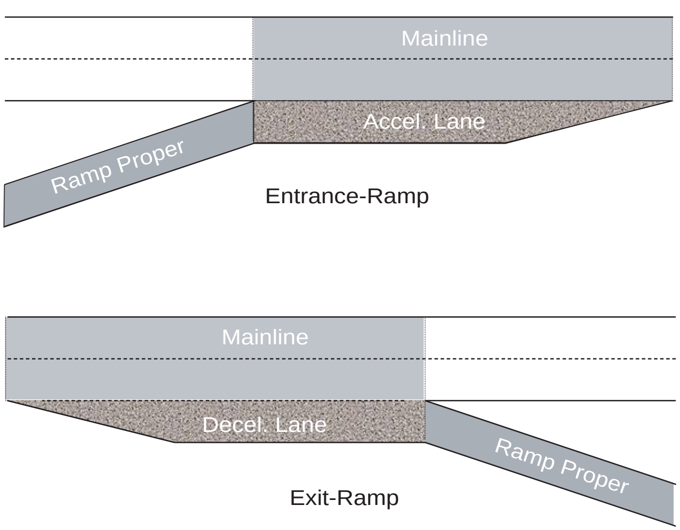


Figure 5. Locations of crashes assigned to entrance and exit ramps.

Without looking at exposure levels for Table 7, but based upon the number of ramps in the database, it appears that operations near entrance ramps experience a greater number of crashes than operations near exit ramps. The following crash frequencies were calculated per ramp for the 3-year analysis period:

- 14.33 total crashes per entrance ramp (4.77 total crashes per site per year),
- 1.39 truck crashes per entrance ramp (0.46 truck crashes per site per year),
- 6.46 total crashes per exit ramp (2.15 total crashes per site per year), and
- 0.64 truck crashes per exit ramp (0.21 truck crashes per site per year).

Table 7 also indicates the location in the vicinity of the freeway mainline ramp terminal where the crashes occurred. For entrance ramps, 87 percent of the total 7,522 crashes were assigned to the mainline, and 93 percent of the 731 truck-related crashes were assigned to the mainline. The smallest proportion of entrance ramp crashes was assigned to the acceleration lane (94 total crashes and 6 truck-related crashes). For any given ramp configuration, the highest proportion of crashes assigned to the acceleration lane was 3 to 4 percent

Table 7. Total crashes and truck crashes by ramp configuration and location.

Location	Ramp configuration														Grand total		
	Diamond		Parclo loop		Free-flow loop		Outer connection		Direct or semi-direct connection		Button hook, scissor, and slip ramps		Others				
	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Truck percent
Entrance ramps																	
Accel. lane	28	1	3	1	3	0	7	0	16	1	2	0	35	3	94	6	6.38
Freeway mainline	2,808	297	445	38	196	11	308	42	355	36	541	50	1,895	208	6,548	682	10.42
Ramp proper	189	6	13	1	55	7	23	1	45	3	28	2	527	23	880	43	4.89
Total	3,025	304	461	40	254	18	338	43	416	40	571	52	2,457	234	7,522	731	9.72
Exit ramps																	
Decel. lane	59	2	2	0	8	0	22	3	41	6	1	0	119	10	252	21	8.33
Freeway mainline	696	77	30	6	116	14	222	16	223	22	75	9	1,169	123	2,531	267	10.55
Ramp proper	269	24	11	2	67	4	108	10	130	10	25	3	162	12	772	65	8.42
Total	1,024	103	43	8	191	18	352	29	394	38	101	12	1,450	145	3,555	353	9.93
Grand total	4,049	407	504	48	445	36	690	72	810	78	672	64	3,907	379	11,077	1,084	9.79

Note: Accel. = acceleration, Decel. = deceleration.

for direct or semi-direct connections. It is interesting that the entrance ramp configuration with the greatest proportion of crashes occurring on the ramp proper is the free-flow loop ramp (22 percent of total crashes and 39 percent of truck crashes). This can be explained possibly by the need to negotiate changes in horizontal alignment, while also possibly accelerating along the ramp. For all entrance ramp configurations combined, 12 percent of total crashes and 6 percent of truck crashes occurred on the ramp proper. Of the 682 truck crashes that occurred on the freeway mainline, it could not be distinguished from the database whether the truck involved in the crash was merging into freeway traffic from the acceleration lane or whether the truck was operating on the freeway mainline and another vehicle was merging onto the freeway.

For the distribution of crashes assigned to the three locations along exit ramps, the majority of crashes were assigned to the freeway mainline (71 percent of total crashes and 76 percent of truck crashes); however, for both total and truck-related crashes, the percentages were lower compared to percentages of crashes assigned to the freeway mainline for entrance ramps. Very few crashes occurred on the deceleration lanes of exit ramps (7 percent of total crashes and 6 percent of truck crashes). Finally, a greater percentage of crashes occurred on the ramp proper of exit ramps (22 percent of total crashes and 18 percent of truck crashes) compared to the ramp proper of entrance ramps. This distribution is logical considering that a significant portion of crashes would be expected along the mainline as vehicles change lanes in an effort to diverge. Also, once on the deceleration lane, few crashes would be expected unless a leading vehicle decelerates at a greater rate than expected or a queue backs up into the deceleration lane (in which case this is not necessarily a deceleration lane problem but more of a storage problem). Finally, the high number of crashes on the ramp proper of exit ramps could be the result of vehicles traveling too fast to negotiate the ramp alignment or the interaction between leading and following vehicles.

Table 7 also indicates that trucks were involved in approximately 10 percent of the total crashes at both freeway mainline ramp terminal entrance and exit ramps. Interestingly, Golob and Regan (2003) conducted a safety evaluation of truck operations along six major urban freeways in Orange County, California. The analysis included a total of approximately 131 centerline miles. Of the 19,202 mainline crashes of these routes in 2000 through 2001, 1,952—or 10.2 percent—involved trucks or tractor-trailers larger than two-axle, four-tire pickups and vans. Thus, based on comparing the percentage of truck crashes to total crashes along different facility types, trucks are involved in approximately the same percentage of crashes along urban freeway mainlines as they are in crashes near freeway mainline ramp terminal entrance and exit ramps (i.e., 10 percent).

Table 8 provides a breakdown of the crash frequency for total crashes and truck crashes based on the area type (urban versus rural) for both entrance and exit ramps. Approximately 38 percent (411) of the ramps included in the analysis were in rural areas, while approximately 62 percent (664) were in urban areas. The total number of crashes that occurred in the vicinity of rural entrance ramps was 886 in the 3-year analysis period, of which 112 involved trucks. For exit ramps, 348 total crashes occurred in the vicinity of exit ramps, of which 46 involved trucks. For urban entrance ramps, 6,636 total crashes occurred in the vicinity of the ramps, of which 619 involved trucks, and for urban exit ramps, 3,207 total crashes occurred in the vicinity of the exit ramps, of which 307 involved trucks. Not taking into consideration exposure, this translates to the following crash rates per year:

- 1.41 total crashes and 0.18 truck crashes per rural entrance ramp,
- 0.58 total crashes and 0.08 truck crashes per rural exit ramp,
- 7.02 total crashes and 0.66 truck crashes per urban entrance ramp, and
- 3.06 total crashes and 0.29 truck crashes per urban exit ramp.

Interestingly, rural exit ramps had the highest percentage of truck crashes (13.22 percent), followed next by rural entrance

Table 8. Total crashes and truck crashes by area type.

Location	Rural			Urban			Grand Total		
	Total	Truck	Truck percent	Total	Truck	Truck percent	Total	Truck	Truck percent
Entrance ramps									
Acceleration lane	10	0	0.00	84	6	7.14	94	6	6.38
Freeway mainline	816	106	12.99	5,732	576	10.05	9,548	682	10.42
Ramp proper	60	6	10.00	820	37	4.51	880	43	4.89
Total	886	112	12.64	6,636	619	9.33	7,522	731	9.72
Total number of entrance ramps	210			315			525		
Exit ramps									
Deceleration lane	23	1	4.35	229	20	8.73	252	21	8.33
Freew ay mainline	193	32	16.58	2,338	235	10.05	2,531	267	10.55
Ramp proper	132	13	9.85	640	52	8.13	772	65	8.42
Total	348	46	13.22	3,207	307	9.57	3,555	353	9.93
Total number of exit ramps	201			349			550		
Grand total	1,234	158	12.80	9,843	926	9.41	11,077	1,084	9.79

ramps (12.64 percent). In urban areas, truck crashes accounted for less than 10 percent of the total crashes on both entrance ramps (9.33 percent) and exit ramps (9.57 percent).

Table 9 shows the distributions of severity levels for total crashes and truck crashes for entrance ramps, exit ramps, and combined. For entrance ramps, truck-related fatal crashes are a slightly higher percentage (0.41 percent) of the total truck crashes than the percentage of fatal crashes for total crashes (0.27 percent). However, it should be pointed out that the proportion of fatal crashes for both total and truck-related crashes is still relatively low (i.e., less than 0.5 percent) in both cases. The proportion of injury crashes for total crashes (36.24 percent) is about 5 percentage points higher than for truck crashes (30.92 percent). Consequently, the proportion of property-damage-only crashes for total crashes (63.49 percent) is about 5 percentage points lower than for truck crashes (68.67 percent). Because of the extra mass associated with trucks, it is reasonable to expect that the proportion of fatal-and-injury truck crashes would be greater than the proportion of fatal-and-injury crashes for total crashes, but this was not the case. The proportions of severity levels for exit ramps were very similar to the proportions for entrance ramps.

The data are not specifically provided in Table 9, but calculations were performed to compare the severity levels of the crash data from the Washington database to national averages for large truck crashes. In 2004, the National Highway Traffic Safety Administration (NHTSA) (2005) reported that one out of eight traffic fatalities resulted from a collision involving a large truck. In the four fatal truck crashes in the Washington database, five individuals were killed, while in the other 27 fatal crashes, 33 fatalities occurred. In other words, approximately 13 percent of all traffic fatalities at freeway-ramp terminals in Washington involved a large truck, compared to the 12 percent of all traffic fatalities reported in 2004 for all facility types combined. NHTSA (2005) also reported that in 2004, large trucks accounted for 8 percent of all vehicles involved in fatal crashes and 4 percent of all vehicles involved in injury and property-damage-only crashes. From the Washington ramp database, large trucks accounted for 11 percent of all vehicles involved in fatal crashes and 7 percent of all vehicles involved in injury and property-damage-only crashes.

Table 10 shows the distribution of total crashes by crash type. Of the 7,522 total crashes that occurred near entrance ramps, 2,383 (32 percent) were single-vehicle crashes and 5,139 (68 percent) were multiple-vehicle crashes. Rear-end crashes (3,627) on the mainline accounted for approximately 48 percent of the crashes at freeway-ramp terminal entrance ramps. As would be expected, sideswipe, same-direction crashes were also common along the mainline at entrance ramps (12 percent [919] of total crashes at entrance ramps). What was unexpected was the high number of single-vehicle, fixed-object crashes that occurred on the freeway mainline (14 percent of total crashes at entrance ramps). One explanation may be that vehicles on the ramp have difficulty merging into the traffic lane and strike an object (e.g., guard rail) just beyond the end of the acceleration lane on the right shoulder. Another explanation may be that merging traffic from the entrance ramp either crosses the mainline lanes or causes a disturbance along the mainline that forces traffic on the mainline to take evasive maneuvers and cause a run-off-the-road-to-the-left crash or median crash. Considering the exit ramp crashes, 36 percent were single-vehicle crashes and 64 percent were multiple-vehicle crashes. Among the multiple-vehicle crashes, rear-end and same-direction sideswipe crashes were the most common types along the mainline, accounting for approximately 38 and 11 percent of total crashes, respectively. Single-vehicle fixed-object crashes along the freeway mainline and along the ramp proper accounted for approximately 12 and 11 percent of total crashes, respectively. These single-vehicle crashes could be the result of vehicles striking a crash attenuation system at the gore between the freeway mainline and ramp and/or vehicles having difficulty negotiating critical curves along the ramp proper due to high speed.

Table 11 shows the distribution of truck crashes by crash type. Of the 731 truck crashes that occurred near entrance ramps, 18 percent were single-vehicle crashes and 82 percent were multiple-vehicle crashes. Rear-end crashes along the mainline accounted for approximately 35 percent of the truck crashes at

Table 9. Total crashes and truck crashes by severity level.

		Severity			Percent		
	Data	Fatal	Injury	PDO	Fatal	Injury	PDO
Entrance ramps	Total crashes	20	2,726	4,776	0.27	36.24	63.49
	Truck crashes	3	226	502	0.41	30.92	68.72
Exit ramps	Total crashes	11	1,318	2,226	0.31	37.07	62.62
	Truck crashes	1	115	237	0.28	32.58	67.14
Total	Total crashes	31	4,044	7,002	0.28	36.51	63.21
	Truck crashes	4	341	739	0.37	31.46	68.17

Note: PDO = property damage only.

Table 10. Total crashes by crash type.

Location	Single-vehicle crashes											Multiple-vehicle crashes							Totals
	Collision with parked motor vehicle	Collision with railroad train	Collision with bicyclist	Collision with pedestrian	Collision with animal	Collision with fixed object	Collision with other object	Other single-vehicle collision	Overturn	Fire or explosion	Other single-vehicle non-collision	Rear-end	Head-on	Rear-to-rear	Angle	Sideswipe, same direction	Sideswipe, opposite direction	Other multiple-vehicle collision	
Entrance ramps																			
Accel. lane	3	0	0	0	0	39	1	0	6	2	4	24	0	0	0	10	0	5	94
Freeway mainline	41	0	0	5	84	1,079	85	16	241	68	62	3,627	3	0	0	919	3	315	6,548
Ramp proper	12	0	1	2	4	514	2	1	101	6	4	153	2	0	0	47	0	31	880
Total	56	0	1	7	88	1,632	88	17	348	76	70	3,804	5	0	0	976	3	351	7,522
Exit ramps																			
Decel. lane	2	0	0	0	2	78	0	0	18	3	2	107	1	0	1	32	0	6	252
Freeway mainline	23	0	0	6	25	425	20	7	72	22	23	1,382	4	0	1	401	0	120	2,531
Ramp proper	13	0	0	1	3	382	1	2	118	29	11	166	2	0	0	35	1	8	772
Total	38	0	0	7	30	885	21	9	208	54	36	1,655	7	0	2	468	1	134	3,555
Grand total	94	0	1	14	118	2,517	109	26	556	130	106	5,459	12	0	2	1,444	4	485	11,077

Table 11. Truck crashes by crash type.

Location	Single-vehicle crashes											Multiple-vehicle crashes							Totals
	Collision with parked motor vehicle	Collision with railroad train	Collision with bicyclist	Collision with pedestrian	Collision with animal	Collision with fixed object	Collision with other object	Other single-vehicle collision	Overturn	Fire or explosion	Other single-vehicle non-collision	Rear-end	Head-on	Rear-to-rear	Angle	Sideswipe, same direction	Sideswipe, opposite direction	Other multiple-vehicle collision	
Entrance ramps																			
Accel. lane	1	0	0	0	0	2	0	0	0	1	0	1	0	0	0	1	0	0	6
Freeway mainline	6	0	0	0	3	48	12	2	8	3	25	260	0	0	0	248	0	67	682
Ramp proper	2	0	0	0	0	9	0	0	11	0	2	8	0	0	0	5	0	6	43
Total	9	0	0	0	3	59	12	2	19	4	27	269	0	0	0	254	0	73	731
Exit ramps																			
Decel. lane	0	0	0	0	0	2	0	0	2	0	1	9	0	0	0	6	0	1	21
Freeway mainline	4	0	0	1	0	13	6	1	3	3	3	112	0	0	1	100	0	20	267
Ramp proper	5	0	0	0	0	14	0	0	14	3	4	12	0	0	0	10	0	3	65
Total	9	0	0	1	0	29	6	1	19	6	8	133	0	0	1	116	0	24	353
Grand total	18	0	0	1	3	88	18	3	38	10	35	402	0	0	1	370	0	97	1,084

entrance ramps, and sideswipe, same-direction crashes along the mainline were also common at freeway-ramp terminal entrance ramps (34 percent of truck crashes at entrance ramps). As with total crashes, fixed-object crashes were the most common type of truck-related single-vehicle crash at entrance ramps. Other multiple-vehicle collisions were another common type of truck crash at entrance ramps. Considering the truck-related exit ramp crashes, 22 percent were single-vehicle crashes and 78 percent were multiple-vehicle crashes. Among the multiple-vehicle crashes, rear-end and sideswipe, same-direction crashes along the mainline were the most common. Single-vehicle fixed-object crashes along the freeway mainline and along the ramp proper each accounted for approximately 4 percent of truck crashes.

Table 12 categorizes the distribution of truck crashes by crash type based upon three ramp truck average daily traffic (RTADT) levels:

- RTADT < 500,
- 500 ≤ RTADT < 1000, and
- RTADT ≥ 1000.

This stratification of RTADT was selected somewhat arbitrarily, but it illustrates that the majority of truck crashes occurred at ramps with RTADT levels above 1,000 trucks/day. In part, this could have to do with exposure level, and it could also be related to the operational conditions of the traffic stream. Ramps with higher RTADT likely experience poorer operating conditions throughout the day because of greater traffic volumes in general, thus reducing the freedom to maneuver within the traffic stream. It could also be a function of the number of ramps included in each categorization.

Table 13 shows the crash rates for total crashes and truck crashes by ramp configuration and ramp location. For entrance ramps, the overall crash rates were approximately the same for total crashes (0.81 crashes/million vehicle miles traveled [MVMT]) and truck crashes (0.80 crashes/MVMT). Similarly for exit ramps, the overall crash rates were approximately equal for total crashes (0.67 crashes/MVMT) and truck crashes (0.70 crashes/MVMT). For the various ramp configurations, there was no consistent trend suggesting that trucks were more or less likely to be involved in a crash along a certain location of the ramp for either entrance or exit ramps. The four ramp configurations where the truck crash rate exceeded the overall total crash rate by the greatest amount included:

- Outer connection entrance ramp (difference between crash rates—0.18 crashes/MVMT),
- Parclo loop exit ramp (difference between crash rates—0.22 crashes/MVMT),
- Free-flow loop exit ramp (difference between crash rates—0.22 crashes/MVMT), and
- Other exit ramp (difference between crash rates—0.19 crashes/MVMT).

Related to Table 13, NHTSA (2005) reported that trucks were involved in fatal crashes at a rate of 2.19 crashes/100 MVMT taking consideration of truck operations along the entire roadway network. Based upon the Washington ramp database, trucks were involved in fatal crashes at a rate of 0.28 crashes/100 MVMT in the vicinity of entrance and exit ramps.

4.3 Key Findings

This section highlights several key findings from the crash analysis, focusing on comparing crash frequencies/rates for all vehicle types combined (i.e., total crashes) to crash frequencies/rates for heavy vehicles (i.e., trucks). The findings from this crash analysis combined with the findings from the observational study described in Section 5, provide the basis for recommending a particular type of design vehicle for freeway mainline ramp terminals. The key findings are as follows:

1. The crash rates for all vehicles compared to the crash rates of trucks are for practical purposes the same at both entrance and exit ramps. At entrance ramps, the crash rate for all vehicles is 0.81 crashes/MVMT compared to 0.80 crashes/MVMT for truck-related crashes. At exit ramps, the crash rate for all vehicles is 0.67 crashes/MVMT compared to 0.70 crashes/MVMT for truck-related crashes.
2. Trucks are involved in approximately 10 percent of all crashes in the vicinity of freeway mainline ramp terminals. As a comparison, trucks are involved in approximately 10 percent of all crashes along the mainline of urban freeways (Golob and Regan, 2003). Thus, truck crashes are neither over-represented nor under-represented near freeway mainline ramp terminals.
3. Rural exit ramps had the highest percentage of truck crashes (13.2 percent), followed by rural entrance ramps (12.6 percent). In urban areas, truck crashes accounted for less than 10 percent of the total crashes on both entrance ramps (9.3 percent) and exit ramps (9.6 percent). Several explanations for having a slightly higher percentage of truck crashes at rural freeway mainline ramp terminals as compared to urban terminals include potentially greater truck exposure levels in rural areas, driver adaptation to the high-speed rural environment, and greater driver attention levels in urban areas.
4. When comparing the severity distributions at the person level, approximately 13 percent of all traffic fatalities at freeway mainline ramp terminals in Washington involved

Table 12. Truck crashes by crash type and RTADT.

Area	Single-vehicle crashes											Multiple-vehicle crashes							Total
	Collision with parked motor vehicle	Collision with railroad train	Collision with bicyclist	Collision with pedestrian	Collision with animal	Collision with fixed object	Collision with other object	Other single-vehicle collision	Overturn	Fire or explosion	Other single-vehicle non-collision	Rear-end	Head-on	Rear-to-rear	Angle	Sideswipe, same direction	Sideswipe, opposite direction	Other multiple-vehicle collision	
RTADT < 500																			
Entrance ramps																			
Acceleration lane	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	1
Freeway mainline	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ramp proper	1	0	0	0	0	3	0	0	1	0	1	0	0	0	0	0	0	2	8
Total	1	0	0	0	0	3	0	0	1	1	1	0	0	0	0	0	0	2	9
Exit ramps																			
Deceleration lane	0	0	0	0	0	0	0	0	1	0	0	1	0	0	0	1	0	0	3
Freeway mainline	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ramp proper	1	0	0	0	0	7	0	0	3	2	1	3	0	0	0	1	0	1	19
Total	1	0	0	0	0	7	0	0	4	2	1	4	0	0	0	2	0	1	22
500 ≤ RTADT < 1,000																			
Entrance ramps																			
Acceleration lane	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1	0	0	2
Freeway mainline	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ramp proper	1	0	0	0	0	3	0	0	2	0	0	4	0	0	0	2	0	1	13
Total	1	0	0	0	0	4	0	0	2	0	0	4	0	0	0	3	0	1	15
Exit ramps																			
Deceleration lane	0	0	0	0	0	1	0	0	1	0	0	1	0	0	0	0	0	1	4
Freeway mainline	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Ramp proper	1	0	0	0	0	4	0	0	8	1	2	2	0	0	0	2	0	0	20
Total	1	0	0	0	0	5	0	0	9	1	2	3	0	0	0	2	0	1	24
RTADT ≥ 1,000																			
Entrance ramps																			
Acceleration lane	1	0	0	0	0	1	0	0	0	0	0	1	0	0	0	0	0	0	3
Freeway mainline	6	0	0	0	3	48	12	2	8	3	25	260	0	0	0	248	0	67	682
Ramp proper	0	0	0	0	0	3	0	0	8	0	1	4	0	0	0	3	0	3	22
Total	7	0	0	0	3	52	12	2	16	3	26	265	0	0	0	251	0	70	707
Exit ramps																			
Deceleration lane	0	0	0	0	0	1	0	0	0	0	1	7	0	0	0	5	0	0	14
Freeway mainline	4	0	0	1	0	13	6	1	3	3	3	112	0	0	1	100	0	20	267
Ramp proper	3	0	0	0	0	3	0	0	3	0	1	7	0	0	0	7	0	2	26
Total	7	0	0	1	0	17	6	1	6	3	5	126	0	0	1	112	0	22	307

Table 13. Total crash rate (crashes/MVMT) and truck crash rate (crashes/MVMT) by ramp configuration and area.

Area	Ramp configuration														Total	
	Diamond		Parclo loop		Free-flow loop		Outer connection		Direct or semi-direct connection		Button hook, scissor, and slip ramps		Others			
	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Total	Truck	Total	Truck
Entrance ramps																
Acceleration lane	0.09	0.03	0.13	0.40	0.16	0.00	0.21	0.00	0.37	0.26	0.08	0.00	0.29	0.26	0.16	0.10
Freeway mainline	0.60	0.63	1.27	1.11	1.14	0.59	0.69	0.96	0.82	0.95	1.43	1.45	1.09	1.36	0.80	0.86
Ramp proper	0.85	0.22	1.41	0.94	1.96	2.08	0.93	0.34	1.54	1.32	1.80	1.49	3.94	1.83	1.90	0.85
Total	0.58	0.57	1.21	1.06	1.16	0.76	0.67	0.85	0.82	0.91	1.35	1.36	1.23	1.32	0.81	0.80
Exit ramps																
Deceleration lane	0.37	0.12	0.35	0.00	0.55	0.00	0.74	0.87	0.96	1.50	0.14	0.00	1.03	1.00	0.68	0.57
Freeway mainline	0.32	0.36	0.44	0.68	1.25	1.80	0.62	0.47	0.73	0.91	0.65	0.75	0.96	1.22	0.59	0.67
Ramp proper	1.00	0.79	1.73	1.95	2.72	2.09	1.68	1.23	1.44	1.22	1.87	1.56	1.28	1.04	1.30	1.03
Total	0.40	0.40	0.53	0.75	1.45	1.67	0.78	0.63	0.90	1.04	0.74	0.81	1.00	1.19	0.67	0.70

a large truck. This is consistent with the national figures reported by NHTSA indicating that approximately 12 percent of all traffic fatalities involve a large truck. Thus, when comparing severity levels at the person level, trucks are not over- or under-represented in the fatal crashes that occur near freeway mainline ramp terminals.

5. When comparing the severity distributions at the vehicle level, trucks are involved in a slightly greater percentage of the fatal crashes (11 percent) and injury and property-damage-only crashes (7 percent) near freeway mainline ramp terminals compared to the national statistics reported by NHTSA for fatal crashes (8 percent) and injury and property-damage-only crashes (4 percent).
6. The entrance ramp configurations where the truck crash rates exceeded the overall total crash rates by the greatest amounts included (a) outer connection, (b) direct or semi-direct connection, and (c) "other" ramps. With the exception of the "other" ramp configuration, where drivers' expectations may be violated, there is no reasonable explanation as to why the truck crash rates exceeded the overall total crash rates by the greatest amounts on these particular entrance ramp configurations.
7. The exit ramp configurations where the truck crash rates exceeded the overall total crash rates by the greatest amounts included (a) parclo loop, (b) free-flow loop, and (c) "other" ramps. In particular, since both parclo and free-flow loop exit ramps have curved alignments, there is concern that trucks may rollover at these types of ramps if trucks are traveling substantially faster than the design speed of the first critical curve.
8. When considering the distribution of crash types at entrance ramps, 48 percent of total crashes are rear-end crashes and 12 percent are sideswipe, same-direction crashes compared to 35 percent for rear-end and 34 percent for sideswipe, same-direction crashes for truck crashes. Combined rear-end and sideswipe, same-direction crashes account for approximately 60 percent of total crashes compared to approximately 69 percent of truck crashes. Thus, trucks are slightly over-represented in these two crash types as might be expected with vehicles operating at lower speeds and acceleration rates.
9. When the RTADT exceeds 1,000 trucks/day, it appears that truck crashes occur more frequently compared to freeway mainline ramp terminals with lower RTADT.

SECTION 5

Observational Study of Freeway Mainline Ramp Terminals

The objective of the observational study was to gather speed and distance information for a large number of merging and diverging vehicles covering a range of ramp types and merge types. From this data, speed profiles of vehicles could be developed to reflect current driver behavior and vehicle performance capabilities. Data from these speed profiles and other corresponding measures were analyzed to compare acceleration and deceleration characteristics of the current driver population and vehicle fleet to the assumptions inherent in the existing AASHTO models for determining minimum acceleration and deceleration lane lengths. This section describes the study locations, the field data collection procedures, the data reduction process, and the analysis results for both entrance and exit ramps.

5.1 Study Locations

Observational studies were conducted at 20 freeway mainline ramp terminals in Kansas, Missouri, Pennsylvania, and Texas. Specific ramps were chosen for inclusion in the study based on the following criteria:

- Include ramps from different geographical areas of the country.
- Include both entrance and exit ramps.
- Include both parallel and tapered SCLs.
- Include both loop and straight (i.e., diamond) ramps.
- Include ramps with a range of grades (0 to 6 percent upgrade/downgrades).
- Include ramps where the freeway conditions range from free-flow to constrained (or forced) throughout portions of the day.
- Operations near the ramps should be independent of upstream/downstream ramps.

In addition to these criteria, the ability to set up the data collection equipment was factored into the site selection process. Table 14 provides the location of each ramp included in the study. Figure 6 shows two straight ramps (i.e., one entrance and one exit) included in the study, and Figure 7 shows two loop ramps (i.e., one entrance and one exit) included in the study. Appendix A (available on the TRB website at http://www.trb.org/Main/Blurbs/167516.aspx) provides aerial views of all ramps included in the study.

The following site characteristics were gathered for potential study locations during the site selection process, prior to data collection, either in the field or from construction plans/profiles or aerial photos in the office:

General Ramp Characteristics:
- Type of ramp (straight/loop),
- Type of merge (parallel/taper),
- Number of lanes,
- Average lane width,
- Average width of right shoulder,
- Average width of left shoulder,
- Radius of controlling curve,
- Vertical profile/grade,
- Posted speed limit and/or advisory speed sign,
- Area type (CBD, urban mixed-use, suburban residential or recreational, suburban commercial),
- Ramp volume,
- Year of ramp volume,
- Ramp vehicle mix (percent trucks), and
- Design speed of ramp.

General Freeway Characteristics:
- Number of lanes (by direction),
- Average lane width,
- Average width of right shoulder,
- Alignment (curve/tangent),
- Posted speed limit,
- Distance to nearest upstream ramp,
- Distance to nearest downstream ramp,
- Freeway volume,
- Year of freeway volume,

Table 14. Study ramps.

Ramp ID	State	Metropolitan area	Freeway	Cross street	Direction
Entrance ramps					
1	KS	Kansas City	I-435	Quivira Rd	WB Entrance
2	MO	Kansas City	I-435	US 24	NB Entrance
3	MO	Kansas City	I-435	63rd St	SB Entrance
4	MO	Kansas City	I-435	Gregory Blvd	NB Entrance
5	PA	Pittsburgh	I-376	Ardmore Blvd	EB Entrance
6	PA	Pittsburgh	I-376	Wm. Penn Hwy	WB Entrance
7	TX	Dallas	I-20	Belt Line Rd	WB Entrance
8	TX	Dallas	I-20	Carrier Pkwy	WB Entrance
9	TX	Dallas	I-635	Midway Rd	WB Entrance
10	TX	Dallas	SH 114	Esters Rd	EB Entrance
11	TX	Dallas	SH 114	Freeport Rd	EB Entrance
Exit ramps					
12	KS	Kansas City	I-435	Quivira Rd	EB Exit
13	KS	Kansas City	I-635	Metropolitan Ave	NB Exit
14	MO	Kansas City	I-70	US 40	WB Exit
15	MO	Kansas City	I-435	US 24	SB Exit
16	MO	Kansas City	I-435	Gregory Blvd	SB Exit
17	TX	Dallas	I-635	Freeport Pkwy	EB Exit
18	TX	Dallas	I-635	Freeport Pkwy	WB Exit
19	TX	Dallas	I-635	Marsh Ln	EB Exit
20	TX	Dallas	I-635	Plano Rd	EB Exit

- Vehicle mix (percent trucks), and
- Design speed of freeway.

Entrance Ramps:
- Length from controlling feature to painted nose,
- Length from painted nose to end of SCL (not including taper), and
- Length of taper.

Exit Ramps:
- Length of taper,
- Length from end of taper to painted nose, and
- Length from painted nose to controlling feature.

5.1.1 Ramp Design and Geometry

Table 15 presents basic design elements for each entrance ramp. Of the 11 entrance ramps studied, nine were straight (i.e., diamond) ramps, and two were loop ramps. Four ramps had tapered merge lanes, as illustrated in Figure 8, and the other seven ramps had parallel merge lanes, as illustrated in Figure 9. The "controlling feature" column indicates whether ramp curvature or the crossroad terminal is the design element that controls when a vehicle can begin accelerating to the merge speed. For example, on a loop ramp, the horizontal curvature limits the vehicle's speed, and full acceleration cannot begin until the vehicle has exited the curve. Speeds on

Figure 6. I-435/Quivira Rd—Ramps 1 and 12. (Image Credit: Google Earth™ Mapping Service).

Figure 7. I-435/US 24—Ramps 2 and 15. (Image Credit: Google Earth™ Mapping Service).

Table 15. Entrance ramp design features.

Ramp	Ramp type	Merge type	Controlling feature	Radius of controlling curve if ≤ 1,000 ft (ft)	Grade	Distance from painted nose to controlling feature (ft)	Speed-change lane length (ft)	Taper length (ft)
I-435/Quivira Rd	Straight	Parallel	Crossroad terminal	1,000	2% upgrade	890	485	235
I-435/US 24	Loop	Parallel	Horizontal alignment	150	1% upgrade	0	640	250
I-435/63rd St	Straight	Parallel	Crossroad terminal	N/A	2% upgrade	1,795	575	200
I-435/Gregory Blvd	Straight	Parallel	Crossroad terminal	N/A	3% upgrade	725	420	205
I-376/Ardmore Blvd	Straight	Parallel	Horizontal alignment	475	4% upgrade	275	325	560
I-376/Wm. Penn Hwy	Straight	Parallel	Horizontal alignment	955	6% upgrade	0	325	200
I-20/Belt Line Rd	Straight	Taper	Crossroad terminal	N/A	2% upgrade	1,390	470	300
I-20/Carrier Pkwy	Straight	Taper	Crossroad terminal	N/A	2% upgrade	905	640	350
I-635/Midway Rd	Straight	Taper	Crossroad terminal	N/A	1% upgrade	700	145	530
SH 114/Esters Rd	Loop	Taper	Horizontal alignment	120	1% downgrade	395	455	560
SH 114/Freeport Rd	Straight	Parallel	Crossroad terminal	N/A	< 1% upgrade	1,850	675	280

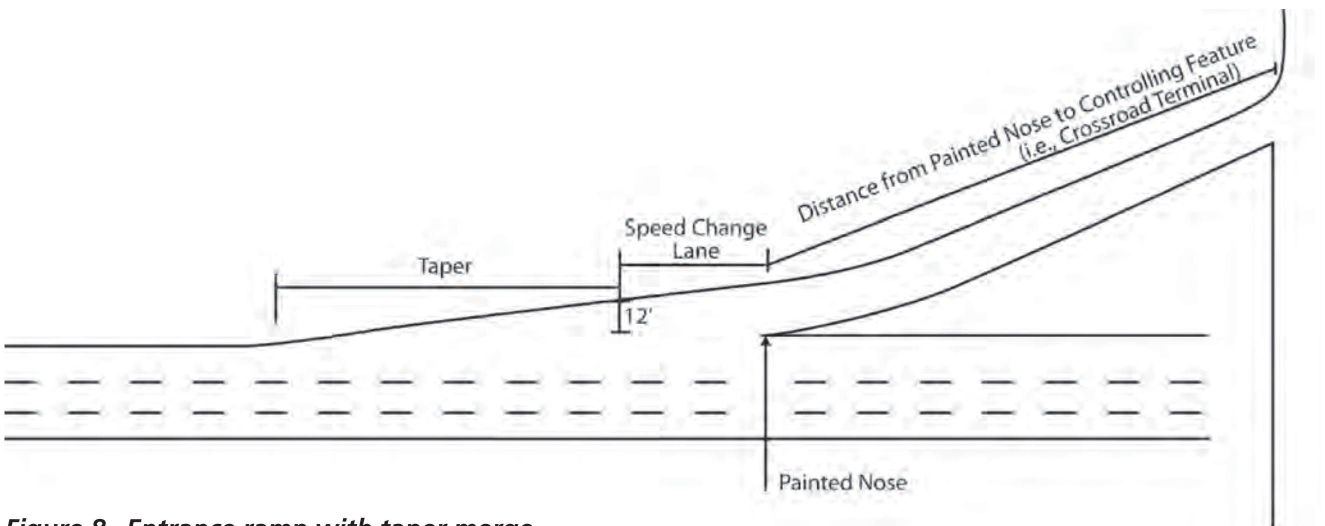


Figure 8. Entrance ramp with taper merge.

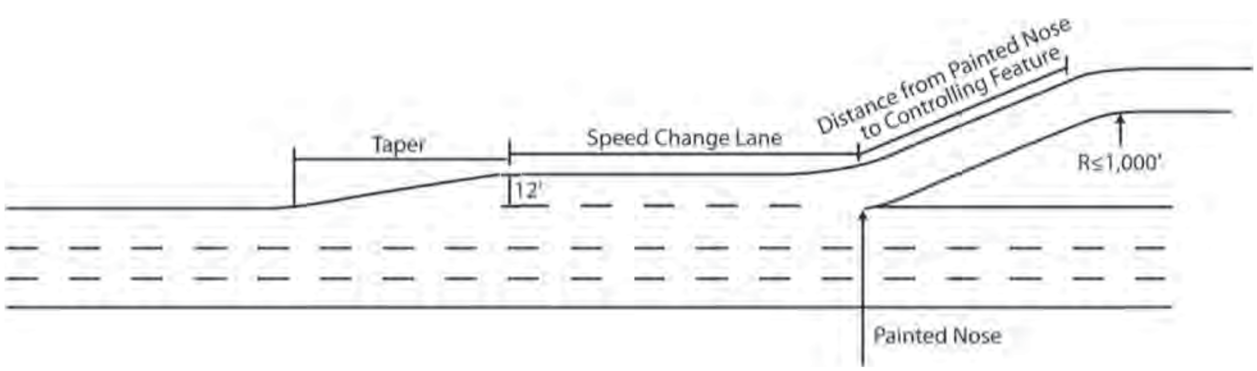


Figure 9. Entrance ramp with parallel merge.

Table 16. Exit ramp design features.

Ramp	Ramp type	Diverge type	Controlling feature	Radius of controlling curve if ≤ 1,000 ft (ft)	Grade	Distance from painted nose to controlling feature (ft)	Speed-change lane length (ft)	Taper length (ft)
I-435/Quivira Rd	Straight	Parallel	Crossroad terminal	N/A	2% downgrade	1,025	615	250
I-635/Metropolitan Ave	Straight	Taper	Crossroad terminal	N/A	6% downgrade	835	125	360
I-70/US 40	Loop	Parallel	Horizontal alignment	180	1% upgrade	255	315	90
I-435/US 24	Loop	Parallel	Horizontal alignment	180	2% downgrade	0	595	270
I-435/Gregory Blvd	Straight	Parallel	Crossroad terminal	N/A	3% downgrade	900	620	190
I-635/Freeport Pkwy	Loop	Taper	Horizontal alignment	200	1% upgrade	410	110	160
I-635/Freeport Pkwy	Loop	Taper	Horizontal alignment	200	1% upgrade	400	100	180
I-635/Marsh Ln	Straight	Taper	Crossroad terminal	N/A	2% upgrade	995	40	135
I-635/Plano Rd	Loop	Taper	Horizontal alignment	445	1% downgrade	340	85	170

straight ramps may also be limited by horizontal alignment if the ramp curves or bends. For this study, the horizontal alignment is only considered controlling if the radius of the curve is equal to or less than 1,000 ft, consistent with current guidance provided in the 2004 *Green Book*. Grade also has an impact on acceleration, especially for large trucks on steep upgrades. The entrance ramps observed in this research had grades ranging from a very slight downgrade to a 6 percent upgrade. Grade was measured in the field and was determined based upon the vertical profile from the painted nose to the end of the SCL.

Table 16 presents basic design elements for each exit ramp. Of the nine exit ramps studied, four were straight ramps, and five were loop ramps. Five ramps had tapered exit lanes, as illustrated in Figure 10, and the other four ramps had parallel exits, as illustrated in Figure 11. For exit ramps, the "controlling feature" column indicates whether ramp curvature or the crossroad terminal is the design element that controls vehicle deceleration. For example, on a loop ramp, the curvature of the loop will require a nearly immediate reduction in speed as the vehicle exits the freeway onto a curve. On a straight ramp, a vehicle may not have to fully decelerate (to a stop or yield) until reaching the crossroad terminal. Deceleration may also be limited by horizontal alignment on a straight ramp if the ramp curves or bends. As on entrance ramps, the horizontal alignment is considered controlling if the radius of the curve is equal to or less than 1,000 ft. Grade also has an impact on deceleration. The exit ramps observed in this research had

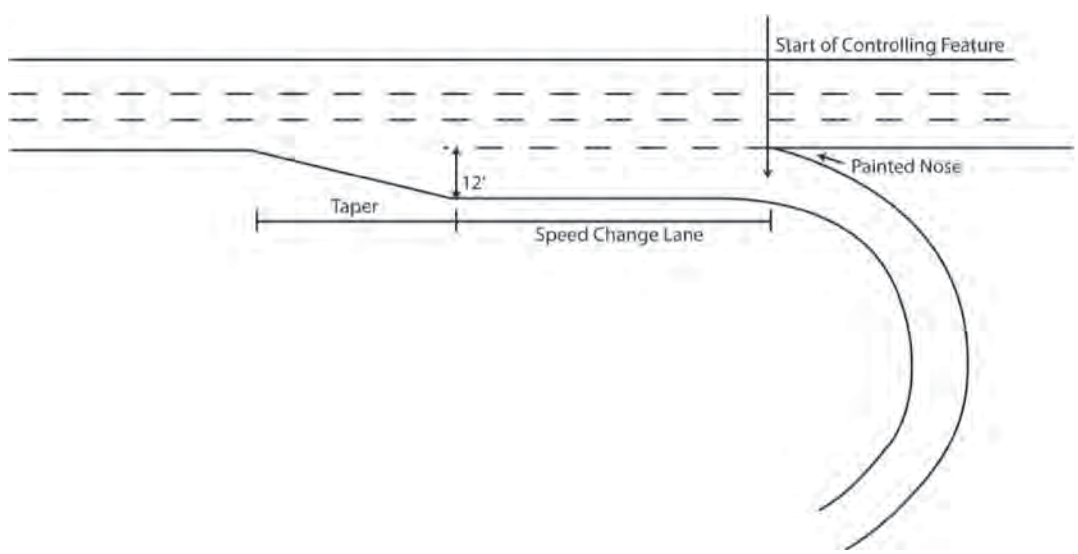


Figure 10. Exit ramp with taper diverge.

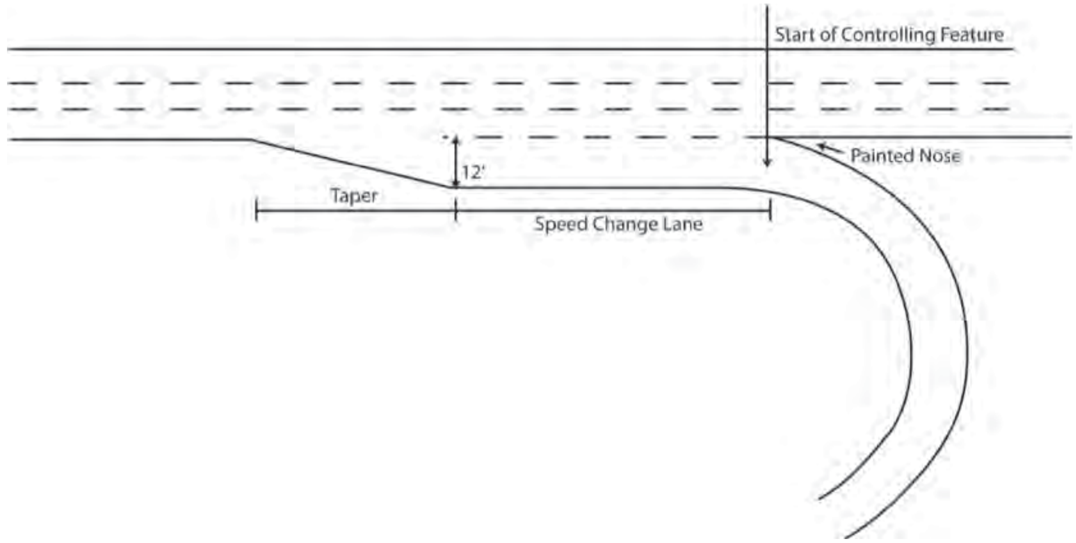


Figure 11. Exit ramp with parallel diverge.

grades ranging from a 6 percent downgrade to a 2 percent upgrade.

Table 15 also provides for each entrance ramp the distance from the controlling feature to the painted nose, the length of the SCL from the painted nose to the beginning of the taper, and the length of the taper. Table 16 provides the equivalent measures for exit ramps—the distance from the beginning of the taper to the end of taper, the length of the SCL from the end of taper to the painted nose, and the length from the painted nose to the controlling feature. In instances where the crossroad terminal is the controlling feature, the length from the painted nose to the controlling feature is the length of the ramp. In instances where the distance from the painted nose to the controlling feature is 0 ft, the horizontal curve with a radius equal to or less than 1,000 ft ends or begins at the painted nose. For an entrance ramp, the SCL begins at the painted nose of the ramp and ends when the lane becomes narrower than 12 ft (i.e., where the taper begins). The taper begins at the end of the SCL and ends where the lane has completely merged back into the freeway lane. For an exit ramp, the SCL begins when the exit lane reaches a width of 12 ft (i.e., where the taper ends), and ends at the painted nose. The taper begins where the exit lane first begins to diverge from the freeway lane and ends where the SCL begins (i.e., where the taper width has reached 12 ft). The painted nose is defined here as the location where the freeway edgeline and ramp edgeline meet. These design features are illustrated in Figures 8 through 11.

5.1.2 Ramp and Freeway Speed Information

Table 17 presents speed information for each of the ramps and associated freeway segments included in the study, including posted speed limit, freeway design speed, and ramp design speed. For ramp design speed, the design speed of the controlling feature was used. Where the horizontal alignment was the controlling feature, this was the design speed of the controlling curve. Where the crossroad terminal was the controlling feature, the stop condition was considered the appropriate design criterion for the ramp.

5.1.3 Comparison of Existing Conditions to *Green Book* Criteria

Each study ramp was evaluated to determine if it met the 2004 *Green Book* design criterion for the minimum length required for acceleration or deceleration as given in *Green Book* Exhibits 10-70 (entrance ramps), 10-73 (exit ramps), and 10-71 (adjustment for grade). For each ramp, the minimum length recommended by the *Green Book* is compared to the sum of the length of the SCL and the distance from the painted nose to the controlling feature. The minimum length recommended by the *Green Book* was determined based on the design speed (rather than average running speed) of the freeway, the ramp design speed (as shown in Table 17), and the grade of the ramp terminal.

Tables 18 and 19 present comparisons of the actual acceleration and deceleration lane lengths to AASHTO criteria. Only three of the 11 entrance ramps meet or exceed the 2004 *Green Book* criteria, while all nine exit ramps meet or exceed the 2004 *Green Book* criteria.

5.2 Field Data Collection Procedures

The general field data collection procedures performed to gather speed and volume data at freeway mainline ramp terminals are described below. Data collection activities were

Table 17. Freeway design, posted speeds, and ramp design speeds.

Ramp	Posted speed limit on freeway (mi/h)	Freeway design speed (mi/h)	Ramp design speed (mi/h)
Entrance ramps			
I-435/Quivira Rd	65	75	Stop condition
I-435/US 24	65	70	25
I-435/63rd St	65	70	Stop condition
I-435/Gregory Blvd	65	70	Stop condition
I-376/Ardmore Blvd	55	65	40
I-376/Wm. Penn Hwy	55	65	55
I-20/Belt Line Rd	60	70	Stop condition
I-20/Carrier Pkwy	60	70	Stop condition
I-635/Midway Rd	60	65	Stop condition
SH 114/Esters Rd	60	70	25
SH 114/Freeport Rd	60	70	Stop condition
Exit ramps			
I-435/Quivira Rd	65	75	Stop condition
I-635/Metropolitan Ave	65	75	Stop condition
I-70/US 40	55	65	30
I-435/US 24	65	70	30
I-435/Gregory Blvd	65	70	Stop condition
I-635/Freeport Pkwy	60	70	30
I-635/Freeport Pkwy	60	70	30
I-635/Marsh Ln	60	65	Stop condition
I-635/Plano Rd	60	65	40

similar at both entrance and exit ramps and for straight and loop ramps. Generally, field data were collected at each study site for 6 hours during a weekday during the following time periods:

- AM or PM peak period—2 hours (i.e., the heavier directional peak period was chosen for data collection),
- AM or PM non-peak period—2 hours, and
- Noon peak—2 hours.

Data were collected using laser guns, traffic classifiers, and video equipment. Laser guns were used to collect speed and distance data of subject vehicles as they traveled along the ramps and acceleration or deceleration lanes. The laser guns

Table 18. Comparison of actual acceleration lengths to 2004 *Green Book* criteria.

Ramp	Controlling feature	Actual acceleration length (ft)	*Green Book* minimum acceleration length (ft)	Grade adjustment factor	Adjusted minimum acceleration length (ft)	Actual length minus adjusted minimum length (ft)	Design status
I-435/Quivira Rd	Crossroad terminal	1,375	1,790	1	1,790	–415	Actual **LESS** than design
I-435/US 24	Horizontal alignment	640	1,420	1	1,420	–780	Actual **LESS** than design
I-435/63rd St	Crossroad terminal	2,370	1,620	1	1,620	750	Actual **GREATER** than design
I-435/Gregory Blvd	Crossroad terminal	1,145	1,620	1.8	2,916	–1,771	Actual **LESS** than design
I-376/Ardmore Blvd	Horizontal alignment	600	770	1.6	1,232	–632	Actual **LESS** than design
I-376/Wm. Penn Hwy	Horizontal alignment	325	370	2.75	1,018	–693	Actual **LESS** than design
I-20/Belt Line Rd	Crossroad terminal	1,860	1,620	1	1,620	240	Actual **GREATER** than design
I-20/Carrier Pkwy	Crossroad terminal	1,545	1,620	1	1,620	–75	Actual **LESS** than design
I-635/Midway Rd	Crossroad terminal	845	1,410	1	1,410	–565	Actual **LESS** than design
SH 114/Esters Rd	Horizontal alignment	850	1,420	1	1,420	–570	Actual **LESS** than design
SH 114/Freeport Rd	Crossroad terminal	2,525	1,620	1	1,620	905	Actual **GREATER** than design

Table 19. Comparison of actual deceleration lengths to 2004 *Green Book* criteria.

Ramp	Controlling feature	Actual deceleration length (ft)	*Green Book* minimum deceleration length (ft)	Grade adjustment factor	Adjusted minimum deceleration length (ft)	Actual length minus adjusted minimum length (ft)	Design status
I-435/Quivira Rd	Crossroad terminal	1,640	660	1	660	980	Actual **GREATER** than Design
I-635/Metropolitan Ave	Crossroad terminal	960	660	1.35	891	69	Actual **GREATER** than Design
I-70/US 40	Horizontal alignment	570	470	1	470	100	Actual **GREATER** than Design
I-435/US 24	Horizontal alignment	595	520	1	520	75	Actual **GREATER** than Design
I-435/Gregory Blvd	Crossroad terminal	1,520	615	1.2	738	782	Actual **GREATER** than Design
I-635/Freeport Pkwy	Horizontal alignment	520	490	1	490	30	Actual **GREATER** than Design
I-635/Freeport Pkwy	Horizontal alignment	500	490	1	490	10	Actual **GREATER** than Design
I-635/Marsh Ln	Crossroad terminal	1,035	570	1	570	465	Actual **GREATER** than Design
I-635/Plano Rd	Horizontal alignment	425	390	1	390	35	Actual **GREATER** than Design

were connected to laptop computers and their readings were automatically recorded in a separate spreadsheet for each vehicle at a rate of approximately three readings per second. The speed and distance data were plotted during post processing to develop speed profiles of individual vehicles along the ramps and acceleration or deceleration lanes. Depending on the geometry of the ramp and available sight distance, either one or two laser guns were used at each site to be able to develop speed profiles for vehicles over as much of the ramp and acceleration or deceleration lane as possible.

Laser guns were handheld and operated by a researcher inside of a vehicle parked off the roadway in a location chosen based on several criteria:

- Safety of data collectors and equipment,
- Minimal impact of presence of data collectors and equipment on driver behavior or desired speeds (Note: Subject vehicles were tracked from the rear as they drove away from the laser gun),
- Visibility of as much of the ramp and merge/diverge area as possible, and
- Minimized angle between the laser gun and the vehicles being tracked.

During a 2-hour study period, speed and distance data were collected for as many ramp vehicles entering or exiting the freeway lanes as possible. Both free-flow and platooned vehicles were recorded as such, and vehicles were classified generally as passenger cars (i.e., including light trucks and sport-utility vehicles) or trucks (i.e., including SU and tractor-semitrailer trucks).

In situations where two lasers were used, the operator of the first laser gun would track a vehicle and then communicate by radio to the operator of the second laser gun the description (i.e., make, model, and color) of the vehicle being tracked. The operator of the second laser gun would track the subject vehicle as soon as it became visible until completion of the maneuver being recorded. The operators would confirm with each other that the vehicle was successfully tracked before storing the data for post processing. During post processing, the separate data files were matched by file name and time stamps and combined to create a single speed profile for each vehicle.

For entrance ramps, the operator of the laser gun began tracking a vehicle (i.e., pulled the trigger of the laser) at the earliest point possible along the ramp and stopped tracking it (i.e., released the trigger of the laser) when the driver-side front tires crossed into the rightmost through lane of the freeway. When the controlling feature of the ramp was a horizontal curve, the operator began tracking vehicles as they exited the controlling curve. The final data points for each vehicle provided its final merge speed and merge location.

For exit ramps, the operator of the laser gun began tracking vehicles (i.e., pulled the trigger on the laser gun) when the passenger-side front tires crossed from the through lane of the freeway into the deceleration lane (or taper) and continued tracking vehicles to the controlling curve or for as long as possible along a straight ramp. Thus, the initial recordings for each vehicle provided its diverge speed from the freeway and its diverge location. The final recordings provided its final speed at the controlling feature (or as near as possible to the controlling feature). Due to the nature of the diverge maneuver and the capability to lock a laser gun on a diverging vehicle, there is more uncertainty in the initial diverge speed and diverge location data presented later compared to corresponding merge speed and merge location data for entrance ramps.

In addition to gathering data for the merging and diverging vehicles, traffic classifiers were used to record vehicle speeds and traffic volumes in the rightmost lane of the freeway at each study site. The traffic classifiers were positioned at three locations at each study site:

- Entrance Ramps
  - *Upstream of the painted nose:* This classifier was positioned approximately 750 to 1,000 ft upstream of the painted nose.
  - *Just prior to the painted nose:* This classifier was positioned approximately 100 ft upstream from the painted nose.
  - *Downstream of the end of taper:* This classifier was positioned approximately 500 to 750 ft from the end of the taper.
- Exit Ramps
  - *Upstream of the taper:* This classifier was positioned approximately 750 to 1,000 ft upstream of the beginning of the taper.
  - *Just prior to the taper:* This classifier was positioned approximately 100 ft prior to the beginning of the taper.
  - *Downstream of the painted nose:* This classifier was positioned approximately 500 to 750 ft downstream from the painted nose.

Average traffic speeds and volumes were recorded for 15-minute intervals continuously during the study.

A video recorder system was used primarily to collect merge/diverge locations of vehicles and to record unusual or critical behavior (e.g., braking, swerving, or use of the shoulder beyond the acceleration lane) in the vicinity of the study sites. A pan-tilt-zoom camera was mounted to a mast arm on a video trailer. The trailer was parked off of the roadway in a location where the camera could view a significant portion of the freeway and the merge/diverge area. A digital video recorder stored the video data, which was later viewed in the office.

At each site, a data collector marked locations along the acceleration or deceleration lane to divide it into thirds. These locations were marked with paint either on the shoulder or near the lanes. Orange traffic cones were placed at the markings for a few minutes prior to each study period. The locations marked by the paint and traffic cones served as reference points that could be distinguished on the video for data reduction purposes. In the office, the videos were viewed to record the locations of merge/diverge maneuvers along the SCLs (i.e., in the first, middle, or last third of the SCL) and to note unusual or critical behavior. Figures 12 and 13 illustrate examples of the general equipment setup for typical straight entrance and exit ramps.

An exception to the data collection procedures described above occurred at the two entrance ramps in Pittsburgh. Rather than collecting speed and volume data for the freeway mainline using traffic classifiers, speed and volume data were collected using non-intrusive radar technology. Speed and volume data were collected at one location near the gore of the entrance ramp.

5.3 Data Processing

The laser gun readings, classifier measurements, and the video data collection required additional processing to ensure data quality. The three types of data were tied together as necessary through synchronization of internal clocks within the equipment.

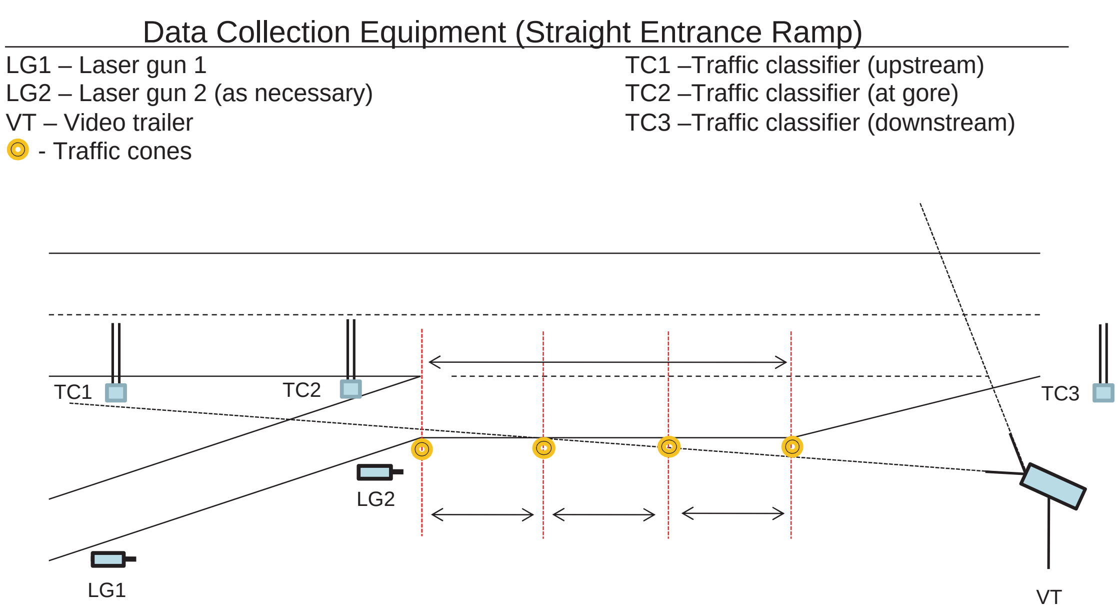


Figure 12. Example of equipment setup at a typical entrance ramp.

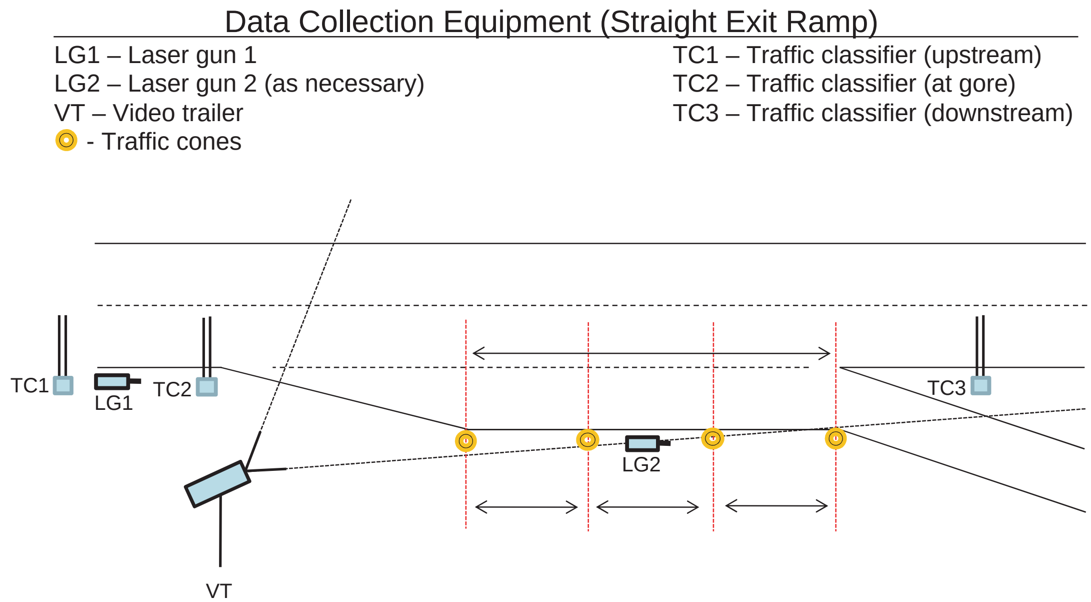


Figure 13. Example of equipment setup at a typical exit ramp.

5.3.1 Laser Gun Data Processing

The laser gun readings were processed using extensive quality checks, extrapolated as needed, and then smoothed using a local linear smoother. This was done to ensure that the profiles analyzed best reflected true vehicle profiles and that sources of measurement variability in the individual profiles were minimized.

The quality checks for each speed profile generated from the laser gun readings included:

- Ensuring vehicle descriptions matched for ramps with multiple lasers,
- Exclusion of erroneous/inconsistent data points due to potential interferences from other vehicles,
- Deletion of profiles with large gaps in readings, and
- Exclusion of data points with invalid distances based on ramp length.

The quality checks were implemented by visual inspection of the speed-distance profiles for each vehicle run.

The merge and diverge locations are based on the point in time when the laser gun measurements ended or began, respectively. In some cases, the laser gun may not have been able to find the vehicle speed at the beginning or ending of the tracking process, resulting in initial or final data points where a time stamp was recorded but without an associated vehicle speed. In these cases, the first or last speed measurement was extrapolated as a constant and the location of the merge/diverge maneuver was calculated assuming that the first recorded speed was constant from the first time stamp until the first valid speed measurement (in the case of exit ramps) or that the last recorded speed was maintained until the point in time when the final measurement was taken (in the case of entrance ramps).

After the quality checks and endpoint extrapolation were implemented, a local smoother was applied to the results. The laser gun values are reported to the nearest whole number, which results in distinct jumps in speed over time/distance. Since this is not representative of the reality of vehicle performance, it was decided that a local linear smoother would better represent changes in speed and acceleration over distance. Using this approach allowed for seamless transition between lasers and reduced the impact of any potential erroneous measurements. The smoothing was performed using the PROC LOESS procedure in SAS v 9.2 with a local linear regression and smoothing parameter of 0.5. An example of a smooth vehicle speed profile is provided in Figure 14.

5.3.2 Classifier Data Processing

The traffic classifier data were summarized into 15-minute intervals for each classifier roadway position. The average vehicle speed and the volume were recorded for each complete time interval. For time intervals that did not cover complete quarter-of-an-hour time periods (i.e., those at the beginning and end of the data collection), volume measures were excluded and average speed was compared to previous or latter time periods for consistency. Incomplete intervals with inconsistent average speed measurements were excluded.

The average speed and volume for the three classifier positions were plotted against the time stamp for the intervals to compare the consistency in the measurements. As expected, for exit ramps the volumes were lower at the downstream position

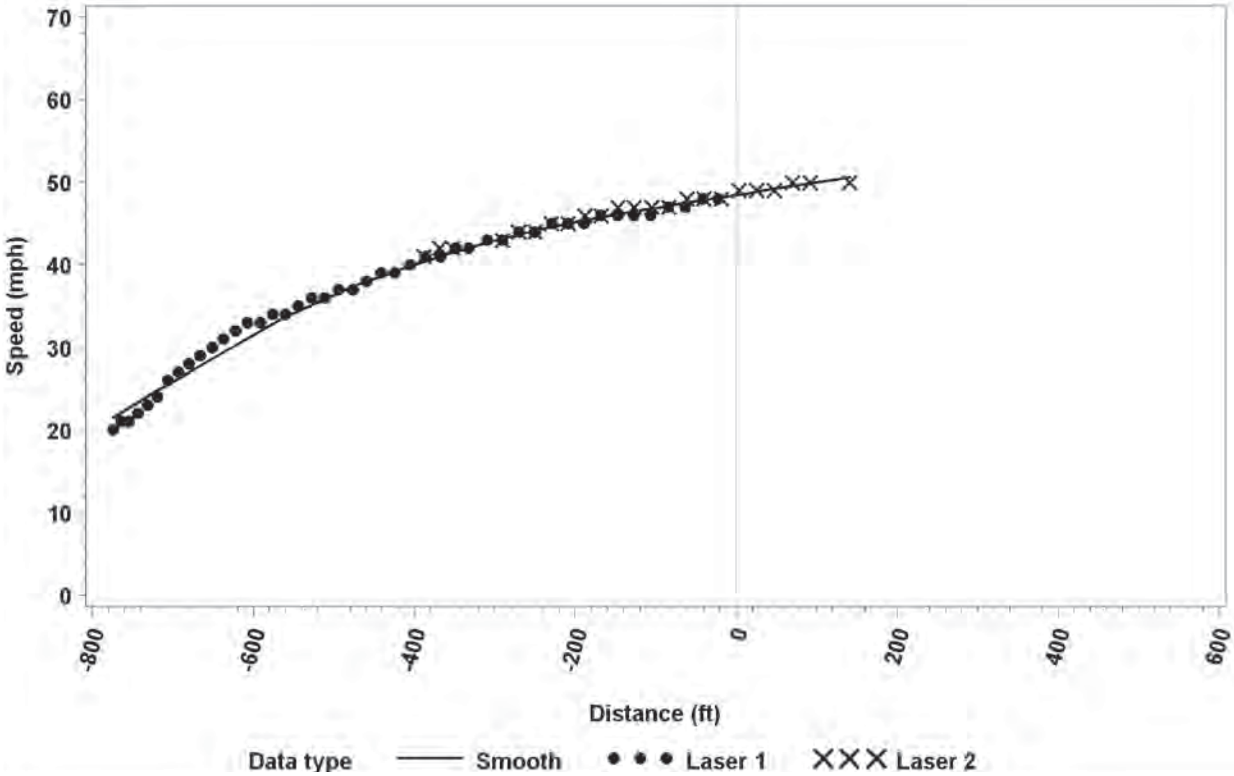


Figure 14. Example of smoothed vehicle speed profile across distance.

and for entrance ramps, the volumes were higher in the downstream. No major anomalies in the average speed between locations were observed.

The painted nose collection point was selected to be representative of the freeway conditions (volume and speed) at the merge or diverge location. The average 15-minute interval speeds were divided into three speed categories:

- Free Merge/Diverge: 15-minute average freeway speed above 50 mi/h;
- Constrained Merge/Diverge: 15-minute average freeway speed between 40 and 50 mi/h, inclusive; and
- Forced Merge/Diverge: 15-minute average freeway speed less than 40 mi/h.

5.3.3 Video Data Processing

Video recordings from each of the study periods were reviewed in the office primarily to collect merge/diverge location information, but also to record unusual or critical behavior such as braking, swerving, or use of the shoulder beyond the end of the taper. For each 2-hour study period, a 15-minute period was selected for detailed data collection. For the given 15-minute time period, the merge/diverge location was recorded for each ramp vehicle entering/exiting the freeway. For entrance ramps, merge locations were grouped into five categories:

- Before the painted nose,
- In the first third of the SCL,
- In the middle third of the SCL,
- In the last third of the SCL, and
- Into or beyond the taper.

Similarly for exit ramps, diverge locations were grouped into five categories:

- Before or within the taper,
- In the first third of the SCL,
- In the middle third of the SCL,
- In the last third of the SCL, and
- Beyond the painted nose.

The merge/diverge locations were estimated based on known distances/locations along the SCL marked with paint (and cones) during data collection.

During the same 15-minute time period, any unusual or critical maneuvers in the vicinity of the freeway mainline ramp terminal were recorded. Several examples of maneuvers that would be considered unusual or critical include:

- A merging vehicle comes to a complete stop within or beyond the taper during free-merge conditions.
- A following (i.e., platooned) vehicle in the SCL stops or decelerates suddenly to avoid a rear-end collision with a slower lead vehicle in the SCL waiting to merge.
- A vehicle merges into the freeway at a lower speed and a following vehicle in the freeway decelerates suddenly or changes lanes to avoid a rear-end collision with the merging vehicle.

5.4 Analysis Results

The analysis considered four measures of effectiveness to evaluate operations near entrance and exit ramps:

- Merge/diverge location,
- Merge/diverge speed,
- Vehicle acceleration/deceleration along the ramp and SCL, and
- Unusual or critical maneuvers in the vicinity of the ramp terminal.

Analyses were performed separately for entrance and exit ramps. For entrance ramps, the following questions were addressed:

1. Were vehicles able to merge into freeway traffic from a point within the SCL, or did they merge prior to the painted nose (i.e., early) or into or beyond the taper (i.e., late)?
2. What was the difference in speed between merging vehicles and freeway vehicles? Were merging vehicles able to enter the freeway traffic at an appropriate speed?
3. Were vehicle acceleration rates greater than, less than, or similar to those assumed in the AASHTO *Green Book?*

For exit ramps, the following questions were addressed:

1. Did vehicles diverge from the freeway traffic at a point along the SCL, or did they diverge prior to or along the taper (i.e., early) or beyond the painted nose (i.e., late)?
2. What was the difference in speed between diverging vehicles and freeway vehicles? Were diverging vehicles reducing their speed before diverging onto the ramp?
3. Were vehicle deceleration rates greater than, less than, or similar to those assumed in the AASHTO *Green Book?*

These questions are explored in depth in the following subsections, first for entrance ramps, and then for exit ramps. The answers to these questions are used to draw general conclusions about how effectively the ramp design accommodated vehicles as they accelerated along the ramp and merged with freeway traffic, or diverged from freeway traffic and decelerated along the ramp.

5.4.1 Entrance Ramps

At entrance ramps, the primary objective of the ramp design is to (1) allow sufficient distance for vehicles to accelerate comfortably from the crossroad terminal (or controlling horizontal alignment, in the case of ramps with a curve radius < 1,000 ft) to freeway speeds by the time a merge is required, and (2) provide sufficient area to allow vehicles to safely find a gap in freeway traffic into which to merge. The distance needed for a vehicle to reach freeway speeds by the time a merge is required is a function of the vehicle's acceleration rate along the ramp and SCL. Ramp design guidelines that assume an acceleration rate greater than what is comfortable or possible for the vehicle fleet may provide lengths that are too short. If an entrance ramp and SCL (the full area over which a vehicle may accelerate before merging) are too short, this may be evident by observations of vehicles waiting to merge beyond the SCL and into or beyond the taper area or by merge speeds that are substantially lower than the speed of the freeway traffic into which they are merging. Both of these measures are considered below. In addition, an analysis is performed to consider the acceleration profiles of the vehicles to determine whether the current vehicle fleet is accelerating as expected in current ramp design guidelines described in the AASHTO *Green Book.*

In summary, under ideal operating conditions along an entrance ramp, vehicles would accelerate to near freeway speed (i.e., within 5 mi/h of the operating speed of the freeway) before merging, and during the merging maneuver vehicles would utilize most of the acceleration lane prior to merging (e.g., vehicles would merge in the middle or last third of the SCL). The greatest concern arises when vehicles merge late and at speeds significantly below the operating speed of the freeway. This type of behavior could be an indication that insufficient length was provided to accelerate to near freeway speeds or that insufficient length was provided for gap acceptance. Lesser concerns are when vehicles merge close to freeway speeds from within the taper area, or when vehicles merge early but at speeds significantly below the operating speed of the freeway. The analyses were designed to compare actual driving practices to these ideal operating conditions and identify areas of concern.

5.4.1.1 Merge Location

For an entrance ramp, vehicles will ideally merge onto the freeway from a point somewhere along the SCL—that is, beyond the painted nose, but prior to the taper. Figures 15 to 20 divide the possible merge locations into five categories: before the painted nose (gore), in the first third of the SCL (early), in the middle third of the SCL (mid), in the last third of the SCL (late), and into or beyond the taper (taper).

The figures below combine the data gathered from the video data reduction and the laser gun data. In Figure 15, the merge location is considered for all vehicle types on all of the study ramps, and broken out by data type (video on the left and laser on the right), and by freeway speeds. The top section shows merge locations for all freeway speed categories combined, the next section shows merge locations during free-flow conditions on the freeway (i.e., freeway speeds > 50 mi/h), the next section shows merge locations during constrained conditions on the freeway (i.e., freeway speeds 40 to 50 mi/h), and the final section shows merge locations during forced conditions on the freeway (i.e., freeway speeds < 40 mi/h).

The proportions of merge location for each speed category shown in Figure 15 are somewhat dissimilar between those obtained from the video data and those obtained from the laser data. For example, under the constrained-merge condition, approximately 5 percent of the vehicles merged in the final third of the SCL or along the taper according to the video data compared to approximately 57 percent of the vehicles according to the laser data. In considering the accuracy and reliability of the data from both data collection sources, the merge locations obtained from the laser data are considered to be more reliable for two reasons. First, the merge locations were directly observed in the field and recorded by the data collectors upon release of the trigger on the laser gun. Second, it is expected that there is more room for error in recording the merge location from the video compared to direct observation in the field due to the angular position of the camera and quality of the video, especially for longer distances between the camera location and the merge locations. Third, the merge location data observed in the field from a sample of vehicles should be more representative of the actual distribution of merge locations throughout the day, compared to selecting merge locations of all vehicles during a given 15-minute period from each 2-hr study period. For these reasons, the remainder of this discussion and information presented in this section includes only the merge positions based upon the laser data.

Figure 15 shows that very few vehicles merged prior to the painted nose, and that for the free-merge and forced-

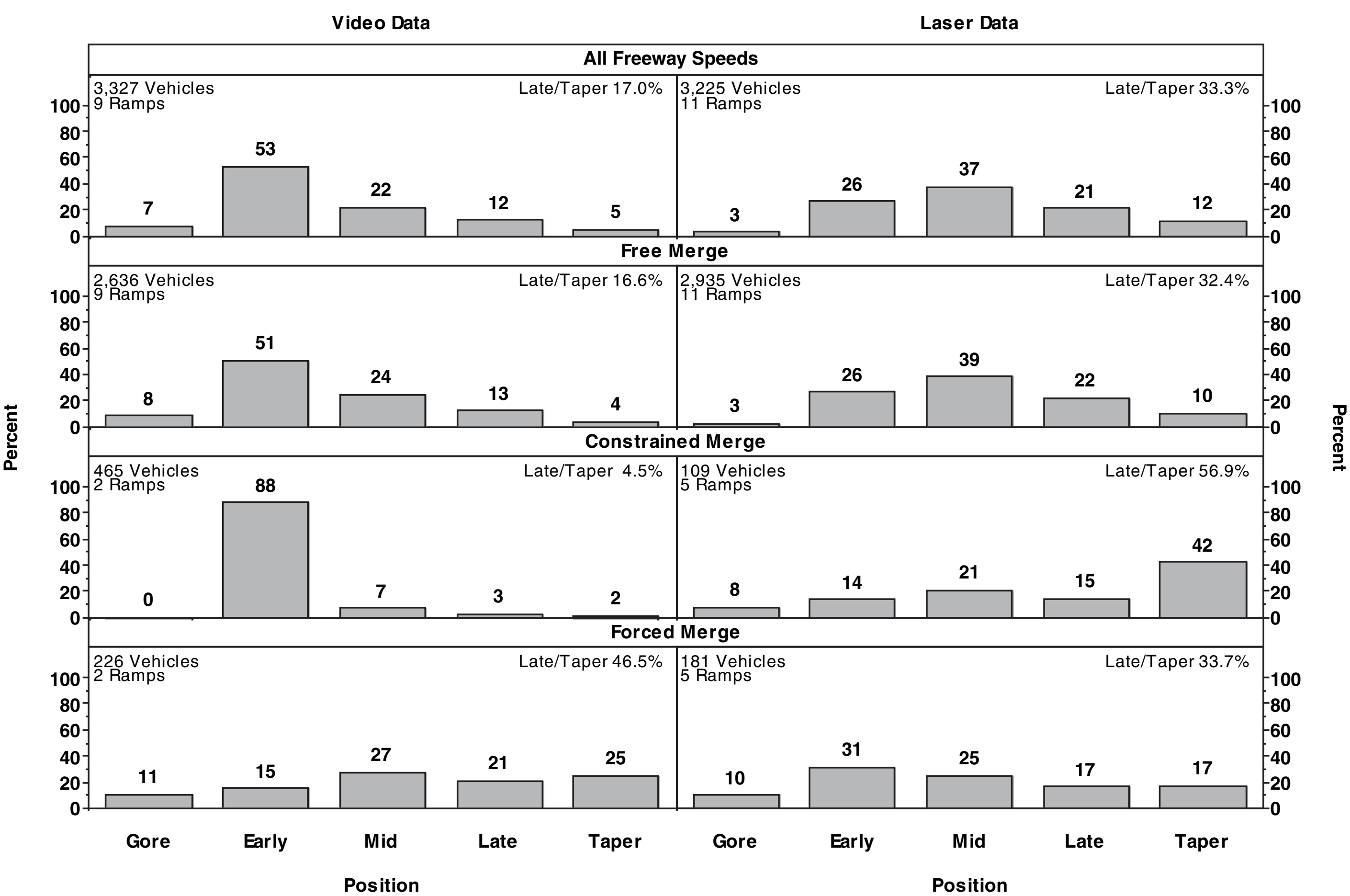


Figure 15. Merge location—all ramps, all vehicle types, by freeway speed categories (all speeds combined, freeway speeds > 50 mi/h, freeway speeds 40 to 50 mi/h, and freeway speeds < 40 mi/h).

merge conditions, a majority of vehicles merge at the early or mid positions, while for the constrained merge, a majority of vehicles merge in the late or taper positions. This observation is intuitive because under free-merge conditions, vehicles should have no trouble finding a gap and should be able to merge as soon as a comfortable merge speed is reached. Under forced-merge conditions, gaps may be less frequent, but merging may take place with a certain amount of order, with each vehicle in the right lane of the freeway allowing one merging vehicle into the lane. Additionally, because freeway traffic is slower, the merging vehicle does not need to accelerate as much and can reach appropriate merging speed earlier. Under constrained merges, however, gaps are likely more difficult to find because they may be infrequent and because freeway traffic has not become congested enough to create a slow, ordered merging process. Figure 15 shows that 42 percent of merges in constrained conditions take place in the taper or beyond, and that an additional 15 percent take place in the final third of the SCL.

Figure 16 shows similar information to that presented in Figure 15, but compares the merge location proportions of passenger cars to those of trucks. In general, the two vehicle types behave similarly, but constrained-merge conditions are more difficult for trucks than for passenger cars. While 41 percent of passenger cars merge in the taper area or beyond under constrained conditions, 56 percent of trucks merge in or beyond the taper. However, caution should be taken when comparing merge locations between passenger cars and trucks under the constrained-merge condition because the percentages for trucks are only based upon nine observations.

Figure 17 compares the merge locations of vehicles merging from loop ramps (i.e., low-speed ramps) to those merging from straight ramps (i.e., high-speed ramps). It appears that those coming from loop ramps tend to merge earlier in the SCL than those merging from straight ramps when the freeway is operating under free-merge conditions. Only two loop ramps were observed, and the observations at those ramps were all taken during free-merge conditions. Therefore, the observations regarding the difference in merge location between loop and straight ramps cannot be made for constrained and forced-merge conditions.

Figure 18 compares the merge locations of vehicles on parallel SCLs to those on taper SCLs. For all speed conditions combined (i.e., free, constrained, and forced), merges from tapered SCLs occur later than those from parallel SCLs. The difference is most extreme under constrained-merge conditions where 61 percent of vehicles on taper SCLs merged at a location beyond where the tapered lane narrowed to 12 ft, while only 12 percent of vehicles on a parallel SCL merged that late. One possible explanation for this is that on tapered SCLs, drivers tend to follow the path of the right edge line as they travel into the freeway lane, therefore crossing into the freeway lane at the point where the taper has narrowed to the approximate width of the vehicle. Parallel SCLs, on the other hand, are marked more similarly to a lane with a more defined end point, encouraging drivers to find a merge location before the taper. For all speed conditions combined, drivers on parallel SCLs merged into the freeway lane within the full length the SCL 94 percent of the time, while only 74 percent of vehicles merged in the freeway lane prior to the taper lane narrowing to 12 ft. When comparing the merge locations of vehicles on parallel SCLs to those on taper SCLs, it has to be recognized that the distance from the painted nose to the taper is longer for parallel SCLs than for tapered SCLs.

In Figure 19, ramps that meet or exceed *Green Book* criteria for ramp and SCL length are compared to ramps that do not. When all merge conditions are considered together, there appears to be very little difference in the distribution of merge locations between the two categories. This similarity holds true when just considering free-merge conditions and forced-merge conditions. However, clear differences are apparent between the two groups when looking specifically at the constrained-merge observations. Under these conditions, vehicles are much more likely to merge late from a ramp that does not meet *Green Book* criteria than from those that do. At ramps that meet or exceed *Green Book* criteria, 11 percent of merges observed occurred in the late or taper areas, while for the ramps that do not meet *Green Book* criteria, 67 percent occurred in the late or taper areas. This may indicate that ramp length is less important when gaps are easy to find or when freeway speeds are low, but that it becomes more significant when freeway speeds are moderate and merge opportunities are more difficult to find. Again, caution should be taken when comparing merge locations under the constrained-merge condition because of a low number of observations (19) for vehicles on ramps that meet or exceed *Green Book* criteria.

Figure 20 compares the distribution of merge locations for free-flow merges (i.e., the speed of the vehicle on the ramp is not impeded by a lead vehicle on the ramp) and platooned merges (i.e., merges made by vehicles following a lead vehicle closely enough that the speed, acceleration, and merge location of the following vehicle may be influenced by the lead vehicle). Considering all merge conditions together, very little difference is evident between the two distributions. However, under constrained-merge conditions, platooned vehicles are much more likely to merge beyond the end of the SCL than free-flowing vehicles (58 percent versus 3 percent in the taper area), and under forced-merge conditions, platooned vehicles tend to merge earlier than free-flow vehicles (73 percent versus 55 percent in the gore, early, and mid areas). This may indicate that when freeway speeds are moderate and merge gaps are difficult to find, the lead vehicle has an advantage in

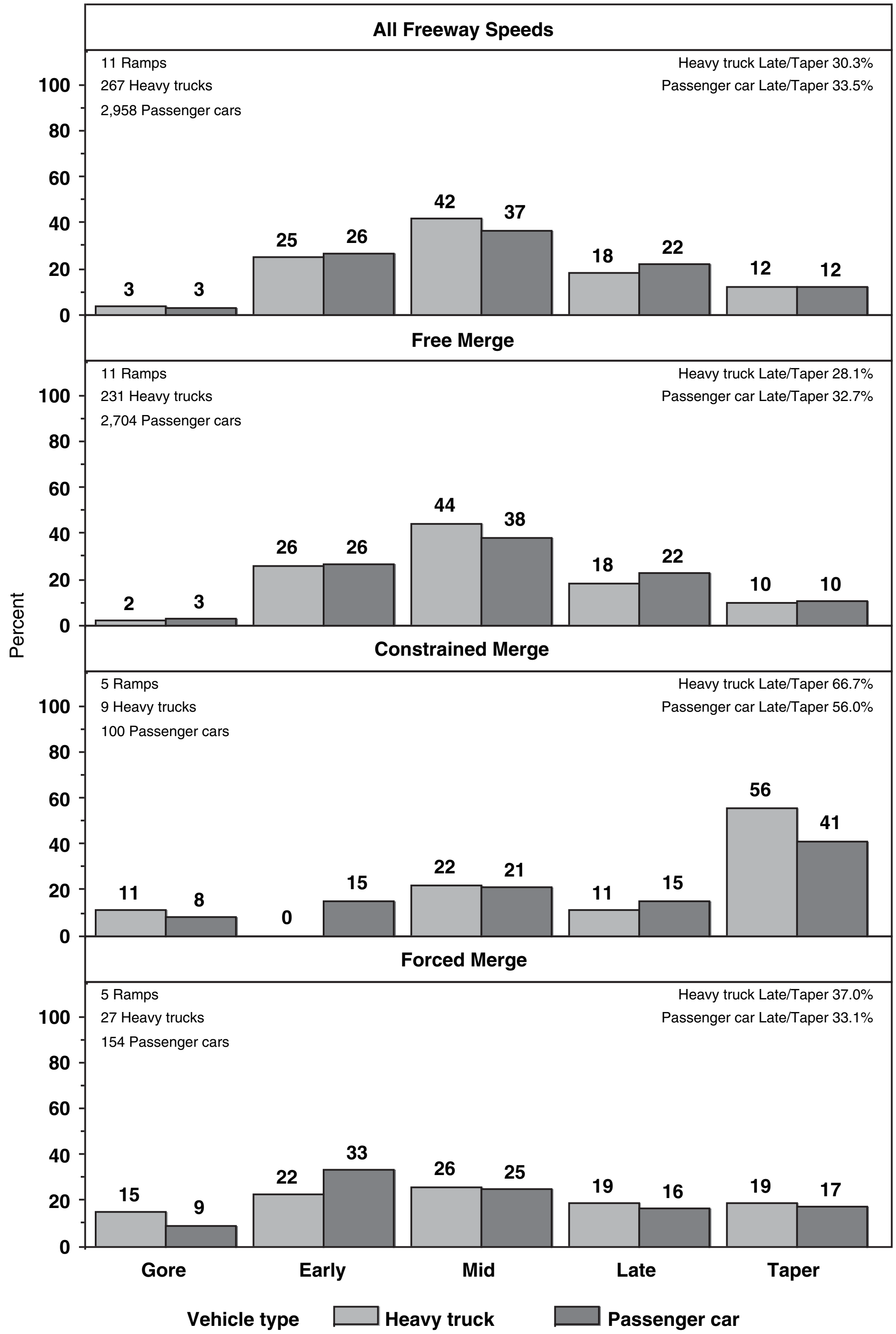


Figure 16. Merge location—all ramps by vehicle type (passenger cars versus trucks).

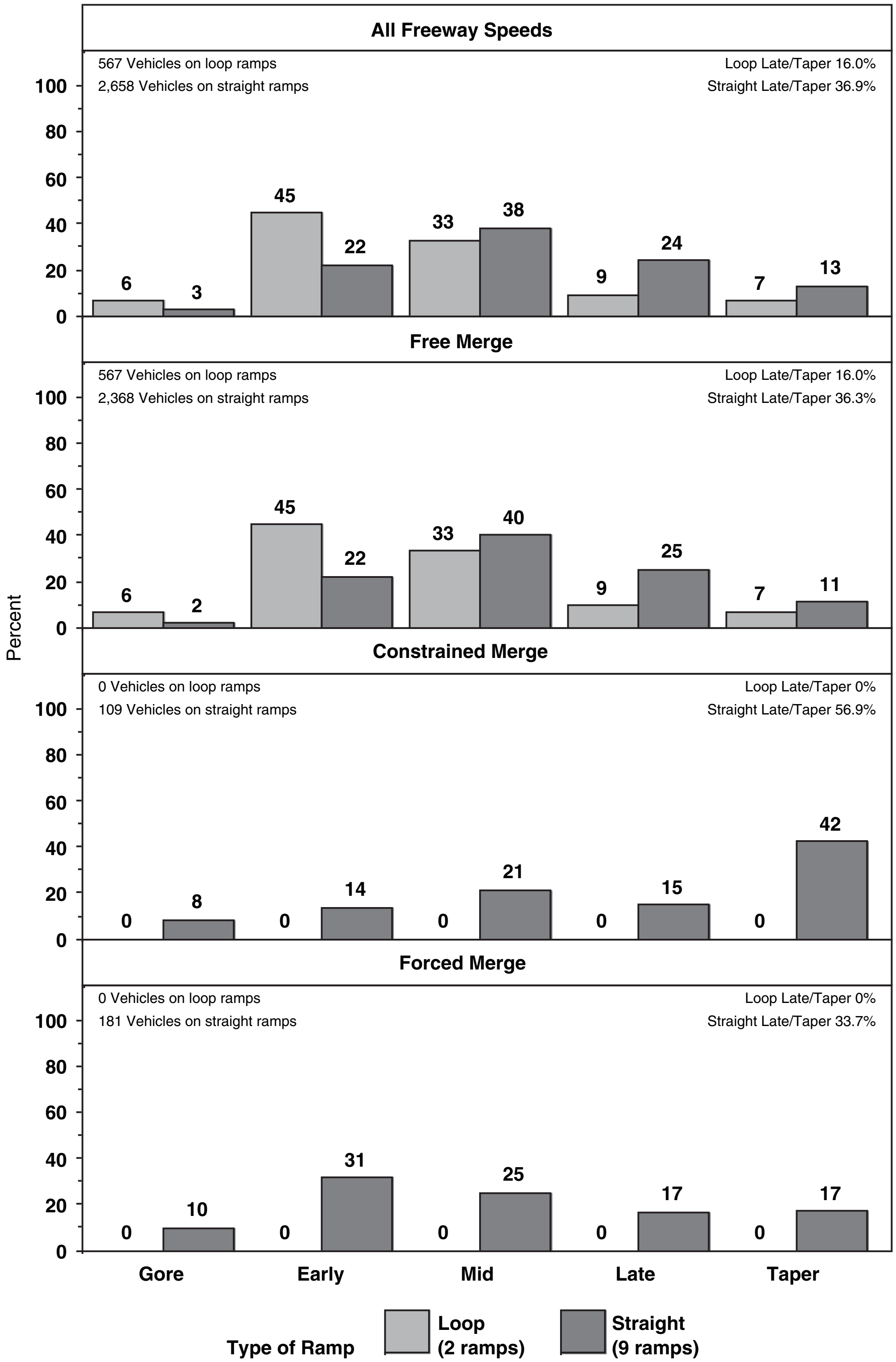


Figure 17. Merge location—by ramp type (straight versus loop).

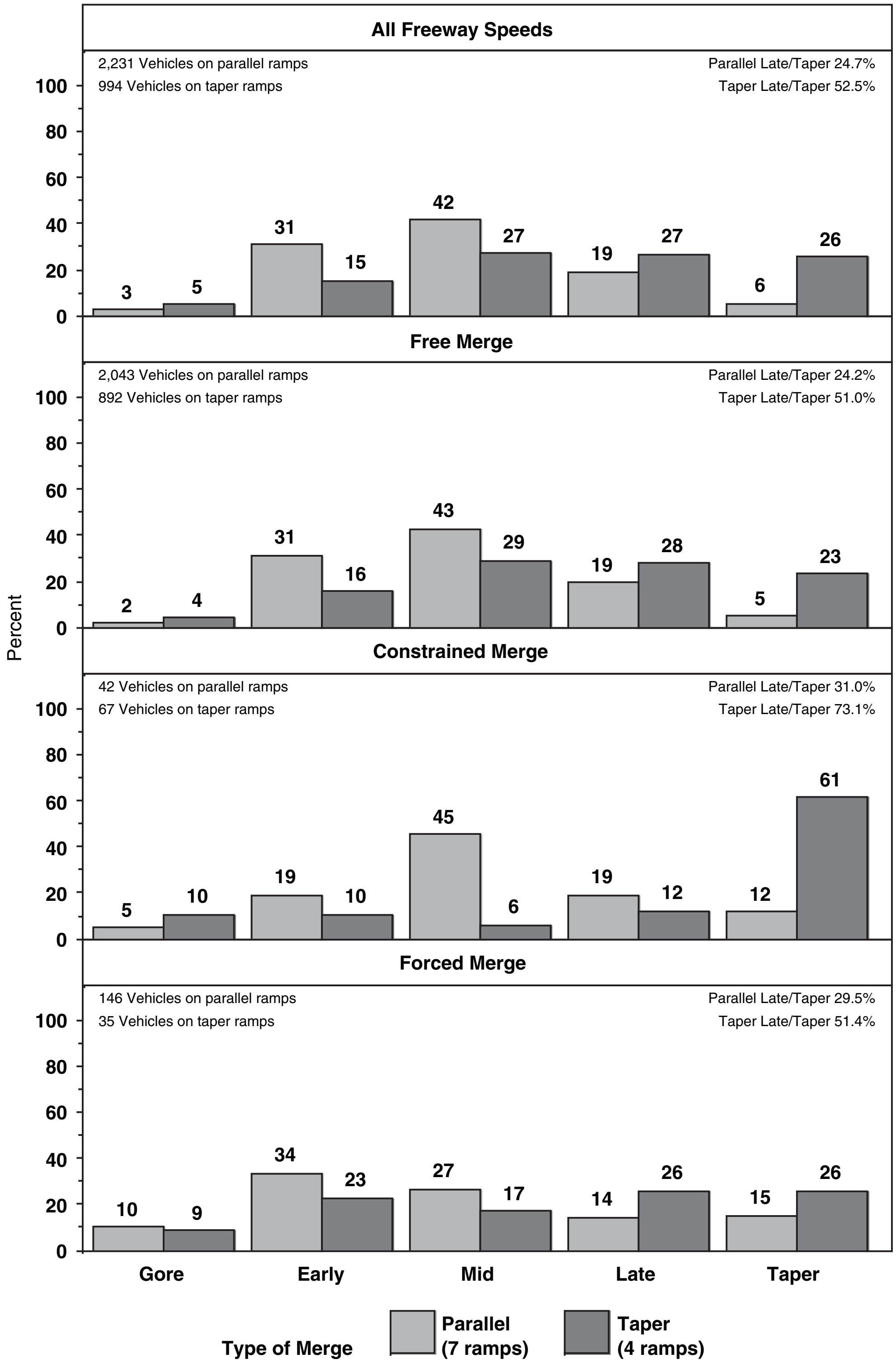


Figure 18. Merge location—by merge type (parallel versus taper).

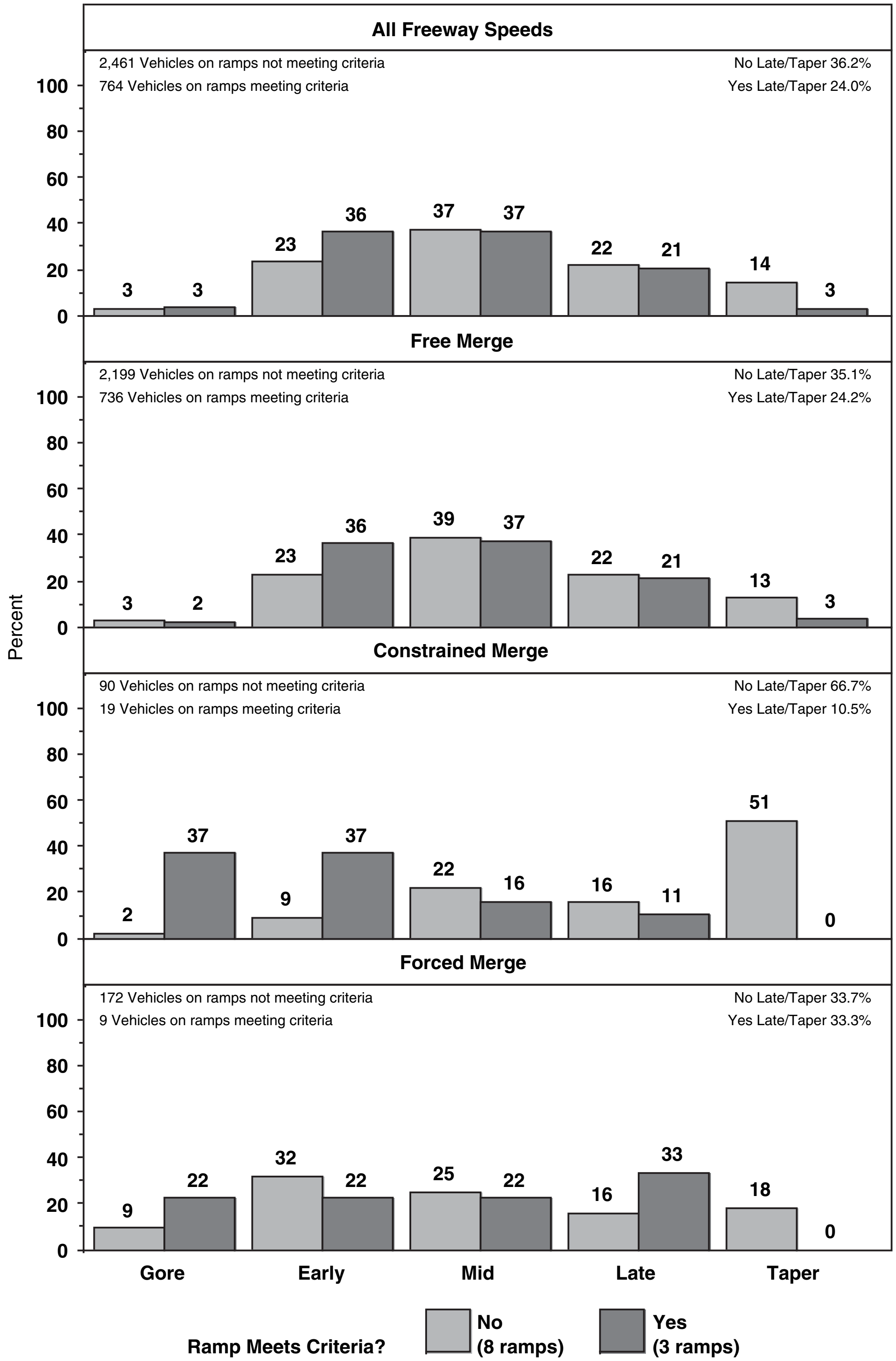


Figure 19. Merge location—existing conditions (ramps meet current criteria: yes versus no).

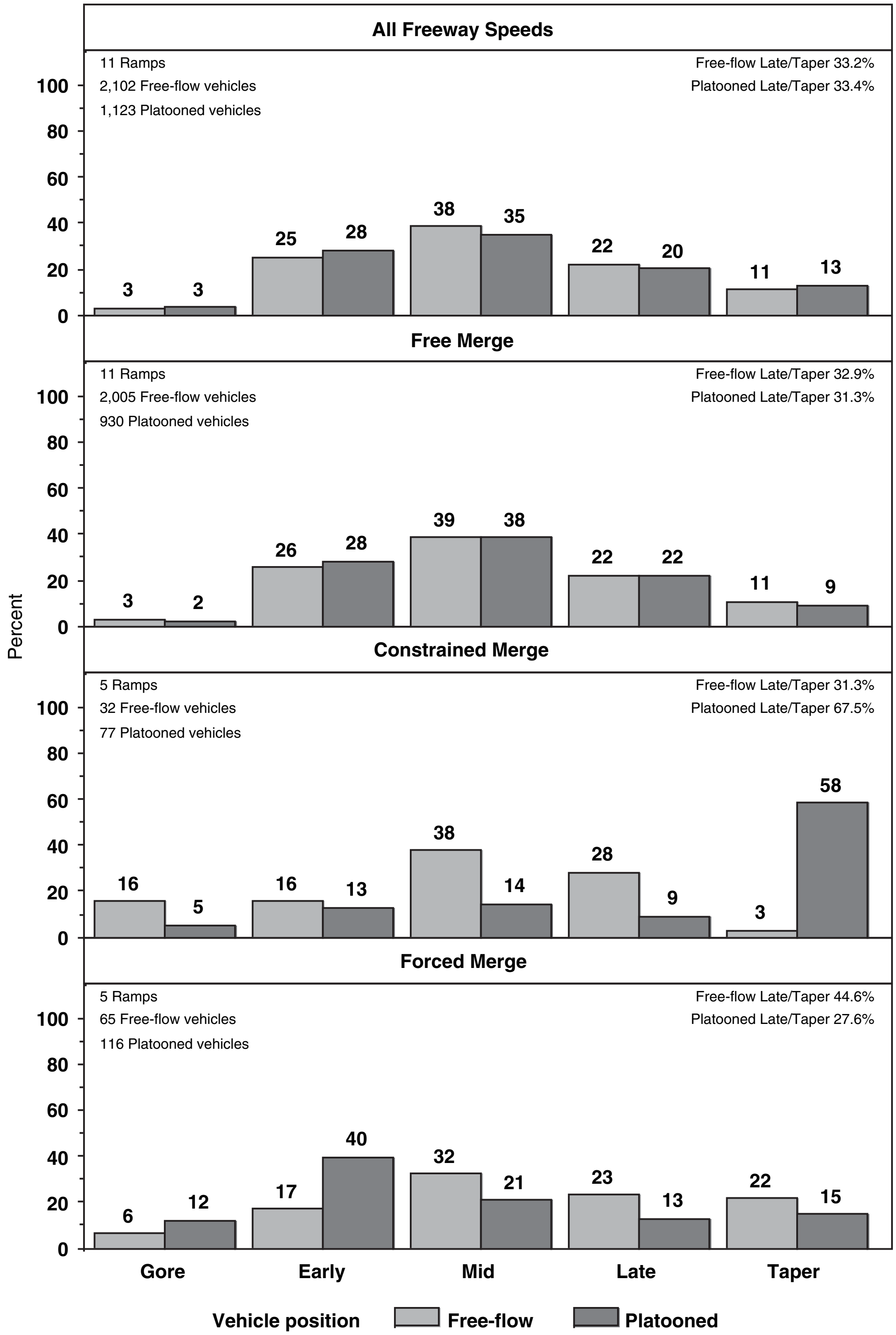


Figure 20. Merge location—by vehicle position (free-flow versus platooned).

taking the first available gap, while in forced-merge conditions, the slower freeway speeds may allow platooned vehicles to merge into freeway traffic at an earlier location than the lead vehicle. It should also be noted that under free-merge conditions, when about twice as many free-flow vehicles as platooned vehicles were observed while under constrained- and forced-merge conditions, the ratio was reversed, with nearly twice as many observations of platooned vehicles as free-flow vehicles merging. This makes sense, since forced and constrained merges tend to take place during peak demand.

Figure 21 shows the percentage of vehicles that merge in the late or taper regions by length of the SCL. In general, a greater percentage of vehicles merge in the late or taper areas at ramps with a short SCL, and fewer vehicles merge later along the SCL when it is long.

The most significant findings from the examination of merge locations are as follows:

- Vehicles tend to merge later under constrained-merge conditions than under free- or forced-merge conditions.
- The distribution of merge locations for trucks is very similar to the distribution of merge locations for passenger cars.
- In free-merge conditions, vehicles tend to merge near the middle of the SCL.
- In forced-merge conditions, vehicles tend to merge earlier in the SCL, but also the distribution of merge locations under forced-merge conditions appears to be the flattest when comparing the three merge conditions. In other words, merge location is most evenly spread among the five merge locations during forced-merge conditions.
- A higher percentage of vehicles merge early on loop ramps compared to straight ramps.
- Vehicles merge later at tapered SCLs than at parallel SCLs. This is most evident under constrained-merge conditions.
- Whether the ramp length met *Green Book* criteria was most important under constrained conditions.
- The distribution of merge locations for free-flow vehicles is very similar to the distribution of merge locations for platooned vehicles, considering all speed categories combined. The greatest difference occurs under constrained-merge

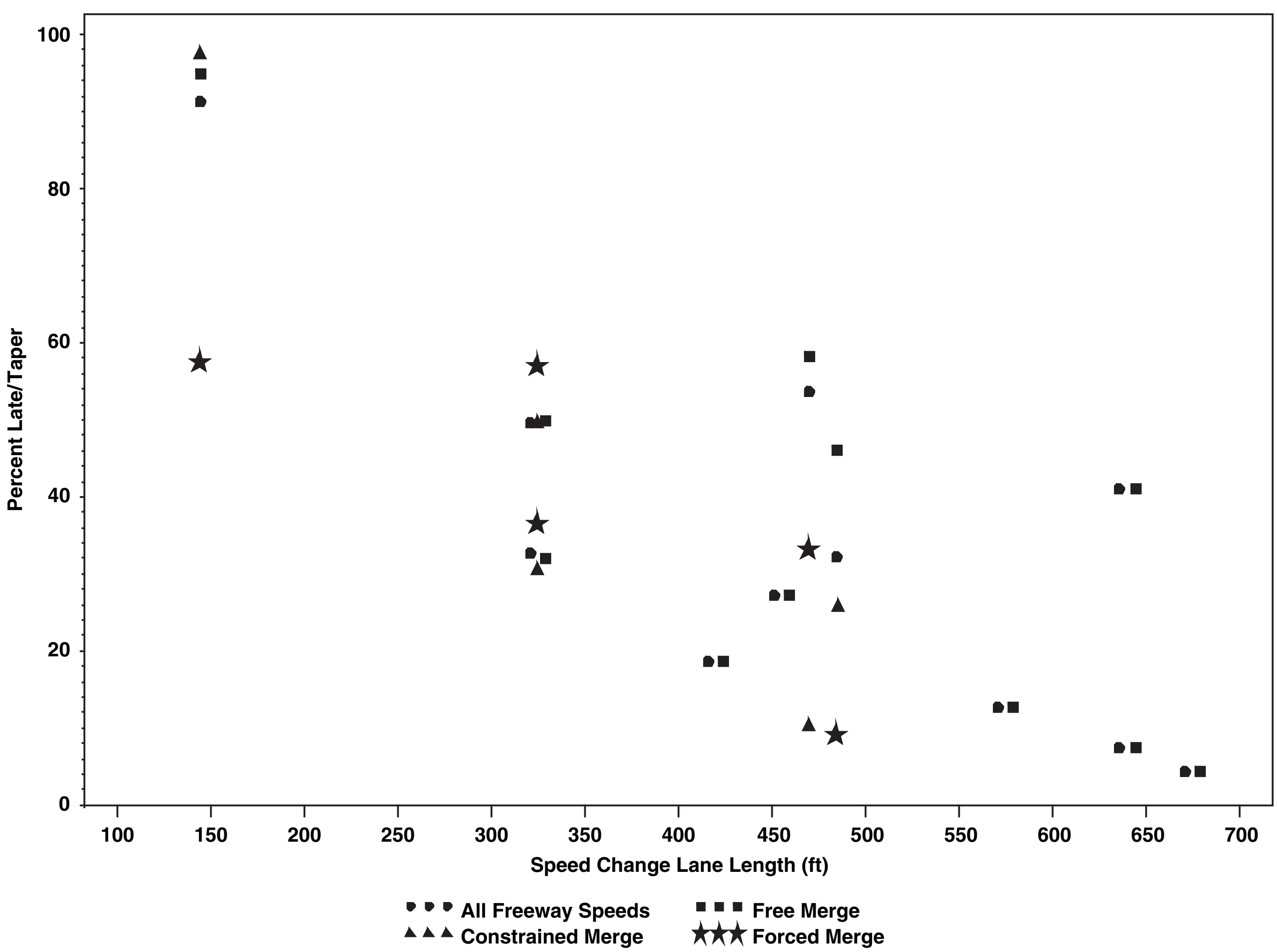


Figure 21. Percent of vehicles merging in the late or taper regions of the SCL by SCL length.

conditions, in which case a larger percentage of platooned vehicles merge later than free-flow vehicles.

5.4.1.2 Merge Speed

The data presented in this section compare the speed of merging vehicles at the time they begin to merge to the speed of freeway traffic in the rightmost lane. Specifically, this is a comparison of final merge speeds of vehicles based upon the laser gun data compared to the average 15-minute free-way speed of the rightmost lane of the freeway at the time of the merge. The *Green Book* states that the "geometrics of the ramp proper should be such that motorists may attain a speed that is within 10 km/h [5 mi/h] of the operating speed of the freeway by the time they reach the point where the left edge of the ramp joins the traveled way of the freeway." It goes on to state that the location described is where the right edge of the ramp is 12 ft from the right edge of the through lane of the freeway.

Table 20 provides a summary of the mean speed differential measured between merging vehicles and freeway traffic for

Table 20. Speed differential by ramp type, merge type, and vehicle type.

Type of ramp	Type of merge	No. of ramps	Vehicle type	No. of vehicles	Merge speed minus freeway speed (mi/h)				P-value for t-test of mean less than - 5 mi/h
					Mean	Std dev	Min	Max	
All freeway speeds									
Loop	Parallel	1	Truck	31	-24.8	5.4	-36.6	-15.6	0.003
			Passenger car	291	-18.0	6.0	-33.5	1.0	0.048
	Taper	1	Truck	8	-10.3	5.1	-18.0	-2.7	0.031
			Passenger car	237	-4.6	7.0	-27.9	13.8	0.559
Straight	Parallel	6	Truck	194	-17.7	8.5	-55.6	11.5	0.001
			Passenger car	1,715	-12.1	7.8	-52.0	23.8	0.015
	Taper	3	Truck	34	-10.2	6.5	-20.0	9.1	0.005
			Passenger car	715	-6.7	9.5	-53.5	16.9	0.029
Free merge: Freeway speed > 50 mi/h									
Loop	Parallel	1	Truck	31	-24.8	5.4	-36.6	-15.6	0.003
			Passenger car	291	-18.0	6.0	-33.5	1.0	0.040
	Taper	1	Truck	8	-10.3	5.1	-18.0	-2.7	0.021
			Passenger car	237	-4.6	7.0	-27.9	13.8	0.636
Straight	Parallel	6	Truck	165	-18.7	7.2	-55.6	3.8	0.001
			Passenger car	1,556	-12.6	6.8	-52.0	23.8	0.016
	Taper	3	Truck	27	-10.6	5.4	-20.0	-1.0	0.001
			Passenger car	620	-6.5	8.5	-53.5	13.5	0.009
Constrained merge: Freeway speed between 40 and 50 mi/h									
Loop	Parallel	0	Truck						
			Passenger car						
	Taper	0	Truck						
			Passenger car						
Straight	Parallel	2	Truck	4	-12.8	5.6	-31.6	5.6	0.344
		3	Passenger car	38	-15.2	13.9	-38.7	12.4	0.158
	Taper	1	Truck	5	-1.6	7.1	-9.4	9.1	0.553
		2	Passenger car	62	-11.5	8.2	-50.3	16.9	0.286
Forced merge: Freeway speed < 40 mi/h									
Loop	Parallel	0	Truck						
			Passenger car						
	Taper	0	Truck						
			Passenger car						
Straight	Parallel	3	Truck	25	-9.6	9.5	-27.2	11.5	0.052
			Passenger car	121	-7.1	10.6	-29.3	21.2	0.027
	Taper	1	Truck	2	-16.5	0.5	-16.8	-16.1	0.116
		2	Passenger car	33	-13.8	6.4	-28.1	-1.9	0.006

the available combinations of ramp type, merge type, vehicle type, and merge condition. For each ramp and merge type, the mean value is the average of the individual ramp averages. Also provided are the standard deviation (pooled across individual ramps), minimum, and maximum values measured for each condition. A negative mean speed in Table 20 indicates that the freeway speed is greater than the merge speed. For conditions where the mean speed differential is greater than or equal to –5 mi/h, vehicles are within the appropriate merge speed according to the *Green Book* guidance; or, stated in another way, whenever the mean speed differential is less than –5 mi/h (e.g., a mean value of –10 mi/h indicates vehicles are merging 10 mi/h below freeway speeds), vehicles are merging into the freeway lane at speeds below the assumed design conditions according to the current design policy, which assumes that merging vehicles will be traveling at a speed within 5 mi/h of the freeway speed.

Considering only observations made during free-merge conditions, the speed differentials for all combinations of geometric design and vehicle types were significantly less than –5 mi/h, except for passenger cars entering the freeway on loop taper entrance ramps. This indicates that most vehicles did not merge at speeds within 5 mi/h of the average freeway operating speed. For all combinations of ramp and merge type, the difference between merge speed and freeway speed was less for cars than for trucks.

Fewer vehicles were observed under constrained-merge and forced-merge conditions, and these observations were only made at straight ramps. Under these conditions, cars and trucks behaved more similarly than under free-merge conditions.

To further evaluate the effect of different ramp characteristics on the difference between merge speed and freeway speed (i.e., speed differential), a mixed effects analysis of variance (ANOVA) model was developed. The model included main effects for ramp type (straight versus loop) and merge type (parallel versus taper). The interaction effect between the two was not included based on comparison of the differences of group means, and it would not have been estimable given the number of loop ramps. The model also includes a random ramp effect to account for individual ramp-to-ramp variability and the fact that repeated measurements were taken at each ramp. The between-ramp variability differs for parallel and taper ramps. The model was estimated using restricted maximum likelihood (REML), and estimates of the differences between ramp types and merge types were obtained using least squares means. The statistical degrees of freedom for evaluation of the parameters were based on the number of ramps, not the number of vehicles.

The mean speed differential for each ramp and merge type combination was evaluated to determine if it was statistically less than –5 mi/h. The results are provided by vehicle type and freeway merge condition in the last column of Table 20. Although there is only one ramp type for each merge type for loop ramps, the ANOVA model was used to evaluate each of the combinations. For passenger cars under free-merge conditions, the following results were obtained from the ANOVA model:

- The mean speed differential for straight ramps (both types of merge) is –10.06 mi/h and is statistically less than –5 mi/h (p-value = 0.01). For loop ramps, the mean of –8.27 mi/h is also statistically less than –5 mi/h at the 90% confidence level (p-value = 0.06).
- For parallel merge ramps (both ramp types), the mean speed differential of –12.74 mi/h is statistically less than –5 mi/h with a p-value of 0.02. The taper ramps have an average estimated differential of –5.59 mi/h and are not statistically less than –5 mi/h (p-value = 0.13).
- Loop ramps tend to have a slighter higher mean merge speed differential (1.97 mi/h) but they are not statistically different from straight ramps (p-value = 0.10).
- Parallel ramps have a lower merge speed differential than taper ramps (–7.15 mi/h) that is statistically significant at the 90% confidence level (p-value = 0.07).

Table 21 examines whether ramp lengths that meet or exceed *Green Book* criteria experience a lower speed differential between merging vehicles and freeway traffic than ramps with lengths shorter than the minimum recommended *Green Book* criteria. At parallel SCLs, both cars and trucks merged at speeds closer to freeway speeds at ramps that met *Green Book* criteria than at ramps that did not meet the criteria. At tapered SCLs, the speed differentials for both cars and trucks were very similar at ramps that met criteria and those that did not. When combining merge types, vehicles merged at speeds much below the average operating speeds of the freeway on ramps that did not meet current *Green Book* criteria than on ramps that met current *Green Book* criteria.

Figure 22 presents a box plot illustrating similar information to that presented in Table 20, but breaks the observations down by merge location. For each merge condition, the first box represents merges that took place at the gore, early, or mid positions, while the second box represents merges that took place at the late or taper positions. For free- and forced-merge conditions, the mean is similar between the two categories of merge location; however, under constrained-merge conditions, vehicles that merged later merged at a much closer speed to that of freeway traffic than those that merged earlier. This was true for both passenger cars and heavy vehicles.

Figures 23 through 26 show subsets of the data presented in Figure 22. Figure 23 presents observations made at loop ramps with parallel SCLs. For this ramp and merge type,

Table 21. Speed differential by existing conditions (ramps meet current criteria: yes versus no).

Ramps meet current criteria	Type of merge	No. of ramps	Vehicle type	N obs	Mean	Std dev	Min	Max
No	Parallel	5	Trucks	198	-22.0	8.1	-55.6	11.5
			Passenger cars	1,522	-16.1	8.0	-52.0	21.2
	Taper	3	Trucks	40	-10.0	6.4	-20.0	9.1
			Passenger cars	701	-5.7	8.7	-53.5	16.9
Yes	Parallel	2	Trucks	27	-10.4	8.2	-33.0	3.8
			Passenger cars	484	-5.1	6.3	-34.4	23.8
	Taper	1	Trucks	2	-10.7	2.7	-12.7	-8.8
			Passenger cars	251	-7.6	9.5	-50.3	13.5
No	Both merge types combined	8	Trucks	238	-17.5	7.9	-55.6	11.5
			Passenger cars	2,223	-12.2	8.2	-53.5	21.2
Yes	Both merge types combined	3	Trucks	29	-10.5	8.1	-33.0	3.8
			Passenger cars	735	-6.0	7.6	-50.3	23.8

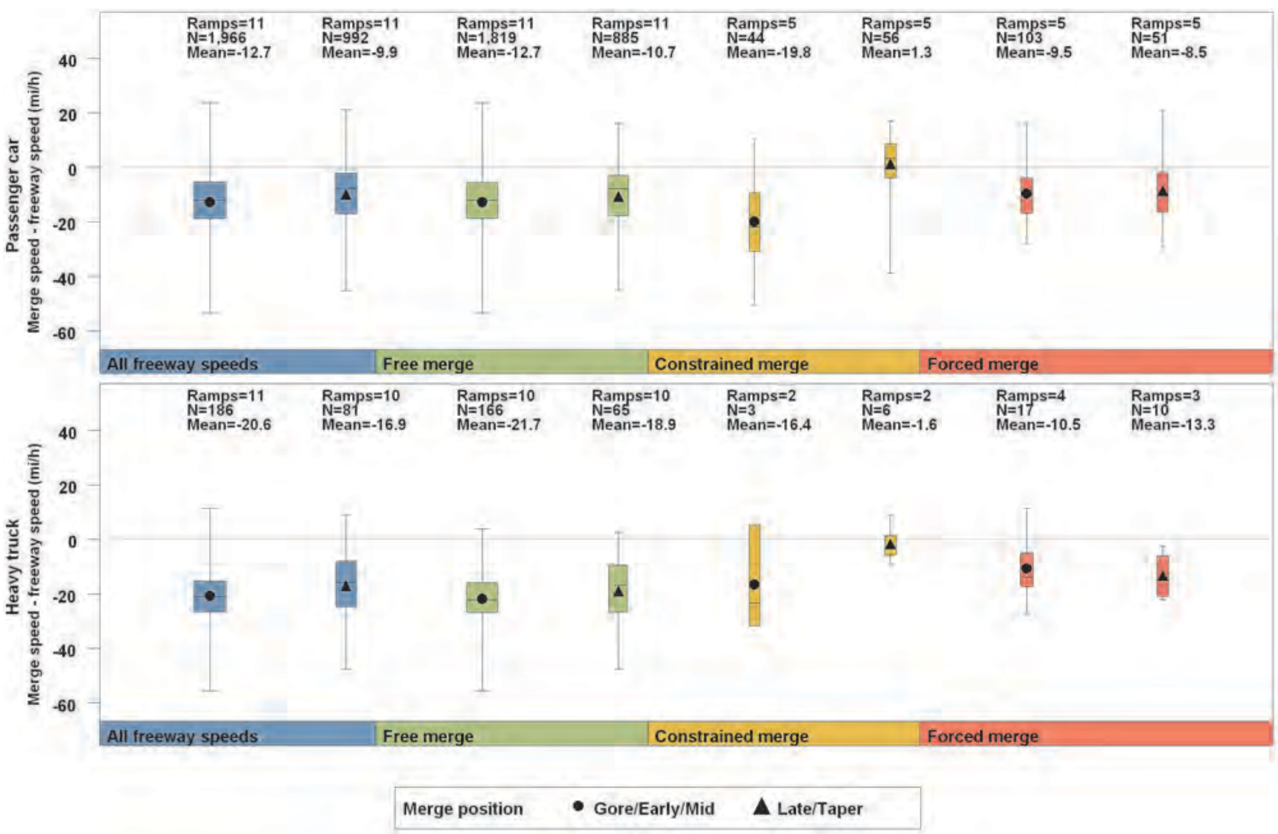


Figure 22. Merge speed differential by vehicle type and merge location for all ramp types.

Figure 23. Merge speed differential by ramp type, merge type, vehicle type, and merge location (ramp type: loop; merge type: parallel).

Figure 24. Merge speed differential by ramp type, merge type, vehicle type, and merge location (ramp type: loop; merge type: taper).

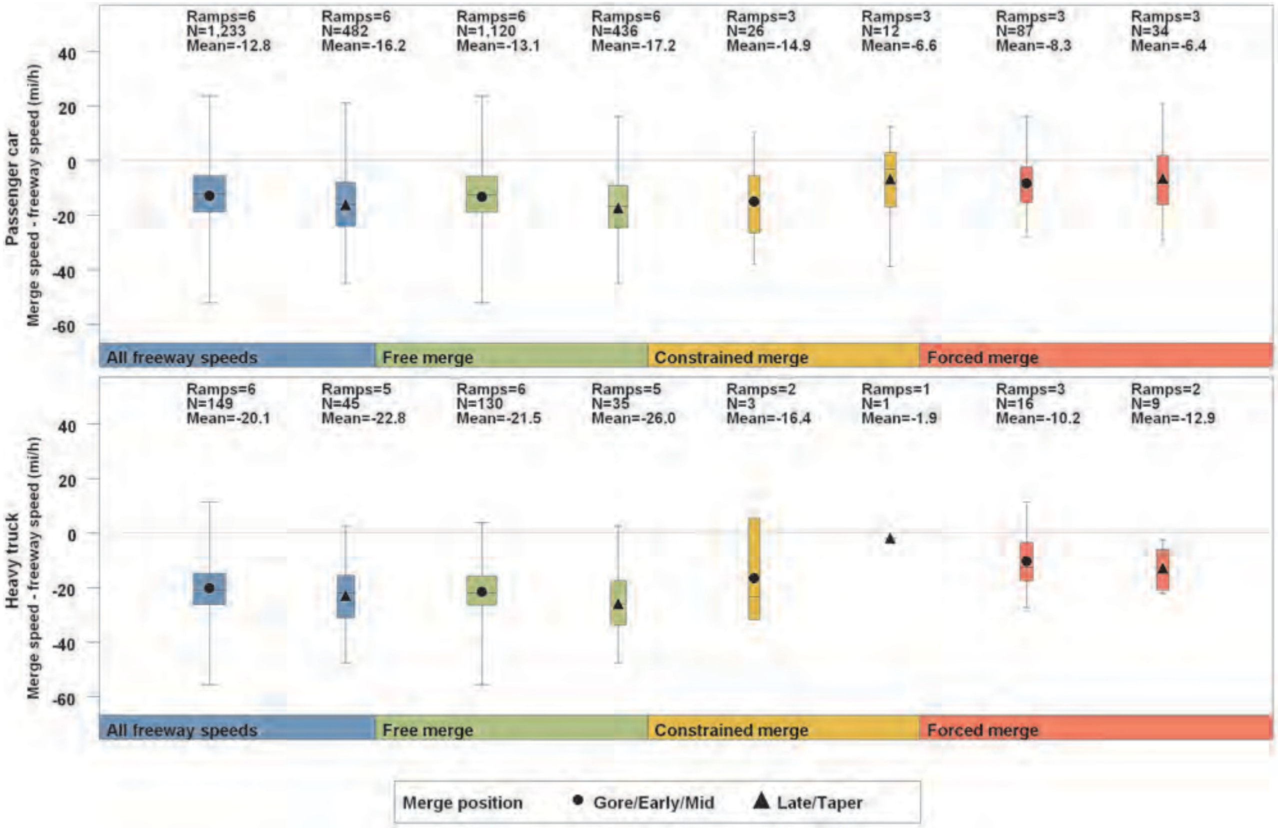


Figure 25. Merge speed differential by ramp type, merge type, vehicle type, and merge location (ramp type: straight; merge type: parallel).

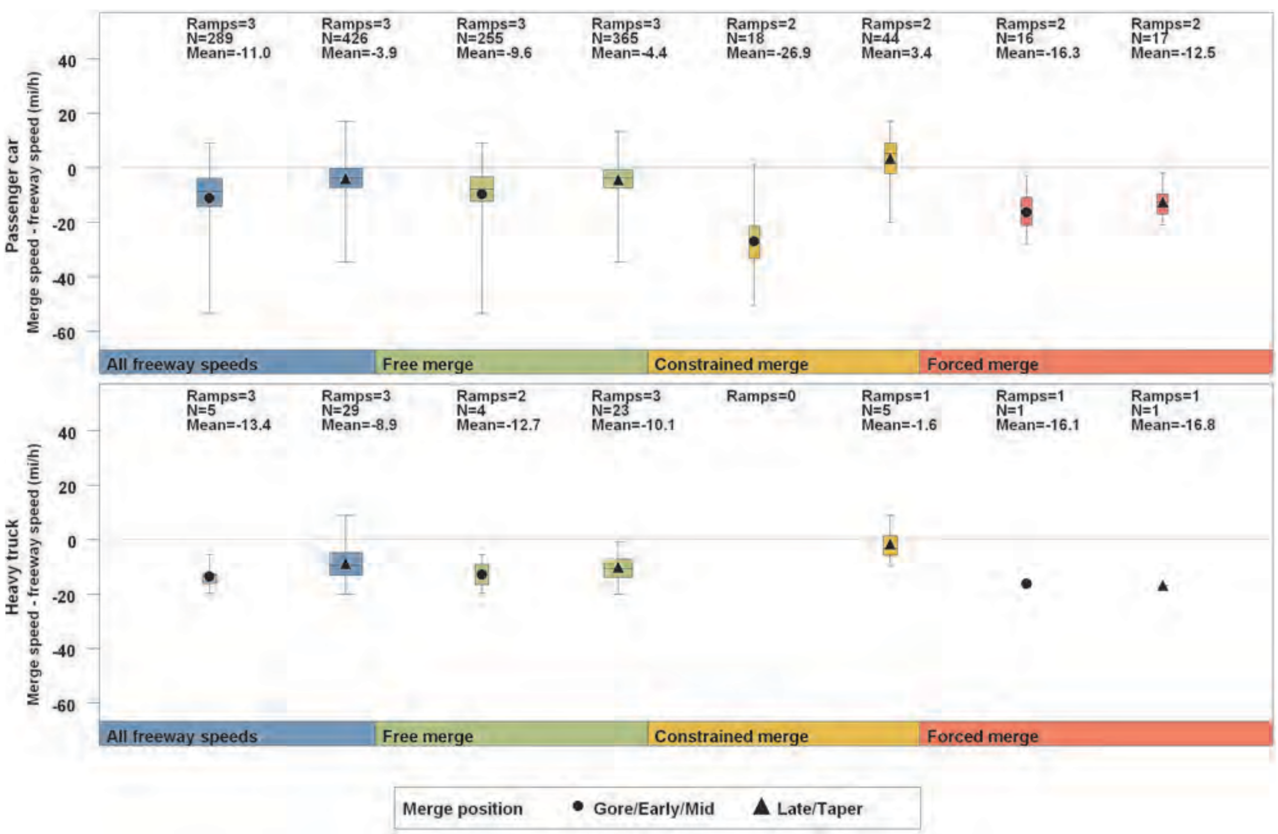


Figure 26. Merge speed differential by ramp type, merge type, vehicle type, and merge location (ramp type: straight; merge type: taper).

observations were only made during free-merge conditions. Trucks have a greater speed differential than passenger cars, and early mergers have a greater speed differential than later mergers.

Figure 24 presents observations made at loop ramps with tapered SCLs, and only under free-merge conditions. Like loop ramps with parallel SCLs, trucks have a greater speed differential than passenger cars, and early mergers have a greater speed differential than later mergers, but for each group of observations, the mean is closer to zero at tapered SCLs than at parallel SCLs.

Figure 25 shows the speed differentials for straight ramps with parallel SCLs. Observations were made under all three merge conditions, although many fewer observations were made under constrained- and forced-merge conditions than under free-merge conditions. The speed differential was greater for vehicles that merged later than earlier (both cars and trucks) under free-flow conditions, but less for late mergers than early mergers under constrained- and forced-merge conditions except for trucks under forced-merge conditions.

Figure 26 shows the speed differentials for straight ramps with tapered SCLs. Observations were made under all three merge conditions, although many fewer observations were made under constrained- and forced-merge conditions than under free-merge conditions. Under all three merge conditions, late mergers merged at speeds closer to freeway traffic than early mergers. This was most evident under constrained-merge conditions.

Figure 27 shows the speed differentials for each of the four ramp type and merge type combinations for only the free-merge conditions. For both straight and loop ramps, those with a tapered SCL experience merge speeds closer to the freeway speed than those with parallel SCLs. This observation holds for both passenger cars and heavy vehicles.

The most significant findings from the examination of merge speeds compared to freeway speeds are as follows:

- Vehicles merge closer to freeway speeds along tapered SCLs compared to parallel SCLs.
- Vehicles merge at speeds closer to freeway speeds under constrained- and forced-merge conditions than under free-merge conditions. This is probably because merging gaps are smaller and less frequent under constrained and forced conditions and require the merging vehicle to be traveling near the freeway speed to be able to accept the available gaps.

Figure 27. Merge speed differential by ramp type, merge type, vehicle type, and merge location (for free-merge conditions only).

- Passenger cars merge at speeds closer to freeway speeds than do trucks. This is expected as trucks do not accelerate as quickly as passenger cars and would be unlikely to reach the same speeds as cars by the merge location.
- When comparing ramp lengths that meet or exceed *Green Book* criteria or are less than *Green Book* criteria, the largest speed differentials occurred at ramps with parallel SCLs. In other words, at parallel SCLs the speed differentials were substantially lower (for both cars and trucks) at the ramps that met *Green Book* criteria than those that do not meet the criteria; while at taper SCLs the speed differentials (for both cars and trucks) were very similar at ramps that met criteria compared to those that did not.
- In general, late mergers merge at speeds closer to freeway speeds than early mergers.
- The speed differential between merge and freeway speeds is about the same when comparing straight and loop ramps.

5.4.1.3 Acceleration Rate

The third measure of performance considered is the acceleration rate of vehicles as they travel along the ramp and SCL to the point where they merge. While the AASHTO *Green Book* does not provide a table of the assumed acceleration rates used to determine guidance for minimum acceleration lane length, these accelerations can be back calculated from Exhibit 10-70, using initial speed, speed reached (i.e., merge speed), and acceleration lane length (see Table 1). The acceleration rates measured in the field can be compared to the *Green Book* accelerations to determine whether vehicles are performing as assumed in the *Green Book*. However, this comparison is not straightforward because vehicle acceleration in the field is a function of several variables including vehicle performance capabilities, driver preference, acceleration of other vehicles, operating conditions on the freeway, and, perhaps most importantly, ramp length. Ramps longer than the minimum length needed to reach the desired merge speed at a comfortable acceleration allow the driver more flexibility in determining when and where to accelerate along the ramp. When ramps are long, drivers may accelerate to very near their merge speed early along the ramp and then reduce their acceleration as they enter the SCL and look for a gap. Accelerations measured near the gore and along the SCL may not be representative of vehicle capabilities or driver comfort levels and preferences, because vehicles may not necessarily accelerate along the ramp at a constant rate. Because it was not possible to capture speed and acceleration along the entire ramp length, the behavior of vehicles near the crossroad terminal is not well documented here. Field-measured accelerations that fall below the assumptions in the *Green Book* may indicate that drivers were completing most of the required acceleration prior to the location where initial speed measurements were taken, rather than that the accelerations assumed in the *Green Book* are too high. For this reason, comparisons of field-measured acceleration to *Green Book* assumptions need to be interpreted carefully and considered in context of the ramp characteristics and merge conditions.

The analysis of acceleration rates includes a general analysis of the acceleration rates measured in the field and comparisons of acceleration rates measured in the field to assumed acceleration rates from the *Green Book* criteria for level grades (i.e., grades of 2 percent or less) and for grades of 3 percent and greater. In addition, the measured speed profiles of trucks on grades of 4 and 6 percent are compared to speed profiles generated using the TSPM, which is based on vehicle performance equations. The TSPM estimates the speed profile for an unimpeded truck on any specified vertical alignment given the truck's weight-to-power ratio. The analysis of acceleration rates is based on the measured speed profiles of 3,225 vehicles.

General Analysis of Acceleration Rates. Figures 28 through 33 provide the acceleration profiles of observed vehicles at each entrance ramp evaluated in this research. Each figure includes two ramps—one on the left and one on the right. The topmost sections show the acceleration profiles for all observed vehicles at the ramp, while the three lower sections show the profiles under the three merge conditions (i.e., free, constrained, and forced). Empty sections indicate that no observations were made for that merge condition at that particular ramp. The acceleration profiles are shown with boxplots, and can be interpreted as follows:

- The x-axis shows distance along the ramp relative to the painted nose, which is marked as zero. For each ramp, the x-axis begins at the location of the controlling feature (crossroad terminal or exit of controlling curve) in the case of straight ramps, or at the location where speeds were first measured in the case of loop ramps.
- At each measurement location, a "box" with "whiskers" provides a summary of the acceleration measured for all of the study vehicles. The colored box indicates where the middle 50 percent of measured acceleration values fall. The whiskers indicate the minimum and maximum acceleration values observed at that location.
- The line running through the boxes indicates the mean acceleration values along the ramp.
- The left-side y-axis provides acceleration values. At zero, the vehicle is maintaining a constant speed. Negative values indicate the vehicle is slowing, while positive values indicate the vehicle is speeding up.
- The s-shaped curve overlaid on the boxplot shows the cumulative percentage of observed vehicles that have merged at each location. The percentage is shown on the

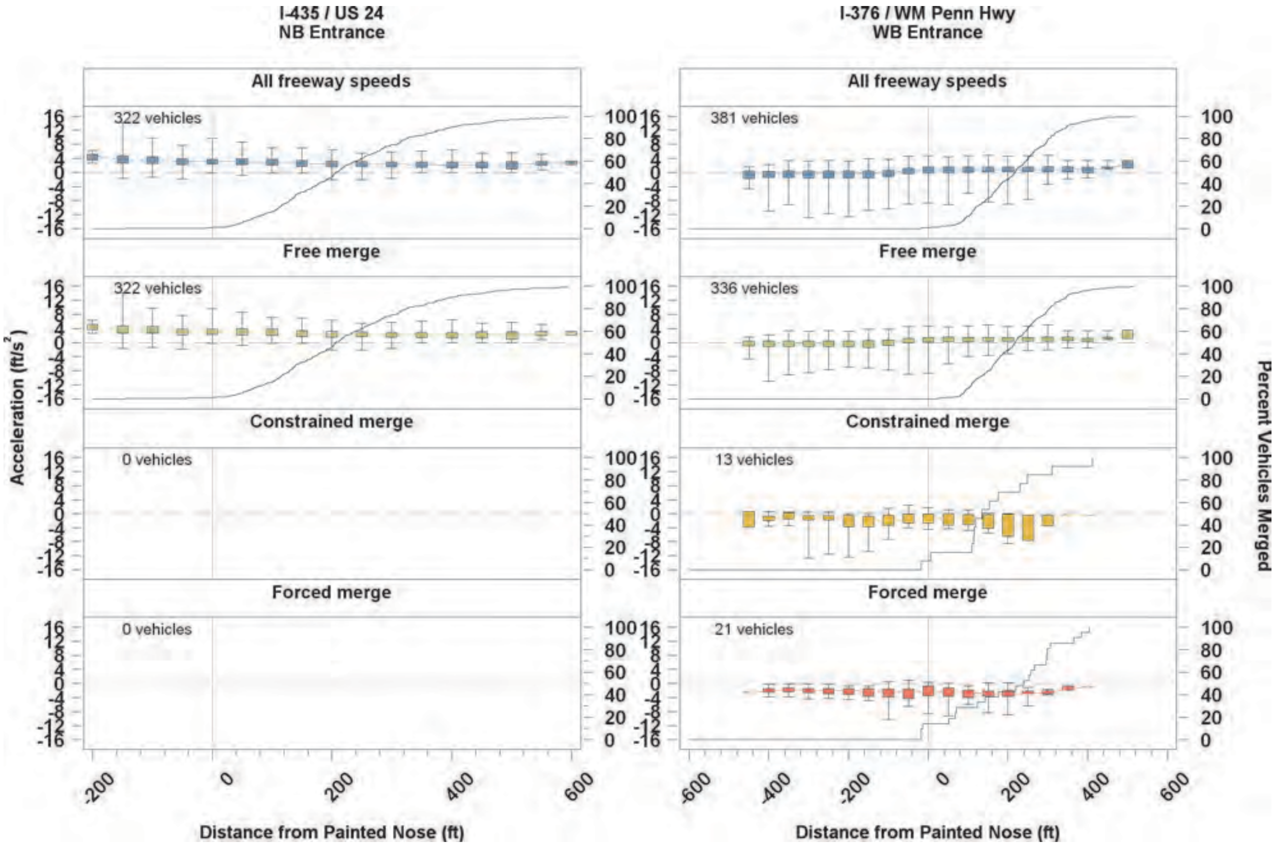


Figure 28. Acceleration profiles and cumulative merge curve by merge condition (NB entrance ramp at I-435/US 24 and WB entrance ramp at I-376/Wm. Penn Hwy).

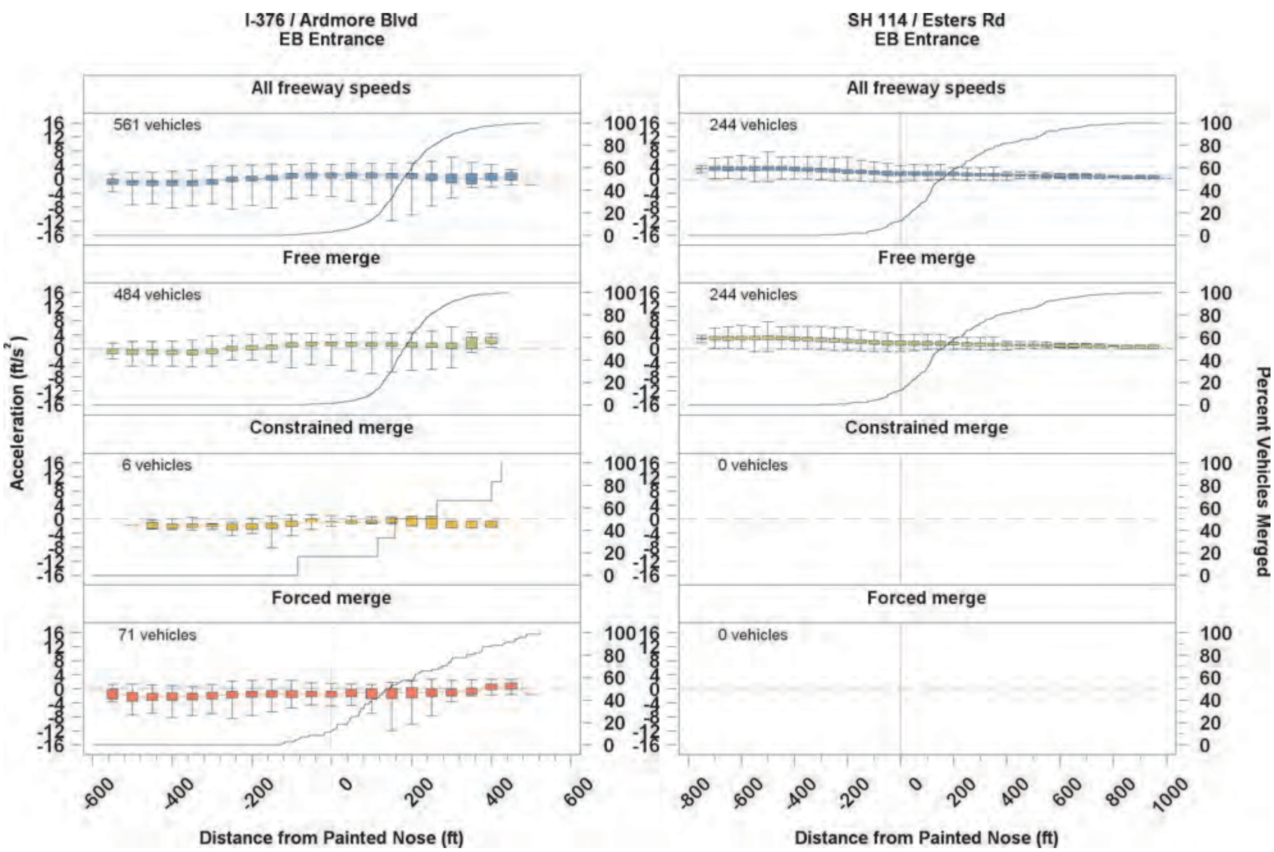


Figure 29. Acceleration profiles and cumulative merge curve by merge condition (EB entrance ramp at I-376/Ardmore Blvd and EB entrance ramp at SH 114/Esters Rd).

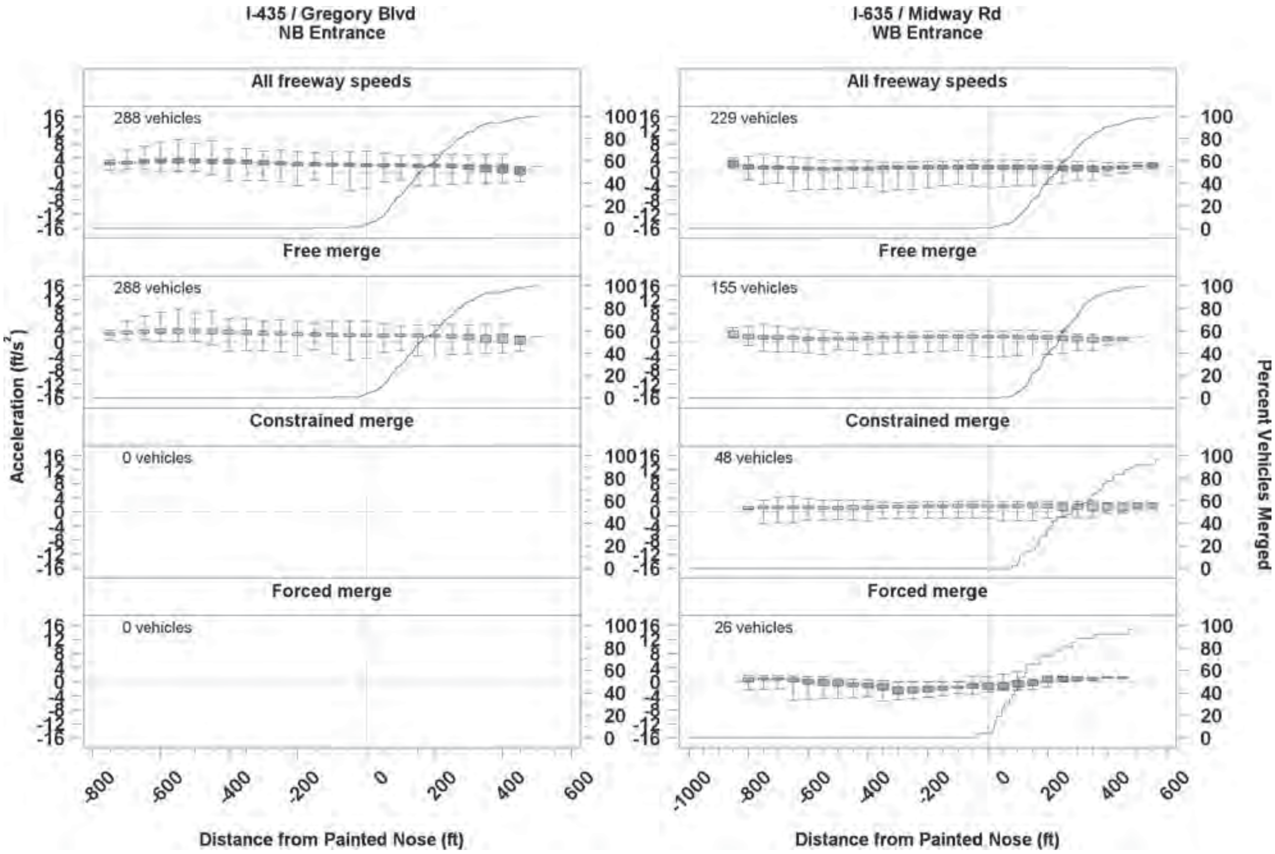


Figure 30. Acceleration profiles and cumulative merge curve by merge condition (NB entrance ramp at I-435/Gregory Blvd and WB entrance ramp at I-635/Midway Rd).

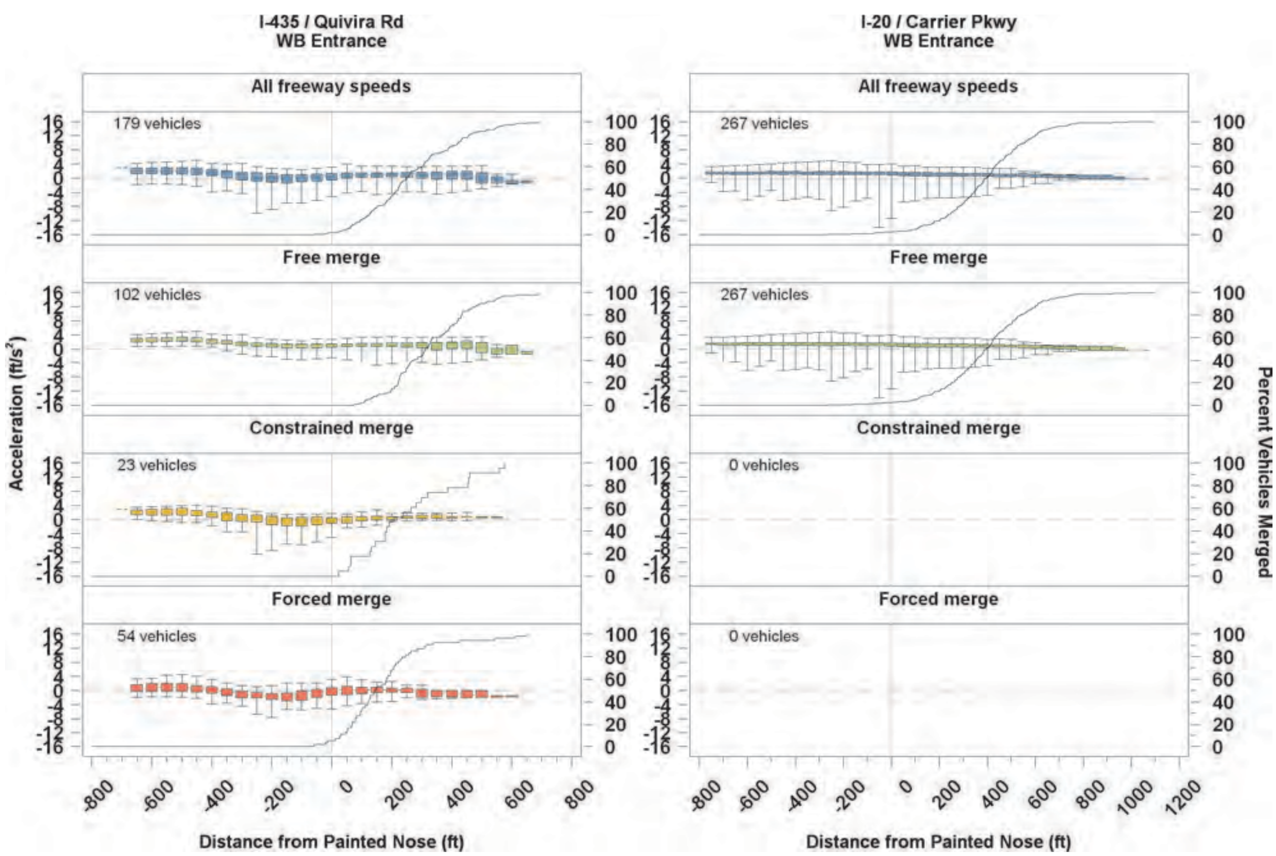


Figure 31. Acceleration profiles and cumulative merge curve by merge condition (WB entrance ramp at I-435/Quivira Rd and WB entrance ramp at I-20/Carrier Pkwy).

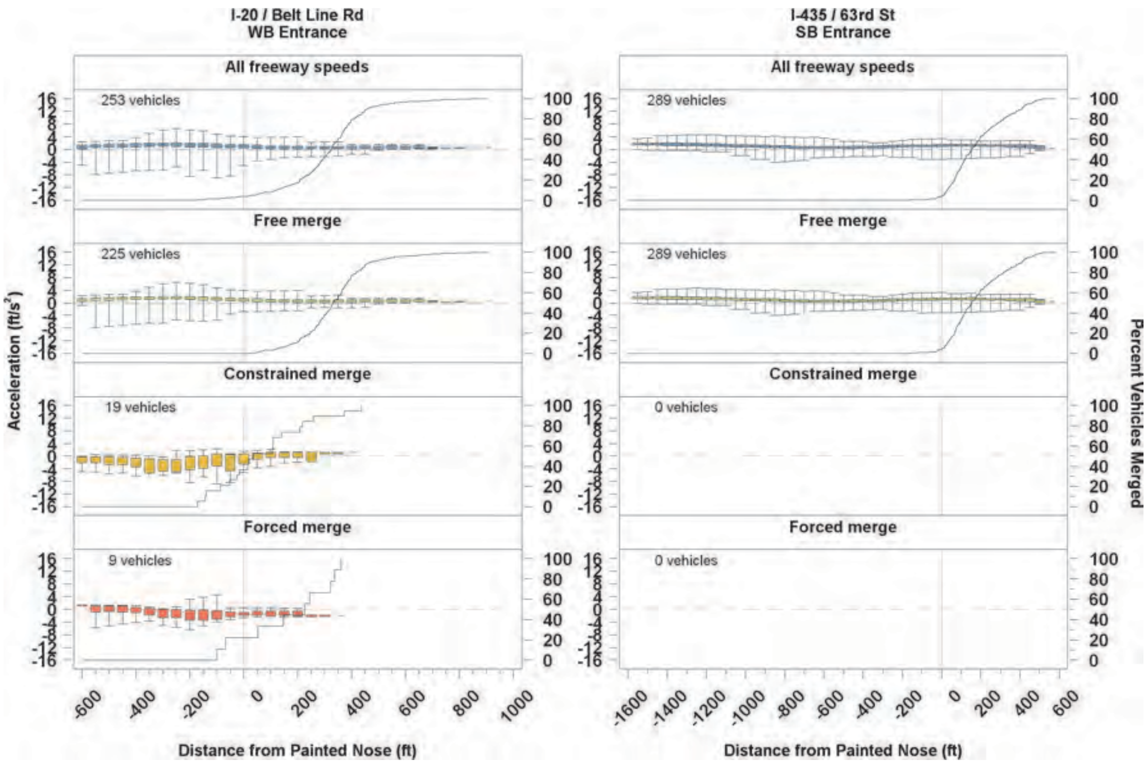


Figure 32. Acceleration profiles and cumulative merge curve by merge condition (WB entrance ramp at I-20/Belt Line Rd and SB entrance ramp at I-435/63rd St).

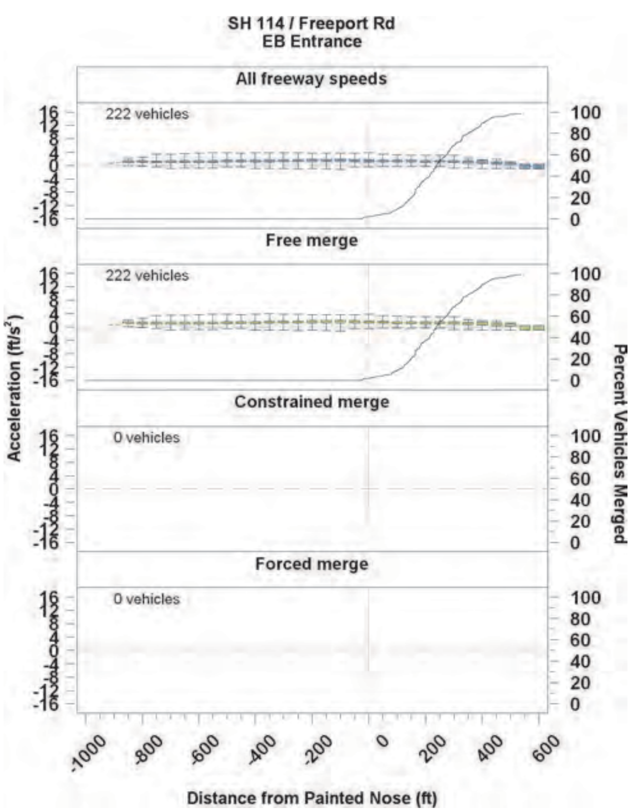


Figure 33. Acceleration profiles and cumulative merge curve by merge condition (EB entrance ramp at SH 114/Freeport Rd).

right-side y-axis. As more vehicles merge, fewer observations are included in the boxes. For example, at the horizontal distance (x-axis) where the s-shaped curve crosses the horizontal dashed line, 50 percent of vehicles have merged and the box and whiskers shown at this location represents only the remaining 50 percent of the original vehicles.

Several general observations from the acceleration profiles (i.e., Figures 28 through 33) are as follows:

- The acceleration rates change along the ramp and SCL.
- The range of acceleration rates decreases proceeding along the SCL.
- Negative accelerations along the ramp and SCL were observed and are most evident under constrained and forced conditions.

Tables 22 through 25 compare ramp length, initial measured speed, and acceleration rates to those recommended or assumed in the *Green Book*. The tables are organized as described below.

Rows. Each row provides information about one entrance ramp. The ramps are organized first by controlling feature, with those controlled by horizontal alignment in the upper section and those controlled by the crossroad terminal in the

Table 22. Observed acceleration rates for free-flow ramp vehicles (free merge and passenger cars only).

Ramp location	Diff. between actual accel. lane length and GB criteria[1] (ft)	No. of vehicles	Initial measurements: Location (ft): From controlling feature	Initial measurements: Location (ft): From painted nose	Initial measurements: Average speed (mi/h)	Initial measurements: Assumed GB speed at controlling feature (mi/h)	Merge measurements: Average distance past painted nose (ft)	Merge measurements: Average speed (mi/h)	Merge measurements: Desired GB speed reached (mi/h)	Acceleration: Assumed GB rate (ft/s²)	Acceleration: Percentiles from initial speed measured (ft/s²): 15th	Acceleration: Percentiles from initial speed measured (ft/s²): 50th
Controlling feature: Horizontal alignment												
I-435/US 24	–780	195	0	0	37.0	22	241	44.6	53	1.77	1.76	2.66
I-376/Wm. Penn Hwy	–693	162	0	0	45.1	44	231	47.0	50	1.65	0.10	0.75
I-376/Ardmore Blvd	–632	200	0	–275	43.4	36	198	47.7	50	1.69	0.22	0.93
SH 114/Esters Rd	–570	196	0	–395	44.9	22	196	55.1	53	1.77	1.20	2.01
Controlling feature: Crossroad terminal												
I-435/Gregory Blvd	–1,771	228	205	–520	31.9	0	176	50.1	53	1.87	1.70	2.39
I-635/Midway Rd	–565	109	200	–500	38.5	0	227	47.6	50	1.92	0.52	1.37
I-435/Quivira Rd	–415	49	370	–520	40.0	0	347	52.1	55	1.83	0.93	1.34
I-20/Carrier Pkwy	–75	215	325	–580	46.5	0	399	55.4	53	1.87	0.54	1.30
I-20/Belt Line Rd	240	172	1,050	–340	51.1	0	337	56.5	53	1.87	0.55	1.00
I-435/63rd St	750	130	795	–1,000	47.3	0	185	56.5	53	1.87	0.49	0.93
SH 114/Freeport Rd	905	179	1,310	–540	49.5	0	257	58.8	53	1.87	1.01	1.33

Note: GB = *Green Book*.

[1] Negative values indicate the actual acceleration lane length is less than minimum recommended by the *Green Book*. Positive values indicate the actual acceleration lane length is greater than minimum recommended by the *Green Book*.

Table 23. Observed acceleration rates for platooned ramp vehicles (free merge and passenger cars only).

Ramp location	Diff. between actual accel. lane length and GB criteria[1] (ft)	No. of vehicles	Initial measurements: Location (ft): From controlling feature	Initial measurements: Location (ft): From painted nose	Initial measurements: Average speed (mi/h)	Initial measurements: Assumed GB speed at controlling feature (mi/h)	Merge measurements: Average distance past painted nose (ft)	Merge measurements: Average speed (mi/h)	Merge measurements: Desired GB speed reached (mi/h)	Acceleration: Assumed GB rate (ft/s^2)	Acceleration: Percentiles from initial speed measured (ft/s^2): 15th	Acceleration: Percentiles from initial speed measured (ft/s^2): 50th
Controlling feature: Horizontal alignment												
I-435/US 24	–780	92	0	0	35.9	22	208	42.2	53	1.77	0.96	2.49
I-376/Wm. Penn Hwy	–693	139	0	0	38.3	44	213	41.0	50	1.65	0.16	1.10
I-376/Ardmore Blvd	–632	210	0	–275	40.3	36	187	44.7	50	1.69	0.08	1.03
SH 114/Esters Rd	–570	41	0	–395	45.2	22	240	55.1	53	1.77	1.06	1.69
Controlling feature: Crossroad terminal												
I-435/Gregory Blvd	–1,771	47	205	–520	31.6	0	178	49.4	53	1.87	1.62	2.33
I-635/Midway Rd	–565	27	200	–500	38.9	0	285	50.9	50	1.92	0.97	1.67
I-435/Quivira Rd	–415	37	370	–520	38.7	0	318	48.7	55	1.83	0.39	1.20
I-20/Carrier Pkwy	–75	45	325	–580	44.7	0	344	50.0	53	1.87	–1.01	1.18
I-20/Belt Line Rd	240	51	1,050	–340	48.8	0	343	53.6	53	1.87	0.18	1.06
I-435/63rd St	750	136	795	–1,000	44.1	0	185	53.1	53	1.87	0.40	0.85
SH 114/Freeport Rd	905	39	1,310	–540	50.0	0	204	59.0	53	1.87	0.96	1.34

[1] Negative values indicate the actual acceleration lane length is less than minimum recommended by the *Green Book*. Positive values indicate the actual acceleration lane length is greater than minimum recommended by the *Green Book*.

Table 24. Observed acceleration rates for free-flow and platooned ramp vehicles combined (free merge and passenger cars only).

Ramp location	Diff. between actual accel. lane length and GB criteria[1] (ft)	No. of vehicles	Initial measurements: Location (ft): From controlling feature	Initial measurements: Location (ft): From painted nose	Initial measurements: Average speed (mi/h)	Initial measurements: Assumed GB speed at controlling feature (mi/h)	Merge measurements: Average distance past painted nose (ft)	Merge measurements: Average speed (mi/h)	Merge measurements: Desired GB speed reached (mi/h)	Acceleration: Assumed GB rate (ft/s^2)	Acceleration: Percentiles from initial speed measured (ft/s^2): 15th	Acceleration: Percentiles from initial speed measured (ft/s^2): 50th
Controlling feature: Horizontal alignment												
I-435/US 24	–780	287	0	0	36.7	22	231	43.8	53	1.77	1.60	2.58
I-376/Wm. Penn Hwy	–693	301	0	0	42.0	44	223	44.2	50	1.65	0.13	0.90
I-376/Ardmore Blvd	–632	410	0	–275	41.8	36	192	46.2	50	1.69	0.15	0.98
SH 114/Esters Rd	–570	237	0	–395	45.0	22	204	55.1	53	1.77	1.12	1.95
Controlling feature: Crossroad terminal												
I-435/Gregory Blvd	–1,771	275	205	–520	31.8	0	177	50.0	53	1.87	1.67	2.39
I-635/Midway Rd	–565	136	200	–500	38.6	0	239	48.3	50	1.92	0.72	1.39
I-435/Quivira Rd	–415	86	370	–520	39.5	0	335	50.6	55	1.83	0.63	1.34
I-20/Carrier Pkwy	–75	260	325	–580	46.2	0	389	54.4	53	1.87	0.46	1.27
I-20/Belt Line Rd	240	223	1,050	–340	50.6	0	338	55.8	53	1.87	0.43	1.01
I-435/63rd St	750	266	795	–1,000	45.7	0	185	54.8	53	1.87	0.43	0.88
SH 114/Freeport Rd	905	218	1,310	–540	49.6	0	247	58.8	53	1.87	0.98	1.34

[1] Negative values indicate the actual acceleration lane length is less than minimum recommended by the *Green Book*. Positive values indicate the actual acceleration lane length is greater than minimum recommended by the *Green Book*.

Table 25. Observed acceleration rates for free-flow ramp vehicles (free merge and trucks only).

Ramp location	Diff. between actual accel. lane length and GB criteria[1] (ft)	No. of vehicles	Initial measurements				Merge measurements			Acceleration		
			Location (ft)		Average speed (mi/h)	Assumed GB speed at controlling feature (mi/h)	Average distance past painted nose (ft)	Average speed (mi/h)	Desired GB speed reached (mi/h)	Assumed GB rate (ft/s^2)	Percentiles from initial speed measured (ft/s^2)	
			From controlling feature	From painted nose							15th	50th
Controlling feature: Horizontal alignment												
I-435/US 24	–780	28	0	0	32.2	22	205	36.7	53	1.77	0.98	1.55
I-376/Wm. Penn Hwy	–693	13	0	0	37.2	44	209	36.7	50	1.65	–1.07	0.30
I-376/Ardmore Blvd	–632	50	0	–275	39.8	36	160	42.7	50	1.69	0.13	0.58
SH 114/Esters Rd	–570	8	0	–395	39.8	22	410	50.2	53	1.77	0.85	1.32
Controlling feature: Crossroad terminal												
I-435/Gregory Blvd	–1,771	13	205	–520	26.2	0	146	39.0	53	1.87	0.73	1.42
I-635/Midway Rd	–565	13	200	–500	29.8	0	191	43.5	50	1.92	1.09	1.58
I-435/Quivira Rd	–415	10	370	–520	34.7	0	291	42.7	55	1.83	0.56	1.13
I-20/Carrier Pkwy	–75	5	325	–580	42.6	0	565	53.1	53	1.87	0.67	1.04
I-20/Belt Line Rd	240	2	1,050	–340	47.1	0	308	51.3	53	1.87	0.58	0.68
I-435/63rd St	750	20	795	–1,000	34.7	0	173	46.1	53	1.87	0.42	0.95
SH 114/Freeport Rd	905	4	1,310	–540	46.1	0	262	56.8	53	1.87	1.15	1.21

[1] Negative values indicate the actual acceleration lane length is less than minimum recommended by the *Green Book*. Positive values indicate the actual acceleration lane length is greater than minimum recommended by the *Green Book*.

lower section. Within the upper section, the first two ramps are loop ramps (where the controlling feature—horizontal curvature—ends at the painted nose), and the second two ramps are straight ramps with a horizontal curve that controls speed. Within each section, the ramps are organized in order of difference between actual length of the ramp and the *Green Book* recommended ramp length.

Columns

Difference between actual acceleration lane length and Green Book *criteria*—Shows the difference between the total length provided from the controlling feature to the painted nose plus the length from the painted nose to the start of the taper (SCL) and the ramp length provided in Exhibit 10-70 of the *Green Book* and adjusted based upon grade using Exhibit 10-71. The *Green Book* criteria length is found using the design speed of the freeway and the radius of the controlling feature. For ramps without controlling horizontal curvature, the stop condition was assumed for the initial speed. Negative values in this column indicate that the actual ramp length is less than the *Green Book* criteria, and positive numbers indicate that actual ramp length exceeds criteria.

Number of vehicles—Total number of vehicle observations included in the acceleration analysis described in the table caption.

Location from controlling feature (initial measurements)—Distance in feet of the initial speed measurement from the controlling feature. For ramps controlled by horizontal alignment, this value is zero because the initial speed measurement was taken as the vehicles exited the feature. For ramps not controlled by horizontal alignment, this is the distance from the crossroad terminal to the location where speeds were first captured.

Location from painted nose (initial measurements)—Distance in feet of the initial speed measurement from the painted nose. For loop ramps, this value is zero because the horizontal curve ends at the painted nose, and this is where initial measurements were taken. For straight ramps, this is the distance upstream of the painted nose where speeds were first measured.

Average speed (initial measurements)—The mean of all observed speeds at the location of initial speed measurement.

Assumed Green Book *speed at the controlling feature (initial measurements)*—The assumed initial speed as shown in Exhibit 10-70 in the *Green Book,* which is based on the design speed of the controlling feature. For example, a ramp controlled by a horizontal curve with a design speed of 30 mi/h has an assumed initial speed of 26 mi/h. For straight ramps not controlled by horizontal alignment, the initial speed is assumed to be zero at the crossroad terminal.

Average distance past painted nose (merge measurements)—Average distance in feet from the painted nose to the merge location of all observed vehicles.

Average speed (merge measurements)—Average merge speed in mi/h of all observed vehicles.

Desired Green Book *speed reached (merge measurements)*—The desired speed in mi/h reached at the time of merge as indicated in Exhibit 10-70 of the *Green Book* based on freeway design speed. For example, a highway with a design speed of 65 mi/h has a speed reached value of 50 mi/h.

Assumed Green Book *rate (acceleration)*—Acceleration in ft/s^2 calculated from the assumed *Green Book* speed at controlling feature (initial speed), desired *Green Book* speed reached (speed reached), and minimum acceleration lane length from Exhibit 10-70 in the *Green Book*.

15th percentile from initial speed measured (acceleration)—Taken from the distribution of the acceleration of each vehicle calculated from the initial measured speed, the measured merge speed, and the distance from where the initial speed was measured to the location of the merge. Eighty-five percent of vehicles exceed this acceleration.

50th percentile from initial speed measured (acceleration)—Taken from the distribution of the acceleration of each vehicle calculated from the initial measured speed, the measured merge speed, and the distance from where the initial speed was measured to the location of the merge. Fifty percent of vehicles exceed this acceleration.

Table 22 presents the information described above for all observed free-flow passenger cars during free-merge conditions. Table 23 presents information for platooned passenger cars during free-merge conditions. Table 24 presents information for all passenger cars (free-flow and platooned combined) during free-merge conditions. Table 25 presents information for free-flow trucks during free-merge conditions. These tables show several important observations:

1. At every ramp except at I-376/Wm. Penn Hwy, the average measured initial speed is greater than the initial speed assumed in the *Green Book.* At ramps controlled by horizontal alignment, this means that vehicles are exiting the controlling curves at higher speeds than what the *Green Book* assumes based on the design speeds of the curves. Because vehicles are exiting the curves at higher speeds, they require less distance from that point to accelerate to merging speeds than what the *Green Book* criteria suggests. For ramps not controlled by horizontal alignment (i.e., straight ramps with no controlling curve), initial speeds are assumed to be zero at the crossroad terminal, but measured initial speeds are not available until several hundred feet along the ramp in most cases, so true speeds at the crossroad terminal are unknown. At these ramps, the measured initial speeds range from 32 to 52 mi/h for passenger cars, indicating that a great deal of acceleration has occurred prior to the initial measured speed. Comparing Table 22

to Table 23, platooned passenger cars are traveling slightly slower than free-flow passenger cars at the location of initial speed measurement, most likely because their speed is limited by the vehicles they are following. Table 25 shows that the initial speed measured for free-flow trucks is around 5 mi/h less than that measured for free-flow passenger cars in Table 22, which indicates that trucks travel at slower speeds around the controlling horizontal curve and accelerate at a lower rate from the crossroad terminal.

2. For free-flow passenger cars under free-merge conditions, the measured average speed at the merge location tends to be a few mi/h less than the desired speed reached assumed in the *Green Book* when the ramp length is less than the recommended, and a few mi/h greater than the *Green Book* desired speed reached when the ramp length is longer than recommended.
3. Acceleration rates calculated from the measured initial speed, measured merge speed, and distance between the two measurements are, in general, lower than the assumed acceleration rates in the *Green Book.* At the 15th percentile, all calculated accelerations are lower than assumed accelerations. At the 50th percentile, a few of the calculated accelerations exceed those assumed in the *Green Book,* and they occur at ramps that are several hundred feet shorter than the recommended minimum *Green Book* criteria.
4. When comparing acceleration rates of free-flow passenger cars and platooned passenger cars, there is no consistent pattern across ramps. At some ramps, the free-flow passenger cars have higher acceleration rates, while at other ramps platooned vehicles have higher acceleration rates.
5. The acceleration rates of free-flow trucks are lower than for free-flow passenger cars, with the exception of two ramps (i.e., I-635/Midway Rd and I-435/63rd St).

Comparison of Acceleration Rates on Level Grade. Table 26 provides a comparison of the acceleration rates derived from the *Green Book* recommended ramp lengths to acceleration rates measured in the field for various vehicle groups under various conditions. Only acceleration rates of vehicles on ramps with flat grades of two percent or less were used in the development of Table 26. The table format is based on *Green Book* Exhibit 10-70, and the values for design speed, speed reached, curve design speed, and initial speed are all taken directly from that figure. Below the presentation of the *Green Book*–assumed acceleration rates are four categories of field-measured acceleration rates:

Condition 1—This category shows the 50th percentile acceleration rate for passenger cars that were not in a platoon on the ramp and entered the freeway when freeway speeds were greater than 50 mi/h. Platooned vehicles are excluded because it is assumed their speed and acceleration choices are constrained by vehicle(s) in front of them.

Condition 2—This category includes the same population of speed profiles as the previous category, but the 15th percentile speed is considered rather than the median speed, to show that 85 percent of vehicles are accelerating at a rate greater than what is shown. Platooned vehicles are excluded because it is assumed their speed and acceleration choices are constrained by vehicle(s) in front of them.

Condition 3—This category looks at heavy vehicles rather than passenger cars, but again, only those that are not platooned on the ramp and that are entering freeway traffic that is moving faster than 50 mi/h. Platooned vehicles are excluded because it is assumed their speed and acceleration choices are constrained by vehicle(s) in front of them.

Condition 4—This category considers all passenger cars (both those in platoons and not in platoons on the ramp) that are entering the freeway when the freeway speed is between 40 mi/h and 50 mi/h. Platooned vehicles are included because constrained-merge conditions typically occur during peak traffic flow periods where a majority of vehicles on the ramp are in platoons. Available observation data for free-flow vehicles in constrained-merge conditions is limited.

Constrained-merge conditions were considered more critical for evaluation than forced-merge conditions based on the findings for the other measures of effectiveness (i.e., merge location and speed differential). In both of those analyses, it was shown that the measures showed a bigger change between free and constrained-merge conditions than between free and forced-merge conditions.

Table 26 compiles field-measured acceleration data for comparison with *Green Book* assumed accelerations. For each condition, the cells in the table were populated by developing a database of the speed profile of each vehicle as measured by the laser gun in the field. This provided vehicle speed at known distances along the ramp for all the recorded vehicles. For each speed profile, an acceleration rate was calculated for the sections of the profile that correspond to a given cell in the table (identified by an initial speed and a speed reached). Since the database provides the location along the ramp where each speed is first reached, the length between initial speed and speed reached is known and can be used to calculate accelerations between the two points. For example, if a given speed profile had an initial speed of 28 mi/h and a merge speed of 42 mi/h, acceleration rates could be calculated for four cells in the table (initial speed of 30 mi/h to speeds reached of 31, 35, and 39 mi/h, and initial speed of 36 mi/h to speed reached of 39 mi/h). This process was repeated for each speed profile, so that each cell in the table included several observations. Only sections of speed profiles that did

Table 26. Comparison of *Green Book* and field acceleration rates on level grades.

Design speed (mi/h)	Speed reached (mi/h)	Acceleration length, L (ft) for entrance curve design speed (mi/h)								
		Stop	15	20	25	30	35	40	45	50
		Initial speed (mi/h)								
		0	14	18	22	26	30	36	40	44
2004 *Green Book* acceleration rates (ft/s^2) (derived from recommended minimum ramp lengths)										
30	23	3.18	2.57	-	–	–	–	–	–	–
35	27	2.81	2.62	2.73	–	–	–	–	–	–
40	31	2.88	2.76	2.55	2.45	2.57	–	–	–	–
45	35	2.36	2.27	2.21	2.11	2.12	2.19	–	–	–
50	39	2.28	2.17	2.12	2.04	2.03	1.92	1.87	–	–
55	43	2.08	1.98	2.03	1.89	1.89	1.86	1.87	1.79	–
60	47	1.99	1.91	1.85	1.83	1.82	1.77	1.79	1.57	1.64
65	50	1.92	1.84	1.79	1.79	1.76	1.73	1.69	1.62	1.65
70	53	1.87	1.81	1.77	1.77	1.71	1.68	1.63	1.59	1.63
75	55	1.83	1.77	1.79	1.74	1.68	1.62	1.61	1.48	1.51
Condition 1: Measured median acceleration rates (ft/s^2) for passenger cars (free-flow and free-merge)										
30	23									
35	27					3.39				
40	31					3.28	2.88			
45	35					3.22	2.97			
50	39					3.24	2.90	2.69		
55	43					3.40	2.72	2.68	2.47	
60	47						2.68	2.66	2.36	1.97
65	50						3.18	2.57	2.36	1.96
70	53							2.62	2.49	2.05
75	55							2.76	2.60	2.12
Condition 2: Measured 15th percentile acceleration rates (ft/s^2) for passenger cars (free-flow and free-merge)										
30	23									
35	27					1.33				
40	31					1.60	1.54			
45	35					1.79	1.51			
50	39					1.92	1.65	1.53		
55	43					1.97	1.55	1.43	1.25	
60	47						1.58	1.45	1.22	1.19
65	50						1.55	1.47	1.28	1.19
70	53							1.48	1.32	1.21
75	55							1.50	1.43	1.26
Condition 3: Measured median acceleration rates (ft/s^2) for trucks (free-flow and free-merge)										
30	23				3.90					
35	27				4.14	2.27				
40	31					2.00	1.66			
45	35					1.97	1.75			
50	39						1.69	1.79		
55	43						1.60	1.62	1.41	
60	47							1.58	1.33	1.48
65	50									
70	53									
75	55									
Condition 4: Measured median acceleration rates (ft/s^2) for passenger cars (free-flow/platoon and constrained merge)										
30	23									
35	27									
40	31									
45	35									
50	39							1.76		
55	43							1.60	1.62	
60	47								1.70	1.76
65	50								1.84	1.87
70	53									
75	55									

not include deceleration were included in the analysis. For example, if a vehicle's initial speed was 43 mi/h, but then fell to 41 mi/h, this section of the speed profile was disregarded. A distribution of all of the individual vehicle accelerations for a given cell in the table was plotted in histogram form. These histograms are presented in Appendix B (available on TRB website at http://www.trb.com/Main/Blurbs/167516.aspx) and show the number of observations for each cell as well as summary statistics. The histograms indicate that the acceleration values are skewed to the left, with long tails toward the higher accelerations. For this reason, median, rather than mean, accelerations provide a better estimate for evaluation. Only values based upon 20 or more observations are presented in Table 26.

Table 26 shows that in ideal conditions (i.e., where the merging vehicle is not in a platoon and the freeway traffic is moving at a speed greater than 50 mi/h), passenger cars can and do accelerate at rates greater than those assumed in the *Green Book.* The Condition 1 acceleration values exceed the corresponding *Green Book* acceleration values where measurements are available. *Green Book* accelerations generally decrease moving to the right and downward through the table. Condition 1 accelerations also decrease moving to the right through the table, but this pattern is not as clear moving downward.

Condition 2 accelerations are considerably lower than those shown in Condition 1, and are only slightly below those assumed in the *Green Book.* This indicates that even the 15 percent of vehicles with the lowest rates of acceleration are accelerating near the assumed values. It should also be noted that drivers may have many reasons for not accelerating at a higher rate, and that the vehicles with the least acceleration may reflect driver preference rather than capabilities. For example, a driver may choose to accelerate at a lower rate when more distance is available to increase speed (on longer ramps) or when freeway traffic is light and merge speed is less critical.

Condition 3 considers acceleration rates for trucks. As expected, these rates are lower than those exhibited by passenger cars, although they are quite close to the 15th percentile values for passenger cars, and in general, are about 10 percent less than the *Green Book* assumed accelerations.

Condition 4 considers passenger cars in constrained-merge conditions. Both free-flow and platooned vehicles are considered because constrained conditions often occur during peak travel times when ramp traffic is heavy. For this reason, very few vehicles were not in a platoon and the number of observations for these cars was quite limited. Even in constrained-merge conditions, vehicles accelerate at rates very close to those assumed in the *Green Book.*

Comparison of Acceleration Rates on Grades of 3 Percent or Greater. Where grades are present on ramps, the minimum SCL lengths listed in *Green Book* Exhibit 10-70 are adjusted as a function of grade using factors listed in *Green Book* Exhibit 10-71 (see Table 2). The adjustment factors are provided as the ratio of length on grade to length on level. Adjustment factors are provided for grades of 3 to 4 percent and 5 to 6 percent and range in value from 1.3 to 3.0 for upgrades.

Table 27 provides a comparison of measured acceleration rates for free-flow passenger cars on ramps with level grade and ramps with grades of 3 percent or greater. Median acceleration rates are provided, and corresponding adjustment factors are calculated as the ratio of the median acceleration rate on level grade to that for the respective categories of grades consistent with the current format provided in *Green Book* Exhibit 10-71 for upgrades. Only values based upon 10 or more observations are presented for grades of 3 percent or more. None of the entrance ramps included in the study had downgrades greater than 2 percent, so insufficient data were available to further investigate acceleration rates on downgrades greater than 2 percent.

Table 28 provides similar comparisons of measured acceleration rates for free-flow trucks. Rather than calculating the adjustment factors based on the median acceleration rate on level grade for trucks, the adjustment factors are based on the median acceleration rates on level grade for passenger cars. Again, only values based upon 10 or more observations are presented for grades of 3 percent or more; as such, insufficient data were available for acceleration rates of trucks on upgrades greater than 4 percent and on downgrades greater than 2 percent.

Comparing grade adjustment factors from *Green Book* Exhibit 10-71 to those provided in Tables 27 and 28 yields the following general observations:

- Many of the adjustment factors for passenger cars on upgrades of 3 to 4 percent are less than one, indicating that passenger cars can and do accelerate at a greater rate on upgrades of 3 to 4 percent than on level grade. This indicates that upgrades of 3 to 4 percent do not limit the vehicle performance capabilities of passenger cars. In addition, the calculated adjustment factors that are greater than one for passenger cars on upgrades of 3 to 4 percent are much less than the corresponding values in *Green Book* Exhibit 10-71.
- The calculated adjustment factors for passenger cars on upgrades of 6 percent are all greater than one. Several of the values are comparable to corresponding values in *Green Book* Exhibit 10-71, while several others are considerably less than values in *Green Book* Exhibit 10-71.
- Most of the calculated adjustment factors for trucks on upgrades of 3 to 4 percent are comparable to corresponding values in *Green Book* Exhibit 10-71, or slightly greater, with exception of one value which is considerably less than the corresponding value in *Green Book* Exhibit 10-71. However, this needs to be carefully interpreted because the acceleration rates of trucks on level grade are less than for

Table 27. Measured acceleration rates for passenger cars on grades and corresponding adjustment factors.

Design speed (mi/h)	Speed reached (mi/h)	Acceleration length, L (ft) for entrance curve design speed (mi/h)								
		Stop	15	20	25	30	35	40	45	50
		Initial speed (mi/h)								
		0	14	18	22	26	30	36	40	44
Measured median acceleration rates (ft/s^2) for passenger cars (free-flow and free-merge) (0 to 2 % grade)										
30	23									
35	27					3.39				
40	31					3.28	2.88			
45	35					3.22	2.97			
50	39					3.24	2.90	2.69		
55	43					3.40	2.72	2.68	2.47	
60	47						2.68	2.66	2.36	1.97
65	50						3.18	2.57	2.36	1.96
70	53							2.62	2.49	2.05
75	55							2.76	2.60	2.12
Measured median acceleration rates (ft/s^2) for passenger cars (free-flow and free-merge) (3 to 4 % grade)										
30	23		2.46	2.83	2.94					
35	27		2.53	3.07	3.12	3.16				
40	31		2.48	3.07	3.18	3.16	3.17			
45	35		2.31	3.01	3.11	3.12	3.06			
50	39		2.41	2.97	3.08	3.04	3.00	2.78		
55	43			2.94	3.09	3.01	2.96	2.81	2.57	
60	47			3.00	3.10	3.02	2.99	2.89	2.83	2.48
65	50			3.11	3.18	3.15	3.11	2.91	2.83	2.67
70	53			3.10	3.27	3.28	3.21	3.07	2.91	2.80
75	55			3.29	3.37	3.38	3.36	3.37	3.21	3.00
Measured median acceleration rates (ft/s^2) for passenger cars (free-flow and free-merge) (6 % grade)										
30	23									
35	27					2.14				
40	31						2.60			
45	35						2.49			
50	39							1.57		
55	43								1.89	
60	47									1.69
65	50									1.76
70	53									
75	55									
SCL adjustment factor as a function of grade (3 to 4% grade)										
30	23									
35	27					1.07				
40	31					1.04	0.91			
45	35					1.03	0.97			
50	39					1.07	0.97	0.97		
55	43					1.13	0.92	0.95	0.96	
60	47						0.89	0.92	0.83	0.79
65	50						1.02	0.88	0.83	0.73
70	53							0.85	0.85	0.73
75	55							0.82	0.81	0.71
SCL adjustment factor as a function of grade (6% grade)										
30	23									
35	27					1.58				
40	31						1.11			
45	35						1.19			
50	39							1.71		
55	43								1.30	
60	47									1.16
65	50									1.11
70	53									
75	55									

Table 28. Measured acceleration rates for trucks on grades and corresponding adjustment factors.

Design speed (mi/h)	Speed reached (mi/h)	Acceleration length, L (ft) for entrance curve design speed (mi/h)								
		Stop	15	20	25	30	35	40	45	50
		Initial speed (mi/h)								
		0	14	18	22	26	30	36	40	44
Measured median acceleration rates (ft/s^2) for trucks (free-flow and free-merge) (0 to 2 % grade)										
30	23				3.90					
35	27				4.14	2.27				
40	31					2.00	1.66			
45	35					1.97	1.75			
50	39						1.69	1.79		
55	43						1.60	1.62	1.41	
60	47							1.58	1.33	
65	50									
70	53									
75	55									
Measured median acceleration rates (ft/s^2) for trucks (free-flow and free-merge) (3 to 4 % grade)										
30	23				2.22					
35	27				1.99	2.02				
40	31					2.17	1.98			
45	35						1.80			
50	39							1.45		
55	43								2.22	
60	47									
65	50									
70	53									
75	55									
SCL adjustment factor as a function of grade (3 to 4 % grade)										
30	23									
35	27					1.68				
40	31					1.51	1.45			
45	35						1.65			
50	39							1.86		
55	43								1.11	
60	47									
65	50									
70	53									
75	55									

passenger cars. Thus, even though the grade adjustment factors based on acceleration capabilities of trucks are comparable to values in the *Green Book,* this does not necessarily mean that by applying the grade adjustment factors for upgrades that the corresponding minimum acceleration lane lengths would accommodate the vehicle performance capabilities of trucks.

Comparison of Truck Acceleration Rates on Grades Using the TSPM. A truck speed profile model (TSPM) that estimates truck performance on grades was developed by Harwood et al. (2003) primarily as a design tool for highway agencies to determine the need for, and to design, truck climbing lanes. The TSPM is a spreadsheet-based tool that applies vehicle performance equations for trucks to estimate truck speed profiles on specified grades. Inputs for the TSPM include both roadway and truck characteristics as follows:

Roadway Characteristics

- Vertical profile—percent grade for specific ranges of position coordinates.
- Elevation above sea level (ft).

Truck Characteristics

- Desired speed (mi/h).
- Initial speed of truck at beginning of analysis section (mi/h).
- Weight-to-power ratio (lb/hp).
- Weight-to-front-area ratio (lb/ft^2).

Measured speed profiles for vehicles traveling on two of the entrance ramps (i.e., I-376/Ardmore Blvd with a 4 percent upgrade and I-376/Wm. Penn Hwy with a 6 percent upgrade) were compared to speed profiles calculated using the TSPM. The vertical alignment of each ramp was input into the TSPM along with the initial speed at the painted nose and the desired speed (i.e., final speed) at the merge location

Table 29. Summary statistics of truck speed profiles compared with TSPM.

Total number of vehicles	127
Total number of free-flow vehicles	75
Total number of platooned vehicles	52
Total number of semitrailers	37
Total number of free-flow semitrailers	24
Total number of platooned semitrailers	13
Total number of free-flow vehicles (matched)	60
Total number of platooned vehicles (matched)	45
Total number of free-flow semitrailers (matched)	19
Total number of platooned semitrailers (matched)	11
Total number of vehicles under free-merge condition	109
Total number of vehicles under free-merge condition (matched)	102
Total number of vehicles under constrained-merge condition	2
Total number of vehicles under constrained-merge condition (matched)	0
Total number of vehicles under forced-merge condition	15
Total number of vehicles under forced-merge condition (matched)	3

as measured in the field. Then a weight-to-power ratio was input and adjusted to obtain a profile that matched the speed profile measured in the field as closely as possible.

Table 29 provides summary statistics of the number and types of vehicles for which speed profiles were compared. In total, the speed profiles of 127 vehicles classified as trucks were compared, including SU trucks, semitrailers, and buses. In many of the cases, the speed profiles from the field closely matched the profiles from the TSPM; however, in several cases a good match could not be found. Figure 34 illustrates a speed profile that was considered a good match to the TSPM data, while Figure 35 illustrates a speed profile that was considered a bad match. Of the 127 vehicles, good matches were found for 105 of the speed profiles.

Figures 36 and 37 illustrate the distribution of weight-to-power ratios for those speed profiles that closely matched the TSPM data. The upper portion of Figure 36 shows the assumed weight-to-power ratios based on TSPM estimates for 60 free-flow trucks, and the bottom portion shows the assumed weight-to-power ratios for 45 trucks under platoon conditions. Similarly, the upper portion of Figure 37 shows the assumed weight-to-power ratios for 19 free-flow semitrailers, and the bottom portion shows the assumed weight-to-power ratios for 11 semitrailers under platoon conditions. Based upon the TSPM estimates, very few of the trucks had weight-to-power ratios greater than 140 lb/hp. Only two of the trucks had a weight-to-power ratio of 200 lb/hp, considered to be the weight-to-power ratio for a typical heavy truck for design of upgrades.

After efforts to calibrate the TSPM to closely match the speed-distance curves for acceleration/deceleration of a typical heavy truck of 200 lb/hp on upgrades and downgrades as illustrated in *Green Book* Exhibits 3-55 and 3-56, speed-distance curves were generated for four weight-to-power ratios of 140, 160, 180, and 200 lb/hp using the TSPM (Figures 38 through 41). A weight-to-power ratio of 140 lb/hp was selected for developing speed-distance curves based upon the distribution of weight-to-power ratios observed in the field at the two locations with grades of 4 percent and 6 percent. The other speed-distance curves for 160 and 180 lb/hp were developed as possible design tools if a lesser weight-to-power ratio is selected for design purposes than the typical 200-lb/hp design vehicle. Selecting a lower weight-to-power ratio for design may be desirable in some instances as a compromise to better accommodate trucks.

Even though speed-distance curves for trucks with weight-to-power ratios as low as 140 lb/hp are provided, it should be noted that there are very few instances where a 140-lb/hp truck can meet the design conditions for minimum acceleration lengths. On a 0 percent grade, a 140-lb/hp truck can accelerate from 14 to 23 mi/h within 140 ft and from 36 to 39 mi/h within 130 ft. For the other design conditions with 0 grade, and for all upgrades, a 140-lb/hp truck cannot meet the design conditions as specified in the *Green Book,* even taking into consideration the grade adjustment factors for upgrades.

Summary of Findings from Examination of Acceleration Rates. The most significant findings from the examination of acceleration rates at entrance ramps are as follows:

- Accelerations of merging vehicles along the ramp and SCL are not constant.

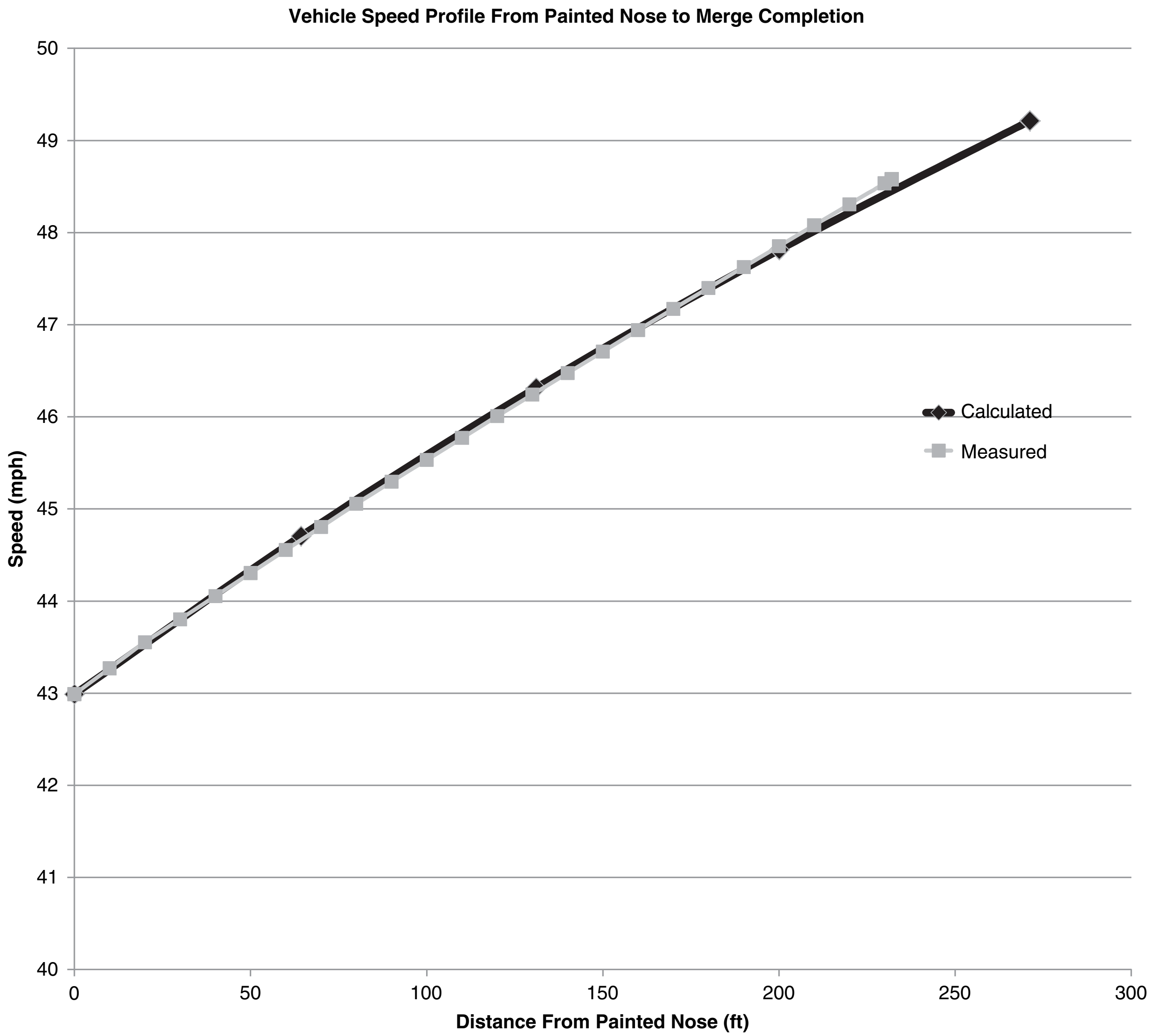


Figure 34. Good match between field and TSPM data.

- Merging vehicles sometimes reach a peak speed along the ramp and/or SCL and decelerate to slower speeds prior to merging onto the freeway. This behavior is most evident under constrained and forced-merge conditions, when freeway speeds are reduced.
- In many cases, vehicles are exiting the controlling feature along an entrance ramp at speeds greater than assumed in the *Green Book*.
- For free-flow passenger cars under free-merge conditions, the measured average speed at the merge location tends to be a few miles per hour less than the desired speed reached assumed in the *Green Book* when the ramp length is less than the recommended, and a few miles per hour greater than the *Green Book* desired speed reached when the ramp length is longer than the recommended.
- As expected, acceleration rates of free-flow trucks are lower than for free-flow passenger cars.
- The median acceleration rates for free-flow passenger cars under free-merge conditions are greater than the assumed acceleration rates in the *Green Book*. Based on the median acceleration rates of free-flow passenger cars measured under free-merge conditions, minimum acceleration lane lengths could be reduced between 16 to 46 percent and still be sufficient for the median vehicle.
- The 15th percentile acceleration rates of free-flow passenger cars under free-merge conditions are less than the assumed acceleration rates in the *Green Book*.
- The median acceleration rates for free-flow trucks under free-merge conditions are less than the assumed acceleration rates in the *Green Book*.

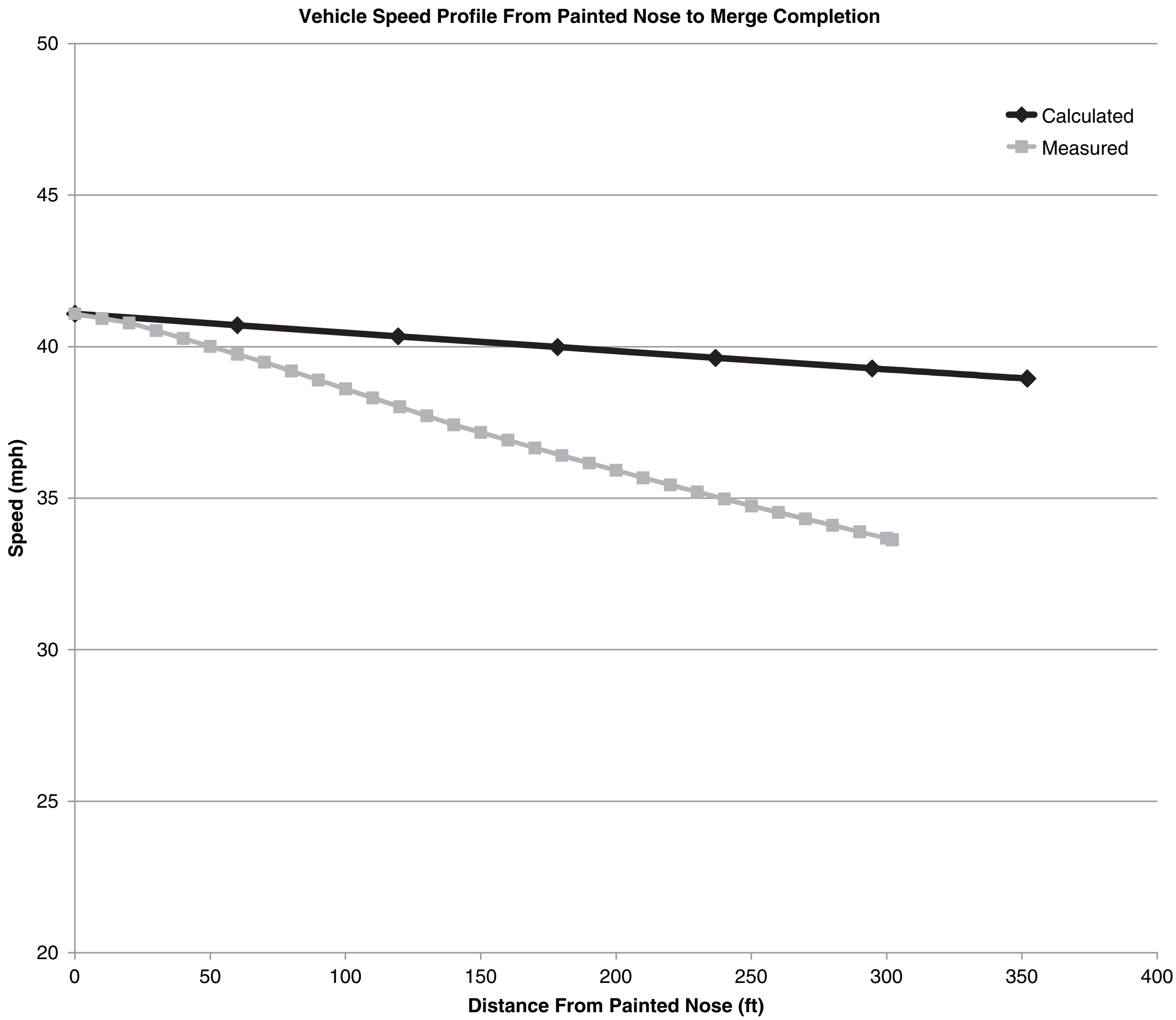


Figure 35. Bad match between field and TSPM data.

- Under constrained conditions, acceleration rates of passenger cars are very close to the assumed acceleration rates in the *Green Book.*
- The *Green Book* generally assumes acceleration rates decrease as initial speeds increase and also decrease as the final speed reached increases. In other words, considering Exhibit 10-70, acceleration rates decrease across rows and down the columns. For free-flow passenger cars under free-merge conditions, the pattern of acceleration rates decreasing as initial speeds increase (i.e., decreasing acceleration rates moving from right to left across rows) was observed, but the pattern of acceleration rates decreasing as speed reached increased (i.e., decreasing acceleration rates from top to bottom within columns) was not observed.
- Upgrades of 3 to 4 percent do not limit the acceleration capabilities of passenger cars. This is illustrated based upon the calculated adjustment factors having values less than one.
- For upgrades of 6 percent, measured acceleration rates of passenger cars are comparable to assumptions in *Green Book* Exhibit 10-71, or in some cases are slightly greater.
- With very few exceptions, a truck with a weight-to-power ratio as low as 140 lb/hp cannot meet the current design conditions in the *Green Book* for minimum acceleration lane lengths for level or upgrades.

5.4.1.4 Critical or Unusual Maneuvers

While reviewing the video recordings, very few critical or unusual maneuvers were observed in the vicinity of the entrance ramps. All of the critical maneuvers that were observed occurred at the same ramp during forced-merge conditions.

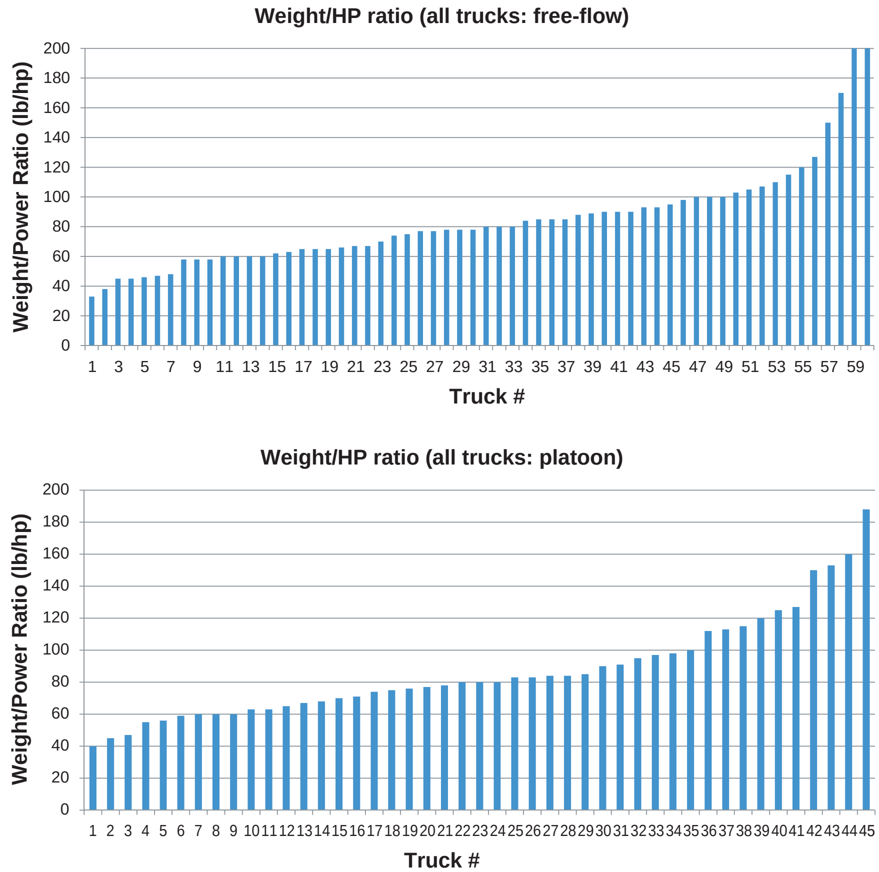


Figure 36. Weight-to-power ratio of all trucks (free-flow and platoon).

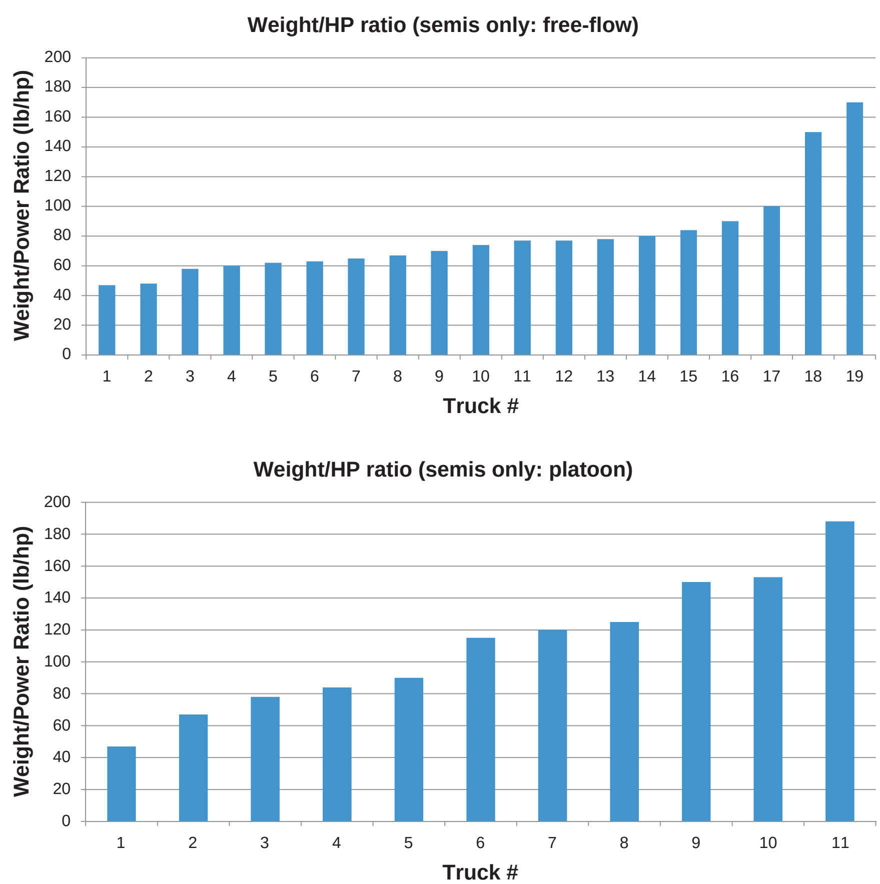


Figure 37. Weight-to-power ratio of all semitrailers (free-flow and platoon).

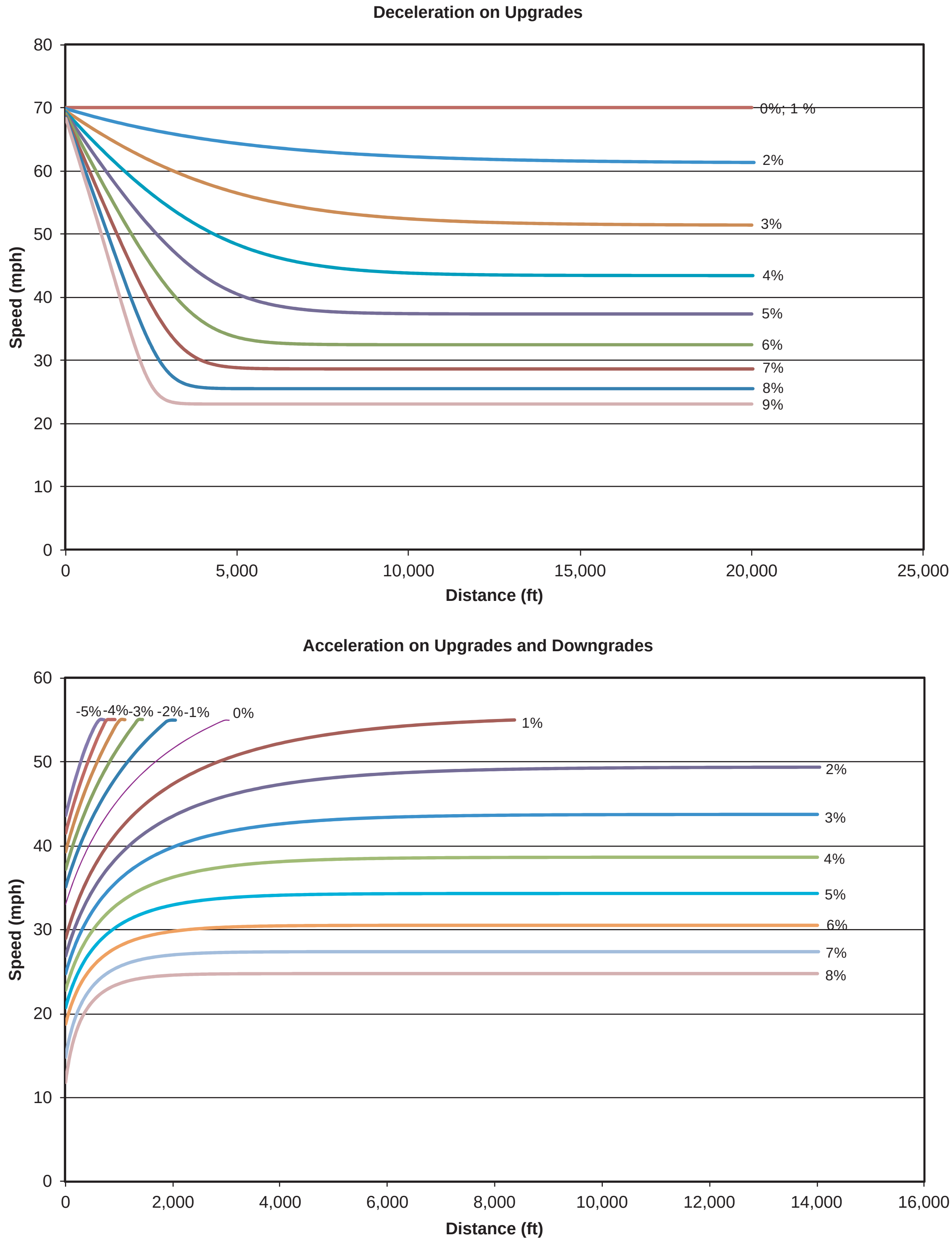
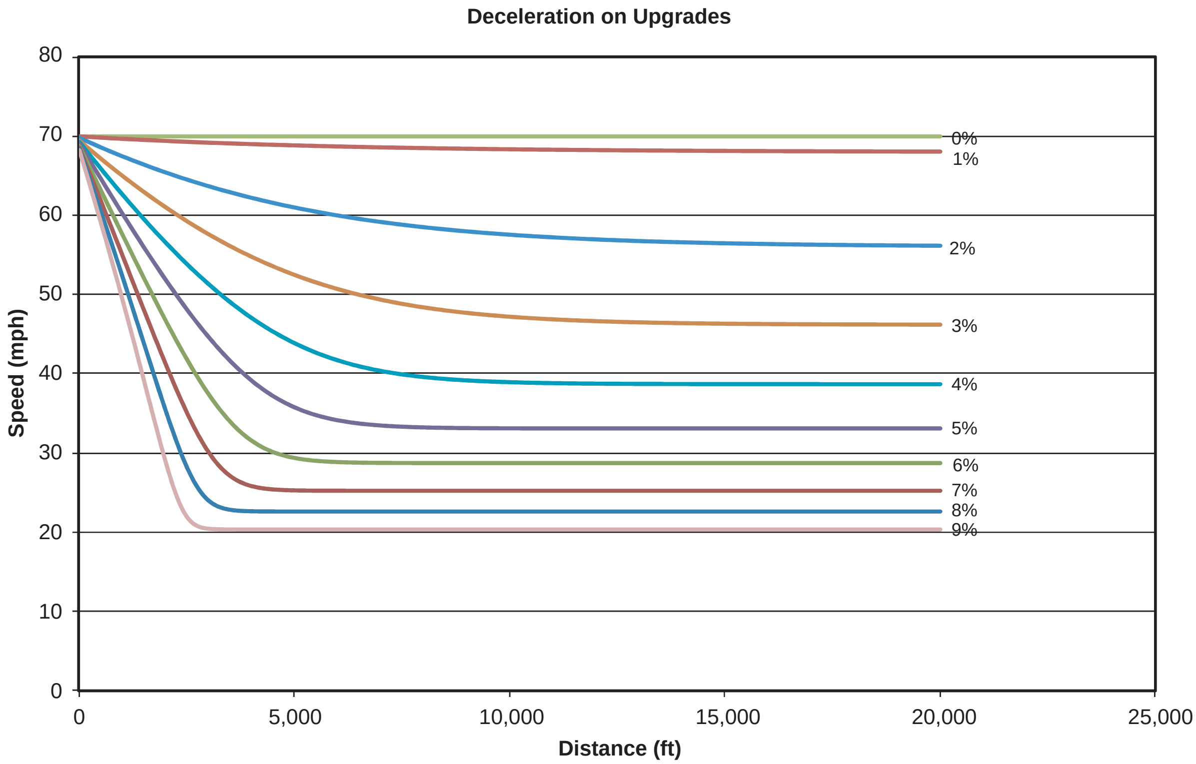


Figure 38. Speed-distance curves for acceleration/deceleration of a 140 lb/hp truck using the TSPM.



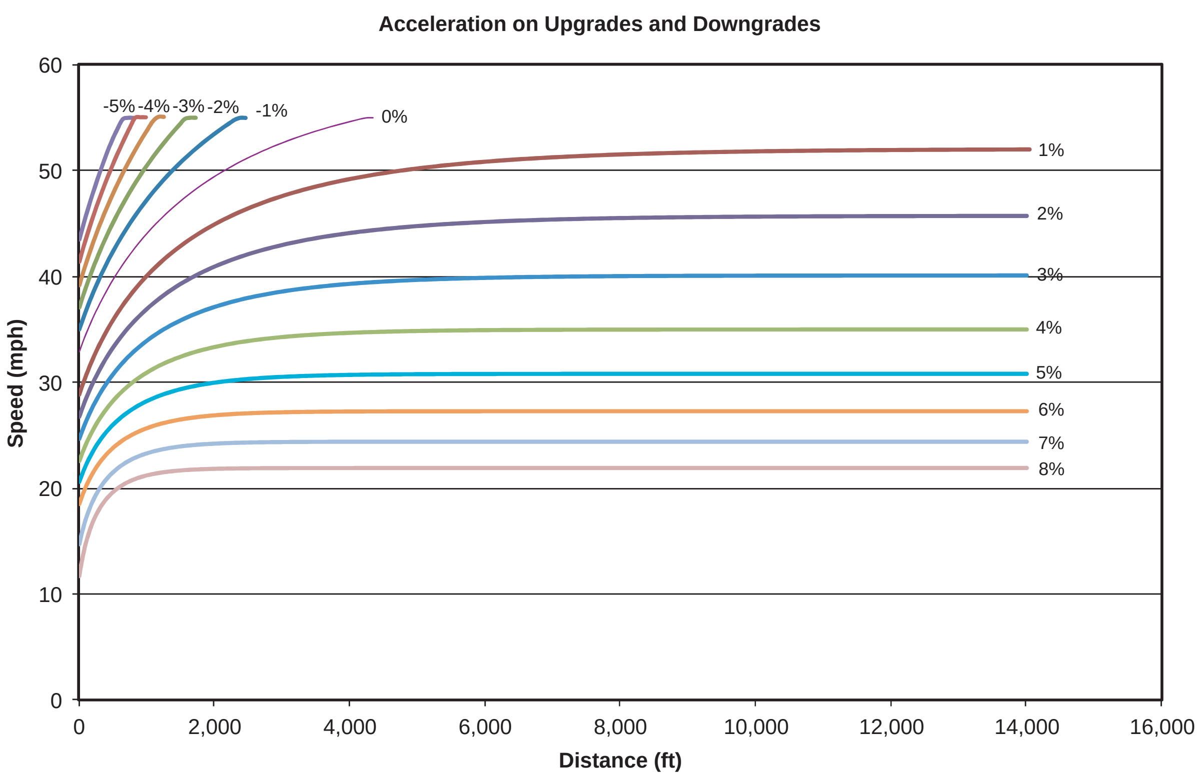


Figure 39. Speed-distance curves for acceleration/deceleration of a 160 lb/hp truck using the TSPM.

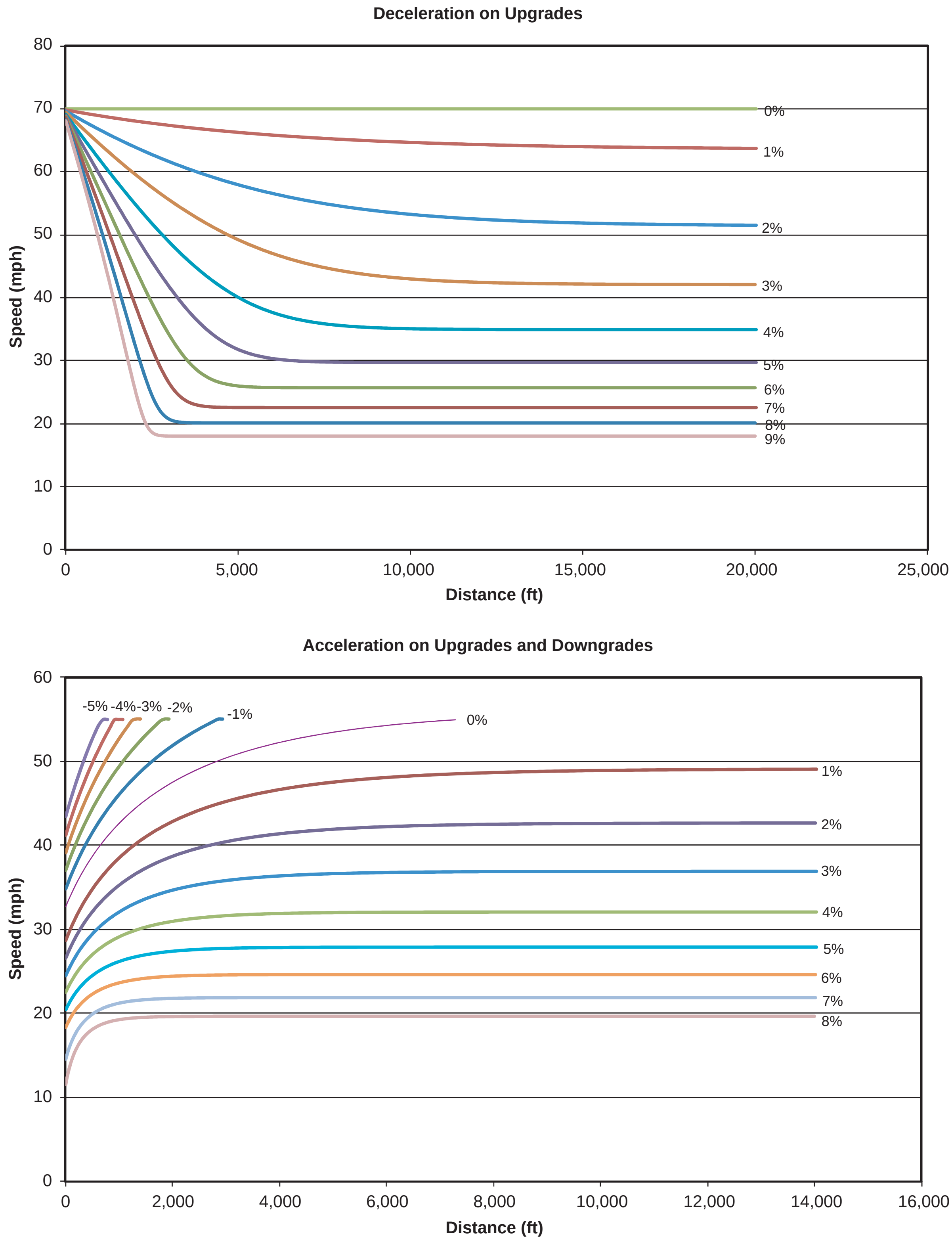


Figure 40. Speed-distance curves for acceleration/deceleration of a 180 lb/hp truck using the TSPM.

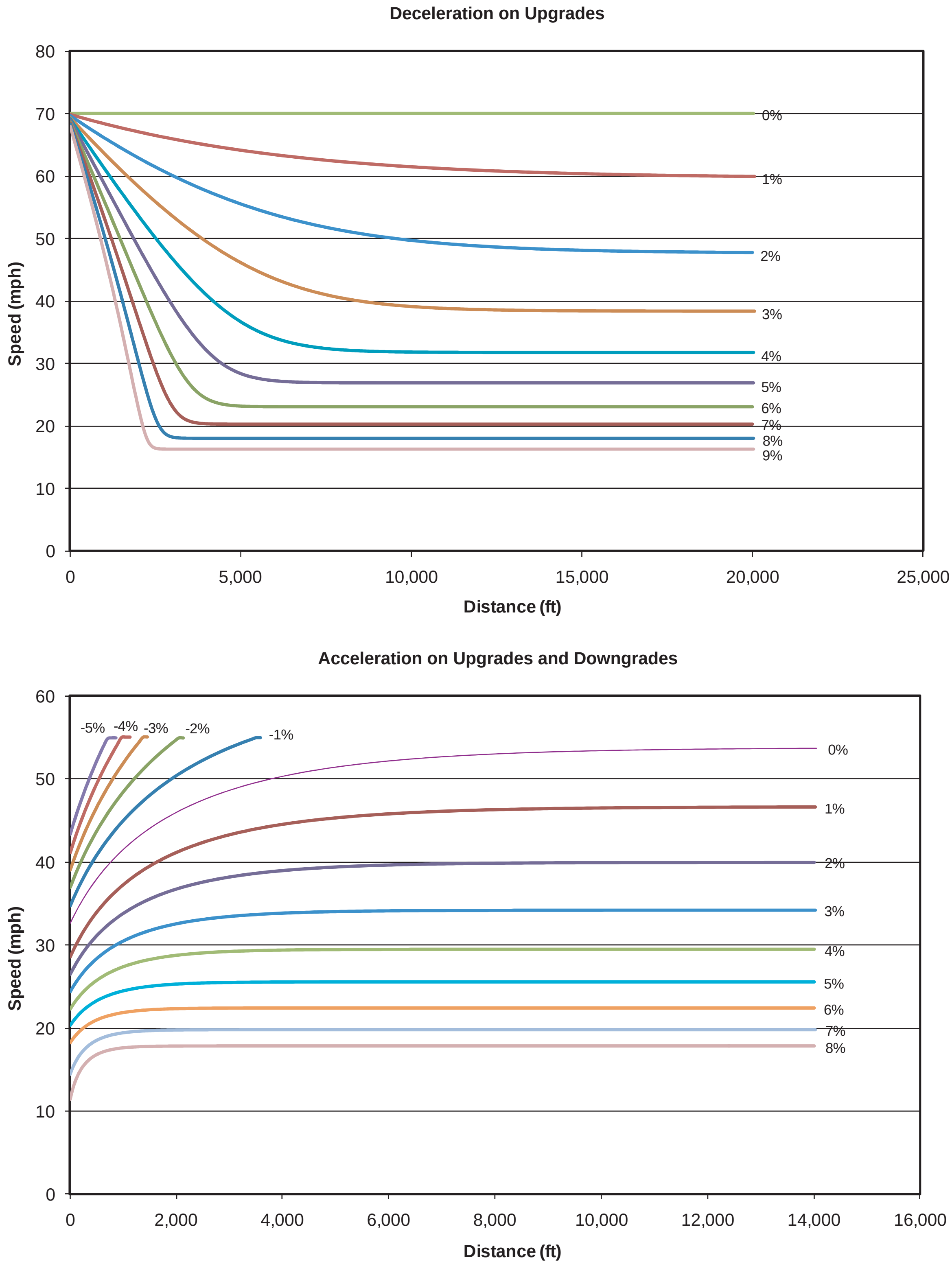


Figure 41. Speed-distance curves for acceleration/deceleration of a 200 lb/hp truck using the TSPM.

Four instances were observed in which a following (i.e., platooned) vehicle in the SCL passed the leading vehicle in the SCL using the right shoulder. In another instance, a following vehicle in the right lane of the freeway changed lanes into the SCL to pass slower vehicles in the freeway lanes.

5.4.2 Exit Ramps

The purpose of an exit ramp is to provide an appropriate means of accessing the adjacent surface streets from a location on a given freeway; the two primary components of exit ramp design that accomplish this purpose are (1) providing sufficient area for vehicles to depart from the main lanes of the freeway and (2) providing sufficient distance for vehicles to decelerate at a comfortable rate from freeway speeds to a speed appropriate for the crossroad terminal or controlling horizontal alignment feature. Similar to the characteristics of an entrance ramp, the distance needed for a vehicle to reach speeds appropriate for the crossroad or controlling feature is a function of the vehicle's deceleration rate along the SCL and the ramp. Ramp design guidelines that assume a deceleration rate greater than what is comfortable for the driver or physically possible for the vehicle may provide lengths that are too short. If the SCL and ramp (the full area over which a vehicle may diverge from the freeway and decelerate before reaching the crossroad or controlling feature) are too short, this may be evident by observations of vehicles diverging upstream of the taper area or by decelerating in the freeway mainlane before entering the SCL. Both of these measures are considered in this analysis. In addition, an analysis is performed to consider the deceleration profiles of the observed vehicles to determine whether the current vehicle fleet is decelerating as expected based on current ramp design guidelines described in the AASHTO *Green Book*.

In summary, under ideal operating conditions along an exit ramp, vehicles begin decelerating after exiting the freeway onto the deceleration lane, and vehicles decelerate along the deceleration lane to an appropriate speed before reaching the controlling feature of the ramp, which could either be a controlling horizontal curve or a crossroad terminal.

The greatest concern arises when vehicles enter the controlling feature of the ramp at too high of speeds. Vehicles decelerating in the freeway lanes prior to diverging is of lesser concern. The analyses were designed to compare actual driving practices to these ideal operating conditions and identify areas of concern.

5.4.2.1 Diverge Location

For an exit ramp, vehicles will ideally diverge from the freeway at a point somewhere along the SCL—that is, beyond the taper, but prior to the painted nose. Similar to the analysis of merge points for vehicles entering the freeway, this analysis divided the possible diverge locations into five categories: upstream of or within the taper (taper), in the first third of the SCL (early), in the middle third of the SCL (mid), in the last third of the SCL (late), and beyond the painted nose (gore). However, in this study, only 10 vehicles observed (less than 0.5 percent of the total vehicles observed) began their diverge maneuvers in the late or gore areas, so the results presented here will be confined to the taper, early, and mid areas.

The taper area is of particular interest because a driver beginning the diverge maneuver either before or within the taper suggests that one of two scenarios is likely:

1. The driver believes that the provided SCL and ramp length are not sufficient to comfortably decelerate to the appropriate speed for the crossroad or controlling feature.
2. The driver wants to decelerate at a lower, more casual rate than that associated with the ramp design and decides to utilize the extra distance in the taper to accomplish this.

The second of these two scenarios will be discussed in more detail as the results from the analysis are presented.

Figure 42 presents the diverge location by vehicle type (i.e., passenger cars versus trucks) on all of the study ramps, based on the laser data, broken out by freeway speeds. The top section shows diverge locations for all freeway speed categories combined, the next section shows diverge locations during free-diverge conditions on the freeway (i.e., freeway speeds greater than 50 mi/h), and the bottom section shows diverge locations during constrained-diverge conditions on the freeway (i.e., freeway speeds between 40 and 50 mi/h). Of the 2,584 vehicles observed, 86 percent exited the freeway in the taper area, 12 percent exited in the first third of the SCL, and 2 percent exited in the middle third of the SCL. In addition, 97 percent of vehicles exited the freeway under free-diverge conditions, while the remaining 3 percent completed their diverge maneuvers under constrained-diverge conditions. Given that no vehicles diverged under forced-diverge conditions, this category is also omitted from the figures in this analysis.

Figure 42 shows that 86 percent of both passenger cars and heavy trucks exited the freeway in the taper area, 12 to 13 percent of both vehicle types exited early, and 1 to 2 percent exited in the middle. In general, the distribution of diverge location is very similar for both vehicle types. Figure 42 also shows that the proportion of vehicles diverging in the first third of the SCL increased under constrained-diverge conditions. This is an intuitive finding because the decreased speeds under constrained-diverge conditions reduce the speed differential between the freeway and the controlling feature of the ramp, which in turn reduces the deceleration rate needed to safely and comfortably travel through the ramp. Under constrained-diverge conditions, some drivers may feel comfortable diverging as early as possible in the taper to enter an

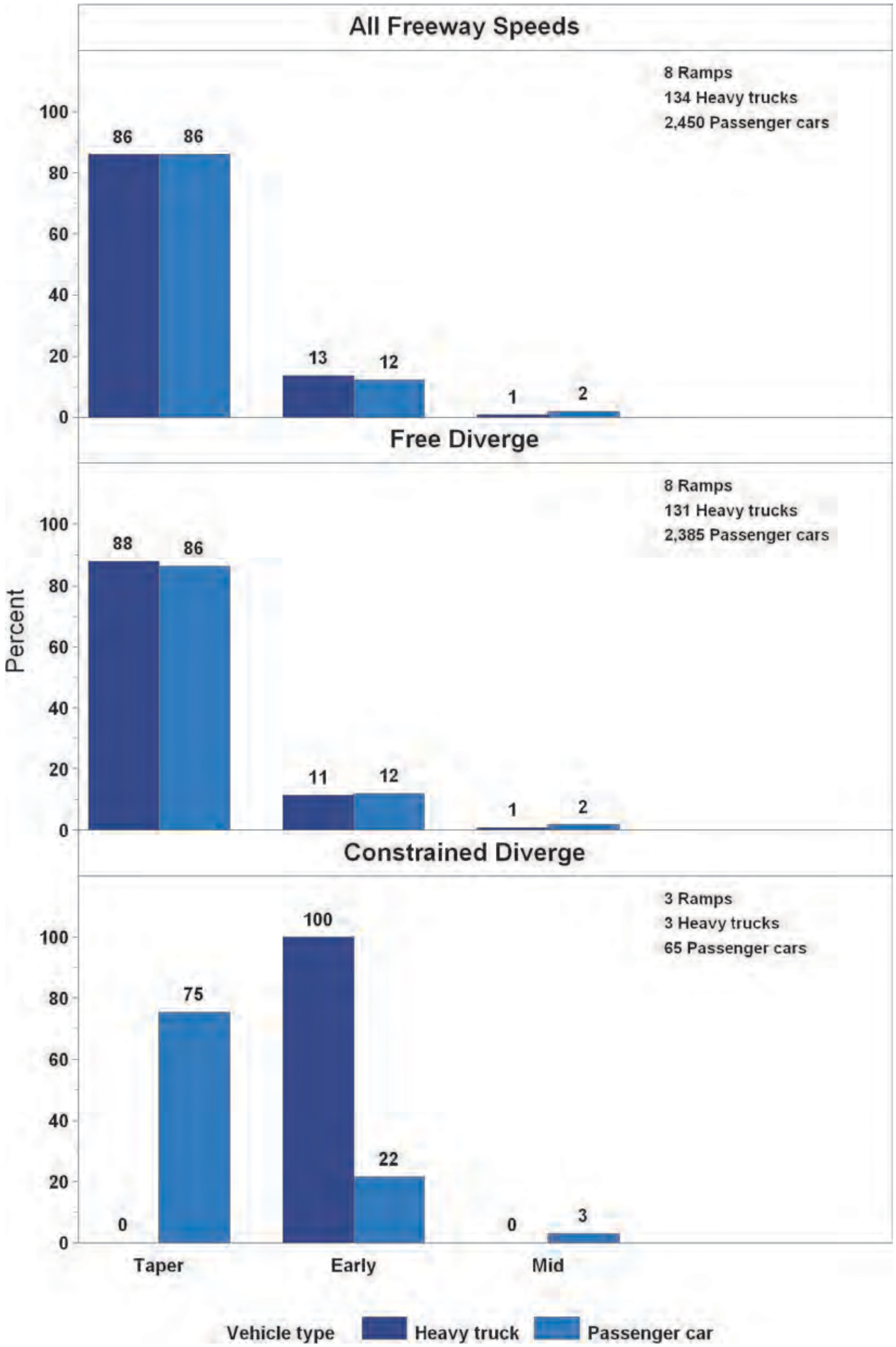


Figure 42. Diverge location—all ramps by vehicle type.

uncongested ramp, but the lower initial speed at the diverge point allows them to exit the freeway at a more casual deceleration rate. It is also important to note that because such a high proportion of the observed vehicles exited under free-diverge conditions within the taper, analysis of the entire population of observed vehicles is heavily influenced by this block.

Figure 43 shows the distribution of diverge location by ramp type. Regardless of freeway speed condition, over 90 percent of vehicles left the freeway in the taper portion of straight ramps. On loop ramps, about 80 percent of vehicles left the freeway in the taper portion of the ramp. The most noticeable change in the distribution of diverge locations occurred at loop ramps between free- and constrained-diverge conditions. Under free-diverge conditions, 81 percent of the vehicles diverged in the taper area and 16 percent diverged early in SCL, while under constrained-diverge conditions, only 42 percent of the vehicles diverged in the taper area and 55 percent diverged early in the SCL. This could be the result of an effect from the ramp geometry; a loop ramp typically has a shorter distance to the controlling feature so during free-diverge conditions when freeway speeds and diverge speeds are higher, drivers diverge earlier to use all available deceleration distance, including the taper, to achieve the appropriate operating speed to negotiate the curve on the ramp.

Figure 44 shows the distribution of diverge location by diverge type. Approximately two-thirds of the 1,106 observed vehicles on parallel exit ramps diverged in the taper area, while 99 percent of the 1,478 vehicles on taper exit ramps did the same. This is an intuitive finding because the SCL on a tapered deceleration lane is typically much shorter than that of a parallel deceleration lane, so it is reasonable that most drivers on a taper exit ramp would diverge within the taper. However, there is a marked difference between distributions under free-diverge conditions and constrained-diverge conditions. The proportion of vehicles exiting on the taper is similar for free- and constrained-diverge conditions on tapered SCLs, but it is substantially lower for constrained-diverge conditions than for free-diverge conditions on parallel SCLs. Similar to the findings discussed above, this may be in part because vehicles do not need as much deceleration distance when traveling at lower freeway speeds prior to their exit maneuver, and may therefore exit within the longer SCL provided at parallel SCLs.

Figure 45 summarizes the data for diverge location based on whether a vehicle began a free-flow diverge or was within a platoon. The figure shows that about 91 percent of platooned vehicles diverged from the freeway within the taper, compared to 85 percent of free-flowing vehicles. The difference between free-flow and platooned vehicles was more pronounced under constrained-diverge conditions, where only about two-thirds of free-flow vehicles used the taper area, compared to 91 percent of platooned vehicles.

The most significant findings from the examination of diverge locations are as follows:

- Over 99.5 percent of diverge maneuvers observed began either before or within the taper or within the first or middle thirds of the SCL.
- Vehicles tend to diverge later under constrained-diverge conditions than under free-diverge conditions.
- The distribution of diverge locations for trucks is very similar to the distribution of diverge locations for passenger cars for free-diverge conditions.
- A higher percentage of vehicles diverge early on straight ramps compared to loop ramps.
- More vehicles diverge later at parallel SCLs than at tapered SCLs, particularly under constrained-diverge conditions.
- A greater proportion of platooned vehicles began their diverge maneuver in the taper area than free-flow vehicles, particularly under constrained-diverge conditions.

5.4.2.2 Diverge Speed

The data presented in this section compares the speed of diverging vehicles at the time they begin to diverge to the speed of freeway traffic in the rightmost lane. Specifically, this is a comparison of initial diverge speeds of vehicles based upon the laser gun data to the average 15-minute freeway speed of the rightmost lane of the freeway at the time of the diverge. The *Green Book* describes the conditions for which an exit ramp should be designed as follows:

> Vehicles should decelerate after clearing the through-traffic lane and before reaching the point limiting design speed for the ramp proper. The length available for deceleration may be assumed to extend from a point where the right edge of the tapered wedge is about 3.6 m [12 ft] from the right edge of the right through lane, to the point of initial curvature of the exit ramp (i.e., the first horizontal curve on the ramp). The length provided between these points should be at least as great as the distance needed to accomplish the appropriate deceleration, which is governed by the speed of traffic on the through lane and the speed to be attained on the ramp. Deceleration may end in a complete stop, as at a crossroad terminal for a diamond interchange, or the critical speed may be governed by the curvature of the ramp roadway.

Researchers analyzed the speeds of observed exiting vehicles and compared them with the prevailing speeds on the freeway at the time of their exit maneuvers, calculating the difference between the speed of an exiting vehicle at the point of divergence and the average speed in the right lane of the freeway during the 15-minute period when the exit maneuver took place. Table 30 summarizes the data on mean speed differential by ramp and diverge type, vehicle type, and diverge condition. The mean value is the average of the

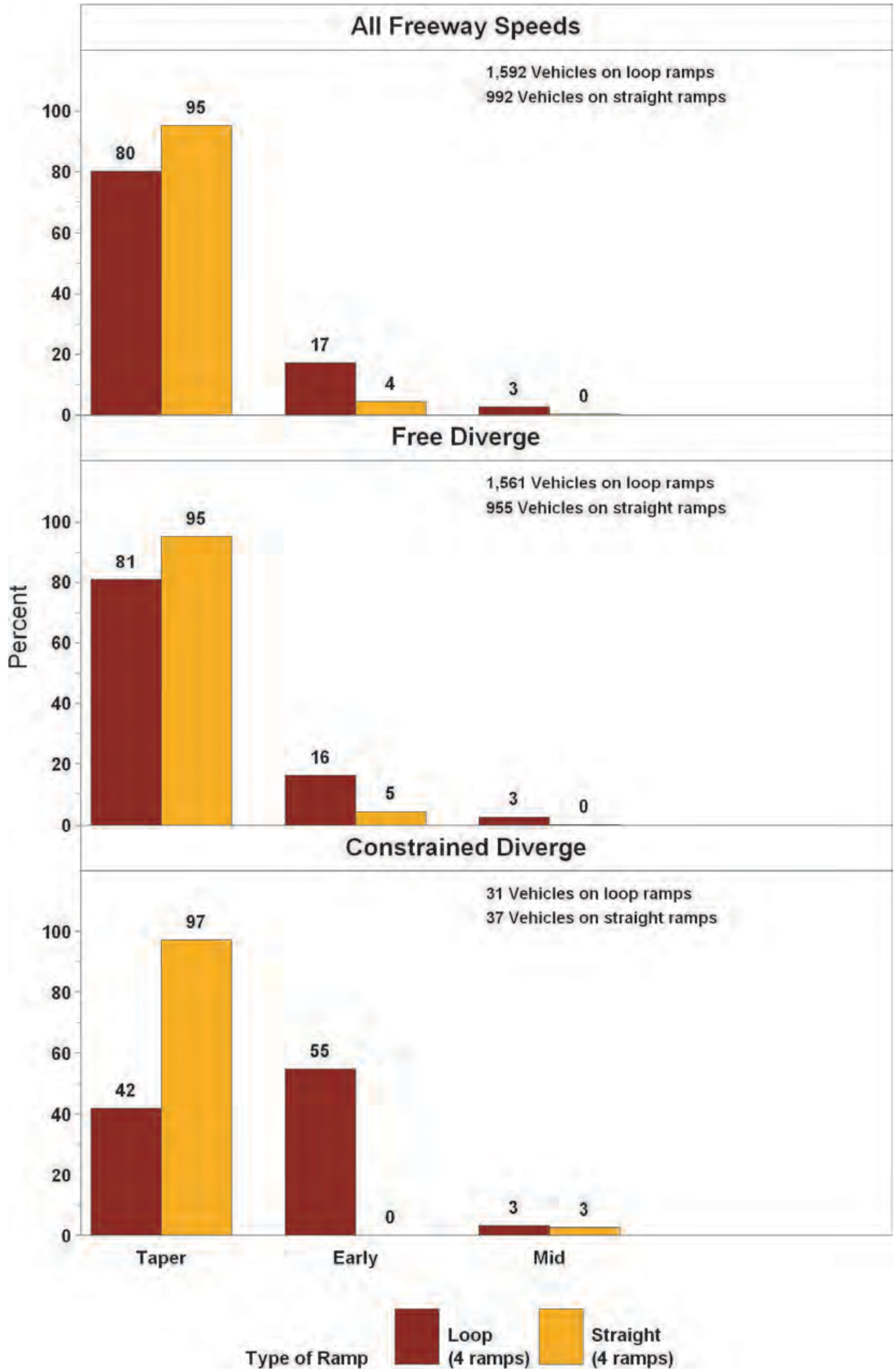


Figure 43. Diverge location—by ramp type.

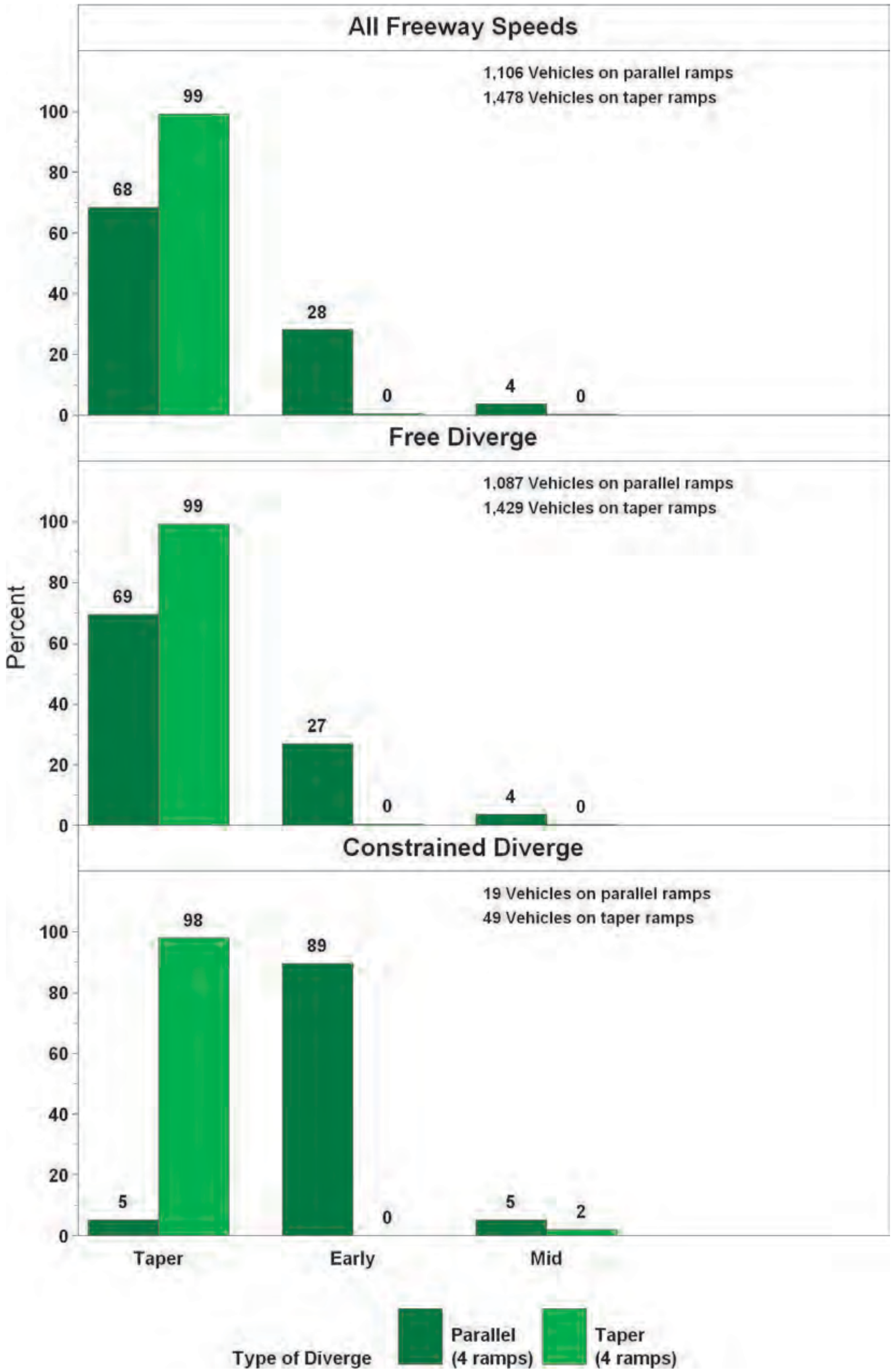


Figure 44. Diverge location—by diverge type.

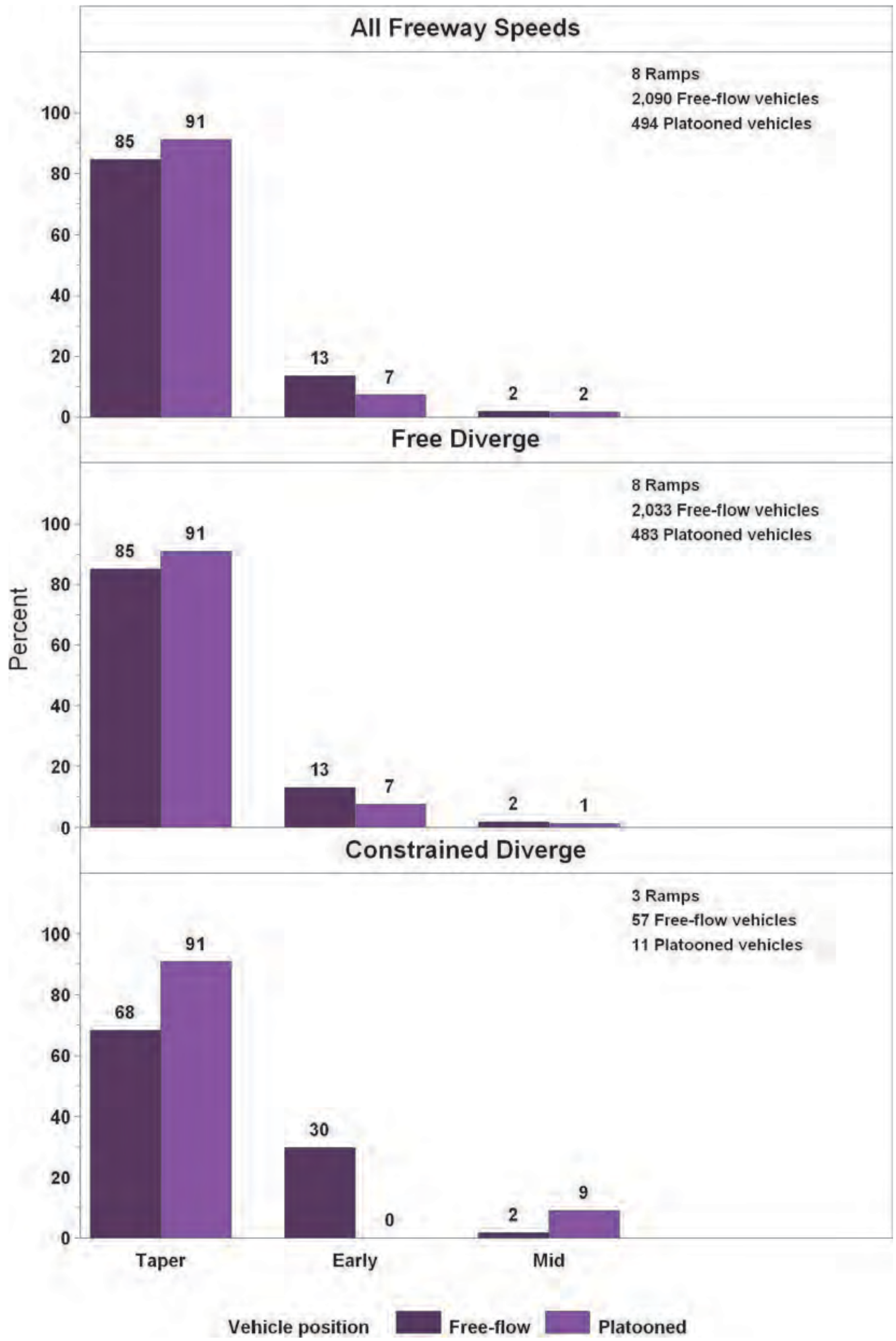


Figure 45. Diverge location—by vehicle position.

Table 30. Diverge speed and freeway speed differential by ramp type, diverge type, vehicle type, and speed category.

Type of ramp	Type of diverge	No. of ramps	Vehicle type	No. of vehicles	Diverge speed minus freeway speed (mi/h): Mean	Std dev	Min	Max	P-value for t-test of mean less than 0 mi/h
All freeway speeds									
Loop	Parallel	2	Truck	46	-7.1	5.1	-18.0	6.1	<0.001
			Passenger car	616	-4.2	4.6	-18.1	11.6	<0.001
	Taper	3	Truck	50	-10.7	4.4	-24.9	-1.6	<0.001
			Passenger car	901	-5.4	4.7	-25.2	13.3	<0.001
Straight	Parallel	2	Truck	18	-5.1	3.3	-14.1	-0.4	<0.001
			Passenger car	433	-1.6	4.6	-18.9	11.6	0.004
	Taper	2	Truck	21	-7.5	6.3	-22.1	3.3	<0.001
			Passenger car	523	-4.1	5.1	-20.8	11.3	<0.001
Free diverge									
Loop	Parallel	2	Truck	43	-7.7	4.8	-18.0	1.1	<0.001
			Passenger car	600	-4.4	4.4	-18.1	11.6	<0.001
	Taper	3	Truck	50	-10.7	4.4	-24.9	-1.6	<0.001
			Passenger car	875	-5.7	4.3	-25.2	11.4	<0.001
Straight	Parallel	2	Truck	18	-5.1	3.3	-14.1	-0.4	<0.001
			Passenger car	433	-1.6	4.6	-18.9	11.6	0.015
	Taper	2	Truck	21	-7.5	6.3	-22.1	3.3	<0.001
			Passenger car	485	-4.1	5.0	-20.8	11.3	<0.001
Constrained diverge									
Loop	Parallel	1	Truck	3	1.0	4.3	-1.6	6.1	
			Passenger car	16	1.6	7.4	-15.3	11.5	
	Taper	1	Truck	0					
			Passenger car	12	4.6	5.9	-5.3	13.3	
Straight	Parallel	0	Truck	0					
			Passenger car	0					
	Taper	1	Truck	0					
			Passenger car	38	-5.3	5.4	-17.7	5.8	

individual ramp averages. Also provided are the standard deviation (pooled across individual ramps), minimum, and maximum values measured for each condition. A negative mean speed indicates that the freeway speed is greater than the diverge speed.

Table 30 illustrates that, in general, the speeds of diverging vehicles were below the average speed on the freeway's right lane. Typically, the difference was between 4 and 7 mi/h, though it was as much as 10.7 mi/h for heavy trucks on loop ramps with taper departures. Similarly, the mean speed differential for trucks is a lower value (i.e., greater negative difference in speed) than the corresponding value for passenger cars in every category. Only under constrained-diverge conditions was the average speed of exiting vehicles higher than the average speed of freeway vehicles. The maximum differential for trucks was lower than that for cars in every category, though the difference between minimum and maximum values was typically larger for cars.

To further evaluate the effect of different ramp characteristics on the difference between diverge speed and freeway speed (speed differential), a mixed effects ANOVA model was developed. The model included main effects for ramp type (straight versus loop) and diverge type (parallel versus taper). The interaction effect between the two was not included based on comparison of the differences of group means, and it would not have been estimable given the number of ramps in the study. The model also includes a random ramp effect to account for individual ramp-to-ramp variability and the fact that repeated measurements were taken at each ramp. The between-ramp variability was assumed to be equivalent for ramp types and diverge types based on comparison of the variability in ramp means. The model was estimated using REML, and estimates of the differences between ramp types and diverge types were obtained using least squares means. Statistical degrees of freedom for evaluation of the parameters was based on the number of ramps, not the number of vehicles.

The mean speed differential for each ramp and diverge type combination was evaluated to determine if it was statistically less than 0 mi/h. The results are provided in the last column of Table 30. Due to the minimal number of vehicles and ramp conditions available for the constrained-diverge condition, no statistical analyses were performed for this operational condition. For passenger cars under free-diverge conditions, the following results were obtained from the ANOVA model:

- The between-ramp standard deviation in the mean diverge speed differential was estimated to be 1.09 mi/h. The within-ramp (vehicle-to-vehicle) standard deviation was estimated to be 4.47 mi/h.
- The mean speed differential for loop ramps of −5.02 mi/h is statistically less than 0 mi/h (p-value < 0.001). For straight ramps, the mean of −2.85 mi/h is also statistically less than 0 mi/h with a p-value = 0.002.
- Parallel and taper diverge ramps (both ramp types) have statistically significant differences of −3.03 mi/h and −4.85 mi/h respectively (both p-values <0.001).
- Loop ramps tend to have a slightly greater mean diverge speed differential (−2.17 mi/h) as compared to straight ramps. The difference is statistically significant with a p-value of 0.03.
- Taper ramps have a slightly greater mean diverge speed differential than parallel ramps (1.82 mi/h) that is statistically significant (p-value = 0.05).

Figure 46 is a graphical representation of the data in Table 30, organized by vehicle type and diverge location. The upper portion of the figure shows the distribution of speed differentials for passenger cars, with the corresponding information for heavy trucks shown in the lower portion. As with the data in Figures 42 through 45, the data for passenger cars under free-diverge conditions exiting at the taper constitute the largest category of observed vehicles. Figure 46 shows that for passenger cars under free-diverge conditions the speed differential increased as the diverge locations approached the painted nose (i.e., vehicles that diverged earlier along the exit diverged closer to freeway speeds than vehicles that diverged later along the deceleration lane). This indicates that passenger cars begin decelerating within the freeway lanes even though they may not necessarily diverge from the freeway lanes at the earliest opportunity. Figure 46 also illustrates that trucks diverged from the freeway at speeds about 2 to 4 mi/h below the speeds of passenger cars, and the variability of speed differentials was less for trucks than for passenger cars.

Figures 47 through 50 show subsets of the data presented in Figure 46. Figure 47 presents observations made at loop ramps with parallel SCLs. For this ramp and diverge type, for passenger cars under free-diverge conditions the speed differential increased as the diverge locations approached the painted nose. For trucks under free-diverge conditions, the speed differential basically was the same for taper and early diverge locations, and under constrained-diverge conditions, diverge speeds were

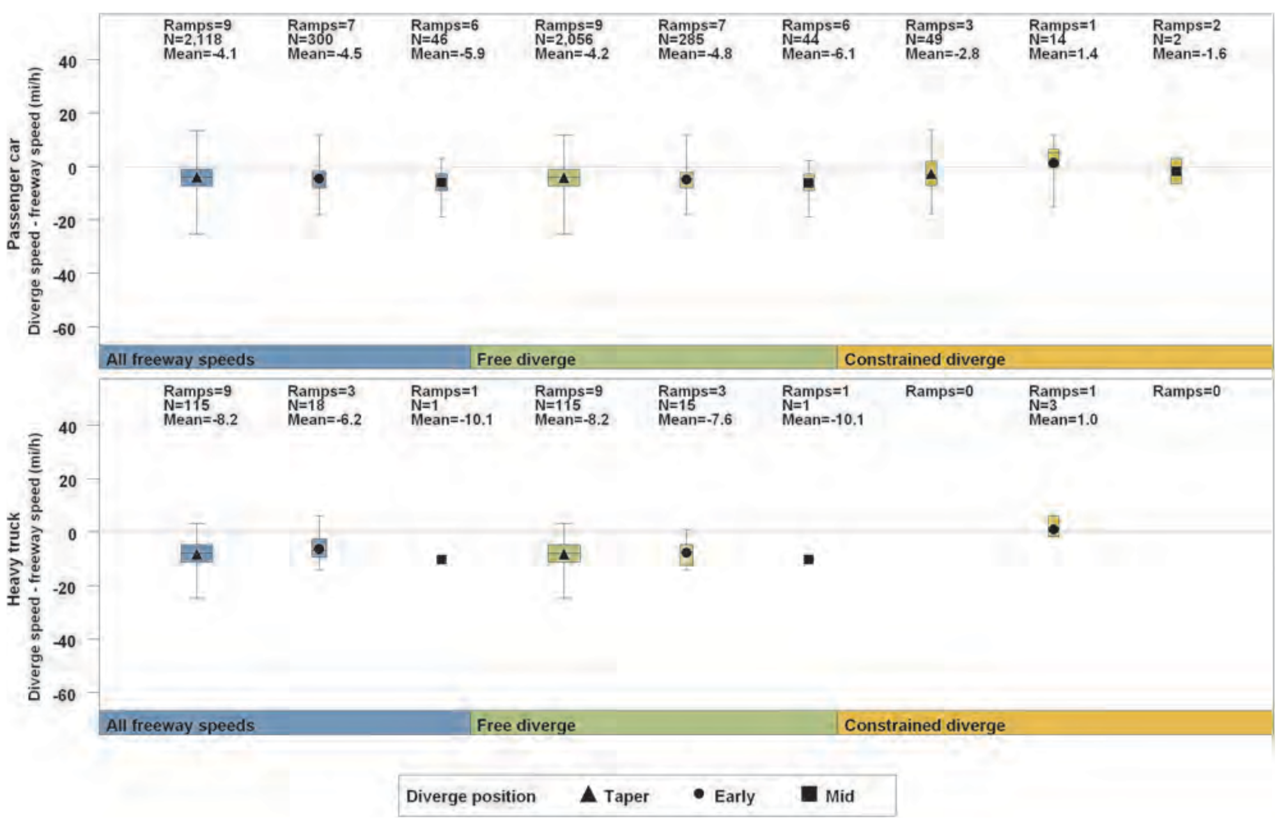


Figure 46. Diverge speed differential by vehicle type and diverge location for all ramp types.

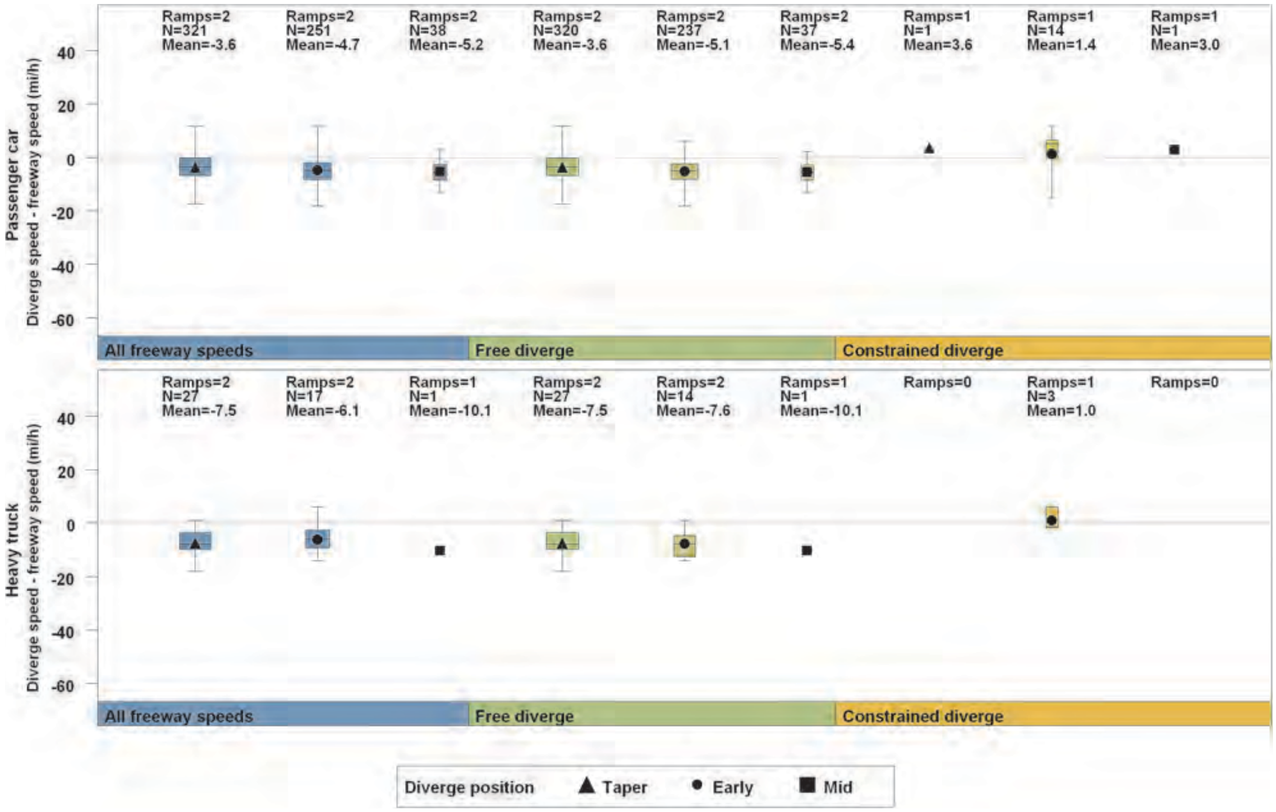


Figure 47. Diverge speed differential by ramp type, diverge type, vehicle type, and diverge location (ramp type: loop; diverge type: parallel).

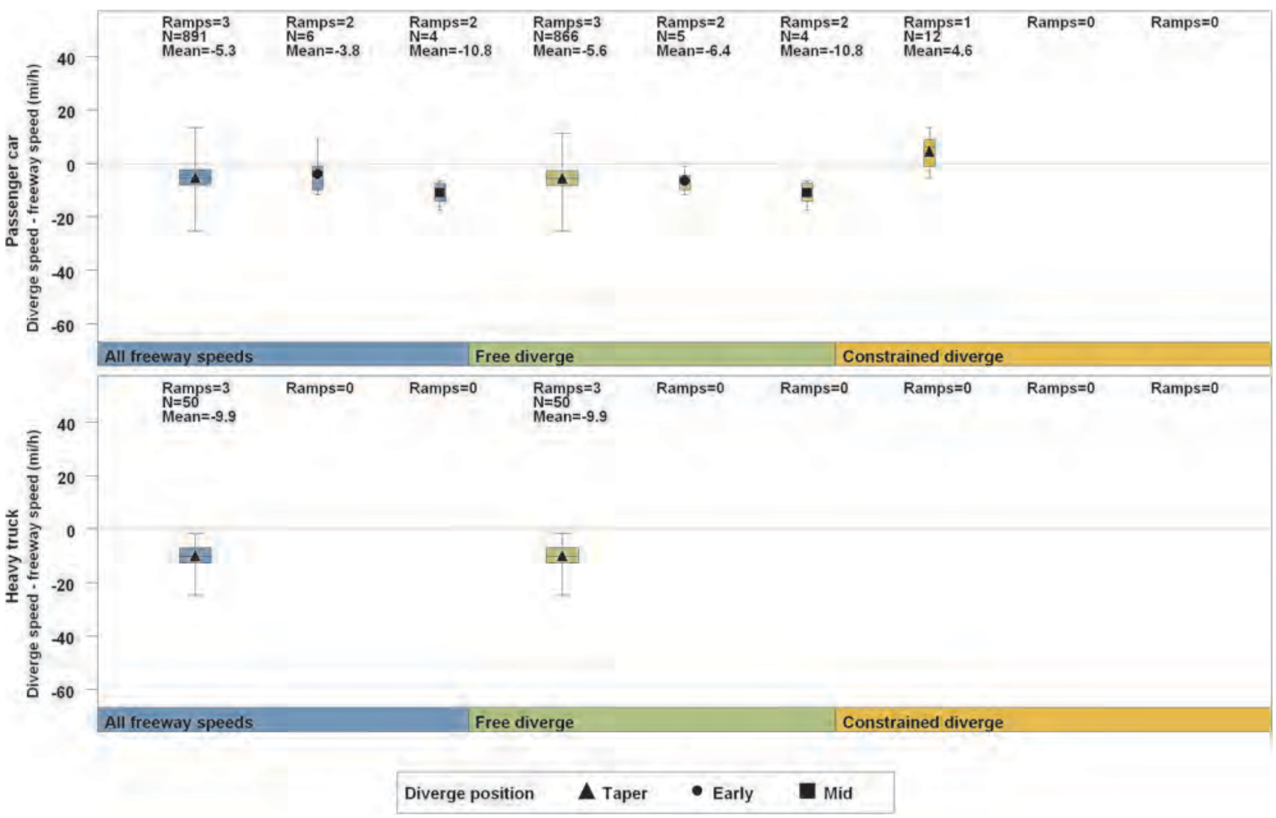


Figure 48. Diverge speed differential by ramp type, diverge type, vehicle type, and diverge location (ramp type: loop; diverge type: taper).

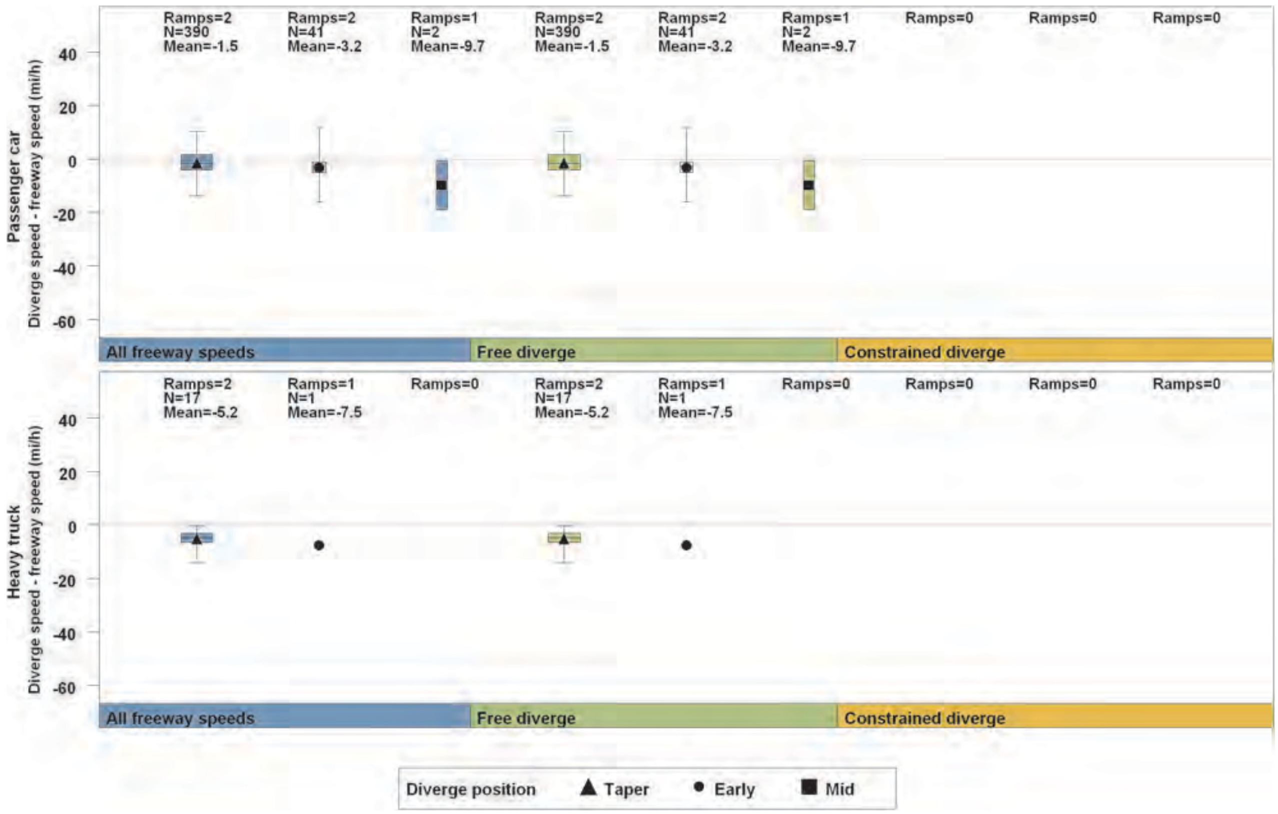


Figure 49. Diverge speed differential by ramp type, diverge type, vehicle type, and diverge location (ramp type: straight; diverge type: parallel).

Figure 50. Diverge speed differential by ramp type, diverge type, vehicle type, and diverge location (ramp type: straight; diverge type: taper).

slightly greater than average freeway speeds for both passenger cars and trucks.

Figure 48 presents observations made at loop ramps with tapered SCLs. For this ramp and diverge type, trucks that diverged at the taper diverged at speeds approximately 4 mi/h slower than passenger cars that diverged at the same location. Again under constrained-diverge conditions, diverge speeds were slightly greater than average freeway speeds for passenger cars.

Figure 49 shows the speed differentials for straight ramps with parallel SCLs. For this ramp and diverge type, observations were made only under free-diverge conditions. Again for passenger cars under free-diverge conditions the speed differential increased as the diverge locations approached the painted nose.

Figure 50 shows the speed differentials for straight ramps with tapered SCLs. For this ramp and diverge type, there was a greater speed differential for passenger cars that diverged at the taper during constrained-diverge conditions than during free-diverge conditions.

Figure 51 shows the speed differentials for each of the four ramp type and merge type combinations for only the free-diverge condition. Again, the trend that is most evident across all combinations is that the speed differential increased as the diverge locations approached the painted nose.

The most notable findings from the examination of diverge speeds compared to freeway speeds are as follows:

- With few exceptions, the speeds of diverging vehicles were between 4 and 7 mi/h below the average speed on the freeway's right lane. For every category, the mean speed of trucks was lower than that of passenger cars. These results indicate that observed drivers are completing some degree of their deceleration within the freeway lane, and the finding is consistent with previous findings that trucks frequently accomplish more of their deceleration in the freeway lane.
- Speed differential increased as the diverge locations approached the painted nose.
- Speeds of diverging vehicles under constrained-diverge conditions were closer to freeway speeds than under free-diverge conditions.

5.4.2.3 Deceleration Rate

The third measure of performance considered is the deceleration rate of vehicles as they travel along the SCL and ramp after the diverge point. As with SCLs on entrance ramps, the AASHTO *Green Book* does not provide a table of the assumed deceleration rates used to determine guidance for minimum

Figure 51. Diverge speed differential by ramp type, diverge type, vehicle type, and diverge location (for free-diverge conditions only).

deceleration lane length. Section 3 of this report provides two methods for back-calculating deceleration rates based on *Green Book* Exhibit 10-73. One approach is based on a two-step process of deceleration, which includes deceleration during coasting and deceleration during braking, and a second approach is based on a constant deceleration over the entire deceleration lane length. The deceleration rates measured in the field can be compared to the *Green Book* rates based on these two differing design conditions to determine whether vehicles are performing as assumed in the *Green Book.* Such a comparison, however, is not a truly direct comparison, because vehicle deceleration in the field is influenced by the same factors as described for acceleration rates in Section 5.4.1.3. For example, ramps longer than the minimum length needed to reach the desired speed at a controlling feature using a comfortable deceleration allow the driver more flexibility in determining when and where to diverge and decelerate along the ramp. When ramps are long, some drivers maintain speeds closer to the prevailing freeway speed in or adjacent to a portion of the SCL and then decelerate more quickly as they enter the SCL and approach the controlling feature. Alternately, some drivers begin their deceleration earlier, but at a lower, more casual rate, and maintain that initial rate throughout the ramp. Therefore, decelerations measured at a given point on the SCL or the ramp may not be representative of vehicle capabilities or driver comfort levels and preferences, because vehicles typically do not decelerate along the ramp at a constant rate. Because it was not possible to capture speed and deceleration along the entire ramp length, the behavior of vehicles near the crossroad terminal is not well documented here. Field-measured deceleration rates that are less than the values calculated from the *Green Book* may indicate that drivers were completing much of the required deceleration in the freeway lane upstream of the beginning of the taper and prior to the location where initial speed measurements were taken, contrary to assuming that the deceleration rates assumed in the *Green Book* are too high. For this reason, comparisons of field-measured deceleration to *Green Book* assumptions must be interpreted carefully and considered in context of the ramp characteristics and diverge conditions.

The analysis of deceleration rates includes:

- A general analysis of deceleration rates.
- Comparison of deceleration rates measured in the field to deceleration rates from the *Green Book* criteria for level grades, assuming a constant deceleration.
- Comparison of deceleration rates measured in the field to deceleration rates from the *Green Book* criteria for level grades, assuming a two-step deceleration process. The focus of this analysis is comparing deceleration rates measured in the field to the assumed deceleration rates during primary braking (i.e., the second portion of the two-step process).
- Comparison of deceleration rates on grades of 3 percent or greater.

The analysis of deceleration rates is based on the measured speed profiles of 2,264 vehicles.

General Analysis of Deceleration Rates. Tables 31 through 33 compare ramp length, measured diverge speed, and deceleration rates to those recommended or assumed in the *Green Book.* The tables are organized as described below.

Rows. Each row provides information about one exit ramp. The ramps are organized first by controlling feature, with those controlled by horizontal alignment in the upper section and those controlled by the crossroad terminal in the lower section. Within each section, the ramps are organized in order of decreasing difference between actual length of the ramp and the *Green Book* recommended ramp length.

Columns

Green Book *(GB) Initial Speed*—The value listed in the *Green Book* for the initial speed of an exit ramp. Conceptually this is the diverge speed of exiting vehicles. The average speed of the freeway is provided, rather than the design speed of the freeway.

Average Diverge Speed—The average speed at the diverge point for the vehicles observed and recorded at the ramp.

Average Diverge Distance—The average distance in feet upstream of the painted nose at which observed vehicles diverged from the freeway at the ramp.

Deceleration Length—SCL length plus distance from painted nose to controlling feature.

Difference from Green Book *Deceleration Length*—The difference in feet between the total length provided from the end of the taper to the painted nose (SCL) plus the length from the painted nose to the controlling feature and the minimum deceleration lane length provided in Exhibit 10-73 of the *Green Book* adjusted for grade. The *Green Book* criteria length is found using the design speed of the freeway and the radius of the controlling feature. For ramps without controlling horizontal curvature, the stop condition was assumed for the final speed. For example, a ramp on a highway with a design speed of 75 mi/h has an assumed diverge speed of 61 mi/h. For a ramp with a stop condition as its design speed, the *Green Book* recommends a deceleration length of 660 ft. The positive numbers in this column indicate that all exit ramps studied had ramp lengths longer than recommended by the *Green Book* criteria.

Green Book *Speed at Controlling Feature*—The average running speed listed in *Green Book* Exhibit 10-73 at the controlling feature for a ramp, based upon the design speed of the exiting curve. The horizontal alignment is considered controlling if the radius of the curve is equal to or less than

1,000 ft, consistent with current guidance provided in the 2004 *Green Book*.

Average Speed at Green Book *Distance*—The average speed recorded for observed vehicles at the distance corresponding to the *Green Book* deceleration length: the average observed speed at the location where the vehicle has traveled the minimum *Green Book* distance from the point of diverge.

Average Speed at Controlling Feature—The average speed recorded for observed vehicles at the controlling feature on each ramp. At some ramps, researchers were unable to collect speed data at the controlling feature (e.g., the final speeds of all profiles were upstream of the controlling feature); therefore, this cell has no value for those ramps.

Average Final Speed—The mean of all observed speeds at the location of the final speed measurement.

Assumed Green Book *Deceleration Rate*—Deceleration in ft/s^2 calculated from the assumed *Green Book* speed at diverge (initial speed), *Green Book* speed at controlling feature, and minimum deceleration lane length from Exhibit 10-73 in the *Green Book*. This is an assumed constant deceleration rate over the minimum deceleration lane length recommended in the *Green Book*.

Deceleration Rate over Green Book *Distance*—The 5th, 15th, and 50th percentile observed deceleration rates over the distance equal to that recommended in *Green Book* Exhibit 10-73 (and adjusted for grade). Taken from the distribution of the deceleration of all vehicles calculated from the measured diverge speed, the measured speed at the *Green Book* distance, and the time taken for each vehicle to travel that distance. The percentile values indicate the deceleration rate that is achieved or exceeded by that percentage of vehicles. For example, Table 31 shows that at I-435/Quivira, 5 percent of observed free-flow passenger cars decelerated at a rate greater than 1.83 ft/s^2 over the distance equal to the corresponding *Green Book* minimum length.

Deceleration Rate to Controlling Feature or Final Speed—The 5th, 15th, and 50th percentile observed deceleration rates from the point of divergence to the controlling feature, or to final reading when speed was not recorded at controlling feature. Taken from the distribution of the deceleration of all vehicles calculated from the measured diverge speed, the measured speed at the controlling feature (or the final measured speed), and the time taken for each vehicle to travel that distance.

Tables 31 through 33 provide details on diverge speeds, diverge location, speeds at controlling feature, final speeds, and deceleration rates based upon observations from the current vehicle fleet and compared to the corresponding values in the *Green Book*. Separate tables are provided for free-flow vehicles (Table 31), platooned vehicles (Table 32), and both combined (Table 33) under free-diverge conditions. Because *Green Book* Exhibit 10-73 is also based strictly on passenger cars, the data in Tables 31 through 33 are for passenger cars only.

Tables 31 through 33 show several important observations, which are summarized as follows:

- The observed speeds at the diverge points are typically at or below those assumed by the *Green Book*.
- The average speed at the diverge point is higher for free-flow vehicles than platooned vehicles on every ramp but one.
- The average observed speeds at the controlling features are higher than the assumed *Green Book* speeds.
- For all vehicle movement categories (free-flow, platoon, and combined), deceleration rates in the 5th, 15th, and 50th percentiles are all closer to zero for the longer ramps (those more than 400 ft longer than the *Green Book* recommendation) than for the shorter ramps (those within 100 ft of the *Green Book* recommendation), which is intuitive because the greater length allows for more casual braking when exiting the freeway. This is illustrated in Figure 52.
- For all ramps and vehicle categories, the observed deceleration rates over the distance equivalent to the *Green Book* deceleration length and the rates over the entire observed exit maneuver are less than the constant deceleration rates needed to match *Green Book* conditions.
- At every ramp, for both free-flow and platooned vehicles, the observed deceleration rates to the controlling feature and to the point of final reading in each percentile are closer to zero than the corresponding deceleration rate over the *Green Book* distance.
- There is no consistent trend in the comparison of deceleration rates between free-flow and platooned vehicles. In some cases, the deceleration rates for free-flow vehicles are greater, while in other cases, the decelerations rates for platooned vehicles are greater.
- On the whole, observed deceleration rates on loop ramps are greater than those on straight ramps, and frequently by a noticeable difference. That difference is most pronounced in the 5th percentile rates for all cars and for free-flow cars (see Figure 53). The noticeable exception is I-635/Metropolitan, which has the highest deceleration rates of all straight ramps in the study. On balance, it should be noted that the loop ramps all have deceleration distances within 100 ft of the *Green Book* distance, as does the ramp at I-635/Metropolitan, so the effect of deceleration length should not be overlooked when considering influences of straight ramps versus ramps with curves.

Table 31. Observed deceleration rates for free-flow ramp vehicles (free-diverge conditions and passenger cars only).

Ramp location	GB initial speed (mi/h)	Average diverge speed (mi/h)	Average diverge distance (ft)	Decel length[1] (ft)	Difference from GB decel (ft)	GB speed at controlling feature (mi/h)	Average speed at GB dist[2] (mi/h)	Average speed at controlling feature (mi/h)	Average final speed (mi/h)	GB decel rate (ft/s^2)	Deceleration rate over *Green Book* distance[3] (ft/s^2)			Deceleration rate to controlling feature or final speed[4] (ft/s^2)		
											5	15	50	5	15	50
Controlling feature: Horizontal alignment																
I-70/US-40	55	51.7	–258	570	100	26	37.3	—[5]	34.8	–5.38	–4.94	–4.35	–3.29	–5.22*	–4.62*	–3.68*
I-435/US-24	61	58.7	–666	595	75	26	51.0	45.6	38.9	–5.70	–3.12	–2.67	–1.73	–3.50	–3.06	–2.30
I-635/Plano	55	53.9	–269	425	35	36	47.1	39.3	39.2	–4.77	–3.87	–3.11	–1.93	–4.33	–3.80	–2.85
I-635 EB/Freeport	58	55.7	–295	520	30	26	47.4	35.5	38.7	–5.56	–3.52	–2.87	–1.90	–4.28	–3.83	–2.98
I-635 WB/Freeport	58	57.0	–348	500	10	26	49.4	—[5]	38.8	–5.56	–3.47	–2.74	–1.67	–4.10*	–3.60*	–2.91*
Controlling feature: Crossroad terminal																
I-435/Quivira	61	62.2	–732	1640	980	0	58.7	29.8	45.2	–6.06	–1.83	–1.26	–0.72	–2.90	–2.82	–2.55
I-435/Gregory	61	60.4	–644	1520	782	0	55.9	29.0	29.9	–4.90	–1.77	–1.59	–0.92	–3.02	–2.74	–2.29
I-635/Marsh	55	50.6	–208	1035	465	0	46.8	—[5]	36.0	–5.71	–2.05	–1.57	–0.83	–2.90*	–2.40*	–1.67*
I-635/Metro	61	60.5	–466	960	69	0	48.2	—[5]	34.6	–4.49	–2.88	–2.36	–1.59	–3.49*	–3.18*	–2.53*

Note: GB = *Green Book*.
[1] Deceleration Length = SCL length + distance from painted nose to controlling feature.
[2] Observed speed at location where vehicle has traveled the minimum *Green Book* distance from the point of diverge.
[3] Deceleration rate over minimum *Green Book* distance beginning at point of diverge.
[4] Deceleration rate from point of diverge to controlling feature, or to final reading when speed was not recorded at controlling feature.
[5] Researchers were unable to collect speed data at the controlling feature; the final speeds of all profiles were upstream of the controlling feature.
* Value is based on final speed readings.

Table 32. Observed deceleration rates for platooned ramp vehicles (free-diverge conditions and passenger cars only).

Ramp location	GB initial speed (mi/h)	Average diverge speed (mi/h)	Average diverge distance (ft)	Decel length[1] (ft)	Difference from GB decel (ft)	GB speed at controlling feature (mi/h)	Average speed at GB dist[2] (mi/h)	Average speed at controlling feature (mi/h)	Average final speed (mi/h)	GB decel rate (ft/s^2)	Deceleration rate over *Green Book* distance[3] (ft/s^2)			Deceleration rate to controlling feature or final speed[4] (ft/s^2)		
											5	15	50	5	15	50
Controlling feature: Horizontal alignment																
I-70/US-40	55	48.8	–244	570	100	26	34.0	—[5]	31.2	–5.38	–4.25	–4.09	–3.71	–4.52*	–4.28*	–3.75*
I-435/US-24	61	55.3	–641	595	75	26	46.4	42.3	36.3	–5.70	–3.33	–2.94	–1.83	–3.58	–3.12	–2.14
I-635/Plano	55	52.5	–256	425	35	36	44.8	36.5	37.5	–5.56	–3.92	–3.38	–1.91	–4.38	–3.99	–2.93
I-635 EB/Freeport	58	53.6	–309	520	30	26	45.7	35.0	37.9	–5.56	–3.29	–2.90	–1.93	–3.41	–2.90	–1.85
I-635 WB/Freeport	58	55.3	–353	500	10	26	46.7	—[5]	36.0	–5.38	–3.91	–2.96	–1.97	–4.11*	–3.68*	–2.86*
Controlling feature: Crossroad terminal																
I-435/Quivira	61	59.6	–718	1,640	980	0	54.7	—[5]	42.3	–6.06	–2.15	–1.54	–0.88	–2.26*	–1.88*	–1.29*
I-435/Gregory	61	60.9	–658	1,520	782	0	54.8	27.2	27.8	–4.90	–1.92	–1.46	–1.08	–3.17	–2.84	–2.47
I-635/Marsh	55	48.3	–196	1,035	465	0	429.	—[5]	33.7	–5.71	–2.34	–1.79	–0.79	–2.97*	–2.26*	–1.34*
I-635/Metro	61	59.8	–507	960	69	0	45.6	—[5]	27.3	–4.49	–3.90	–2.70	–1.70	–4.33*	–3.88*	–2.86*

[1] Deceleration Length = SCL length + distance from painted nose to controlling feature.
[2] Observed speed at location where vehicle has traveled the minimum *Green Book* distance from the point of diverge.
[3] Deceleration rate over minimum *Green Book* distance beginning at point of diverge.
[4] Deceleration rate from point of diverge to controlling feature, or to final reading when speed was not recorded at controlling feature.
[5] Researchers were unable to collect speed data at the controlling feature; the final speeds of all profiles were upstream of the controlling feature.
* Value is based on final speed readings.

Table 33. Observed deceleration rates for free-flow and platooned ramp vehicles (free diverge and passenger cars only).

Ramp location	GB initial speed (mi/h)	Average diverge speed (mi/h)	Average diverge distance (ft)	Decel length[1] (ft)	Difference from GB decel (ft)	GB speed at controlling feature (mi/h)	Average speed at GB dist[2] (mi/h)	Average speed at controlling feature (mi/h)	Average final speed (mi/h)	GB decel rate (ft/s^2)	Deceleration rate over *Green Book* distance[3] (ft/s^2)			Deceleration rate to controlling feature or final speed[4] (ft/s^2)		
											5	15	50	5	15	50
Controlling feature: Horizontal alignment																
I-70/US-40	55	51.5	–257	570	100	26	36.6	—[5]	34.5	–5.38	–4.93	–4.33	–3.29	–5.21*	–4.60*	–3.69*
I-435/US-24	61	58.0	–661	595	75	26	50.0	44.9	38.3	–5.70	–3.24	–2.73	–1.73	–3.54	–3.09	–2.24
I-635/Plano	55	53.8	–267	425	35	36	46.9	38.8	39.0	–5.56	–3.87	–3.20	–1.92	–4.38	–3.85	–2.86
I-635 EB/Freeport	58	55.2	–298	520	30	26	47.0	35.4	38.5	–5.38	–3.46	–2.90	–1.91	–4.26	–3.81	–2.96
I-635 WB/Freeport	58	56.6	–349	500	10	26	48.8	—[5]	38.2	–5.38	–3.58	–2.84	–1.76	–4.11*	–3.62*	–2.88*
Controlling feature: Crossroad terminal																
I-435/Quivira	61	61.4	–728	1640	980	0	57.4	29.8	44.2	–6.06	–1.88	–1.42	–0.78	–2.90	–2.82	–2.55
I-435/Gregory	61	60.5	–646	1520	782	0	54.7	26.2	25.0	–4.90	–1.77	–1.57	–0.93	–3.07	–2.76	–2.34
I-635/Marsh	55	50.1	–205	1035	465	0	45.9	—[5]	35.4	–5.71	–2.13	–1.64	–0.82	–2.92*	–2.39*	–1.60*
I-635/Metro	61	60.5	–470	960	69	0	48.0	—[5]	33.9	–4.49	–2.89	–2.38	–1.60	–3.76*	–3.22*	–2.55*

[1] Deceleration Length = SCL length + distance from painted nose to controlling feature.
[2] Observed speed at location where vehicle has traveled the minimum *Green Book* distance from the point of diverge.
[3] Deceleration rate over minimum *Green Book* distance beginning at point of diverge.
[4] Deceleration rate from point of diverge to controlling feature, or to final reading when speed was not recorded at controlling feature.
[5] Researchers were unable to collect speed data at the controlling feature; the final speeds of all profiles were upstream of the controlling feature.
* Value is based on final speed readings.

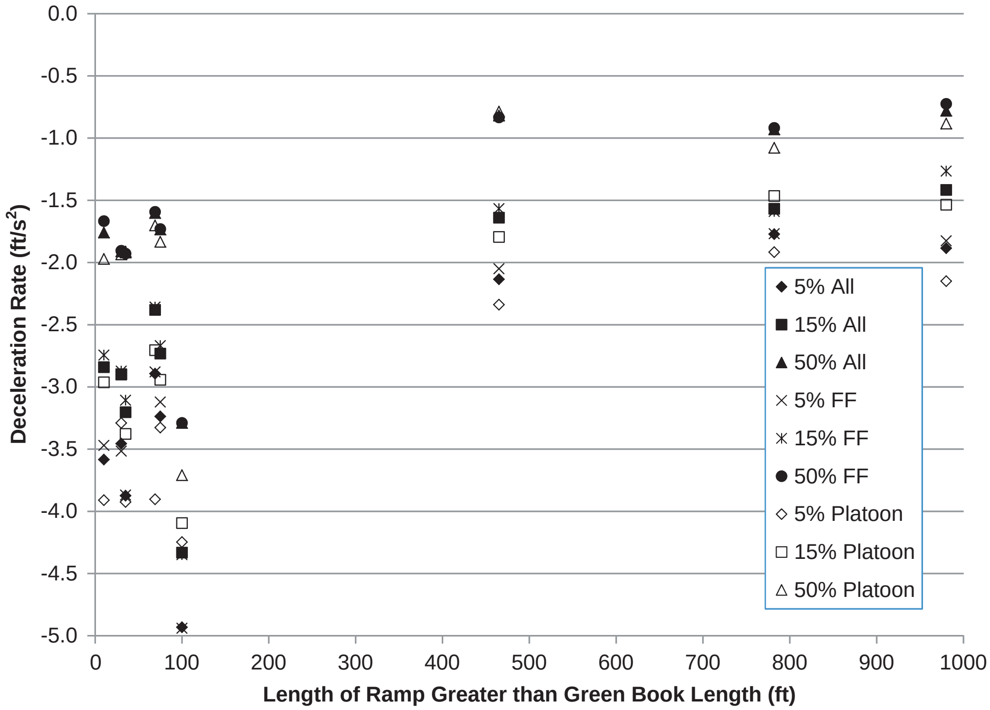


Figure 52. Comparison of observed passenger car deceleration rates by ramp length.

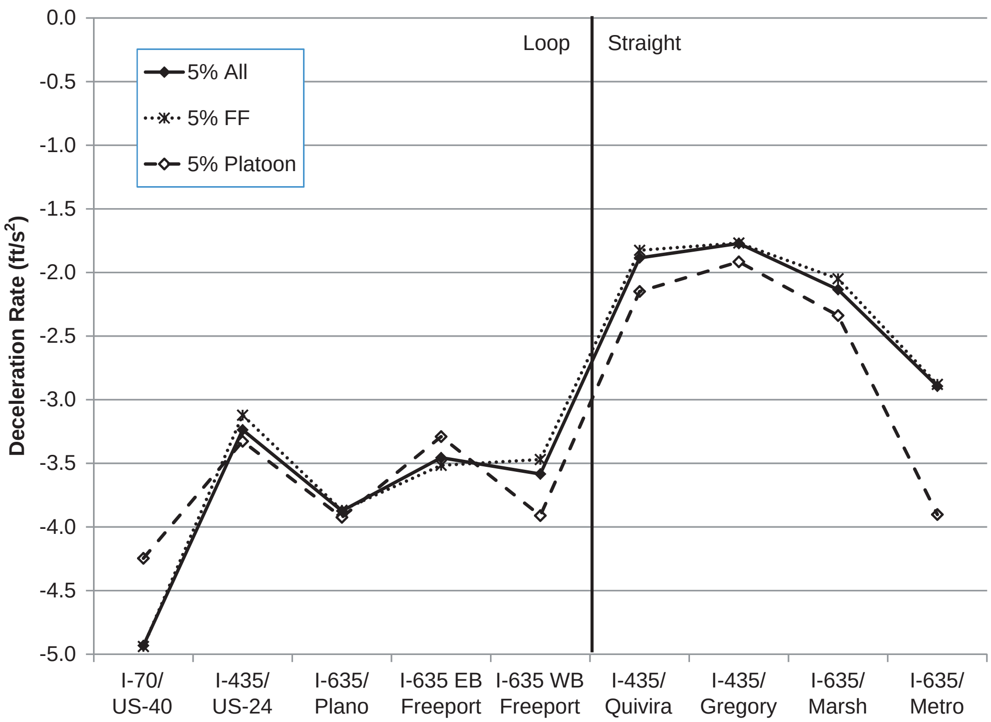


Figure 53. Comparison of observed passenger car 5th percentile deceleration rates on loop ramps and straight ramps.

Comparison of Deceleration Rates: Deceleration Rates from *Green Book* Criteria for Level Grades Assuming a Constant Deceleration. Table 34 provides a comparison of the deceleration rates derived from the *Green Book* recommended deceleration lane lengths for level grades to deceleration rates measured in the field for various vehicle groups under various conditions. The table format is based on *Green Book* Exhibit 10-73, and the values for design speed, speed reached, curve design speed, and initial speed are all taken directly from that table. Below the presentation of the *Green Book* deceleration rates assuming a constant deceleration are three categories of field-measured deceleration rates:

Condition 1—This category shows the average deceleration rate for free-flow passenger cars that exited the freeway when freeway speeds were greater than 50 mi/h. Platooned vehicles are excluded because it is assumed their speeds and deceleration choices are constrained by vehicle(s) in front of them.

Condition 2—This category shows the average deceleration data for free-flow heavy vehicles (rather than passenger cars) exiting freeway traffic that is moving faster than 50 mi/h. Platooned vehicles are excluded because it is assumed their speeds and deceleration choices are constrained by vehicle(s) in front of them.

Table 34. Comparison of *Green Book* and field deceleration rates on level grades.

Design speed (mi/h)	Speed reached (mi/h)	Deceleration length, L (ft) for design speed of exit curve (mi/h)								
		Stop	15	20	25	30	35	40	45	50
		For average running speed on exit curve (mi/h)								
		0	14	18	22	26	30	36	40	44
2004 *Green Book* deceleration rates (ft/s^2) (derived from recommended minimum ramp lengths)										
30	28	–3.59	–3.16	–2.91	–2.30	–	–	–	–	–
35	32	–3.93	–3.56	–3.59	–3.14	–2.50	–	–	–	–
40	36	–4.36	–4.01	–3.95	–3.72	–3.60	–2.75	–	–	–
45	40	–4.47	–4.31	–4.22	–4.07	–3.98	–3.42	–	–	–
50	44	–4.79	–4.62	–4.50	–4.40	–4.30	–3.91	–3.06	–2.07	–
55	48	–5.16	–4.98	–4.84	–4.77	–4.61	–4.31	–3.80	–3.22	–
60	52	–5.49	–5.39	–5.33	–5.19	–5.07	–4.79	–4.33	–3.96	–3.44
65	55	–5.71	–5.63	–5.59	–5.47	–5.38	–5.19	–4.77	–4.51	–4.18
70	58	–5.88	–5.78	–5.74	–5.63	–5.56	–5.41	–5.06	–4.86	–4.52
75	61	–6.06	–5.97	–5.89	–5.80	–5.70	–5.67	–5.32	–5.18	–4.92
Condition 1: Measured average deceleration rates (ft/s^2) for passenger cars (free flow and free diverge)										
30	28									
35	32									
40	36									
45	40						–1.13	–0.85		
50	44						–1.67	–1.37	–0.96	
55	48						–1.93	–1.64	–1.28	
60	52				–3.19	–3.15	–2.47	–2.41	–2.10	–1.71
65	55			–3.67	–3.59	–3.30	–2.61	–2.81	–2.44	–2.01
70	58						–3.91	–3.42	–3.04	–2.58
75	61							–2.82	–3.07	–2.57
Condition 2: Measured average deceleration rates (ft/s^2) for trucks (free flow and free diverge)										
30	28									
35	32									
40	36									
45	40									
50	44				–1.65	–1.89	–1.90	–1.83	–1.73	
55	48						–2.35	–1.90	–1.65	–1.75
60	52				–3.67	–3.05	–2.80	–2.57	–2.20	–2.03
65	55				–3.77	–3.83	–3.19	–2.59	–2.27	–1.81
70	58			–2.16	–2.09	–2.65	–3.12	–2.99	–2.72	–2.38
75	61				–2.32	–2.26	–2.13	–1.90	–1.69	–1.55
Condition 3: measured average deceleration rates (ft/s^2) for passenger cars (free flow/platoon and constrained diverge)										
30	28									
35	32									
40	36									
45	40							–1.17		
50	44						–1.30	–0.88	–0.52	
55	48						–3.00	–1.63	–1.37	
60	52							–2.12	–2.06	–1.84
65	55							–4.10	–3.30	–2.94
70	58								–4.47	–4.19
75	61									

Condition 3—This category considers all passenger cars (i.e., both free-flow and platooned) that are exiting the freeway under constrained-diverge conditions, when the freeway speed is between 40 and 50 mi/h. Platooned vehicles are included because constrained-diverge conditions typically occur during peak traffic flow periods where a majority of vehicles on the ramp are in platoons.

Table 34 compiles field-measured deceleration data for comparison with *Green Book* assumed deceleration rates. For each condition, the values in the table were populated by developing a database of the speed profile of each vehicle as measured by the laser gun in the field. This provided each vehicle's speed at known distances along the ramp. For each speed profile, a deceleration rate was calculated for the sections of the profile that correspond to a given value in the table (identified by an initial speed and a speed reached). Since the database provides the location along the ramp where each speed is first reached, the length between initial speed and speed reached is known and can be used to calculate decelerations between the two points. For example, if a given speed profile had a diverge speed of 44 mi/h and a final speed of 22 mi/h, deceleration rates could be calculated for five values in the table (initial speed of 44 mi/h to speed reached of 40, 36, 30, 26, and 22 mi/h). This process was repeated for each speed profile, so that each value in the table included several observations. Only deceleration rates of vehicles on ramps with flat grades of two percent or less were used in the development of Table 34. Only values based upon 5 or more observations are presented in Table 34.

Table 34 shows that in the ideal situation of Condition 1, the observed passenger cars decelerated at rates less than those assumed in the *Green Book.* The Condition 1 deceleration values are lower than the corresponding *Green Book* deceleration values where measurements were available. The magnitude of *Green Book* deceleration rates generally decreases moving to the right and upward through the table; this is generally the trend in Condition 1 as well.

Condition 2 deceleration rates for trucks follow the same trend as rates for cars and are also lower than the *Green Book* values. The magnitudes of Condition 2 deceleration rates are frequently greater than those shown in Condition 1 at lower diverge speeds (55 mi/h and below), but not necessarily so at high diverge speeds.

Condition 3 considers passenger cars in constrained-diverge conditions. The magnitudes of deceleration rates in constrained-diverge conditions are almost always higher than the rates in free-diverge conditions, but even in constrained-diverge conditions, vehicles decelerated at rates less than those assumed in the *Green Book.*

Comparison of Deceleration Rates: Deceleration Rates from *Green Book* Criteria for Level Grades Assuming a Two-Step Deceleration Process. This section compares deceleration rates measured in the field to deceleration rates based on *Green Book* criteria for level grades, assuming a two-step deceleration process. The analysis focuses on comparing deceleration rates measured in the field to the assumed deceleration rates during primary braking. Based upon results of the behavioral study (see Section 6), the first 1.0 s of data after the beginning of the diverge maneuver was excluded from this analysis. This was done to better capture the deceleration activity that takes place during braking. Excluding the first 1.0 s of data helps ensure that the coasting that began in the freeway lane was complete. This analysis approach deviates slightly from the basic two-step process for establishing design criteria for deceleration lanes assumed by AASHTO. AASHTO assumes 3.0 s for the deceleration time without braking (i.e., coasting), but a coasting time of 1.0 s after initiating the diverge maneuver is more consistent with the data from the behavioral study. Only deceleration rates of vehicles on ramps with flat grades of two percent or less were used in this analysis.

In general, drivers in the observational study decelerated in a manner equivalent to a constant rate of 2.73 ft/s^2. This rate is lower than those in *Green Book* Exhibit 10-73 for all but a few combinations of initial speed and final speed, where rates are typically between 5.00 and 9.00 ft/s^2.

The following tables provide summaries of the characteristics associated with the deceleration rates observed during primary braking. Table 35 shows the difference in rates by metropolitan area, and indicates that there was not a substantial difference between results from study sites in Dallas and Kansas City.

An analysis of deceleration rates by the location at which the diverge maneuver began showed that deceleration rates increased substantially as the diverge point moved closer to the painted nose. This is intuitive, because greater deceleration is needed to accomplish the same speed change in a reduced distance.

Table 36 shows deceleration rates as divided by presence of leading vehicles. Free-flow vehicles had a somewhat higher deceleration rate than platooned vehicles, indicative of the higher speeds at which free-flow vehicles travel and the need to reduce speed by a greater amount.

Table 35. Average deceleration rates by location.

Location	Number of vehicles	Average deceleration rate (ft/s^2)
Dallas	1,275	–2.68
I-635/Marsh	313	–1.63
I-635/Plano	302	–2.89
I-635 EB/Freeport	232	–2.93
I-635 WB/Freeport	428	–3.17
Kansas City	989	–2.79
I-435/US-24	363	–2.91
I-435/Quivira	327	–1.41
I-70/US-40	299	–4.15
Total	2,264	–2.73

Table 36. Average deceleration rates by presence of lead vehicle.

Lead vehicle status	Number of vehicles	Average deceleration rate (ft/s^2)
Free-flow	1,803	–2.83
Platoon	461	–2.34
Total	2,264	–2.73

Table 37 shows the distribution of deceleration rate by vehicle type. There was not a great deal of difference between rates for passenger cars and rates for heavy trucks overall (–2.72 and –2.82 ft/s^2, respectively), though the rate for trucks was slightly greater. Comparison of rates for a given initial speed also suggests that trucks decelerated at greater rates than cars. The reason that the overall average rates for cars and for trucks are so similar is likely a result of there being more trucks with lower initial speeds. It should also be noted that, at the lowest initial speeds, passenger cars actually accelerated (shown as positive values in Table 37), indicating that drivers entered the ramp at speeds lower than the operating speed of the controlling feature, and they felt comfortable enough to increase their speed and still negotiate the ramp.

Tables 38 through 40 show deceleration rates by ramp characteristics. Table 38 shows a noticeable difference in rates on loop ramps and straight ramps, with loop ramps having a greater deceleration by 0.6 ft/s^2. Parallel SCLs had

Table 37. Average deceleration rates by vehicle type.

Initial speed (mi/h)	Passenger cars		Heavy vehicles	
	Average deceleration rate (ft/s^2)	Count	Average deceleration rate (ft/s^2)	Count
24	2.77	1		
29	0.28	1		
30	0.59	1		
31	0.00	1		
32	0.21	2		
34	–0.16	2		
35	–0.36	1		
36	–0.43	8		
37	–0.39	5	–0.56	1
38	–0.77	4		
39	–0.67	11		
40	–0.94	10		
41	–0.98	3		
42	–0.87	7	–1.75	1
43	–1.12	18	–1.77	2
44	–1.87	19	–2.98	4
45	–1.67	22	–2.25	5
46	–1.92	36	–1.70	1
47	–2.10	54	–1.91	6
48	–2.44	59	–2.82	6
49	–2.19	81	–2.91	11
50	–2.55	95	–2.15	4
51	–2.56	94	–2.80	8
52	–2.69	126	–2.83	13
53	–2.71	147	–2.94	9
54	–3.02	145	–3.53	7
55	–2.92	154	–2.78	8
56	–3.01	138	–2.96	2
57	–2.98	138	–2.87	5
58	–3.18	157	–3.25	8
59	–3.09	100	–3.49	3
60	–3.19	90	–3.91	4
61	–2.98	100	–2.06	2
62	–3.08	86	–3.76	2
63	–2.96	60		
64	–2.61	52		
65	–2.92	27		
66	–2.51	32		
67	–2.80	17		
68	–2.47	12		
69	–2.22	11		
70	–2.40	11		
71	–2.70	7		
72	–2.79	4		
73	–2.85	1		
74	–1.67	2		

Table 38. Average deceleration rates by ramp type.

Ramp type	Number of vehicles	Average deceleration rate (ft/s^2)
Loop	1,325	–2.99
Straight	939	–2.35
Total	2,264	–2.73

Table 39. Average deceleration rates by diverge type.

Diverge type	Number of vehicles	Average deceleration rate (ft/s^2)
Parallel	662	–3.47
Taper	1,602	–2.42
Total	2,264	–2.73

Table 40. Average deceleration rates by ramp and diverge type.

Ramp/diverge type	Number of vehicles	Average deceleration rate (ft/s^2)
Loop		
Parallel	363	–2.91
Taper	962	–3.03
Straight		
Parallel	299	–4.15
Taper	640	–1.52
Total	2,264	–2.73

a substantially higher rate than tapered SCLs (1.25 ft/s^2), as shown in Table 39. This finding was not initially as expected, but could be explained by drivers on parallel SCLs making a more focused deceleration action within the SCL that they could not perform with a tapered diverge; on parallel SCL drivers possibly completed more of their deceleration in the form of coasting on the freeway mainlane.

Table 40 shows results for combinations of ramp type and diverge type. The disparity between parallel and tapered diverge types was most apparent on straight ramps, where the deceleration rate for the former was more than twice the latter. A similar explanation is plausible, in that drivers have more latitude to decelerate within the SCL on a parallel SCL than on a tapered SCL. In addition, deceleration can be more gradual on a straight ramp than on a loop ramp because the controlling feature is typically further downstream than on a loop ramp. The average deceleration rates on both diverge types for loop ramps were higher than the overall average and had much more similar values than the corresponding rates for straight ramps.

Table 41 provides the comparison of deceleration rates by diverge condition. The results show drivers decelerate at greater rates within the deceleration lane during free-diverge conditions compared to during constrained-diverge conditions. This is probably most reflective of the higher initial diverge speeds observed during free-diverge conditions.

Figure 54 shows the observed deceleration rates as compared to the initial speed at the beginning of the diverge maneuver. For the lowest initial speeds (i.e., speeds 30 mi/h and below), there was actually a small acceleration, reflecting that the driver was already at or below a speed deemed appropriate or comfortable for traversing the ramp when the diverge maneuver was initiated. Above 30 mi/h, the deceleration rate generally increased as the speed increased, though not on a linear basis. A simple trendline for the data shows a strong relationship with the natural log of initial speed. Initial speeds below 35 mi/h or above 70 mi/h were very rare, so the shape of the trendline at each end may be less reliable than for speeds where more data were available, but the general trend has a high level of confidence.

Table 41. Average deceleration rates by diverge conditions.

Diverge conditions	Number of vehicles	Average deceleration rate (ft/s^2)
Constrained	63	–1.85
Free	2,201	–2.75
Total	2,264	–2.73

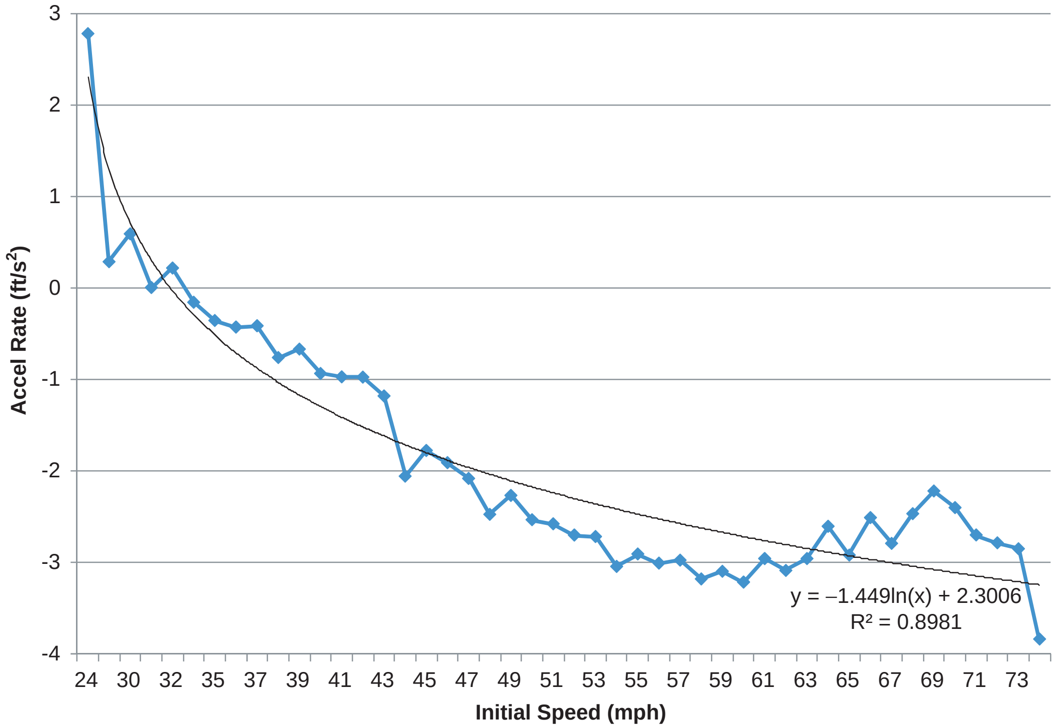


Figure 54. Average deceleration rate by initial speed.

Comparison of Deceleration Rates on Grades of 3 Percent or Greater. Two of the exit ramp locations had grades that could not be considered in the level-grade category with the other nine ramps: I-435 at Gregory (3 percent downgrade) and I-635 at Metropolitan (6 percent downgrade). Because of the steeper grade, these two ramps were withheld from the previous analyses.

Table 42 shows the average deceleration rates by location for the steeper ramps. It illustrates that at the I-435/Gregory exit ramp, the average deceleration rate was 0.32 ft/s² less than the average deceleration rate at the I-635/Metropolitan exit ramp. The exit ramp at Metropolitan is steeper and shorter than the ramp at Gregory; the combination of influences of length and grade encouraged drivers to decelerate more sharply at Metropolitan to accomplish the desired speed change. Even with the increased grade, however, both ramps have average deceleration rates within the range of rates found at the nine exit ramps on level grade. The deceleration rate for the Metropolitan ramp is 0.02 ft/s² below the average for the nine level ramps, and the rate for the Gregory ramp is greater than two of the level ramps.

Table 42. Average deceleration rates by location for ramps on steep grades.

Location	Number of vehicles	Average deceleration rate (ft/s²)
Kansas City		
I-435/Gregory	133	–2.39
I-635/Metropolitan	237	–2.71
Total	370	–2.59

Summary of Findings from Examination of Deceleration Rates. The most noteworthy findings from the examination of deceleration rates are as follows:

- Decelerations of diverging vehicles along the SCL and ramp are not constant.
- In every combination of diverge speed and final speed, vehicles are decelerating at rates lower than those assumed by the *Green Book*. This is likely due in part to the study site ramps being longer than the *Green Book* minimum lengths and to vehicles decelerating in the freeway lane upstream of the exit taper.
- Ramps more than 400 ft longer than the *Green Book* minimum have lower deceleration rates than ramps within 100 ft of the *Green Book* length.
- Observed deceleration rates on loop ramps were greater than those on straight ramps.
- There was little practical difference in deceleration rates by metropolitan area in the field study.
- Deceleration rates increased substantially as the diverge point moved closer to the painted nose.
- Free-flow vehicles had a somewhat higher deceleration rate than platooned vehicles.

- There was very little difference between deceleration rates for passenger cars and heavy trucks overall; however, comparison of rates for a given initial speed suggest that trucks decelerated at a greater rate than cars.
- There was a noticeable difference between deceleration rates on loop ramps and straight ramps, with loop ramps having greater deceleration.
- Parallel SCLs had a substantially higher deceleration rate than tapered SCLs. The disparity between parallel and tapered diverge types was most apparent on straight ramps, where the deceleration rate for the former was more than twice the latter.
- For initial speeds above 30 mi/h, the deceleration rate generally increased as the speed increased, having a strong relationship with the natural log of initial speed.

5.4.2.4 Critical or Unusual Maneuvers

While reviewing the video recordings, no critical or unusual maneuvers were observed in the vicinity of the exit ramps.

SECTION 6

Behavioral Study of Freeway Mainline Ramp Terminals

This portion of the research focused on investigating driver behaviors while performing merge and diverge maneuvers onto and off of freeways. There are a number of ways to explore behavioral tendencies and patterns, both directly and indirectly. This study focused on several indirect behavioral measures, specifically looking at speed, acceleration/deceleration, use of throttle and brake, glance activity, and presence of a leading vehicle during the merge/diverge maneuver.

The behavioral study was designed to collect detailed information on a limited number of drivers, in contrast to more generalized information gathered on a large number of vehicles in the observational study. By observing drivers in close detail, it is possible to identify behavioral patterns and influences that determine how drivers operate their vehicles on freeway ramps, thus providing further information to determine whether the assumptions and data used to support existing design guidelines are appropriate for current conditions.

This section describes the processes used to collect and reduce the data for the behavioral study, and it describes the various types of data collected and the analysis results.

6.1 Data Collection

The behavioral study took place in the Dallas-Ft. Worth Metroplex area of Texas, using a data collection protocol approved by institutional review boards at Texas A&M University and MRIGlobal. Flyers were distributed to various agencies/organizations to recruit 12 subjects to participate in the study. Subjects were paid for their participation.

The Texas A&M Transportation Institute (TTI) has developed an instrumented vehicle to facilitate robust data collection in both test-track and public road environments. A 2006 Toyota Highlander is equipped with multiple integrated systems to record various data relating driver behavior, traffic conditions, and vehicle performance. All on-board equipment is managed by a data acquisition system on a central computer. This computer is responsible for integrating the many streams of data that can be collected through the vehicle. The computer stores basic driving data such as brake and throttle position and steering wheel angle, gathered through potentiometers located on the pedals and steering column. A global positioning system (GPS) provides real-time data on the exact position of the vehicle, enabling the calculation of location, distance traveled, and velocity.

A series of video cameras provided information on adjacent traffic conditions and in-vehicle driver behaviors. One in-vehicle camera was positioned to monitor drivers' head turns and glance direction. Another camera was positioned to monitor foot activity on the pedals. While the main source of pedal activity came from the potentiometers, the video record of the feet provided an opportunity to check the source of any anomalies in the pedal potentiometer data.

Researchers recruited 12 subjects to drive the instrumented vehicle through the predetermined course. The 12 subject drivers were recruited from the general population in the Dallas/Ft. Worth area; they were not affiliated with TTI, nor had they previously driven the instrumented vehicle. Subjects met at a designated hotel in the area for a briefing session to complete an informed consent form and a demographics questionnaire and receive pre-driving instructions. The script for the verbal instructions to each subject is provided in Appendix C (available on the TRB website at http://www.trb.org/Main/Blurbs/167516.aspx). Three subjects drove for approximately 90 minutes each, during each day of the experiment; two subjects drove in the morning and one subject drove in the afternoon. A typical day's schedule had subjects beginning their tasks at 8:00 AM, 10:30 AM, and 1:30 PM. This allowed the subjects to avoid a majority of the daily commuter traffic.

The subjects were given instructions during the briefing, prior to beginning the driving course; they were told before they started driving and were reminded after they started driving that they were in complete control of the vehicle at all times and were responsible for its safe operation. Two researchers were inside the vehicle during the study to offer additional instructions as needed: one researcher sat in the front passenger seat, giving directions and acting as a safety observer, while the other researcher served as the data recorder, sitting in the rear passenger seat and operating the computer. Subjects were instructed to drive normally, obey the speed limit, and follow the driving directions offered during the course, but they were not told that freeway merging and diverging were the focus of the study. At the conclusion of the driving course, subjects were paid $50 for their time.

The subjects drove from the hotel parking lot to I-635 via neighborhood roads. During this time the subjects acclimated themselves to the instrumented vehicle. The predefined route (see Figure 55) took the subjects on I-635 as far west as Royal Lane and on I-35E as far south as Medical District Drive. On the freeway, the subjects were given directions on when to exit as they passed certain pre-selected state or federal guide signs. On the surface streets, subjects were given advance driving directions and repeated driving directions, if needed. Data were collected at nine entrance ramps and nine exit ramps; a summary of site characteristics for these ramps is provided in Table 43. All subjects drove all 18 ramps except for Subject 7, who was instructed to bypass Ramps 16 and 17 due to an incident blocking both ramps.

As the subjects drove through each ramp, two notes were input into the computer; the first note indicated which ramp was approaching and the second note indicated which ramp was just completed. These notes helped to identify the boundaries of each ramp within the data file. In addition, four keystroke data points were recorded on each ramp. For entrance ramps these points were:

1. Beginning of ramp or end of controlling feature,
2. Point at which the ramp edgeline changes from yellow to white,
3. Merge location, and
4. End of taper.

For exit ramps they were:

1. Beginning of taper,
2. Diverge location,

Figure 55. Behavioral study driving route (Image Credit: Google Earth™ Mapping Service).

Table 43. Characteristics of ramps used in behavioral study driving route.

Ramp			Ramp type	Merge/ diverge type	Length (ft)			Design speed (mi/h)	
No.	Location	Type			Painted nose to ctrl feat	SCL	Taper	Freeway	Ramp
1	I-635/Josey Ln	WB Entrance	Straight	Parallel	680	710	0*	65	0***
2	I-635/Freeport Pkwy	WB Exit	Loop	Taper	400	100	180	70	25
3	I-635/Royal Ln	EB Entrance	Straight	Taper	1,660	345	490	70	0***
4	I-635/Freeport Pkwy	EB Exit	Loop	Taper	410	110	160	70	25
5	SH 114/Freeport Pkwy	EB Entrance	Straight	Parallel	1,850	675	280	70	0***
6	SH 114/Belt Line Rd	EB Exit	Straight	Taper	1,010	310	585	70	0***
7	SH 114/Belt Line Rd	EB Entrance	Straight	Taper	810	650	385	70	0***
8	SH 114/Rochelle Blvd	EB Exit	Straight	Taper	580	70	160	70	0***
9	SH 114/Rochelle Blvd	EB Entrance	Straight	Parallel	1,010	1,120	0*	70	0***
10	SH 183/Regal Row	EB Exit	Straight	Parallel	665	355	0**	70	0***
11	I-35E/Mockingbird Ln	SB Entrance	Straight	Taper	660	180	250	65	0***
12	I-35E/Medical District Dr	SB Exit	Straight	Taper	235	50	100	65	45
13	I-35E/Inwood Rd	NB Entrance	Straight	Parallel	230	200	450	65	45
14	I-635/Marsh Ln	EB Exit	Straight	Taper	995	40	135	65	35
15	I-635/Midway Rd	WB Entrance	Straight	Taper	700	145	530	65	0***
16	I-635/Webb Chapel Rd	WB Exit	Straight	Taper	650	80	185	65	0***
17	I-635/Marsh Ln	EB Entrance	Straight	Taper	1,115	300	630	65	0***
18	I-635/Midway Rd	EB Exit	Straight	Taper	980	35	170	65	0***

* The path of these entrance ramps carried through to the adjacent downstream exit ramp; therefore, there was no lane departure taper on these ramps.
** This exit ramp was an "exit-only" freeway lane that was carried through to the ramp proper; therefore, there was no lane addition taper on this ramp.
*** For the purposes of comparison with *Green Book* Exhibits 10-70 and 10-73, these ramps have an estimated design speed of 0 mi/h, corresponding to a stop condition.

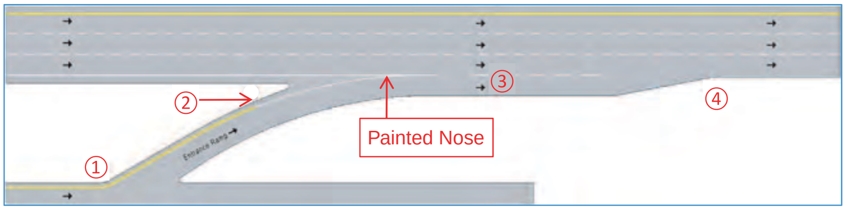
3. Point at which the ramp edgeline changes from white to yellow, and
4. End of ramp or beginning of controlling feature.

Figure 56 illustrates generalized entrance and exit ramps with the four key points highlighted. While the subjects drove the course, sensors on the vehicle recorded cumulative elapsed time, distance traveled, displacement of the throttle and brake pedals, X-Y-Z coordinates from GPS, velocity, and direction traveled. In addition, video cameras recorded the driver's facial and head movements, the driver's foot movements and pedal activity, and the driver's view of the environment in front of the vehicle.

6.2 Data Reduction

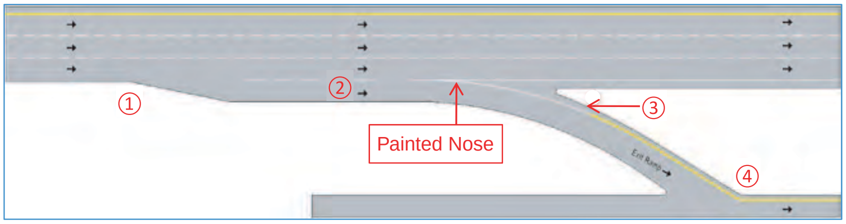
After post processing the spreadsheet file for each subject, researchers reviewed the data in the spreadsheets and the corresponding video recordings to quantify glance activity and pedal activity, determine the presence of lead vehicles, and develop speed-distance plots for each of the 18 ramps on the driving course.

Technicians reviewed the video recordings of the subjects' behavior while merging onto the nine entrance ramps to observe glances that they made in the center or side mirror or over their shoulder. For each merge maneuver, a technician watched the movement of the driver's head, shoulders, and torso, to classify four different categories of glances, as shown in Table 44.



- 1 – Beginning of ramp or end of controlling feature
- 2 – Point at which ramp edgeline changes from yellow to white
- 3 – Merge point, defined as the location where the vehicle's left tires crossed the lane line into the freeway mainlane
- 4 – End of taper

(a) Entrance Ramp



- 1 – Beginning of taper
- 2 – Diverge point, defined as the location where the vehicle's right tires crossed the lane line into the SCL
- 3 – Point at which ramp edgeline changes from white to yellow
- 4 – End of ramp or beginning of controlling feature

(b) Exit Ramp

Figure 56. Key points along mainline freeway-ramp terminals.

Technicians also reviewed the video to classify three types of pedal activities related to foot movements, using the terms in Table 45.

Using the locations of the key points on each ramp, researchers identified when each driver passed the beginning of the ramp, passed the change in edgeline, made the lane change to initiate the merge/diverge maneuver, and passed the end of the ramp. Using those four key points on each ramp, researchers divided each ramp into four stages and then plotted the data from 10 s prior to Point 1 to 10 s after Point 4. An example of such a plot for an entrance ramp is shown in Figure 57. The zero distance along the x-axis (i.e., cumulative distance in feet) corresponds to the painted nose of each ramp.

Figure 57 shows the speed of the instrumented vehicle as it increases from 25 mi/h at a point 1,250 ft upstream of the painted nose to 60 mi/h at a point 1,250 ft downstream of the

Table 44. Head movement codes and definitions.

Code	Definition
A	Relaxed body, head turn (i.e., subject's back against the driver's seat, only the subject's head is turned; typical glance into a mirror)
B	Relaxed body, shoulder turn (i.e., subject's back against the driver's seat, the subject's head and shoulders are turned; may be a glance into a mirror or through the window)
C	Non-relaxed body, head turn (i.e., subject's back turned away from the driver's seat, only the subject's head is turned; typical glance through the window)
D	Non-relaxed body, shoulder turn (i.e., subject's back against the driver's seat, the subject's head and shoulders are turned; typical glance through the window)

Table 45. Foot movement codes and definitions.

Code	Definition
Accel	Gas pedal (i.e., throttle) is depressed
Decel	Brake pedal is depressed
Coast	Foot is positioned between pedals for greater than 0.3 s

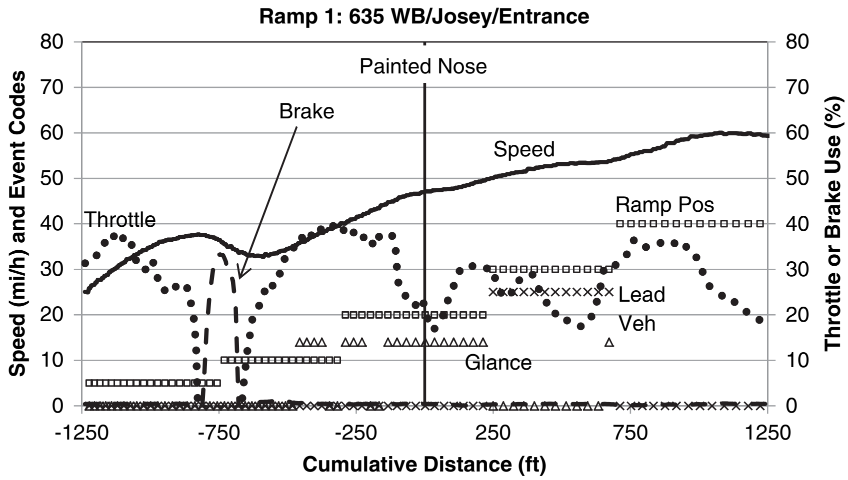


Figure 57. Speed-distance plot for freeway entrance ramp.

painted nose. It also shows the following driver and vehicle characteristics corresponding to those times and locations:

- The stage of the ramp being traversed, expressed as a numerical code:
  - 5 = within 10 s before passing Point 1,
  - 10 = Stage 1, between Points 1 and 2,
  - 20 = Stage 2, between Points 2 and 3,
  - 30 = Stage 3, between Points 3 and 4, and
  - 40 = Stage 4, within 10 s after passing Point 4;
- Throttle and brake pedal use;
- The presence of a lead vehicle ahead of the instrumented vehicle near enough to affect the subject's desired speed;
- The occurrence of any glances by the subject into a mirror or through the side window.

Figure 57 shows that this subject entered the ramp approximately 750 ft upstream of the painted nose, passed the change in edgeline about 500 ft later, and began the lane change to merge about 250 ft downstream of the painted nose, with the end of the taper approximately 700 ft downstream of the painted nose. Within that distance, the subject depressed the brake pedal once, with as much as 32 percent activation, in proximity to the start of the ramp. During the remainder of the merge activity, the subject activated the throttle, between 18 percent and 40 percent for most of the ramp distance. The driver completed four glances, almost exclusively prior to merging onto the freeway. There was a lead vehicle ahead of the instrumented vehicle for a distance of approximately 400 ft, roughly corresponding to Stage 3; this indicates that the subject accepted a gap in traffic and merged onto the freeway behind another vehicle already in the freeway mainlane. Researchers generated a plot for each subject and each ramp; the 216 plots and their corresponding data led to the subsequent analyses discussed in the remainder of this section.

6.3 Entrance Ramp Analyses

6.3.1 Glance Activity

Researchers observed a total of 308 glances by the 12 subjects on the nine entrance ramps. For glances of all types shown in Table 44, researchers measured the time duration of each glance and calculated the corresponding distance traveled and change in speed. Of the 308 glances, 145 of them began prior to the painted nose, as shown in Figure 58. Tables 46 and 47 present summaries of glance data by ramp and by subject, respectively.

The glance summary data indicate that the average glance by a merging driver is typically about 2.5 to 3.0 s long, though some glances are very small and others are very lengthy, as drivers occasionally look into their mirrors for extended periods of time with imperceptible breaks. The first glances on a given ramp were frequently longer than subsequent glances, suggesting that later glances were commonly used to confirm

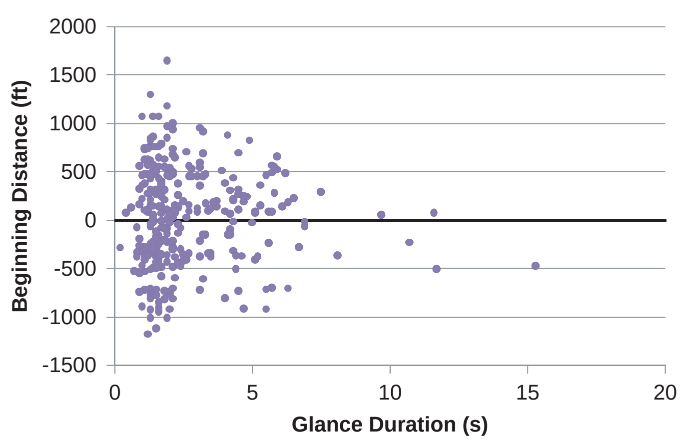


Figure 58. Glances by subject drivers in relative position to the painted nose.

Table 46. Summary data for glance behavior by entrance ramp.

Ramp no.	Average no. glances/ subject	Min glance duration (s)	Max glance duration (s)	Average glance duration (s)	Average distance (ft)	Average speed increase per glance for all subjects (mi/h)
1	2.7	0.6	15.3	3.0	177	2.8
3	4.3	0.4	6.9	2.3	149	2.7
5	2.7	0.9	11.6	2.8	212	2.6
7	2.3	0.8	7.5	2.9	196	3.9
9	3.5	0.2	5.9	2.3	160	3.2
11	3.2	0.8	9.7	3.0	100	2.3
13	2.4	0.9	10.7	2.5	176	2.2
15	3.0	0.8	6.3	2.2	127	2.2
17	1.8	0.7	5.8	2.5	119	0.4
All	2.9	0.2	15.3	2.6	155	2.5

the appropriateness of a gap identified previously. Based on the data summarized in the tables, a merging driver traveled 100 to 200 ft and increased speed by 2 to 3 mi/h during an average glance.

6.3.2 Use of Speed-Change Lane

This analysis investigated the proportion of the SCL each subject used at each ramp. For example, if a subject's merge point was 350 ft downstream of the painted nose, and the SCL length was 500 ft, the amount of SCL used was (350/500) = 70 percent.

Figure 59 illustrates the SCL usage data at entrance ramps on the driving course. Each marker represents the portion of SCL used by one subject on an entrance ramp; the solid line connects the average SCL percent values for each ramp. The data table attached to the chart shows the summary statistics for each ramp.

The data in Figure 59 suggest that on ramps with SCLs less than 350 ft in length (Ramps 3, 11, 13, 15, and 17), subject drivers used all of the provided SCL length and completed their merge in the taper. For the four ramps with SCLs longer than 600 ft (Ramps 1, 5, 7, and 9), most subjects completed their merge within the SCL.

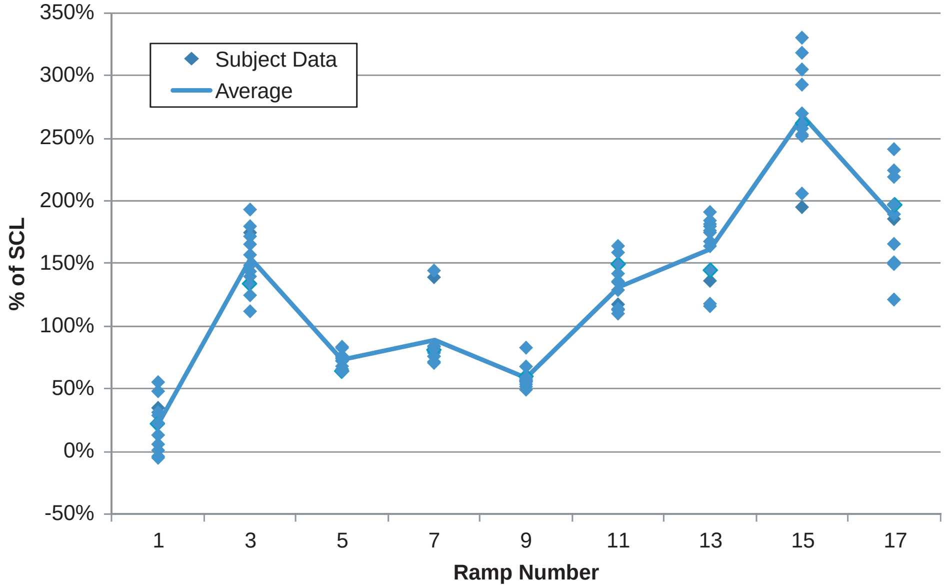
Ramp 1 has a noticeably lower average than the other eight ramps. This suggests that drivers treated this ramp differently. Given that it was the first ramp encountered on the driving course for each subject, earlier merging on Ramp 1 may have been a function of the subjects adjusting to driving on the freeway in the instrumented vehicle. The negative sign on the minimum value for Ramp 1 indicates that this driver merged onto the freeway early, upstream of the painted nose.

6.3.3 Acceleration

Researchers used the speed and time data from each subject to develop speed profiles for each subject on each ramp, which could then be used to evaluate acceleration patterns. Figure 60 shows the recorded acceleration rates for each subject on each ramp, represented as constant acceleration from the beginning of the entrance ramp (Point 1 in Figure 56a) to the merge point (Point 3 in Figure 56a). Each marker rep-

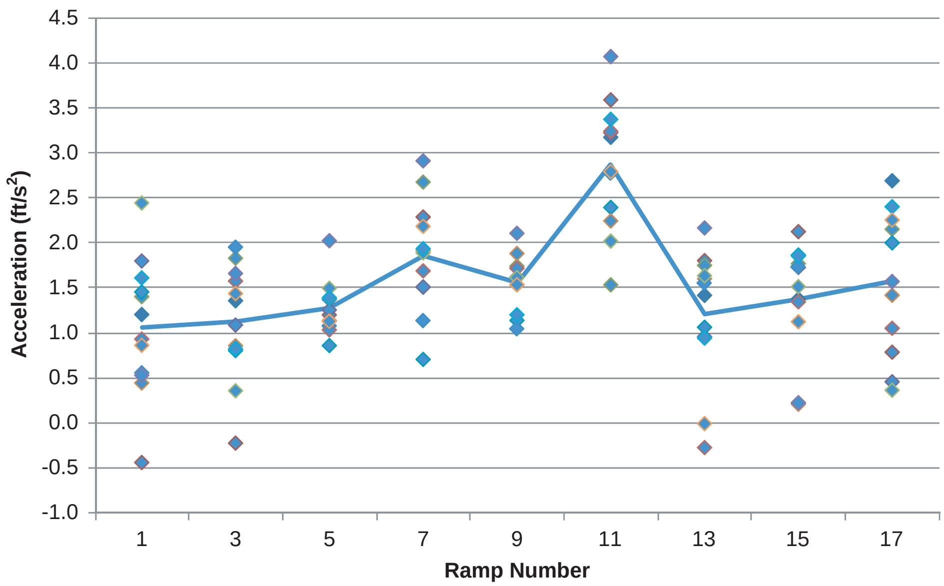
Table 47. Summary data for glance behavior by subject.

Subject no .	Average no. glances/ ramp	Min glance duration (s)	Max glance duration (s)	Average glance duration (s)	Average distance (ft)	Average speed increase per glance for all ramps (mi/h)
1	0.4	1.5	2.6	2.2	140	2.8
2	2.8	1.1	8.1	2.9	178	2.9
3	3.7	1.1	11.7	3.5	229	4.0
4	2.9	0.6	5.8	2.5	154	2.6
5	2.6	1.3	6.7	3.0	176	2.5
6	5.0	0.8	11.6	2.7	140	2.2
7	2.6	0.2	15.3	4.0	234	4.0
8	4.2	0.9	5.5	2.1	115	1.7
9	3.3	0.9	6.3	2.2	140	2.2
10	2.6	0.8	3.0	1.5	95	1.6
11	2.2	0.4	3.3	1.6	106	1.5
12	2.0	0.9	5.8	2.7	164	3.2
All	2.9	0.2	15.3	2.6	155	2.5



SCL length (ft)	710	345	675	650	1120	180	200	145	300
Taper/parallel	Taper	Taper	Parallel	Taper	Parallel	Taper	Parallel	Taper	Taper
Straight/curve	Straight	Straight	Straight	Straight	Straight	Straight	Straight	Straight	Straight
Min SCL %	–5.3%	112.0%	64.1%	70.4%	49.5%	109.8%	115.7%	195.1%	121.3%
Max SCL %	55.0%	192.8%	83.5%	144.1%	82.5%	163.7%	191.0%	330.4%	240.9%
Avg SCL %	21.0%	153.6%	72.7%	88.7%	58.7%	131.7%	161.1%	267.3%	186.5%
Std. deviation	19.7%	24.1%	6.5%	25.1%	8.7%	18.9%	26.0%	42.7%	34.6%

Figure 59. SCL use at entrance ramps.



Avg Observed Accel (ft/s^2)	1.07	1.13	1.28	1.86	1.56	2.87	1.22	1.38	1.57
Avg Accel Distance (ft)	914	1,441	1,340	908	1,270	734	585	1,116	1,358

Figure 60. Acceleration rates at entrance ramps.

resents the acceleration rate of one subject on an entrance ramp; the solid line connects the average rate for each ramp. The data table attached to the chart shows the summary statistics for each ramp. The first row of the data table shows the average observed acceleration rate for all subjects to accelerate from the beginning of the ramp (Point 1) to the merge point (Point 3). The average distance between Points 1 and 3 on each ramp is listed in the second row. The average acceleration rates were typically between 1.0 and 1.5 ft/s^2, though rates at Ramps 7 and 11 were somewhat higher. Ramp 11 is unique because it lies just upstream of another entrance ramp and its merging area; thus, it is necessary for drivers entering at Ramp 11 to ensure they have attained the prevailing freeway speed to facilitate merging prior to the subsequent ramp. This feature encourages drivers to accelerate at a higher rate than on other ramps. Causes for the higher acceleration rate at Ramp 7 are not as clear; it could be related to the fact that drivers entered the ramp immediately after exiting the freeway, with no driving time on local roads. However, as great an effect is not shown in Ramp 9, which also was a part of an "off-and-on" sequence of events in the driving route.

Table 48 shows a comparison between the observed acceleration rates and distances and minimum acceleration lengths and assumed acceleration rates from Exhibit 10-70 of the *Green Book.* Because the entrance ramps on the driving route were straight ramps, the controlling feature at the beginning of the ramp was always the crossroad terminal. At some sites, the terminal was at a signal-controlled intersection, while at others the terminal was located on a one-way frontage road. For the latter condition, it was not necessary for vehicles to stop before entering the ramp. Thus, the speed of the instrumented vehicle at Point 1 was not always equal to zero. The observed rates are less than the *Green Book* rates at all locations except Ramp 11. This indicates that drivers are typically more casual in their acceleration under uncongested conditions than the *Green Book* rates suggest.

The observed acceleration distances are also shorter than those offered by the *Green Book,* which initially seems counterintuitive in conjunction with lower acceleration rates. A review of the data indicates that subjects in this study commonly merged at speeds lower than the merge speed assumed by the *Green Book.* In addition, the *Green Book* length extends to the point at which the width of the SCL decreases below 12 ft, while the observed distances were measured to the point of merge, which were often upstream of the point at which the 12-ft threshold was crossed. Thus, while some subjects used much of the SCL, many did not, producing average acceleration distances shorter than those listed in the *Green Book.* This suggests that the *Green Book* distances are sufficient to accommodate typical merge maneuvers by this population of 12 subject drivers.

6.4 Exit Ramp Analyses

6.4.1 Coasting

The objective of this analysis was to examine the validity of the assumption in the *Green Book* methodology that drivers decelerate in gear (i.e., coast) for 3 s prior to applying the brake when exiting a freeway. The 1965 *Blue Book* (AASHO, 1965) states that deceleration is a two-step process: first, the accelerator pedal is released (for a length of time assumed for 3 s) and the vehicle slows in gear without the use of brakes, and second, the brakes are applied. Two graphs were included in Figure VII-15 of the 1965 *Blue Book* to provide these distances. Previous research (Fitzpatrick and Zimmerman, 2007) concluded that the graphs were based on data from studies conducted in the 1930s and 1940s, but the underlying methodology was carried through to the current edition of the *Green Book.*

Using the reduced and processed data, researchers further examined the details of the data for exit ramp coasting. In the review of coasting data, researchers examined three time values for each subject on each ramp:

- **Throttle Release Time:** The elapsed time between the occurrence of peak speed and the deactivation of throttle (i.e., the time spent to remove the foot from the pedal).

Table 48. Comparison of constant acceleration rates and distances.

Ramp no.	Parallel/ taper	Design speed (mi/h)		Observed average		*Green Book* Exhibit 10-70	
		Freeway	Ramp	Accel (ft/s^2)	Distance (ft)	Accel (ft/s^2)	Distance (ft)
1	Taper	65	Stop Condition1	.07	914	1.92	1,410
3	Taper	70	Stop Condition	1.13	1,441	1.87	1,620
5	Parallel	70	Stop Condition	1.28	1,340	1.87	1,620
7	Taper	70	Stop Condition	1.86	908	1.87	1,620
9	Parallel	70	Stop Condition	1.56	1,270	1.87	1,620
11	Taper	65	Stop Condition	2.87	734	1.92	1,410
13	Parallel	65	45	1.22	585	1.62	600
15	Taper	65	Stop Condition	1.38	1,116	1.92	1,410
17	Taper	65	Stop Condition	1.57	1,358	1.92	1,410

Table 49. Summary statistics for coasting data.

	Throttle release time (s)		No pedal time (s)		Throttle release +no pedal (s)	
Ramp No.	Average	95% CI	Average	95% CI	Average	95% CI
2	1.47	(0.70,3.10)	1.00	(0.60,1.66)	3.21	(2.17,4.74)
4	1.08	(0.47,2.45)	1.27	(0.72,2.23)	3.02	(1.92,4.73)
6	1.42	(0.98,2.07)	1.26	(0.69,2.31)	3.29	(2.45,4.41)
8	2.56	(1.12,5.83)	0.78	(0.54,1.12)	3.86	(2.22,6.71)
10	1.76	(0.95,3.25)	0.80	(0.46,1.42)	3.25	(2.25,4.69)
12	0.88	(0.33,2.33)	0.89	(0.52,1.53)	2.09	(1.01,4.32)
14	2.74	(1.08,6.95)	1.27	(0.66,2.47)	6.17	(3.93,9.69)
16[1]	0.61	(0.16,2.29)	0.40	(0.17,0.94)	0.91	(0.23,3.57)
18	2.51	(1.56,4.04)	1.54	(0.80,3.00)	4.55	(2.88,7.17)
All	1.53	(1.05,2.23)	0.98	(0.74,1.30)	3.07	(2.07,4.55)

[1] Only six times were recorded; 12 were recorded for all other ramps.

- **No Pedal Time:** The elapsed time between the deactivation of throttle and the activation of brake (i.e., the amount of time when neither throttle nor brake was in use).
- **Throttle Release + No Pedal:** The sum of the previous two time values.

Visual exploration of the data reveals that the times for each ramp follow a lognormal distribution rather than a normal distribution (skewed to the right) and the center differs by ramp. This finding is intuitive since the minimum times bounded below by 0 and 50 percent of the Throttle Release + No Pedal times are less than 3.5 s, but values as high as 13.3, 16.1, and 32.4 s were also observed. The geometric mean and 95 percent confidence intervals for the three times are presented in Table 49 for each ramp. Observed times less than 0.1 s were rounded up to 0.1 s. The results for all ramps are based on the random effects model described below.

Review of the data in Table 49 indicates that the No Pedal times of subject drivers were noticeably shorter than the 3.0 s assumed by the *Green Book,* never averaging more than 1.54 s for any ramp. The minimum No Pedal time recorded was less than 0.1 s, and the maximum was 8.8 s. Adding the Throttle Release time to the No Pedal time produces results larger than 3 s on all but two ramps.

To determine the statistical significance of the results as compared to the assumed 3-s average coasting time, a random effects model was estimated for each of the times based on the log transformed values. The model includes an intercept term and a random effect for each ramp. The model does not account for the correlation between measurements from the same driver on exit ramps due to insufficient degrees of freedom available for testing. The variability in time within each ramp is assumed to be equivalent for all ramps. The estimated overall average time was compared to the 3 s assumed time. The results of the t-test for the No Pedal time are presented in Table 50, and the results for the Throttle Release + No Pedal times are presented in Table 51. The results show that average No Pedal time of 0.98 s is statistically different from 3 s, while the average Throttle Release + No Pedal time of 3.07 s is not statistically different from 3 s. Further investigation of the coasting data indicates 2 s of the coasting time typically occurs prior to the diverge maneuver, and 1 s of the coasting time occurs within the SCL following the diverge maneuver.

6.4.2 Use of Speed-Change Lane

This analysis investigated how much of the SCL each subject used at each exit ramp. This was calculated by dividing the measured distance between Point 2 and the painted nose by the length of the SCL. Figure 61 shows the location at which drivers entered the SCL on exit ramps on the driving course, expressed as a percentage of distance into the SCL. Each marker in Figure 61 represents the distance into the

Table 50. Results of random effects model test for average no pedal time = 3 s (log–scale transformed).

Average log time (log s)	Average time (s)	Standard Error (log s)	DF	t-statistic	P-value
-0.02	0.98	0.12	8	-9.18	<0.01

Note: DF = degrees of freedom.

Table 51. Results of random effects model test for average throttle release + no pedal time = 3 s (log–scale transformed).

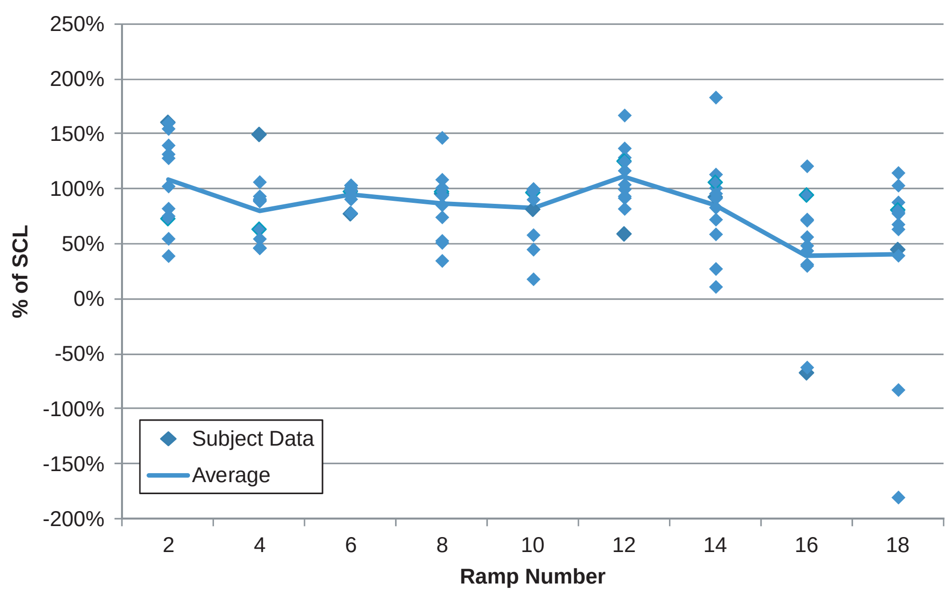
Average log time (log s)	Average time (s)	Standard Error (log s)	DF	t-statistic	P-value
1.12	3.07	0.17	8	0.13	0.90

SCL for one subject on an exit ramp, and the solid line connects the average SCL distance values for each ramp. The data table attached to the chart shows summary statistics for each ramp.

Diverging drivers in this study tended to complete their maneuvers within the second half of the SCL or later. The later drivers did not travel in the provided SCL, but instead completed their diverge maneuver beyond the painted nose; they are represented in Figure 61 as greater than 100 percent of SCL. The latter finding is somewhat related to the SCL length, since drivers on the two ramps with SCLs longer than 300 ft (Ramps 6 and 10), generally completed their maneuvers within the SCL, while the incidences of late diverges were more common on the ramps 110 ft or shorter.

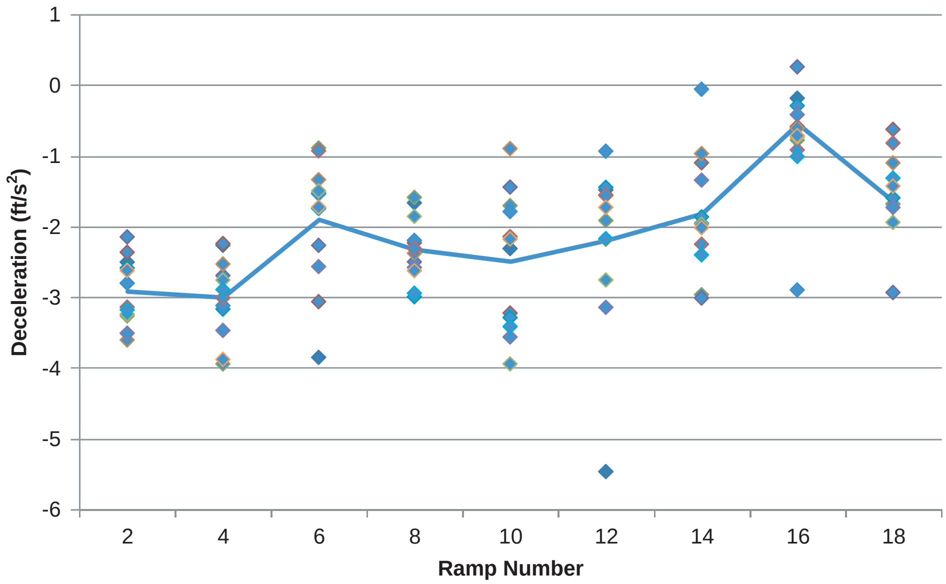
Ramp 10 is also unique because the approach to Ramp 10 is a through lane that changes to an exit-only lane. Thus, there is no taper upstream of the SCL at Ramp 10; only a change in lane line from a dotted or "skip-stripe" line to a solid line denotes the beginning of the SCL.

It appears that there may be a bit of a learning process as drivers see ramps that are similar in type. Ramps 16 and 18 have two of the shortest SCLs in the study, but they have the lowest average diverge point. The averages are influenced by two drivers at each ramp who began their diverge in the taper, upstream of the SCL and represented in Figure 61 as less than 0 percent of SCL. However, there is a lower frequency of late merges at these two ramps than in other ramps with SCLs of similar length.



SCL Length (ft)	100	110	310	70	355	50	40	80	35
Taper/Parallel	Taper	Taper	Taper	Taper	Parallel	Taper	Taper	Taper	Taper
Straight/Curve	Curve	Straight	Curve	Straight	Straight	Straight	Straight	Straight	Straight
Min SCL %	39.0%	45.8%	77.0%	34.4%	17.7%	58.7%	10.2%	–67.4%	–181.2%
Max SCL %	160.6%	149.1%	103.5%	146.7%	99.8%	167.0%	183.2%	120.7%	113.9%
Avg SCL %	108.3%	80.2%	94.8%	86.1%	82.1%	110.6%	86.0%	39.6%	40.9%
Std. Deviation	42.9%	30.8%	8.9%	30.1%	27.3%	28.5%	43.9%	58.4%	86.2%

Figure 61. SCL use at exit ramps.



Avg Observed Decel (ft/s^2)	–2.90	–2.99	–1.90	–2.31	–2.48	–2.19	–1.82	–0.54	–1.64
Avg Decel Distance (ft)	575	474	682	587	808	236	615	669	965

Figure 62. Deceleration rates at exit ramps.

6.4.3 Deceleration

Similar to the acceleration analysis for entrance ramps, researchers used the speed and time data from each subject to develop speed profiles for each subject on each exit ramp, which were then used to evaluate deceleration patterns. Figure 62 shows the recorded deceleration rates for each subject on each ramp, represented as constant deceleration from the diverge point (Point 2 in Figure 56b) to the critical feature (Point 4 in Figure 56b). Each marker represents the deceleration rate of one subject on an individual exit ramp; the solid line connects the average rate for each ramp. The data table attached to the chart shows the summary statistics for each ramp. The first row of the data table shows the average observed deceleration rate for all subjects to decelerate from the diverge point (Point 2) to the end of the ramp (Point 4). The average distance between Points 2 and 4 on each ramp is listed in the second row. The average deceleration rates were typically between –1.5 and –3.0 ft/s^2, though the rate at Ramp 16 is somewhat less. Ramp 16 is unique because the ramp terminates at a one-way frontage road approximately 600 ft upstream of the signalized intersection of the adjacent crossroad. Thus, in the typically uncongested conditions in which subjects traveled through this ramp, there was an added distance of approximately 600 ft in which to decelerate, which reduced the need for more pronounced deceleration on the ramp. Ramp 12 was the only other exit ramp to terminate at a frontage road, and the frontage road volumes at that location were much higher, reducing the possibility of using the frontage road for substantial deceleration.

Table 52 shows a comparison between the observed deceleration rates and distances and minimum deceleration lengths and assumed deceleration rates from Exhibit 10-73 of the *Green Book*. Deceleration rates based on *Green Book* criteria are provided assuming a constant deceleration and the assumed deceleration rates during primary braking.

Comparison of the observed rates and *Green Book* rates shows that the observed rates are less than the *Green Book* rates at all locations. A review of the speed profiles indicates that every subject began deceleration prior to entering the SCL at every location; this means that the needed deceleration was accomplished over a greater distance than the length of the SCL and the distance to the controlling feature. As a result, drivers decelerated more gradually than the rates implied by the *Green Book*.

Further review of the data indicates that, while average deceleration rates are useful for summarizing the data, the actual deceleration rates are variable, commonly increasing as the driver approaches the end of the ramp. While the final deceleration rates are not necessarily equal to those obtained from *Green Book* values, they are typically closer in value than the average rates shown in Table 52. Given these deceleration characteristics of subject drivers, the deceleration length values provided in the *Green Book* appear sufficient to accommodate typical diverge maneuvers. Although the observed

Table 52. Comparison of constant deceleration rates and distances.

Ramp no.	Diverge type	Design speed (mi/h) Freeway	Design speed (mi/h) Ramp	Observed average Distance (ft)	Observed average Decel (ft/s^2)	*Green Book* Exhibit 10-72 Distance (ft)	*Green Book* Exhibit 10-72 Constant decel (ft/s^2)	*Green Book* Exhibit 10-72 Decel during braking (ft/s^2)
2	Taper	70	25	575	–2.90	550	–5.63	–7.77
4	Taper	70	25	474	–2.99	550	–5.63	–7.77
6	Taper	70	Stop Condition	682	–1.90	615	–5.88	–7.40
8	Taper	70	Stop Condition	587	–2.31	615	–5.88	–7.40
10	Parallel	70	Stop Condition	808	–2.48	615	–5.88	–7.40
12	Taper	65	45	236	–2.19	340	–4.51	–7.66
14	Taper	65	35	615	–1.82	440	–5.19	–7.49
16	Taper	65	Stop Condition	669	–0.54	570	–5.71	–7.55
18	Taper	65	Stop Condition	965	–1.64	570	–5.71	–7.55

deceleration values were less than assumed in the *Green Book,* none of the drivers appeared uncomfortable and likely could have tolerated greater deceleration if necessary.

6.5 Summary of Significant Findings

The most significant findings from the examination of behavioral data are as follows:

- In uncongested or lightly congested conditions, a typical glance into a mirror or over the shoulder by a driver merging onto the freeway is typically about 2.5 to 3.0 s, but the driver tends to take three such glances on a given entrance ramp. During the typical glance, the driver travels between 100 and 200 ft and increases speed by approximately 2.5 mi/h.
- Merging drivers in uncongested or lightly congested conditions tended to use at least half of the SCL provided when entering the freeway, and often used the taper area to complete their merging maneuvers.
- Observed acceleration rates were lower than the assumed acceleration rates from the *Green Book.* Additionally, observed acceleration distances were shorter than those offered by the *Green Book.* This seems counterintuitive, but subjects in this study commonly merged at speeds lower than the merge speed assumed by the *Green Book.*
- The assumption in the *Green Book* that drivers decelerate in gear (i.e., coast) for 3 s was not statistically different from the observations for the drivers in this study, if the *Green Book* definition of coasting includes the time used to remove the driver's foot from the throttle. If coasting is defined as the activation of neither throttle nor brake, the appropriate value for this driver population is approximately 0.98 s.
- Diverging drivers tended to use the freeway through lane for a large portion of their deceleration when exiting the freeway, and seldom entered the SCL within the first 50 percent of the provided length. Drivers on the two ramps with SCLs longer than 300 ft, generally completed their maneuvers within the SCL, while the incidences of late diverges were more common on the ramps 110 ft or shorter.
- Every subject began deceleration prior to entering the SCL at every location; this means that the needed deceleration was accomplished over a greater distance than the length of the SCL and the distance to the controlling feature. As a result, drivers decelerated more gradually than the rates implied by the *Green Book.*
- Actual deceleration rates are variable, commonly increasing as the driver approaches the end of the ramp.
- Deceleration length values provided in the *Green Book* appear sufficient to accommodate typical diverge maneuvers by this subject driver population. Although the observed deceleration values were less than assumed in the *Green Book,* none of the drivers appeared uncomfortable, and there was no indication that the drivers could not tolerate greater deceleration if necessary as assumed in the *Green Book* criteria.

SECTION 7

Conclusions and Proposals

This section provides the conclusions and proposals based upon the findings of this research. General conclusions are first provided related to the design of both entrance and exit ramps. Then, conclusions are provided specifically related to entrance ramps, followed by conclusions specifically for exit ramps. Finally, potential changes proposed for consideration in the next edition of the *Green Book* are provided, followed by future research needs.

General Conclusions

- Passenger cars should remain the principal design vehicle for freeway mainline ramp terminals. This is consistent with current AASHTO policy. This conclusion is based primarily upon the analysis of truck-related crashes, which indicates that truck crashes are not overrepresented near freeway mainline ramp terminals. In addition, during the observational study, no critical maneuvers were observed as trucks merged and diverged onto and off of the freeway without much difficulty. This suggests that truck drivers and the general driver population adjust their behavior as necessary to accommodate larger, heavier vehicles at freeway mainline ramp terminals.

  Ultimately, it comes down to assessing the risks associated with not providing the additional acceleration/deceleration lengths to accommodate the performance capabilities of trucks and making a decision based upon benefits and costs given available resources. No economic analysis calculations were performed in this research because of the lack of data, but given the level of information currently available from the research findings, passenger cars appear to be the appropriate design vehicle for most freeway-ramp terminals.

  The exception to this rule is at freeway mainline ramp terminals where the truck volumes on the ramps are substantial, in which case further consideration should be given to more fully accommodating trucks within the design. A value of 1,000 ramp trucks/day appears to be a good threshold for defining when the truck volume is significant, but this value should not be considered an absolute.
- Considering the three potential freeway operational design conditions evaluated (i.e., free, constrained, and forced merge/diverge), freeway mainline ramp terminals should be designed based upon free-merge/diverge conditions (i.e., free-flow conditions). By designing for free-merge/diverge conditions, sufficient length is provided to accommodate merging/diverging behaviors during more congested operating conditions. This is consistent with current AASHTO policy.
- It is most appropriate to design freeway mainline ramp terminals based upon average operating speeds of vehicles, rather than design speeds. Again, this is consistent with current AASHTO policy. Designs based upon design speeds would provide terminals that are over-designed.

Conclusions Specific to Entrance Ramps

- Merging vehicles do not accelerate at a constant rate along the length of the ramp and SCL.
- A clear view of the freeway and mainline traffic is important to accommodate merging vehicles. On average, drivers glance into the mirror or over their shoulder three times before merging onto the freeway. Drivers begin glancing at the freeway and mainline traffic prior to reaching the painted nose. While drivers are taking a glance at the freeway traffic, they continue to accelerate along the ramp and SCL.
- Vehicles exit curves on ramps at speeds much higher than the values given for "initial speed" in *Green Book* Exhibit 10-70. This indicates that vehicle performance and driver preferences have changed since these values were determined.
- In free-merge conditions, many vehicles choose to enter the freeway at speeds much lower than the speed of freeway traffic. It appears that drivers simply choose not to use the

full length of the ramp and SCL for acceleration when gaps are abundant and merging is not difficult.

- Constrained-merge conditions appear to be the most difficult for drivers, as reaching freeway speeds becomes more critical since gaps are smaller and do not provide as much opportunity for accelerating in the freeway lane. Additionally, drivers have a more difficult task identifying appropriate gaps. In these conditions, some drivers take the first available gap they find even if they have not reached an ideal merge speed, while other drivers use the full length of the SCL, and in some cases the taper, to reach an ideal speed near the speed of the freeway traffic before merging onto the freeway.
- As expected, heavy vehicles do not perform as well as passenger cars at entrance ramps. Their acceleration rates are lower, and they merge onto the freeway at lower speeds. However, at ramps with a small proportion of truck traffic, their merging behavior does not appear to negatively impact the overall operation of the ramp terminals.
- Vehicles are more likely to use the full length of a tapered SCL to accelerate to near freeway speeds before merging in contrast to parallel SCLs, where vehicles may merge earlier along the ramp and at lower speeds.
- A potential disbenefit of a parallel SCL is that vehicles from the freeway mainline may use the adjacent SCL to pass vehicles within the freeway lanes, particularly during congested conditions. With the geometry of a tapered SCL, such undesirable maneuvers by freeway vehicles are less likely.
- There is no substantive difference in the operational performance between low-speed (loop) and high-speed (straight) ramps under free-merge conditions.
- Based upon vehicle capabilities and driver preferences, many vehicles are capable of accelerating at higher rates than the assumed acceleration rates used to determine minimum acceleration lane lengths for entrance terminals in the *Green Book.* However, because most situations do not require that vehicles accelerate to the speeds assumed with the design, many drivers choose to accelerate at lower rates than assumed within AASHTO policy.
- Only three of the 11 entrance ramps studied met or exceeded the *Green Book* recommendation for minimum acceleration lane lengths. Despite this, all the ramps appeared to have acceptable operational performance. This is evidence that the current design guidance is conservative.
- Upgrades as steep as 3 to 4 percent do not impact the acceleration capabilities of passenger cars, at least over lengths necessary for entrance terminals. As grades increase to 5 or 6 percent, the acceleration rates of passenger cars tend to decrease.
- The conceptual approach that assumes constant acceleration used in AASHTO policy is a reasonable approach for determining minimum acceleration lane lengths for design. In addition, the current values provided in *Green Book* Exhibit 10-70 are conservative estimates for minimum acceleration lane lengths, given the current vehicle fleet and driver population, and do not need to be modified. In particular, they provide sufficient length for vehicles to merge onto the freeway under a range of freeway operating conditions. It is also concluded that in situations where free-merge (i.e., free-flow) conditions are expected for the foreseeable future and constraints make it difficult to provide the recommended minimum acceleration lengths, the minimum acceleration lane lengths can be reduced by 15 percent without causing expected operational problems. Reducing minimum acceleration lane lengths by 15 percent is even conservative considering that, based upon the median acceleration rates of free-flow passenger cars measured under free-merge conditions, minimum acceleration lane lengths could be reduced between 16 to 46 percent and still be sufficient for 50 percent of vehicles.

Conclusions Specific to Exit Ramps

- Most diverge maneuvers begin before or within the taper or within the first or middle thirds of the SCL. Few diverge maneuvers take place in the final third of the SCL or beyond the painted nose.
- Vehicles that diverge earlier along the deceleration lane diverge closer to freeway speeds than vehicles that diverge later along the deceleration lane, closer to the painted nose.
- Deceleration rates of diverging vehicles along the SCL and ramp are not constant.
- Where the deceleration lane length is longer than the *Green Book*–recommended minimum length, most vehicles decelerate at rates lower than those assumed by the *Green Book.* This is due in varying degrees to the additional length provided and to vehicles decelerating in the freeway lane prior to initiating the diverge maneuver.
- Deceleration rates of exiting vehicles are greater for vehicles that diverge closer to the painted nose than for vehicles that diverge further upstream from the painted nose.
- Free-flow vehicles decelerate at greater rates than platooned vehicles. This is most likely due to higher initial diverge speeds for free-flow vehicles.
- When exiting the freeway, trucks decelerate at rates very comparable to those of passenger cars. In addition, trucks typically diverge from the freeway at lower speeds than passenger cars.
- Crash rates for trucks are higher at parclo, free-flow, and "other" ramp configurations than at diamond; outer connection; direct or semi-direct connection; and button hook, scissor, and slip ramp configurations.
- Drivers exiting on loop ramps tend to reduce their speed in the freeway lane more, and decelerate along the SCL

at a greater rate, than drivers exiting on straight ramps. This may be due to the visual perceptions of drivers as they approach the horizontal curvature of a loop ramp.

- The geometry of parallel deceleration lanes generally leads to substantially higher deceleration rates than on tapered deceleration lanes. This may be the result of vehicles diverging slightly closer to freeway speeds along parallel deceleration lanes than along tapered deceleration lanes. The disparity between deceleration rates is most apparent on straight ramps.
- For initial speeds above 30 mi/h, deceleration rates along the SCL and ramp generally increase as the speed increases, having a strong relationship with the natural log of initial speed.
- AASHTO policy assumes a two-step process for establishing design criteria for minimum deceleration lane lengths. Deceleration is accomplished first while coasting in gear without the use of brakes and then during the application of the brake. AASHTO assumes 3.0 s for the coasting period. This is consistent with the amount of time spent coasting during diverge maneuvers in the behavioral study, assuming the coasting time includes the time spent releasing the throttle until application of the brake. Thus, 3.0 s of coasting time is a valid assumption for describing the diverge maneuver. However, 2 s of the coasting time typically occurs prior to the diverge maneuver while in the freeway, and 1 s of the coasting time occurs within the SCL following the diverge maneuver.
- The conceptual approach used in AASHTO policy for the design of exit ramps, which assumes a two-step process of deceleration, is a reasonable approach for determining minimum deceleration lane lengths for design. In addition, the current values provided in *Green Book* Exhibit 10-73 are conservative estimates for minimum deceleration lane lengths, given the current vehicle fleet and driver population, and do not need to be modified. No critical or unusual diverge maneuvers were observed at the study sites that met and exceeded the current design criteria. In addition, at these study locations, vehicles decelerated at rates well within the capabilities of the vehicle fleet and driver preferences. This was in part due to some deceleration by diverging vehicles in the freeway mainline prior to the diverge maneuver. Given this last point, it is beneficial to have a conservative design process that does not assume that vehicles begin decelerating in the freeway mainline.

Potential Changes Proposed for Consideration in the Next Edition of the *Green Book*

- Include a statement in the *Green Book* text that tapered SCLs are preferred over parallel SCLs at entrance ramps because vehicles tend to merge closer to freeway speeds at tapered SCLs, and, if parallel SCLs are used, they are most appropriate at ramps expected to experience constrained- or forced-merge conditions because they provide greater flexibility in selecting a merge location along the SCL.
- Include a statement in the text accompanying *Green Book* Exhibit 10-70 that the design values in the exhibit are conservative, and that in situations where free-merge (i.e., free-flow) conditions are expected for the foreseeable future and constraints make it difficult to provide the recommended minimum acceleration lengths presented in the *Green Book,* minimum acceleration lane lengths may be reduced by 15 percent without causing any expected operational problems.
- Include additional exhibits that provide speed-distance curves for trucks in a range from 140 to 200 lb/hp. This will provide the designer with more flexibility to select an appropriate heavy vehicle for design, and, in some cases, find a better compromise between designing for passenger cars and designing for trucks, especially at entrance terminals with substantial truck volumes. These additional exhibits should be provided in Chapter 3 of the *Green Book* because they could be used for more general purposes than the design of freeway mainline ramp terminals, rather than just for acceleration lanes. A reference to the new exhibits should be included in Chapter 10 of the *Green Book.*
- Modify *Green Book* Exhibit 10-70 to indicate that the initial speed (or design speed) is based upon the controlling feature, which, in the case of a straight ramp, is the crossroad terminal rather than a horizontal curve, and within the exhibit, replace "Speed reached" with "Merge speed." In addition, an estimate should be provided in the text for the speed that vehicles enter the ramp at the crossroad terminal.
- Emphasize in the *Green Book* text that the values presented for minimum deceleration lane length on exit ramps are a conservative estimate that do not account for any deceleration in the mainline freeway lanes. While some drivers do accomplish a considerable portion of their deceleration on the freeway, such that a shorter deceleration lane length would be operationally sufficient, it is prudent for the designer not to assume deceleration in the mainline freeway lanes in the design of an exit ramp.
- Mention within the text that providing deceleration lanes longer than the minimum values listed in *Green Book* Exhibit 10-73 may promote more casual deceleration by exiting drivers, particularly under uncongested or lightly congested conditions. This is not necessarily a negative result, but it does change the operational characteristics of the ramp, as those drivers will maintain higher speeds further into the SCL and possibly into the ramp proper.
- Modify *Green Book* Exhibit 10-73 and accompanying text to indicate that the final speed (or design speed) of the exit ramp is based upon the controlling feature, which, in the case of a straight ramp, is the crossroad terminal rather

than a horizontal curve. Also within the exhibit, replace "Speed reached" with "Diverge speed."

- Modify *Green Book* Exhibits 10-70 and 10-73 to be more consistent in format. Exhibit 10-70 provides the desired taper rate as a note, and Exhibit 10-73 does not. Exhibit 10-73 provides the definitions of each speed term, and Exhibit 10-70 does not.

Future Research Needs

- The data set collected and analyzed for this study indicated that the "initial speed" and "speed reached" values shown in *Green Book* Exhibit 10-70 are lower than the actual speeds for both. In many cases, vehicles exit the controlling feature on the ramp at speeds greater than the design speed of the ramp, indicating that vehicle and driver performance have evolved from what they were when both the design speeds were determined and when the exhibit was created. Additional research is recommended to determine more appropriate values for the "initial speed" row and the "speed reached" column in the exhibit.
- A study similar to the one performed in this project should be conducted to cover a wider range of design conditions in *Green Book* Exhibit 10-70 than was possible in this study. In particular, a future study should include ramps with entrance curve design speeds in the range of 15 to 25 mi/h. Also, the study should measure speeds of vehicles beginning at the crossroad terminal.
- A study on the "speed reached" (i.e., diverge speed) for given values of freeway design speed in *Green Book* Exhibit 10-73 would be beneficial to determine whether these values reflect not only current vehicle performance but also common driver behavior. In particular, a future study should include ramps with highway design speeds in the range of 30 to 60 mi/h.
- The study on diverge speed should also include a review of higher running speeds on the ramp proper, as straight ramps with no controlling curve allow drivers to maintain higher speeds as they approach the crossroad terminal, particularly where that terminal does not require a stop or yield (e.g., intersection with a one-way frontage road).
- The results of this study suggest that deceleration length, occurrence of freeway deceleration, and diverge speed may be influencing factors that have measurable effects on deceleration rate. Research to quantify these effects, as well as to identify possible relationships between and among those influencing factors, is recommended.
- This research focuses on single-lane freeway mainline ramp terminals that are outside the influence of upstream or downstream operations of other freeway mainline ramp terminals. This research does not specifically address freeway mainline ramp terminals with two or more lanes, weaving areas where a continuous auxiliary lane is provided between entrance and exit terminals, nor issues specifically related to ramp metering. Additional research should be conducted to further investigate these topics.

SECTION 8

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Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation