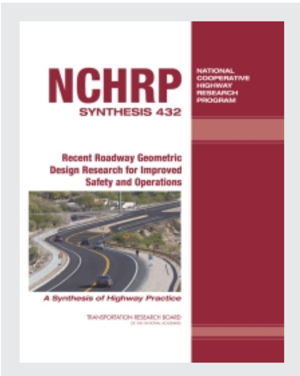

Recent Roadway Geometric Design Research for Improved Safety and Operations

DETAILS

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP SYNTHESIS 432

Recent Roadway Geometric Design Research for Improved Safety and Operations

A Synthesis of Highway Practice

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Subscriber Categories
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TRANSPORTATION RESEARCH BOARD

WASHINGTON, D.C.
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

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The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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The project that is the subject of this report was a part of the National Cooperative Highway Research Program, conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council.

The members of the technical panel selected to monitor this project and to review this report were chosen for their special competencies and with regard for appropriate balance. The report was reviewed by the technical panel and accepted for publication according to procedures established and overseen by the Transportation Research Board and approved by the Governing Board of the National Research Council.

The opinions and conclusions expressed or implied in this report are those of the researchers who performed the research and are not necessarily those of the Transportation Research Board, the National Research Council, or the program sponsors.

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Cover figure: Several geometric improvements were constructed on Sunrise Drive in Tucson, Arizona, to improve roadway safety and operations. *Photo credit:* Khang Nguyen, Kittelson & Associates.

FOREWORD

Highway administrators, engineers, and researchers often face problems for which information already exists, either in documented form or as undocumented experience and practice. This information may be fragmented, scattered, and unevaluated. As a consequence, full knowledge of what has been learned about a problem may not be brought to bear on its solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem.

There is information on nearly every subject of concern to highway administrators and engineers. Much of it derives from research or from the work of practitioners faced with problems in their day-to-day work. To provide a systematic means for assembling and evaluating such useful information and to make it available to the entire highway community, the American Association of State Highway and Transportation Officials—through the mechanism of the National Cooperative Highway Research Program—authorized the Transportation Research Board to undertake a continuing study. This study, NCHRP Project 20-05, "Synthesis of Information Related to Highway Problems," searches out and synthesizes useful knowledge from all available sources and prepares concise, documented reports on specific topics. Reports from this endeavor constitute an NCHRP report series, *Synthesis of Highway Practice*.

This synthesis series reports on current knowledge and practice, in a compact format, without the detailed directions usually found in handbooks or design manuals. Each report in the series provides a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems.

PREFACE

By Jo Allen Gause
Senior Program Officer
Transportation Research Board

This synthesis report is an update of *NCHRP Synthesis of Highway Practice 299* on the same topic published in 2001. It reviews and summarizes selected roadway geometric design literature completed and published from 2001 through early 2011, particularly research that identified impacts on safety and operations. The report is structured to correspond to chapters in AASHTO's *A Policy on Geometric Design of Highways and Streets*, more commonly referred to as the *Green Book*.

Information for the synthesis study was collected using an extensive literature review and analysis.

Marcus A. Brewer, Texas Transportation Institute, College Station, Texas, collected and synthesized the information and wrote the report. The members of the topic panel are acknowledged on the preceding page. This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

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.

RECENT ROADWAY GEOMETRIC DESIGN RESEARCH FOR IMPROVED SAFETY AND OPERATIONS

SUMMARY

Since the publication of *NCHRP Synthesis 299* in 2001, a considerable amount of research related to operational and safety effects of highway and street geometrics has been completed. Application of this knowledge is sometimes limited because of the sheer volume of information that exists and the rapid pace in which it is produced and published. Geometric design research results are scattered across a variety of different tools and publications that are not easily accessible to highway and street geometric designers and geometric design policy makers. This synthesis identifies and summarizes roadway geometric design literature completed and published from 2000 through early 2011, particularly research that identified impacts on safety and operations. Findings within the synthesis are presented in groups similar to key chapters and sections within AASHTO's *A Policy on Geometric Design of Highways and Streets* (commonly referred to as the *Green Book*). Some key findings are provided here.

It is important to note that the recommendations included in the list of findings from the literature shown here and throughout the report are those of the authors cited. Before any revisions to AASHTO's *Green Book* were to be made on the basis of these recommendations, they would need to be considered on the basis of the rigor of the research and logic that underlie them. No endorsement of these recommendations is implied by their inclusion in the listing of findings from the literature.

- Review of existing long-standing guidelines was a common theme, evaluating whether changes in vehicle performance or driver behavior necessitated changes in design practices. In many cases, vehicle performance did not affect the perceived appropriateness of guidelines, although changes in headlamp performance did prompt a recommended change in sag curve design. Driver behavior, however, was the source of several suggested changes, including perception–reaction time for stopping sight distance and consideration of older drivers for intersection sight distance.
- During this period, finding ways to make intersections more efficient was also a frequent topic of research. The use of modern roundabouts in the United States has grown tremendously, leading to two comprehensive FHWA *Informational Guides,* which are summarized in the body of the report. Innovative intersection designs that seek to improve capacity by adjusting left-turn movements were also often investigated. These designs were often shown to have increased capacity under certain conditions, but they typically require additional right-of-way and increased construction costs to install.
- Many of the research topics found in the assembled body of knowledge were not directly investigating the characteristics of a particular geometric design element; rather, common topics were traffic control devices, access management techniques, or other treatments that had a relationship with one or more design elements, and the research investigated what effects, if any, the design had on the treatment, or vice-versa.
- A growing trend is research that attempts to quantify the safety effects of geometric design elements. Crash modification factors and similar metrics have been developed in an attempt to directly relate safety to design; the first edition of AASHTO's *Highway Safety Manual* is a comprehensive source of such measures on a wide variety of treatments and countermeasures, including those that are geometric in nature.

This report is a synthesis of research, not of current or implementable practice. Therefore, the study did not employ a survey or questionnaire on current practices, as is typical for NCHRP synthesis projects. The study used two approaches to identify information: (1) a review of the literature contained in national databases, and (2) a request to state design and traffic engineers to supply additional information on studies conducted within their jurisdictions. The national literature review represented the vast majority of the effort for this synthesis study. TRB's Transportation Research Information System (TRIS), the Transport online database, and the TRB online publications catalog were all used to identify potential sources from papers and reports published during the previous decade.

Findings from research conducted during the decade addressed a variety of issues related to geometric design. A selection of key findings included:

- Dimensions of commonly used trucks have changed in recent years, prompting recommendations to revise the dimensions of those vehicles in the *Green Book* (Harwood et al. 2003a).
- Along with changes in dimensions have come changes in performance; however, design guidelines are sufficient to accommodate their performance for many design elements (Harwood et al. 2003a).
- Posted speed limit and anticipated operating speed were frequently associated with the selection of design speed (Fitzpatrick and Carlson 2002).
- Observation of driving behavior revealed that the strongest indicator of operating speed was posted speed limit. "Design speed appeared to have minimal impact on operating speeds unless a tight horizontal radius or a low K-value was present" (Fitzpatrick et al. 2003a).
- New values for stopping sight distance and new design controls for vertical curves were recommended, based on a perception–reaction time of 2.5 s, a 10th percentile deceleration rate of 11.2 ft/s^2, a 10th percentile driver eye height of 3.5 ft, and a 10th percentile object height of 2.0 ft (Fambro et al. 2000).
- Increased consistency between AASHTO design standards for passing sight distance and *Manual on Uniform Traffic Control Devices* (*MUTCD*) pavement marking practices was recommended, specifically accomplished by using the *MUTCD* criteria for marking passing/no-passing zones on two-lane roads in the *Green Book*'s passing sight distance (PSD) design process. In addition to providing the desired consistency between PSD design and marking practices, two-lane highways could be designed to operate safely with the *MUTCD* criteria (Harwood et al. 2008).
- Lane widths of 11 or 12 ft provide optimal safety benefit for common values of total paved width on rural two-lane roads. Although 12-ft lanes appear to be the optimal design for 26- to 32-ft total paved widths, 11-ft lanes perform equally well or better than 12-ft lanes for 34- to 36-ft total paved widths (Gross et al. 2009).
- Crash data on roads treated with centerline rumble strips or shoulder rumble strips revealed noticeable crash reductions on all classes of roads (rural and urban two-lane roads and freeways). Shoulder rumble strips placed as close to the edgeline as possible maximize safety benefits. The safety benefits of centerline rumble strips for roadways on horizontal curves and on tangent sections are for practical purposes the same (Torbic et al. 2009).
- A minimum skew angle of 15 degrees can accommodate age-related performance deficits at intersections where right-of-way is restricted (Staplin et al. 2002).
- Adding "left-turn lanes is effective in improving safety at signalized and unsignalized intersections," reducing crashes between 10% and 44%. Positive results can also be expected for right-turn lanes, with reductions in total intersection accidents between 4% and 14% (Harwood et al. 2002).
- A series of projects during the decade led to the publication of two FHWA *Informational Guides* containing recommendations and guidelines for all aspects of roundabout design.
- A number of innovative intersection designs were considered, many of which showed benefits in capacity and/or delay, but the additional right-of-way needed to construct each of these innovative designs is a potential drawback.

- "ADA requires that new and altered facilities constructed by, on behalf of, or for the use of state and local government entities be designed and constructed to be readily accessible to and usable by individuals with disabilities" (Rodegerdts et al. 2004).
- Spacing assessments indicate that ramp spacing of less than 900 ft is likely not geometrically feasible. That spacing value increases up to 1,600 ft for entrance–exit ramp pairs (Ray et al. 2011).

Results from the research synthesized in this document recommended a number of changes to the AASHTO *Green Book,* the *MUTCD,* and other guidance documents. The research also produced two FHWA *Informational Guides* on roundabouts and contributed to other guides on access management, pedestrian and bicycle accommodation, designing for older drivers, the *Highway Capacity Manual,* and the *Highway Safety Manual.* Discussion of the relationship between this synthesis report and other documents, along with relevant cross references, is also provided.

CHAPTER ONE

INTRODUCTION

BACKGROUND

The national policy for geometric design in the United States is the AASHTO *A Policy on Geometric Design of Highways and Streets* (commonly referred to as the *Green Book*). This document has been updated numerous times, with revisions based on new information from research findings. As of the writing of this report, the *Green Book* had most recently been revised in 2001, which included a substantial reformatting of the document from previous editions, and in 2004, to incorporate additional guidance based on recently completed research.

During the decade since 2000, a great deal of geometric design-related research was conducted on a wide variety of topics and issues. Results from the research were produced to inform the profession not only on design techniques and processes, but also on the safety and operational effects of those designs and how they influence other activities, such as maintenance. As a result, there was a significant addition to the body of research related to safety and operational impacts of roadway geometric design decisions during the decade. The ability of practitioners to apply this knowledge can be limited because of the sheer volume of information that exists and the rapid pace in which it is produced and published. In addition, geometric design research results are scattered across a variety of different tools and publications, some of which are not easily accessible to designers and policy makers. To avoid the significant time investments that would otherwise be required to find, critique, and implement the research results into practice, a synthesis of recent research was needed. A standard for such a synthesis was established with the publication in 2001 of *NCHRP Synthesis 299* (Fitzpatrick and Wooldridge 2001). NCHRP Synthesis Project 20-05 funded a study to develop an updated synthesis to build on the previous work of *NCHRP Synthesis 299*; the result of the study is this document.

This synthesis study reviewed and summarized the geometric design research published between 2000 and early 2011, particularly research with improved safety and operations implications. The following topics were addressed in the review:

- Design speed,
- Additional design controls and criteria (e.g., vehicles, consistency, and driver characteristics),
- Horizontal alignment,
- Vertical alignment,
- Cross section (including relevant roadside elements),
- Intersections (including channelization, roundabouts, and recent innovative intersection designs),
- Interchanges,
- Design consistency,
- Access management, and
- Pedestrian and bicycle issues.

STUDY OBJECTIVE

The objective of this study was to identify and summarize roadway geometric design literature completed and published from 2000 through early 2011, particularly research that identified impacts on safety and operations. To identify such information the study used two approaches: a review of the relevant literature contained in national databases, and a request to state design and traffic engineers for additional information on studies conducted within their jurisdictions.

The national literature review represented the vast majority of the effort for this synthesis study. TRB's Transportation Research Information System (TRIS), the Transport online database, and the TRB online publications catalog were all used to identify potential sources from papers and reports published during this period. Other sources of information included responses to the request for information from state departments of transportation (DOTs), input from the Synthesis Study Topic Panel, and the author's personal knowledge of recently completed research. The information collection from state DOT design and traffic engineers was requested by members of the AASHTO Subcommittees on Design and Traffic Engineering.

This document is a synthesis of research, not of current or implementable practice. Therefore, the study did not employ a survey or questionnaire on current practices, as is typical for such projects. An example of such a survey, documenting recent state DOT practices on design guidance and standards for non-freeway resurfacing, restoration, and rehabilitation (3R) projects, can be found in *NCHRP Synthesis 417* (McGee 2011).

ORGANIZATION OF REPORT

This synthesis report consists of the introduction, five chapters that summarize the findings from the literature, and a concluding chapter with suggestions for future research. This introductory chapter provides an overview of the project, including background information. Chapters two through six provide summaries and relevant tables and figures of the findings from the literature. These chapters are organized to match the presentation of material in the *Green Book*:

- Chapter two—Design Controls and Criteria
- Chapter three—Elements of Design
- Chapter four—Cross-Section Elements
- Chapter five—Intersections
- Chapter six—Interchanges

The final chapter of the report is a summary of key findings from the literature, along with potential issues to be considered for future research.

It is important to note that the recommendations included in the findings from the literature throughout this report are those of the authors cited. Before any revisions to AASHTO's *Green Book* were to be made on the basis of these recommendations, they would need to be considered on the basis of the rigor of the research and logic that underlie them. No endorsement of these recommendations is implied by their inclusion in the findings from the literature.

RELATIONSHIP TO OTHER DOCUMENTS

Results from the research synthesized in this document recommended a number of changes to the AASHTO *Green Book,* the *Manual on Uniform Traffic Control Devices* (*MUTCD*), and other guidance documents. The research also produced two FHWA *Informational Guides* on roundabouts and contributed to other guides on access management, pedestrian and bicycle accommodation, designing for older drivers, the *Highway Capacity Manual* (*HCM*), and the *Highway Safety Manual.* Discussion of the relationship between this synthesis report and other documents, along with relevant cross references, is also provided.

It should be emphasized that this synthesis report does not contain the applicable policies or design tools; it is confined to summarizing the literature. References to appropriate research reports, policies, and guidance documents are provided throughout the document, compiled into a comprehensive list of references, which follows the Conclusions (chapter seven). Additional, relevant sources are provided in a Bibliography following the References; these sources are not specifically included in the body of the report, but may provide readers additional information on issues and topics relevant to those contained in the report.

CHAPTER TWO

DESIGN CONTROLS AND CRITERIA

OVERVIEW

The advent of a new definition for design speed in the 2001 *Green Book* led to some ideas for research about the effects of that new definition. Researchers used that definition to investigate relationships among design speed, operating speed, and posted speed. Also, multiple studies reviewed selections of the *Green Book* to determine if long-standing guidelines were still applicable to modern vehicles and drivers. Special attention was paid to trucks, to determine if roadways primarily designed for passenger cars would still accommodate increasing numbers of heavy vehicles.

DESIGN VEHICLES

Trucks are an important consideration in the geometric design of highways. Many highway geometric design policies are based on vehicle characteristics. Truck characteristics are often a key consideration in determining the recommended values of such criteria. Harwood et al. (2003b) conducted a research project to "review the characteristics of trucks in the current U.S. truck fleet, as well as possible changes to the truck fleet, and [recommend] appropriate changes to highway geometric design policy to ensure that highways can reasonably accommodate trucks."

The authors recommended several changes in the design vehicles presented in the *Green Book,* specifically:

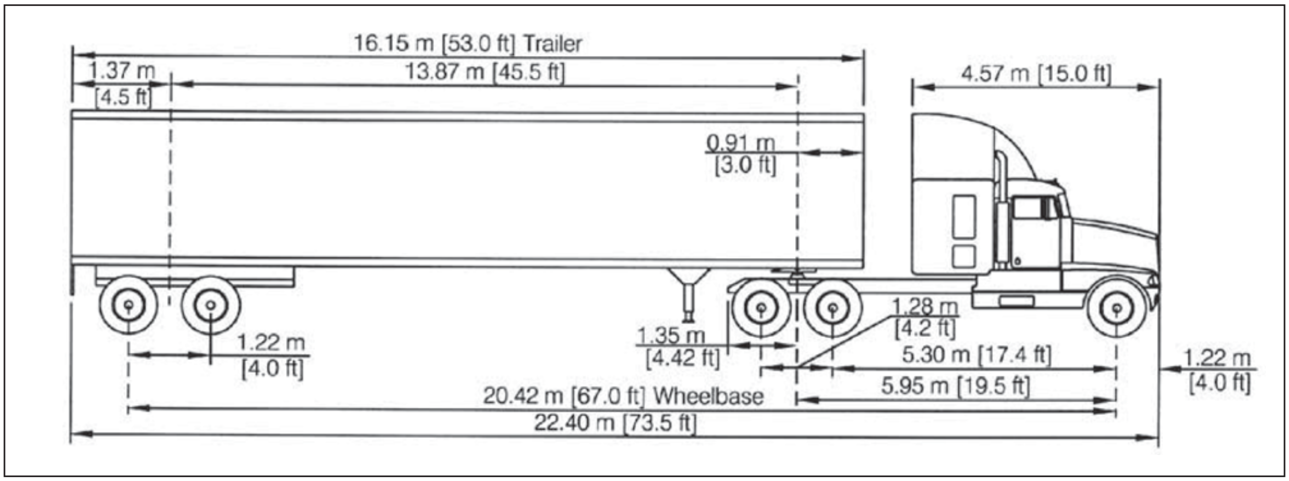
- That the then "current WB-15 [WB-50] design vehicle be dropped because it [was] no longer common on U.S. roads."
- "The kingpin-to-center-of-rear-tandem distance for the WB-19 [WB-62] design vehicle be increased from 12.3 to 12.5 m [40.5 to 41 ft]."
- "The WB-20 [WB-65] design vehicle should be dropped from the *Green Book* and the WB-20 [WB-67] design vehicle" (shown in Figure 1a) used in its place.
- "A three-axle truck, the SU-8 [SU-25] design vehicle" (shown in Figure 1b), "and a Rocky Mountain Double, the WB-28D [WB-92D] design vehicle" (shown in Figure 1c) be added to the *Green Book.*

The researchers did not identify a need to update the *Green Book* design criteria for sight distance, lane width, horizontal curves, cross-slope breaks, or vertical clearance to better accommodate trucks. In each case, their evaluation determined that the current geometric design criteria could reasonably accommodate trucks. The research did develop "a spreadsheet program, known as the truck speed profile model, [designed to] estimate the truck speed profile on any specified upgrade, considering any truck weight/power ratio, any initial truck speed, and any vertical profile. Field studies were also conducted to better quantify the weight/power ratios of the current truck fleet; the results of [those] field studies [indicated] that trucks in western states have better performance than in eastern states and the truck population on freeways generally has better performance than the truck population on two-lane highways."

Easa and El Halim (2006) conducted research to establish minimum radius requirements on the basis of vehicle stability for trucks on three-dimensional (3-D) reverse horizontal curves with intermediate tangents. With vehicle simulation software, vehicle dynamics were recorded for the base case of two-dimensional (2-D) simple curves and for reverse curves superimposed with different vertical alignments (upgrade, downgrade, crest curve, and sag curve). They conducted simulation for two maximum superelevation rates, three design vehicles, and different vertical grades. Two mathematical models were developed for flat and 3-D reverse curves. The models provided the minimum radius of the sharper arc of the reverse curve as a function of design speed, maximum superelevation, ratio of flatter to sharper curve radius, design vehicle, and intermediate tangent length. Their results indicated that an increase in the minimum radius of existing design guides (between 5% and 27%) was required to compensate for the effects of reverse curvature and vertical alignment and maintain the same comfort level specified in the design guides. They concluded that the required increase could be reduced by using longer intermediate tangents, and they presented design requirements for the spiral length of reverse curves.

DESIGN SPEED

Fitzpatrick and Carlson (2002) reviewed current practices for selection of design speed after the release of the 2001 *Green Book* and its revised definition of design speed. They found that practices varied widely, including the use of functional classification, consideration of location (i.e., rural or urban), terrain, *Green Book* procedure, legal speed limit (possibly with a value of 5 or 10 mph added), anticipated volume and/or operating speed, adjacent development, costs, and design



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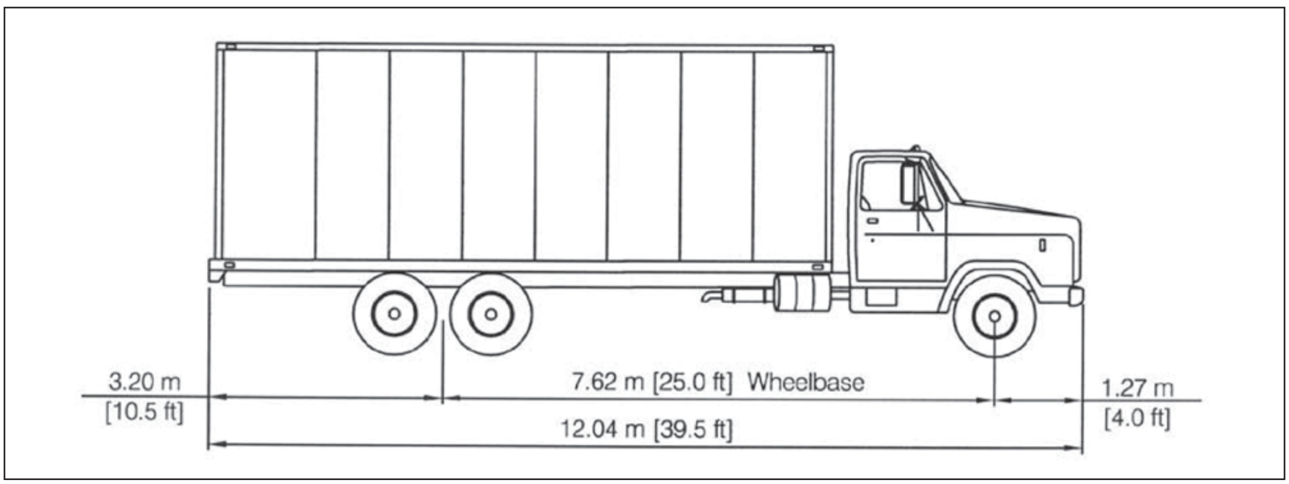

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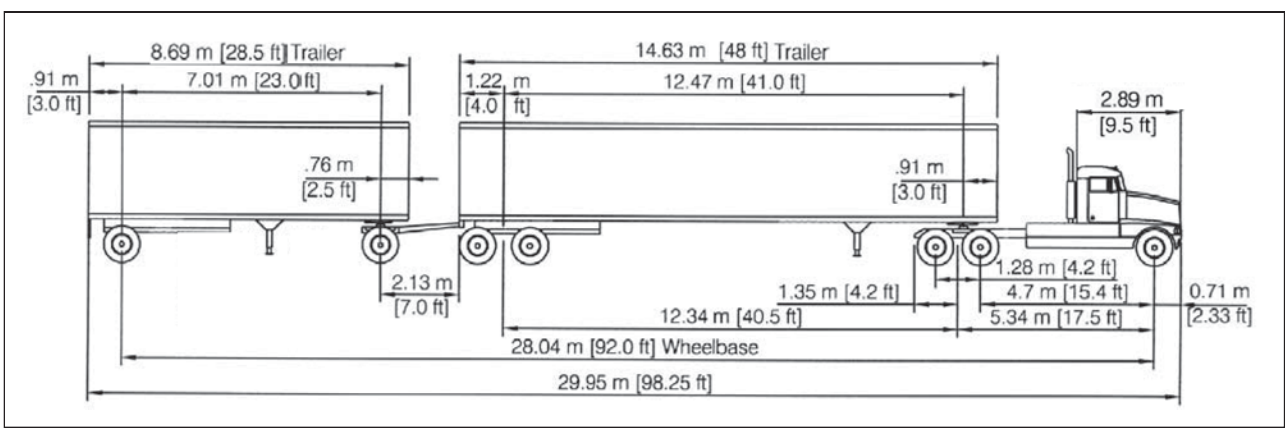


(c)

FIGURE 1 Dimensions of recommended design vehicles: (a) Interstate semitrailer [WB-20 (WB-67)] design vehicle, (b) three-axle single-unit [SU-8 (SU-25)] design vehicle, (c) Rocky Mountain double combination [WB-28D (WB-92D)] design vehicle (Harwood et al. 2003b).

consistency. They found that as many as half of the states surveyed used posted speed or operating speed in their considerations, although the *Green Book* process did not explicitly include them. Techniques they recommended for future revisions to the design speed selection process included:

- Consideration of anticipated posted or operating speed;
- A feedback loop;
- Modifying values recommended for different functional classes, rural versus urban, or terrain; and
- Explicit consideration of tangent length as a design element.

Under NCHRP Project 15-18 (Fitzpatrick et al. 2003a), "the Texas Transportation Institute compiled and analyzed industry definitions for speed-related terms and recommended more consistent definitions for the *Green Book* and the *MUTCD*. The researchers surveyed state and local practices for establishing design speeds and speed limits and synthesized information on the relationships between speed, geometric design elements, and highway operations. Next, researchers critically reviewed geometric design elements to determine if they should be based on speed and identified alternative [design-element] selection criteria. Geometric, traffic, and speed data were collected at numerous sites around the United States and analyzed to identify relationships between the various factors and speeds on urban and suburban sections away from signals, stop signs, and horizontal curves (all elements previously found to affect operating speeds)."

The work of the NCHRP 15-18 team was documented in *NCHRP Report 504* (Fitzpatrick et al. 2003a) In addition to including the survey of practice and information on the relationships between speed and various geometric and traffic factors, the report lists suggested refinements to the *Green Book* in the following areas: design speed definitions, information on posted speed and its relationship with operating speed and design speed, how design speed values were selected in the United States (noting that anticipated posted speed and anticipated operating speed were also used in addition to the process in the then-current edition of the *Green Book,* which is based on terrain, functional class, and rural versus urban), changes to functional class material, and additional discussion on speed prediction and feedback loops. Among the findings documented in *NCHRP Report 504* are the following:

- The "strongest relationship found in NCHRP Project 15-18 was between operating speed and posted speed limit. No other roadway variable [including design speed] was statistically significant at a 5 percent alpha level."
- "Design speed [appeared] to have minimal impact on operating speeds unless a tight horizontal radius or a low K-value [was] present. Large variance in operating speed was found for a given inferred design speed on rural two-lane highways."
- Other notable relationships between operating speed and roadway variables were identified as follows:
  - Access density showed a strong relationship with 85th percentile speed, with higher speeds being associated with lower access densities.
  - Lower speeds occurred as pedestrian activity increased.
  - The absence of either centerline or edgeline markings was associated with lower speeds.
  - Speeds were lower where on-street parking was permitted.
  - When no median was present, speeds were slightly lower than when a raised, depressed, or two-way left-turn lanes (TWLTL) median was present, with a few exceptions.
  - There was no evidence that the presence of curb and gutter resulted in lower speeds for a facility.
- Results from a mailout survey indicated that most states used *Green Book* definitions in the design of roadways, "but far fewer respondents indicated that it was their preferred definition."
- Most design elements and their values were either directly or indirectly selected based on design speed. In several situations, the type of roadway was used to determine the design element value or feature; however, the type of roadway was strongly associated with the operating speed of the facility.
- The relationship with operating speed was identified for several design elements. In some cases, such as for horizontal curves, the relationship was strong, and in other cases, such as for lane width, the relationship was weak. In all cases when a relationship between the design element and operation speed existed there were ranges when the influence of the design element on speed was minimal.
- "While the relationship between a design element and operating speed may be weak, the consequences of selecting a particular value may have safety implications. A safety review [indicated] that there [were] known relationships between safety and design [features] and that the selection of the design feature [varied] based on the operating speed of the facility. Therefore, the design elements investigated within this study should be selected with some consideration of the anticipated operating speed of the facility. In some cases the consideration would take the form of selecting a design element value within a range that has minimal influence on operating speed or that would not adversely affect safety, while in other cases the selection of a design element value would be directly related to the anticipated operating speed."

Based on their findings, researchers recommended the following changes to the *Green Book* in *NCHRP Report 504:*

- "Add discussion on posted speed limit to encourage a better understanding of the relationship between 85th percentile speed and posted speed limit (i.e., posted

speed limits [were] generally set between 4 and 8 mph less than the measured 85th percentile speed, and only 23% to 64% of vehicles operated below the posted speed limit in urban areas in field studies)."
- "Change text to recognize freeways as a unique functional class. Encourage the recognition that the look of a roadway (e.g., ramps, wide shoulders, and medians) is associated with the anticipated speeds on the facility."
- "Add comments in the design speed discussion to identify that the following may affect operating speed: radius, grade, access density, median presence, on-street parking, pedestrian activity, and signal density."
- "Add information on the state of the practice for selecting design speed values, [because] anticipated operating speed and anticipated posted speed limit [were] being used by a notable percentage of the states [surveyed]."
- "Introduce the concept of speed prediction and feedback loops, [with] reference to FHWA-[sponsored] work on the [Interactive Highway Safety Design Model] IHSDM."

Garrick and Wang (2005) examined context-based alternatives to the use of design speed as a controlling criterion for design of streets and highways. They concluded that there were two main areas of concern—how to better define context and how to design for appropriate operations (including speed)—that must be addressed in developing a more coherent and context-based approach to design. They discussed the need for an overarching design framework that integrates all facets required for good design; their framework consisted of a four-step process: define the context, characterize the function, select the road typology, and determine the design details. They believed that this framework would be essential when designing truly context-based thoroughfares that facilitate the operational and safety issues of all users and that also address the issues of context and livability that affect how well streets or roads function as places.

Wang et al. (2006) investigated the relationship between the speed choices of drivers and their associated low-speed (e.g., speed limits ranging from 30 to 40 mph) urban roadway environments by analyzing second-by-second in-vehicle global positioning system data from more than 200 randomly selected vehicles in Georgia. The authors developed operating-speed models for low-speed urban street segments on the bases of roadway alignment, cross-section characteristics, roadside features, and adjacent land uses; their goal was that the model could "help highway designers and planners better understand expected operating speeds when they design and evaluate low-speed urban roadways." The authors concluded that the following variables were significant at the 95th percentile: number of lanes, the density and offsets of roadside objects, the density of T-intersections and driveways, raised curb presence, sidewalk presence, on-street parking, and land uses. They suggested that the posted speed limit not be included in the model because of its strong correlation to design speed, and that the posted speed limit was highly correlated with the intercept variable and the number of lanes variable in their model. In addition, they found that several significant variables in their tangent model became statistically insignificant when posted speed limit was included. Their major findings included the following:

- The number of lanes per direction of travel had the most significant influence on drivers' speeds at tangent locations.
- On-street parking and sidewalks were "the second and third [most] significant variables that [affected] drivers' speeds on tangent" sections of low-speed urban streets.
- Drivers selected lower speeds with an increase in the density of trees or utility poles, or with a decrease in their offsets.
- Drivers tended to select lower speeds with an increase in density of driveways or T-intersections.

Donnell et al. (2009) employed another term for practitioners to consider, "referred to as 'inferred design speed.' Inferred design speed is applicable only to features and elements that have a criterion based on [a] designated design speed (e.g., vertical curvature, sight distance, superelevation)." The inferred design speed of a feature will be different from the designated design speed when the actual value is different from the criterion-limiting (minimum or maximum) value. For example, the inferred design speed for a combination of radius and superelevation is the maximum speed for which the limiting speed-based side friction value is not exceeded for the designed rate of superelevation and the inferred design speed; as such, it is determined through an iterative process. The inferred design speed for a horizontal curve may also be limited by horizontal offsets to sight obstructions on the inside of a horizontal curve. The inferred design speed for a crest vertical curve is the maximum speed for which the available stopping sight distance (SSD) is not exceeded by the required SSD. The inferred design speed may also be limited by a combination of lane width and average daily traffic (ADT). The inferred design speed can be greater than, equal to, or less than the designated design speed.

DESIGN CONSISTENCY

Under NCHRP Project 15-17, Wooldridge et al. (2003) "reviewed the domestic and international literature on geometric design consistency and developed a comprehensive list of geometric design features for high-speed, rural, two-lane roads that can reduce geometric consistency or violate driver expectancy. They then identified the most critical roadway features or combinations of features and considered how they might affect driver performance. A data collection and analysis plan was developed to formulate relationships between key parameters of the features and driver performance." As part of the research, the team recommended a definition for design consistency: "Design consistency is the conformance of a highway's geometric and operational features with driver expectancy."

The 15-17 project team then developed a set of rules for evaluating the design consistency of selected conditions. Following their evaluation of several case studies, they developed a list of data needs for future evaluations of selected design elements, as follows:

- Cross section,
- Horizontal alignment,
- Vertical alignment,
- Railroad grade crossings,
- Narrow bridges,
- Driveways,
- Preview sight distance,
- Climbing and passing lanes, and
- Frequency of decisions

The data needed to evaluate a roadway design using the developed design consistency rules largely consisted of information the researchers deemed to be readily available to the designer, through field measurements and speed models. In some cases, however, additional information may be necessary to evaluate older alignments. All of the rules, data needs, and the research team's related recommendations for revisions to the 2001 *Green Book* are summarized in *NCHRP Report 502* (Wooldridge et al. 2003).

Cafiso et al. (2005) developed a model based on fuzzy logic techniques to classify roadway elements by using three safety criteria (design consistency, operating speed consistency, and driving dynamics) to obtain a more careful evaluation of inconsistencies between highway design elements for redesigns, 3R projects, and existing alignments. For each criterion, the inconsistencies were included in three fuzzy sets (good, fair, poor), with differing degrees of membership. By defining linear membership functions, the researchers classified road sections and then determined a prioritization scale of maintenance interventions. Their procedure was intended to be applied to large databases of road networks to identify the more dangerous design elements that need interventions to improve highway safety and to allocate resources under limited budget conditions.

DRIVER CHARACTERISTICS

NCHRP's *Human Factors Guidelines for Road Systems* (*HFG*) (Campbell et al. 2008) states that designers and traffic engineers need to examine the roadway environment for information conflicts that may mislead or confuse road users. They must anticipate what information the road user requires and where it is needed so that appropriate design elements or traffic control can be integrated into the design and operational plans. Missing information is not helpful to the road user. As stated another way in the report, designers and traffic engineers must also seek road environments that are self-explaining, quickly understood, and easy for users to act upon.

The *HFG* recommends that the highway designer and the traffic engineer examine the road environment in incremental steps similar to those steps taken by a road user to ensure that the user will not be overloaded with temporal tasks and decisions. In short, good human factor principles must be integrated into the design of the road system. The sizes of the iterative and incremental steps are not going to be the same for all road environments, and they will vary depending on the road user, the type of highway, the operations, and the environment. The iterative steps, however, must overlap from one section to the next to ensure continuity of the travel path and that no potentially meaningful information for road users will be overlooked. Highway designers and traffic engineers must jointly examine the road environment; that is, lane alignment (roadway and intersections), signing (advisory, regulatory, and guidance), and operations (normal and work zones) relative to the likelihood users will be able to perform the required tasks safely and efficiently within the time and space available. The *HFG* discusses these elements primarily in terms of the driver, but similar principles are also discussed in relation to the nonmotorized road user. No specific recommendations were given for changes in *Green Book* methodology, but the *HFG* provides additional guidance based on empirical data and expert judgment.

WORK ZONE CONSIDERATIONS

NCHRP Report 581 (Mahoney et al. 2004) discusses the procedure for establishing an appropriate design speed for work zones, which they define as "a selected speed used to determine [specific work zone] geometric design features." A value equal to or slightly greater than the target speed (i.e., the desirable free-flow operating speed) is appropriate for work zone design speed. In the report, work zone design speed is applicable to radius of curvature and superelevation and, when the work zone design speed is less than 40 mph, it is also used to determine appropriate sight distance. Work zone design speed may also be used in computing the minimum length of sag vertical curves. Other speed parameters (e.g., speed limit and anticipated 85th percentile speed) are also referenced in some design guidelines. The authors conclude that the establishment of a target speed and work zone design speed, design of temporary traffic control, and potential selection of speed management measures are related. It is important that speed-related decisions within specific domains (i.e., design, regulatory, and speed management) be consistent with an overall strategy.

SUMMARY OF KEY FINDINGS

This section summarizes key findings from this chapter. This is an annotated summary; conclusions and recommendations are those of the authors of the references cited.

Design Vehicles

- Dimensions of commonly used trucks have changed in recent years, prompting recommendations to revise the dimensions of those vehicles in the *Green Book* (Harwood et al. 2003b).
- Along with the changes in dimensions have come changes in performance; however, research indicated that design criteria for sight distance, lane width, horizontal curves, cross-slope breaks, and vertical clearance were sufficient to accommodate the performance of trucks (Harwood et al. 2003b).

Design Speed

- A review of design speed practices indicated that posted speed limit and anticipated operating speed were frequently associated with the selection of design speed (Fitzpatrick and Carlson 2002).
- Observation of driving behavior revealed that the strongest indicator of operating speed was posted speed limit. Design speed appeared to have minimal impact on operating speeds unless a tight horizontal radius or a low K-value was present (Fitzpatrick et al. 2003a).
- Multiple studies examined the possibility of selecting a design speed based more heavily on the context of the environment in which the roadway was located. A primary area of concern, however, was how to define the context to be considered (Garrick and Wang 2005; Wang et al. 2006).

Driver Characteristics

- A study of human factors related to the driving task suggested that designers and traffic engineers must examine the roadway environment for information conflicts that may mislead or confuse road users (Campbell et al. 2008).
- The study concluded that designers and traffic engineers must also seek road environments that are self-explaining, quickly understood, and easy for users to act up (Campbell et al. 2008).

CHAPTER THREE

ELEMENTS OF DESIGN

OVERVIEW

Research during the decade led to new recommendations for SSD, which were later analyzed for probability of hazard (POH). An updated look at passing sight distance (PSD) compared *Green Book* guidelines with *MUTCD* guidelines for passing zone markings. With the increasing availability of appropriate technological design aids, a new emphasis on 3-D modeling was promoted to consider the interactions between horizontal and vertical alignments and their effects on the driver. Additional methods for 2-D analyses were also investigated. Researchers also revisited truck performance on crest curves and headlight performance on sag curves to compare current traffic characteristics with existing guidelines. Estimated safety benefits of selected design elements are also discussed in this chapter, based on the material contained in the AASHTO *Highway Safety Manual* (*HSM*); this chapter does not contain a comprehensive reproduction of the *HSM* guidance, but does provide examples. Readers desiring to obtain full details on the safety effects of various design elements and treatments, including the appropriate methodology for the proper application of *HSM* guidance, should consult that document.

SIGHT DISTANCE

Stopping Sight Distance

Fambro et al. (2000) developed a SSD model to update the values used in the then-current 1994 *Green Book*. A comparison of the existing SSD model with those used by other countries showed that AASHTO's SSD values and vertical curve lengths were longer than those used in most other countries. The researchers conducted field studies involving more than 50 drivers, 3,000 braking maneuvers, and 1,000 driver eye heights. Field tests were conducted under a variety of geometric, weather, and surprise conditions; under closed-course and open-roadway conditions; and with and without antilock braking systems. From the results of those field studies, they determined that 2.5 s was the 90th percentile value for perception–reaction time (PRT) and that 3.4 m/s^2 (11.2 ft/s^2) was the 10th percentile deceleration rate. In addition, they identified 1080 mm (3.5 ft) as the 10th percentile driver eye height and 600 mm (2.0 ft) as the 10th percentile object height. Using these as design values, they recommended revised SSDs for design as shown in Table 1. Based on those distances, the authors also recommended new design controls for vertical curves, reproduced in Table 2.

Wang (2007) analyzed the placement design of ramp control signals in relation to the satisfaction of a driver's comfortable cone of vision for stopped vehicles at the stop line and the satisfaction of SSD for approaching vehicles in accordance with *MUTCD*. He derived relationships between location of stop line, location of signal standard, ramp geometry, and approaching speeds, and he developed sample lookup design charts to facilitate the development and evaluation of signal placement design. A brief analysis using these relationships concluded that signal standards placed in alignment with the stop line would violate not only the comfortable cone of vision of stopped drivers but also the SSD of approaching vehicles. He concluded that for a loop on-ramp with a 300-ft radius, standard-mounted signals on the left side of the ramp should be placed at least 22 ft downstream of the stop line to satisfy the requirements of both the stopped and approaching vehicles. In contrast, he added, signals on the right side of the loop ramp could satisfy only the stopped vehicles but not the approaching vehicles if placed at least 44 ft downstream of the stop line. Therefore, for a loop ramp with a smaller radius (approximately 300 ft or less), two signal indications are needed to satisfy the *MUTCD*'s requirements, with one at the left side of the ramp curve to provide sufficient sight distance for the approaching vehicles and one at the right side of the ramp curve to provide sufficient viewing angle for the stopped vehicles. He added that signals placed on the left side of the on-ramp curve of a loop ramp (even with a radius greater than 300 ft) are more critical than those on the right side, especially when the approaching SSD is important.

Sarhan and Hassan (2008) sought to develop a reliability-based probabilistic approach that was well suited to replace deterministic highway design practice. In their study, reliability analysis was used to estimate the POH that might result from insufficiency of SSD. As an application, they checked the available sight distance against the required SSD on an assumed road segment. Variation of the design parameters was addressed with Monte Carlo simulation using 100,000 sets of design parameters based on distributions available in the literature. They also developed a computer program to use these sets of design parameters to calculate the profiles of available and required SSD in 2- and 3-D projections as well as the profile of POH. They applied their approach to a horizontal curve with 100-km/h (62-mph) design speed overlapping with flat grade, crest curves, and sag curves in a cut section where the side slope would restrict the sightline.

TABLE 1
RECOMMENDED STOPPING SIGHT DISTANCES FOR DESIGN

Initial Speed		Perception-Brake Reaction			Deceleration		Braking Distance		Stopping Sight Distance for Design	
			Distance							
km/h	mph	Time, s	m	ft	m/s^2	ft/s^2	m	ft	m	ft
30	18.6	2.5	20.8	68.2	3.4	11.2	10.2	33.5	31.0	101.7
40	24.9	2.5	27.8	91.2	3.4	11.2	18.2	59.7	45.9	150.6
50	31.1	2.5	34.7	113.8	3.4	11.2	28.4	93.2	63.1	207.0
60	37.3	2.5	41.7	136.8	3.4	11.2	40.8	133.9	82.5	270.7
70	43.5	2.5	48.6	159.4	3.4	11.2	55.6	182.4	104.2	341.9
80	49.7	2.5	55.6	182.4	3.4	11.2	72.6	238.2	128.2	420.6
90	55.9	2.5	62.5	205.1	3.4	11.2	91.9	301.5	154.4	506.6
100	62.1	2.5	69.4	227.7	3.4	11.2	113.5	372.4	182.9	600.1
110	68.4	2.5	76.4	250.7	3.4	11.2	137.3	450.5	213.7	701.1
120	74.6	2.5	83.3	273.3	3.4	11.2	163.4	536.1	246.7	809.4

Source: Fambro et al. (2000).

They determined that their analysis showed that the current deterministic approach yielded very conservative estimates of available and required SSD, resulting in very low POH (0.302%). An application example also showed the change of POH with the change of vertical alignment parameters. They concluded that although changes in vertical alignment caused a significant change in POH in relative terms, the absolute value of POH remained low, indicating that current design practice may be uneconomical. However, the significance of the different values of POH in terms of safety implications remains a subject for further investigation.

Passing Sight Distance

Carlson et al. (2005) investigated characteristics of daytime high-speed passing maneuvers along a straight and flat 15-mi section of a rural two-lane, two-way highway. The posted speed limit on this highway in Texas was 70 mph, and the researchers recorded characteristics of passing maneuvers from their own vehicle, which was driven at speeds of 55, 60, and 65 mph to encourage passing by adjacent drivers. They recorded 105 single-vehicle daytime passing maneuvers, and they developed speed profiles of the passing vehicles for each of the three studied speeds. The researchers then compared their findings with AASHTO's assumptions and criteria for minimum PSD for two-lane, two-way highways. In particular, their analysis focused on the elements associated with a passing vehicle while it occupied the opposing lane of travel. The specific elements that were studied included average passing speed, speed differential between passing and passed vehicles, distance traveled while making the pass, and total elapsed time. Their general findings provided support for the AASHTO PSD model, and the researchers concluded that the model provided reasonable results for the assumptions made. However, they added, the assumptions may need to be updated or have more flexibility added. For instance, for a 70 mph design speed, the assumed speed of the overtaken vehicle was 54 mph in the AASHTO PSD model, which was verified in this study. However, they also concluded that the then-current AASHTO PSD model would provide inadequate PSD values for speeds of overtaken vehicles that were greater than those assumed (e.g., 60 or 65 mph).

Under NCHRP Project 15-26, Harwood et al. (2008) evaluated current methods for determining minimum PSD requirements. Based on their results, the research team assessed the guidance on PSD provided in the *Green Book* and the *MUTCD*. The assessment considered safety concerns on two-lane high-

TABLE 2
RECOMMENDED DESIGN CONTROLS FOR VERTICAL CURVES

Initial Speed		Stopping Sight Distance for Design		Rate of Vertical Curvature, *K* (length per % of algebraic difference in grade A)			
				Crest Curves		Sag Curves	
km/h	mph	m	ft	m	ft	m	ft
30	18.6	31.0	101.7	2	6.6	5	16.4
40	24.9	45.9	150.6	4	13.1	8	26.2
50	31.1	63.1	207.0	7	23.0	12	39.4
60	37.3	82.5	270.7	11	36.1	17	55.8
70	43.5	104.2	341.9	17	55.8	23	75.5
80	49.7	128.2	420.6	25	82.0	29	95.1
90	55.9	154.4	506.6	37	121.4	37	121.4
100	62.1	182.9	600.1	51	167.3	45	147.6
110	68.4	213.7	701.1	70	229.7	53	173.9
120	74.6	246.7	809.4	93	305.1	62	203.4

Source: Fambro et al. (2000).

TABLE 3
PASSING SIGHT DISTANCE FOR DESIGN OF TWO-LANE HIGHWAYS

Metric				U.S. Customary			
Design Speed (km/h)	Assumed Speed of Passed Vehicle (km/h)	Assumed Speed of Passing Vehicle (km/h)	Passing Sight Distance (m)	Design Speed (mph)	Assumed Speed of Passed Vehicle (mph)	Assumed Speed of Passing Vehicle (mph)	Passing Sight Distance (ft)
30	11	30	120	20	8	20	400
40	21	40	140	25	13	25	450
50	31	50	160	30	18	30	500
60	41	60	180	35	23	35	550
70	51	70	210	40	28	40	600
80	61	80	245	45	33	45	700
90	71	90	280	50	38	50	800
100	81	100	320	55	43	55	900
110	91	110	355	60	48	60	1,000
120	101	120	395	65	53	65	1,100
130	111	130	440	70	58	70	1,200
				75	63	75	1,300
				80	68	80	1,400

Source: Harwood et al. (2008).

ways, driver behavior, and the possible influence of longer trucks and older drivers. The findings of the research, documented in *NCHRP Report 605,* presented recommendations to bring consistency between PSD design standards and pavement marking practices. Researchers recommended that the *MUTCD* PSD criteria used for marking of passing and no-passing zones on two-lane roads be used in PSD design in the AASHTO *Green Book.* The support for the decision was that, in addition to providing the desired consistency between PSD design and marking practices, the research found that two-lane highways could be designed to operate safely with the *MUTCD* criteria. The researchers added that the longer PSD criteria presented in the then-current edition of the *Green Book* might provide improved traffic operational efficiency, but they were often considered to be impractical. Their recommendations for PSD design values are presented in Table 3.

HORIZONTAL ALIGNMENT

Bidulka et al. (2002) investigated the effect of overlapping vertical alignment on the horizontal curvature perceived by the driver. Initially, their hypothesis was "that overlapping crest curves made the horizontal curvature appear sharper and overlapping sag curves made the horizontal curvature appear less sharp." Researchers noted those drivers' responses to both static and dynamic computer-generated 3-D images of roadways indicated that this hypothesis was valid and that it was more evident in the case of sag curves. They concluded that erroneous perceptions, as influenced by vertical curves, increased as (1) the sight distance increased, (2) the horizontal curve radius increased, and (3) the length of vertical curve per 1% change in grade decreased. They reported that driver characteristics did not appear to affect the horizontal curve perception.

Hassan et al. (2002) continued the experiment in an attempt to quantify the extent of the driver's erroneous perception. Drivers were shown an image of a horizontal curve overlapping a vertical curve (test curve) and a number of horizontal curves overlapping flat vertical grades and with different radii (reference curves), as shown in Figure 2. The driver was then asked to state which reference curve had a radius closest to that of the test curve. Analysis of drivers' responses led the researchers to conclude that drivers would drive faster on horizontal curves in sag combinations and slower on horizontal curves in crest combinations. They added that the approach tangent, generally a downgrade in sag combinations and an upgrade in crest combinations, would further encourage the tendency to drive faster on sag combinations and slower on crest combinations. They recommended that designers establish the profile and predicted operating speed of an alignment based on a 3-D model, rather than a traditional 2-D model.

Lamm et al. (2002) developed a process to evaluate the safety of horizontal alignment on two-lane rural roads. The primary parameter in their methodology was the change in curvature rate of a given curve, which they tested against several databases of accident rates and accident cost rates and found to be a major descriptor of safety. They developed quantitative ranges for three safety criteria (design consistency, operating speed consistency, and driving dynamic consistency) and associated them with design classes for good, fair, and poor practices with respect to accidents. They concluded that the use of their methodology would allow designers to predict the potential accident risks and safety problems of a particular alignment and make changes to remedy them or develop countermeasures to mitigate them.

Schurr et al. (2002) studied circular horizontal curves on rural two-lane highways in Nebraska to determine relationships among design speed, operating speed, and posted speeds for developing horizontal alignment design guidelines. They found that for drivers at their study sites [curves

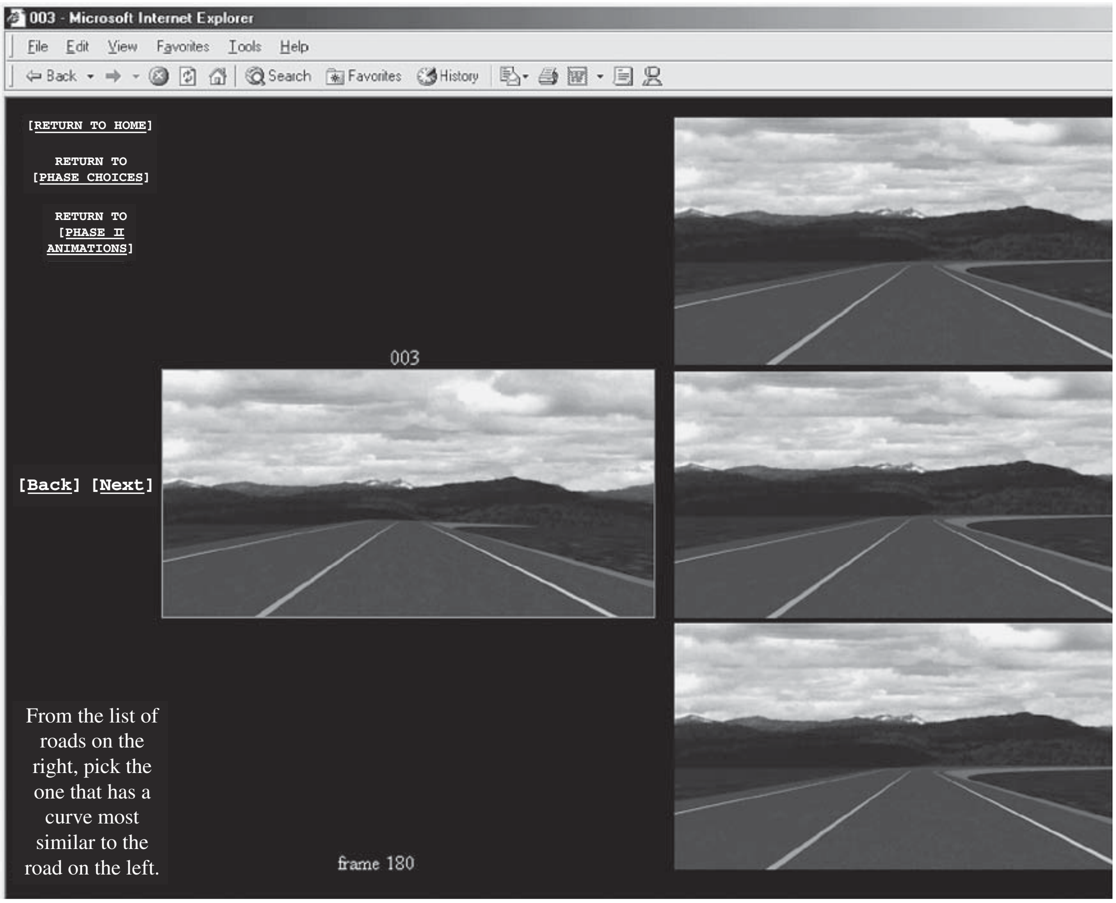


FIGURE 2 Presentation of horizontal curve images for comparison (Hassan et al. 2002).

with radii greater than or equal to 350 m (1,146 ft)], as the deflection angle increased, speed measures (mean, 85th percentile, and 95th percentile) decreased. They concluded from this finding that motorists may view a large change in direction as a motivation to slow their speed. They also found that as curve length increased, their speed measures increased, leading them to conclude that drivers were motivated to increase their speed as a curve lengthens, suggesting they may become more comfortable at higher speeds because they have more time to adjust their vehicle path to a constant radius. They also concluded that grade has an influence on the upper-percentage range of vehicle speeds, because the 85th percentile speed decreased as approach grade increased at their study sites. Finally, they found that as ADT increased, 95th percentile speed decreased; the authors speculated that roadways with higher ADT values may be perceived by drivers as having a higher likelihood of speed enforcement. The report contains their recommendation for using a series of equations to estimate mean, 85th percentile, and 95th percentile operating speeds at approach locations and midpoints of horizontal curves in Nebraska or curves with similar characteristics. Those equations are shown in Table 4.

Schurr et al. (2005) also developed a model to describe design speed profiles of vehicles traversing horizontal curves on approaches to stop-controlled intersections on two-lane

TABLE 4
EQUATIONS FOR ESTIMATING OPERATING SPEEDS AT HORIZONTAL CURVES

Speed	Approach Location	Midpoint of Curve
Mean	$V = 51.7 + 0.508\ V_p$	$V = 67.4 - 0.1126\Delta + 0.02243\ L + 0.276\ V_p$
85th Percentile	$V = 70.2 + 0.434\ V_p - 0.001307\ T_{ADT}$	$V = 103.3 - 0.1253\Delta + 0.0238\ L + 1.039\ G_1$
95th Percentile	$V = 84.4 + 0.352V_p - 0.001399\ T_{ADT}$	$V = 113.9 - 0.122\Delta + 0.0178\ L + 0.00184\ T_{ADT}$

Source: Schurr et al. (2002).
Notes:
V = speed (km/h) of free-flow passenger cars at that location; V_p = posted speed limit (km/h); T_{ADT} = traffic volume (vehicles/day); Δ = deflection angle (decimal degrees); L = arc length of curve (m); G_1 = approach grade (percent).

two-way rural highways. They used the model to create a procedure for designing horizontal curves that would accommodate vehicles transitioning from high speeds to a stop. Based on speed profile data from 15 study sites in Nebraska, the researchers concluded that posted speed, median type, presence of rumble bars, roadway surface condition, and degree of rutting did not significantly affect the vehicle speed profiles at these sites at a 95% confidence level. They also concluded that the intercepts of the regression lines for approaches with and without horizontal curves were significantly different in the case of heavy vehicles. The speed of heavy vehicles on tangent approaches was generally about 8 mph higher than on sites that exhibited horizontal curvature, although the rate of deceleration remained almost the same until vehicles were near the stop. Passenger cars exhibited no statistically significant difference between curved and tangent alignments. Researchers used the results of the study to develop a procedure for determining the minimum curve radius appropriate for a roadway alignment approaching a stop ensuring that (1) the visual expectations of the driver were met, (2) the comfort of the passengers within the vehicle was optimized, (3) the curve design used a simple curve with no spirals, (4) the vehicle speed within the limits of the curve were reasonable, (5) sufficient braking distance to the stop was available, and (6) deceleration rates were reasonable.

Cafiso et al. (2005b) sought to determine design inconsistencies on existing two-lane rural roads in Italy with the use of actual driving behavior and to verify their agreement with a consistency evaluation model. They developed a data collection method using a sample of test drivers operating an instrumented vehicle on a pre-determined route. From this study they concluded the following:

- A coordinate sequence of curves did not produce an unexpected driving event even if short bending radii were adopted.
- Geometric inconsistency produced by a sharp curve following a long tangent produced tense driving behavior, as observed on curves with radii of 120 m (394 ft) and 80 m (262 ft).
- Driving inconsistencies were highlighted by high-speed gradients of about 2 m/s^2 (6.5 ft/s^2), transversal accelerations of 0.3 g, and local maximum curvatures of the car path higher than those required by horizontal alignment. These values of deceleration reached with a light braking action were higher than the 0.80 to 0.85 m/s^2 (2.62 to 2.79 ft/s^2) generally assumed with regard to driving behavior in speed profile diagrams.
- Maneuvers were caused by the driver's need to suddenly correct his or her driving behavior owing to an unexpected alignment and could produce a dangerous situation if bad pavement conditions or unexpected events occur.
- The lack of transition curves was also a contributing factor in geometric inconsistency.

Lyles and Taylor (2006) stated that, "historically, the horizontal curve is the most critical geometric design element that influences driver behavior and has the most potential for crashes." They added that "research has indicated that the average accident rate for horizontal curves is about three times the average accident rate for highway tangents and the average run-off-the-road crash rate for highway curves is about four times that of highway tangents." They stated that many curve-related crashes were the result of drivers approaching and entering the curve at a speed that was too fast for the alignment. A study of driver behavior and errors on a selection of horizontal curves led them to conclude the following:

- Drivers approaching curves routinely exceeded the posted speed limit as well as the posted advisory speed, where applicable.
- Drivers had more errors at curves where they had limited or no visibility of the curves when the traffic control devices (TCDs) were first visible.
- Drivers made more errors on horizontal curves that were adjacent to vertical curves, particularly crests that obscured a downstream horizontal curve.
- There were increased errors when curves were combined with other elements, especially intersections.

Many design standards recommend the use of spiral curves in the transition design. Perco (2006) conducted a study to evaluate effects of a long spiral transition on the driver's curve perception and safety. He analyzed driving paths on 12 transitions with and without spiral curves, and concluded that the results confirmed a negative effect of excessive spiral length on driver behavior. His analysis results showed that the most desirable spiral length, which offered advantages in comparison with a tangent-to-curve transition, was equal to the distance traveled during the steering time. He developed a model to estimate the desirable spiral length for transitions of sharp horizontal curves on two-lane rural roads, based on the data collected in three studies. Starting from the radius of the impending curve, the model calculated the desirable spiral length and provided a description of actual driver behavior, as observed in field surveys. Perco concluded that the choice of the spiral length based on this model was useful because the estimated length was consistent with the real distance traveled by the vehicle during the steering action, which ensured optimal operating conditions for drivers.

VERTICAL ALIGNMENT

Hassan (2004) described the development of two models to determine the required SSD on crest and sag vertical curves. By comparing profiles of available SSD and required SSD on examples of vertical curves, Hassan concluded that current North American design practices might yield segments of the vertical curve where the driver's view is constrained to a distance shorter than the required SSD. He developed new models based on longitudinal friction and on acceleration, then developed an alternative design procedure based on the models,

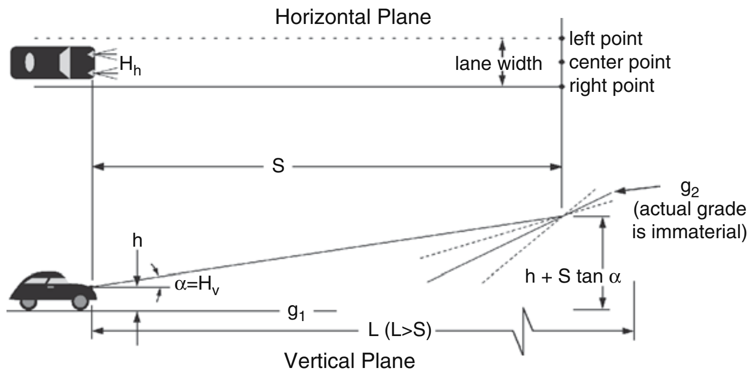


FIGURE 3 Illustration of headlamp analysis approach (Hawkins and Gogula 2008).

which he used to determine recommendations for minimum lengths of crest and sag vertical curves. Depending on the approach grade, the new values of minimum curve length could be greater than or less than values obtained through conventional design procedures; design aids were therefore provided in tabular form to facilitate use by designers.

Torbic et al. (2005) conducted a study to determine the distribution of truck weight/power ratios in the current truck fleet in several regions of the United States, and compare them with the 120 kg/kW (200 lb/hp) value recommended in the 2001 *Green Book.* The researchers collected data on truck crawl speeds at locations in California, Colorado, and Pennsylvania and concluded that a "weight/power ratio of 102 to 108 kg/kW (170 to 180 lb/hp) would be appropriate for freeways in California and Colorado, and a weight/power ratio of 126 kg/kW (210 lb/hp) would be more appropriate in Pennsylvania." They also determined that truck performance on two-lane highways was sufficiently different from freeways to recommend different ratios for those roads: a 108 kg/kW (180 lb/hp) design vehicle in Colorado, and 150 to 168 kg/kW (250 to 280 lb/hp) for California and Pennsylvania. According to the researchers, all of these ratios represented the 85th percentile of the truck population that was studied; therefore, most of the truck population performed substantially better.

Motivated by changes in headlamp design in recent decades, Hawkins and Gogula (2008) reviewed existing sag curve design criteria to determine if revisions to the design procedure were appropriate. They compared theoretical and field measurements of the levels of illuminance falling across the road surface, provided by sealed-beam and modern headlamps, as illustrated in Figure 3. The results of their analysis indicated that modern headlamps provided significantly less light above the horizontal than sealed-beam headlamps, indicating a potential need to modify the design equations for sag vertical curves. According to their theoretical analysis, the upward divergent headlamp angle used in the sag curve design equation should be reduced from 1° to between 0.75° and 0.90°. They stated that results from field analysis indicated a significant difference in illuminance levels from the theoretical analysis, but also indicated a need to reduce the headlamp angle used in sag curve design.

Easa (2008) developed a single-arc unsymmetrical vertical curve that takes the form of a cubic instead of parabolic function. The curve has a rate of change in grade that gradually varies between the start and end of the vertical curve, which eliminates the sudden change in curvature of traditional two-arc unsymmetrical vertical curves. He developed sight distance relationships for the new single-arc crest curve, which established the sight distance profile for the new curve and shows a substantial improvement over the abrupt-type sight distance profiles of two-arc curves. Included in the description of the single-arc curve characteristics are length requirements to satisfy AASHTO stopping, passing, and decision sight distance guidelines.

The *Highway Safety Manual* (AASHTO 2010) provides guidance on the effect of grades on expected safety of roadway segments. The base condition for grade is a generally level roadway. Table 5 presents the crash modification factors (CMFs) for grades based on an analysis of rural two-lane, two-way highway grades in Utah. The CMFs in the table are applied to each individual grade segment on the roadway being

TABLE 5
CRASH MODIFICATION FACTORS FOR GRADE OF ROADWAY SEGMENTS

Approximate Grade (%)	Level Grade (≤3%)	Moderate Terrain (3% < grade ≤ 6%)	Steep Terrain (>6%)
CMF	1.00	1.10	1.16

Source: AASHTO (2010).

evaluated without respect to the sign of the grade. The sign of the grade is irrelevant because each grade on a rural two-lane, two-way highway is an upgrade for one direction of travel and a downgrade for the other. The grade factors are applied to the entire grade from one point of vertical intersection to the next. The CMFs apply to total roadway segment crashes.

WORK ZONE CONSIDERATIONS

NCHRP Report 581 (Mahoney et al. 2004) discusses several elements of design and their relation to work zones. The authors state that although extended sight distances throughout work zones are desirable, the underlying need for decision sight distance (because of an unexpected or difficult-to-perceive information source or condition) should be avoided in designing construction work zones. Temporary traffic control and other driver information strategies are used in conjunction with extended sight distance to mitigate work zone conditions that are atypical or involve complex driver decisions. They concluded "that extended sight distance approaching and within work zones is desirable from an operations perspective. Safety issues also point to [the need for] some minimum sight distance." For work zone design speeds less than 40 mph, the SSD values tabulated in the *Green Book* and corresponding to work zone design speed were recommended. For work zone design speeds of 40 mph and greater, the *Green Book* design-speed-corresponding values did not necessarily represent the minimum values that could be accepted; a minimum sight distance of 300 ft was recommended using a driver eye height of 3.5 ft and an object height of 2.0 ft.

Maximum superelevation rates (e_{max}) are typically selected as a matter of policy rather than for specific projects. Absent other considerations, the e_{max} used for permanent roadways is appropriate for construction work zones. Superelevating roadway curves necessitates superelevation transitions, which bring alignment and other (e.g., drainage) complications. For these reasons, it is common design practice to provide curves that are sufficiently flat to not require the introduction of superelevation. Mahoney et al. (2004) discuss the use of Methods 2 and 5 from the *Green Book* for determining appropriate superelevation distributions.

NCHRP Report 581 (Mahoney et al. 2004) also states that, in general, the same maximum grade criteria applicable to the highway under construction should be applied to work zone roads. However, marginally exceeding these criteria is often justified in consideration of all factors. Grades below the maximum are desirable. When designing work zone temporary roadways, the potential effect of grades on operations and capacity should be considered. When speeds are substantially reduced in advance of a temporary roadway (e.g., in conjunction with a reduction in the number of lanes), the work zone capacity may be controlled by heavy vehicles attempting to accelerate on grade, which, in turn, influences queue formation. The authors found that the most common basis for agency work zone sight distance design criteria is the set of SSD values from the *Green Book*. For this case, minimum crest and sag vertical curve lengths are determined from Exhibits 3-71 and 3-74, respectively, in the 2004 *Green Book*.

SUMMARY OF KEY FINDINGS

This section summarizes key findings from the research noted in this chapter. This is an annotated summary; conclusions and recommendations are those of the authors of the references cited.

Stopping Sight Distance

- New values for SSD and new design controls for vertical curves were recommended, based on a PRT of 2.5 s, a 10th percentile deceleration rate of 11.2 ft/s^2, a 10th percentile driver eye height of 3.5 ft and a 10th percentile object height of 2.0 ft (Fambro et al. 2000).
- Ramp control signals placed on the left side of a curve of a loop on-ramp (even with a radius greater than 300 ft) are more critical for accommodating SSD than those on the right side (Wang 2007).
- The method of selecting SSD values deterministically yielded conservative estimates of available and required SSD, resulting in a very low probability (0.302%) of hazard (Sarhan and Hassan 2008).

Passing Sight Distance

- An analysis of observed passing maneuvers provided support for the AASHTO PSD model, and the model provided reasonable results for the assumptions made. However, the model's assumptions may need to be updated or accommodate more flexibility for speeds higher than 55 mph (Carlson et al. 2005).
- Increased consistency between AASHTO PSD design standards and *MUTCD* pavement marking practices was recommended, specifically accomplished by using the *MUTCD* criteria for marking passing/no-passing zones on two-lane roads in the *Green Book*'s PSD design process. In addition to providing the desired consistency between PSD design and marking practices, two-lane highways could be designed to operate safely with the *MUTCD* criteria (Harwood et al. 2008).

Horizontal Alignment

- Erroneous perceptions by drivers approaching horizontal curves, as influenced by vertical curves, increased as (1) the sight distance increased, (2) the horizontal curve radius increased, and (3) the length of vertical curve per 1% change in grade decreased. Drivers tend to drive faster on horizontal curves in sag combinations and slower on horizontal curves in crest combinations.

Designers can establish the profile and predicted operating speed of an alignment based on a 3-D model, rather than a traditional 2-D model (Bidulka et al. 2002; Hassan et al. 2002).

- For drivers on curves with radii greater than or equal to 350 m (1,146 ft), as the deflection angle increased, speed measures (mean, 85th percentile, and 95th percentile) decreased; as a result, motorists may view a large change in direction as a motivation to slow their speed. In addition, as curve length increased, speed measures increased, suggesting that drivers may become more comfortable at higher speeds because they have more time to adjust their vehicle path to a constant radius. Grade has an influence on the upper-percentage range of vehicle speeds, because the 85th percentile speed decreased as approach grade increased (Schurr et al. 2002).
- A study of driver behavior and errors on a selection of horizontal curves led Lyles and Taylor (2006) to conclude the following:
  - Where applicable, drivers approaching curves routinely exceeded the posted speed limit as well as the posted advisory speed.
  - Drivers had more errors at curves where they had limited or no visibility of the curves when the TCDs were first visible.
  - Drivers made more errors on horizontal curves that were adjacent to vertical curves, particularly crests that obscured a downstream horizontal curve.
  - There were increased errors when curves were combined with other elements, especially intersections.

Vertical Alignment

- Current North American design practices might yield segments of the vertical curve where the driver's view is constrained to a distance shorter than the required SSD. An alternative design procedure is recommended based on a new model that incorporated longitudinal friction and acceleration, which produced new recommended values for minimum lengths of crest and sag vertical curves (Hassan 2004).
- A weight/power ratio of 102 to 108 kg/kW (170 to 180 lb/hp) would be appropriate for freeways in California and Colorado, and a weight/power ratio of 126 kg/kW (210 lb/hp) would be more appropriate in Pennsylvania, as compared with the 120 kg/kW (200 lb/hp) value recommended in the 2001 *Green Book* (Torbic et al. 2005).
- The upward divergent headlamp angle used in the sag curve design equation should be reduced from 1° to between 0.75° and 0.90° (Hawkins and Gogula 2008).

CHAPTER FOUR

CROSS-SECTION ELEMENTS

OVERVIEW

Researchers investigated operational and safety characteristics of passing lanes on two-lane highways, which grew in popularity during the decade in an attempt to maximize performance without widening to traditional four-lane highways. At the same time, the concept of the "road diet" was introduced to reduce traffic speeds and provide space for pedestrian or bicycle amenities without widening the right-of-way. The effects of the width of travel lanes and TWLTLs were examined. Shoulder treatments such as rumble strips received attention for their potential in reducing crashes, and researchers looked at new median and roadside treatments.

ALLOCATION OF TRAVELED WAY WIDTH

An NCHRP-sponsored scan tour looked at characteristics of 2+1 roads in several European countries to determine the potential applications of the design for use in the United States (Potts and Harwood 2003). A 2+1 road design has a continuous three-lane cross section with alternating passing lanes. Although specifics of the designs in the respective countries varied somewhat, the authors made comparisons of some of the key design and operational criteria, which are summarized in Table 6.

The NCHRP authors concluded that the benefits of 2+1 roads in Europe validated a recommendation for their use in the United States, to serve as an intermediate treatment between an alignment with periodic passing lanes and a full four-lane alignment. They also recommended that 2+1 roads were most suitable for level and rolling terrain, with installations to be considered on roadways with traffic flow rates of no more than 1,200 veh/hr in a single direction. The authors discouraged the use of cable barrier as a separator, and they recommended that major intersections be located in the buffer or transition areas between opposing passing lanes, with the center lane used as a turning lane.

Gattis et al. (2006) reported on a study of passing lane operations in Arkansas. The focus was on segments of continuous three-lane cross sections with alternating passing lanes; for example, three-lane alternate passing or 2+1. They examined the effects of passing lane length on platooning, passing, speed, and passing lane crash rates. Five sets of field data were collected at four rural sites, and it was determined that platooning decreased and eventually stabilized after a vehicle entered the passing lane. They observed that passing activity was greatest at the beginning of the segments and the greatest benefits of decreased platooning and increased safety occurred within the first 0.9 mi of a passing lane segment. Speed patterns were found to vary among sites, but average speed rose when a vehicle entered the passing lane section. Their study of crash rates included "five years of crash data from 19 sites; [they concluded that] even though the volumes for the passing lane segments were higher than the state average volume for rural two-lane roads, the passing lane crash rates were generally lower than the statewide average crash rate for rural two-lane roads."

A study of similar roadways in Texas, called "Super 2" highways, found that passing lanes were beneficial at volumes approaching 15,000 vehicles per day, particularly on rolling terrain; the presence of passing lanes improved delay and percent time spent following (Brewer et al. 2011). Most passing occurred within the first mile of a passing lane, so additional length may be less useful than additional lanes in a Super 2 corridor, particularly at lower volumes. Empirical Bayes analysis of crash data showed that there was a statistically significant crash reduction of 35% for segment-only (i.e., nonintersection) injury crashes on the study corridors, as compared with the expected number of crashes without passing lanes. In lieu of guidelines related to specific ADT values, researchers recommended including general principles for Super 2 design as part of their proposed revisions to the Texas *Roadway Design Manual,* such as avoiding intersections with state highways and high-volume county roads within passing lanes, consideration of terrain and right-of-way in determining alignment and placement of passing lanes, avoiding the termination of passing lanes on uphill grades, and discouraging passing lane lengths longer than 4 mi.

Volume 4 of *NCHRP Report 500* (Neuman et al. 2003b) discusses the use of center TWLTL on four-lane and two-lane roads to reduce the likelihood of head-on and rear-end collisions.

> It can be accomplished either by the conversion of four-lane undivided arterials to three-lane roadways with a center left-turn lane or by the more conventional reconstruction of a two-lane road to include the TWLTL. Since the latter could be a costly conversion because it may require new right-of-way, the four-lane road conversion is considered more appropriate to the AASHTO emphasis on low-cost alternatives. However, where right-of-way

TABLE 6
COMPARISON OF EUROPEAN 2+1 ROAD CHARACTERISTICS

	Germany	Finland	Sweden
Critical Transition Length, m (ft)	180 (590)	500 (1,600)	300 (1,000)
Non-Critical Transition Length, m (ft)	30–50 (100–160)	50 (160)	100 (330)
Typical Passing Lane Length, km (mi)	1.0–1.4 (0.6–0.9)	1.5 (0.9)	1.0–2.0 (0.6–1.2)
Separation between Opposing Traffic m (ft)	0.5 (1.6)	0.3 (1.0)	1.25–2.0 (4.1–6.6)
Fatal+Injury Crash Rate, per 10^6 veh-km (10^6 veh-mi)	0.16 (0.26)	0.09 (0.14)	0.50 (0.80)
Typical Volumes, veh/day	15,000–25,000	14,000–25,000	4,000–20,000

Source: Potts and Harwood (2003).
Note: Sweden's 2+1 roads are separated by cable barrier, and their crash rates are specifically reported as crashes per million axle pair-km.

cost is not a major consideration, the inclusion of TWLTLs on existing two-lane roads may be an even more effective treatment for head-on collisions since more of such collisions would likely occur on two-lane roads than on four-lane roads.

The development of TWLTLs is usually for traffic operations rather than safety concerns. TWLTLs are usually implemented to improve access. When they are used in response to a safety concern, the use is traditionally to reduce driveway-related turning and rear-end collisions. However, because studies have also indicated a positive effect on head-on crashes, the strategy is included here. The principle behind the use of TWLTLs in this context is to provide a buffer between opposing directions of travel. The strategy is intended to reduce head-on crashes by keeping vehicles from encroaching into opposing traffic lanes through the use of the buffer.

If available right-of-way, construction budget, and traffic volumes allow, incorporating TWLTLs into the design of new and reconstructed roads is more efficient than converting roadways later.

Managed Lanes

Transportation agencies in many jurisdictions have examined new ways to maximize the use of existing infrastructure to improve capacity and reduce congestion, particularly on urban freeway corridors. One such method is the use of managed lanes, whereby one or more lanes in the corridor are reserved for use that is limited to specific types of vehicles, such as high-occupancy vehicles (HOV), buses, motorcycles, or toll-paying drivers. Kuhn et al. (2005) conducted a multi-year study on a wide variety of planning, design, and operational issues related to managed lanes. A chapter in their *Managed Lanes Handbook* contains recommendations and guidelines for design elements of freeway managed lanes. In general, they recommended that the features of the managed lane be commensurate with the design vehicle that is selected to be appropriate for the facility. They recommended that the designer use the AASHTO *Green Book* templates in determining turning paths, lateral and vertical clearances, bus stops, and other elements associated with a project. In particular, the design process might also account for the path of the vehicle overhang beyond the outside turning radius.

According to Kuhn et al. (2005), "in most cases, the design speed of managed lanes will be the same as that used on the adjacent general-purpose lanes. However, there may be limited instances where the design speed of the managed lanes is lower than the adjacent general-purpose lanes, owing to the geometrics of the managed lanes facility or other limitations. The designated design speed of the facility should relate to the maximum speed the facility is expected to accommodate. Further, the design speed should accommodate the vast majority of users (e.g., the anticipated 85th percentile speed)."

The *Managed Lanes Handbook* also contains recommendations on horizontal and vertical clearance and curvature, gradient, SSD, cross slope, superelevation, and minimum turning radius. Their recommended guidelines are summarized in Table 7.

Hard Shoulder Running

In the United States, the primary use of shoulders has been as a safety refuge area; however, in recent years there has been an increasing trend for transportation agencies to explore the use of shoulders as travel lanes during peak periods as a congestion management strategy. Also called "hard shoulder running," the limited use of the shoulder as a travel lane has been primarily reserved for special users of the roadway system, most often transit vehicles. Overall, experience using shoulders for interim use has been positive in the United States, and more agencies are considering the strategy to address growing congestion on their urban freeway networks. Several states have deployed temporary shoulder use for all vehicles on congested corridors with success. Kuhn (2010) describes several uses of hard shoulder running in the United States, as noted in the following paragraphs.

In San Diego, California, along I-805/SR 52, transit vehicles may use the freeway shoulder during congested periods, when general-purpose lane traffic slows to 30 mph or lower.

TABLE 7
SUMMARY OF MANAGED LANES MAINLINE DESIGN CRITERIA

Design Speed	U.S. Customary Desirable (70 mph)	U.S. Customary Reduced (50 mph)	Metric Desirable (110 km/h)	Metric Reduced (80 km/h)
Alignment				
Stopping distance	730 ft	425 ft	220 m	130 m
Horizontal curvature (radius)	2,050–2,345 ft	835–930 ft	560–635 m	250–280 m
Maximum superelevation	0.04 ft/ft	0.06 ft/ft	0.04 m/m	0.06 m/m
Rate of vertical curvature				
Crest, *k*	247	84	74	26
Sag, *k*	181	96	55	30
Gradients				
Maximum (%)	4.0	5.0	4.0	5.0
Minimum (%)	0.5	0.3	0.5	0.3
Clearance				
Vertical	16.5 ft	14.5 ft	5 m	4.4 m
Lateral	4 ft	2 ft	1.2 m	0.6 m
Lane Width				
Travel lanes	12 ft	11 ft	3.6 m	3.4 m
Cross Slope				
Maximum	0.020 ft/ft	0.020 ft/ft	0.020 m/m	0.020 m/m
Minimum	0.015 ft/ft	0.015 ft/ft	0.015 m/m	0.015 m/m
Superelevation: Dependent on curve radii and design speed [0.10 ft/ft (0.10 m/m) maximum].				

Source: Kuhn et al. (2005).

They may travel no more than 10 mph faster than traffic in general traffic lanes. The cross section of the shoulder is at least 10-ft wide throughout the deployment area. Pavement markings to indicate the operational strategy include text indicating "Transit Lane Authorized Buses Only" (Martin 2006).

The Florida DOT, Miami–Dade Transit, and the Miami Dade Expressway Authority operate several shoulder use applications in the Miami region. Along the Florida Turnpike, SR 826, and SR 836, buses are allowed to use the shoulder when the freeway is congested. Implemented in 2005, the program stipulates that buses may travel no faster than 35 mph on the shoulder when open to transit. The typical cross section is a minimum 10-ft width, with a 12-ft width in high volume areas, and cross slopes of 2% to 6%. Pavement markings indicate "Watch for Buses on Shoulder" (Martin 2006).

On GA 400 in Alpharetta, Georgia, buses are allowed to use the freeway shoulder in an effort to provide access between a local transit rail station and a park-and-ride lot. The Georgia Regional Transportation Authority and Georgia DOT operate the facility, which is functional whenever traffic slows to 35 mph or less. Buses can travel no more than 15 mph faster than general-purpose lane traffic and are required to reenter general traffic lanes before interchanges. A construction project was necessary to upgrade the travel surface by widening the shoulder by 2 ft and providing reinforcement of the shoulder pavement (Martin 2006).

An extensive system around Minneapolis–St. Paul, Minnesota, is also focused on buses. The operational strategy is considered interim, and some deployments have been removed since their inception. When operational, buses must yield to any vehicle entering, merging, or exiting through the shoulder, and buses must reenter the main lanes when the shoulder is obstructed. Typically, buses may use the shoulder any time that traffic in the adjacent mainlines is moving at less than 35 mph. Buses may travel no more than 15 mph faster than mainline traffic with a 35 mph maximum allowed speed on the shoulder. The typical minimum shoulder width is 10 ft, with an 11.5-ft minimum at bridges, and a 12-ft minimum on new construction. Typically, buses travel through the entrance and exit ramps; where queues are long at ramps with metering, buses typically merge with traffic on the ramp and return to the shoulder after the ramp (Kuhn 2010).

One segment of I-35W in Minneapolis has a unique combination of strategies. Known as priced dynamic shoulder lanes (PDSL), the left shoulder is open during the peak periods; transit and carpools use the shoulder for free and MnPASS customers can use the shoulder for a fee. As shown in Figure 4, the left shoulder is open to traffic, with overhead sign gantries indicating its operational status. When the general-purpose lanes become congested, the shoulder is opened and the speed limit on the general-purpose lanes is reduced (Kuhn 2010).

In dedicated shoulder-lane operations, either general-purpose or HOV-specific capacity has been added through the permanent conversion of shoulders. Most HOV applications use the interior lane for HOV operations, whereas the exterior shoulder is used for general-purpose traffic so as to maintain the same number of general-purpose lanes that existed before implementation. A typical HOV application would convert a three-lane freeway with 12-ft lanes, 10-ft exterior shoulder, and 8-ft interior shoulder to 11-ft general-purpose lanes, 14-ft (including buffer striping) HOV lane, 5-ft exterior shoulder, and 2-ft interior shoulder (Kuhn 2010).

FIGURE 4 Open priced dynamic shoulder lane (*Credit*: Minnesota Department of Transportation).

LANE WIDTH

Potts et al. (2007b) investigated the relationship between lane width and safety for roadway segments and intersection approaches on urban and suburban arterials. Their research found no general indication that the use of lanes narrower than 12 ft (3.6 m) on urban and suburban arterials increased crash frequencies. Researchers stated that this finding suggested that geometric design policies can provide substantial flexibility for use of lane widths narrower than 12 ft (3.6 m). They added that inconsistent results suggested increased crash frequencies with narrower lanes in three specific design situations:

- Lane widths of 10 ft (3.0 m) or less on four-lane undivided arterials.
- Lane widths of 9 ft (2.7 m) or less on four-lane divided arterials.
- Lane widths of 10 ft (3.0 m) or less on approaches to four-leg stop-controlled arterial intersections.

The researchers recommended that "narrower lanes should be used cautiously in these three situations unless local experience indicates otherwise."

Gross et al. (2009) studied a variety of crash data and roadway characteristics to determine the safety effectiveness of specific combinations of lane and shoulder width on rural, two-lane, undivided roads. In general, all else being equal, results were consistent with previous research efforts, showing crash reductions for wider paved widths, wider lanes, and wider shoulders. More specific to the research objective, CMFs were provided for various lane–shoulder configurations. Individual state analyses did not indicate a clear preference for lane or shoulder width given a fixed paved width, but combined with findings from previous research, researchers described some potential trends:

- For 26- to 32-ft total paved widths, 12-ft lanes provided the optimal safety benefit. The CMF ranged from 0.94 to 0.97, indicating a 3% to 6% crash reduction for 12-ft lanes compared with 10-ft lanes.
- For 34-ft total paved width, 11-ft lanes provided the optimal safety benefit. The CMF for 11-ft lanes was 0.78 compared with the 10-ft baseline.
- For 36-ft total paved width, 11- or 12-ft lanes provided the optimal safety benefit. The CMF was 0.95 for 11- and 12-ft lanes compared with the 10-ft baseline.

These results applied, in general, to rural, two-lane roads with traffic volumes greater than 1,000 vehicles per day and posted speeds of 25 mph or greater. Although 12-ft lanes appeared to be the optimal design for 26- to 32-ft total paved widths, 11-ft lanes performed equally well or better than 12-ft lanes for 34- to 36-ft total paved widths.

The *Highway Safety Manual* (AASHTO 2010) provides CMFs for lane width on two-lane highway segments, which are presented in Table 8. The base value for the lane width CMF is 12 ft. For lane widths with 0.5-ft increments that are not depicted specifically in Table 8, a CMF value can be interpolated because there is a linear transition between the various AADT effects. A corresponding chart is also provided as a figure in the *HCM.*

Number of Lanes

Kononov et al. (2008) explored the relationship between safety and congestion on urban freeways by examining the shape of the safety performance functions (SPFs). SPFs are crash prediction models that relate traffic exposure, measured in AADT, to safety, measured in the number of accidents over a unit of time (e.g., accidents per mile per year). They found that to that point "the focus of most SPF modeling efforts had been on the statistical technique and the underlying prob-

TABLE 8
CRASH MODIFICATION FACTORS FOR LANE WIDTH ON ROADWAY SEGMENTS

	AADT (vehicles per day)		
Lane Width	<400	400 to 2,000	>2,000
9 ft or less	1.05	$1.05 + 2.81 \times 10^{-4}$ (AADT – 400)	1.50
10 ft	1.02	$1.02 + 1.75 \times 10^{-4}$ (AADT – 400)	1.30
11 ft	1.01	$1.01 + 2.50 \times 10^{-5}$ (AADT – 400)	1.05
12 ft or more	1.00	1.00	1.00

Source: AASHTO (2010) .
Note: The collision types for which this CMF is applicable include single-vehicle run-off-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes.

ability distribution, with only limited consideration given to the nature of the phenomenon itself." Their relationship of safety to the degree of congestion suggested that safety deteriorated with the degradation in the quality of service expressed through the level of service. Their assessment was that practitioners generally believed the additional capacity afforded by additional lanes was associated with more safety, but how much safety and for what time period were generally not considered. Comparison of SPFs of multilane freeways suggested that adding lanes may initially result in a temporary safety improvement that disappears as congestion increases. They found that total as well as injury and fatal crash rates increased with AADT and that it was significantly safer to travel on urban freeways that operate at level-of-service (LOS)-C or better during the peak period than on more congested facilities. As AADT increased, the slope of SPF, described by its first derivative, became steeper, reflecting that crashes were increasing at a faster rate than would be expected from a freeway with fewer lanes. As the number of lanes increased, so did the opportunity for drivers to maneuver around slower traffic. Increased maneuverability tended to increase the average speed of traffic, but at the same time it increased the speed differential and the number of crashes related to lane changes, such as sideswipes and rear-end crashes.

Road Diet

Huang et al. (2002) investigated the effects on crashes and injuries through conversion of an undivided four-lane road to three lanes and a TWLTL, also known as a "road diet." They reviewed before-and-after crash data from 12 road diet sites and 25 comparison sites in California and Washington, and found that "the percent of road diet crashes occurring during the 'after' period was about 6% lower than that of the matched comparison sites." However, a separate analysis in which a negative binomial model was used to control for possible differential changes in ADT, study period, and other factors indicated no significant treatment effect. Crash severity was virtually the same at road diets and comparison sites, whereas there were some differences in crash type distributions between road diets and comparison sites, they found none between the "before" and "after" periods. They concluded that conversion to a road diet should be made on a case-by-case basis in which traffic flow, vehicle capacity, and safety are all considered. They also recommended that the effects of road diets be further evaluated under a variety of traffic and roadway conditions.

Pawlovich et al. (2006) used a Bayesian approach to evaluate the effects of the "road diet" on crashes in Iowa. Their methodology incorporated both monthly crash data and estimated volumes for 30 sites—15 treatment and 15 comparison—for more than 23 years (1982 to 2004). Their results indicated a 25.2% reduction in crash frequency per mile and an 18.8% reduction in crash rate. The authors stated that their results from the Iowa study fit practitioner experience and agreed with another Iowa study that used a simple before-and-after approach on the same sites.

NCHRP Report 617 (Harkey et al. 2008) presents the findings of a research project to develop CMFs for traffic engineering and Intelligent Transportation System improvements. One such improvement was the "road diet." Researchers estimated the change in total crashes owing to the conversion and use Empirical Bayes methodology to compare the results with previous studies. They reviewed geometric, traffic, and crash data for 45 treatment sites and 347 reference sites in Iowa, Washington, and California, and found significant effects on crashes. Their recommendations for CMFs are shown in Table 9.

Resurfacing

The research team on NCHRP Project 3-56 (Harwood et al. 2003a) developed a process for allocating resources to maximize the effectiveness of 3R projects in improving safety and traffic operations on the nonfreeway highway network. They developed a program called the Resurfacing Safety Resource Allocation Program (RSRAP) designed to allow highway agencies to maximize the cost-effectiveness of the funds spent on 3R projects by improving safety on nonfreeway facilities while maintaining the structural integrity and ride quality of the highway pavement. To do this, their process considered:

- "A specific set of highway sections that are in need of resurfacing either at the present time or within the relatively near future;
- A specific set of improvement alternatives for each candidate site, including doing nothing, resurfacing only, and various combinations of safety improvements for the site; and
- A limit on the funds available for improvements to the set of highway locations."

The RSRAP procedure considers other treatments in addition to resurfacing, such as lane width changes, turning lane improvements, and shoulder widening. Among their findings, the research team concluded that:

- Resource allocation methods provided an effective method for highway agencies to decide when safety improvements are to be made in conjunction with pavement resurfacing projects.
- For a given set of sites, resource allocation methods provided an optimal mix of resurfacing treatments with and without accompanying safety improvements that provided greater benefits than any fixed strategy.
- Resurfacing without accompanying geometric improvements may cause a small, short-term increase in accidents resulting from increased speeds; however, the evidence for this effect was conflicting. An optional feature in the RSRAP software allowed the user to

TABLE 9
RECOMMENDED CRASH MODIFICATION FACTOR FOR ROAD DIET TREATMENT

TREATMENT: Convert Undivided Four-Lane Road to Three-Lane and TWLTL (Road Diet)	CMF Level of Predictive Certainty: High			
METHODOLOGY: Empirical Bayes Before–After	Crash Type Studied and Estimated Effect			
REFERENCE: NCHRP Project 17-25 research results	State/Site Characteristics	Crash Type	Number of Treated Sites	CMF (std. error)
STUDY SITES: • 15 urban locations in Iowa with a mean length of 1.02 miles, a minimum and maximum length of 0.24 and 1.72 miles. AADT after conversion ranged from 3,718 to 13,908. • 30 urban locations from Washington and California studied previously with a mean length of 0.84 miles, a minimum and maximum length of 0.08 and 2.54 miles. AADT after conversion ranged from 6,194 to 26,376.	*Iowa* Predominately U.S. and state routes within small urban areas (average population of 17,000)	Total Crashes	15 15 miles	0.53 (0.02)
	California/Washington Predominately corridors within suburban areas surrounding larger cities (average population of 269,000)	Total Crashes	30 30 miles	0.81 (0.03)
	All Sites	Total Crashes	45 45 miles	0.71 (0.02)
	FOOTNOTES: [1]Huang et al. (2002). [2]Pawlovich et al. (2006).			
COMMENTS: • The study conducted was a reanalysis of data from two prior studies.[1,2] • The reanalysis of the Washington/California data indicated a 19% decrease in total crashes. The reanalysis of the Iowa data showed a reduction of 47% in total crashes. If the characteristics of the treated site can be defined on the basis of road and area type (as shown above), the CMFs of 0.53 and 0.81 should be used. Otherwise, it is recommended that the aggregate CMF of 0.71 be applied.				

Source: Harkey et al. (2008).

include this short-term effect if desired. The increase in accidents following resurfacing was assumed to occur only at sites with existing lane widths of less than 11 ft and existing shoulder widths of less than 6 ft.

Work Zone Considerations

Changes in lane width, particularly lane constrictions, are often used in conjunction with lane shifts, lane closures, and shoulder closures. The authors of *NCHRP Report 581* (Mahoney et al. 2007) discussed some aspects of lane width for designers to consider in work zones on high-speed roadways, defined as those with free-flow speeds of 50 mph or more. They mention that it is common practice to reference "travel lane width" as the key lane constriction decision variable. However, operations in one travel lane can be influenced by operations in adjacent lanes. Additionally, adjoining travel lanes occasionally have different widths; therefore, it may be more appropriate for design guidance to address traveled way width. For example, they suggested that a 10-ft travel lane adjacent to a 12-ft travel lane is generally more desirable than a 10-ft travel lane adjacent to a travel lane of the same width. Although their desirable traveled way width resulted in 12-ft travel lanes, 11-ft lane widths were common in work zones, and lanes narrower than 10 ft were generally not used for work zones on high-speed roads. They offered the information presented in Table 10 as an example framework to determine minimum traveled way width in a work zone on a high-speed roadway.

SHOULDERS

Width

NCHRP Report 633 (Stamatiadis et al. 2009) presented recommendations for CMFs for shoulder width and median width for four-lane roads with 12-ft lanes. The authors' recommended CMFs for average shoulder width are shown in Table 11. Recommendations for median width CMFs are provided in the section on medians elsewhere in this chapter.

FHWA's *Highway Design Handbook for Older Drivers and Pedestrians* (Staplin et al. 2002) recommends that "for horizontal curves on two-lane nonresidential facilities that have 3 degrees of curvature, the width of the lane plus the paved shoulder be at least 5.5 m (18 ft) throughout the length of the curve." The *Handbook's* authors cite previous research stating that "older drivers, as a result of age-related declines in motor ability, have been found to be deficient in coordinating the control movements involved in lanekeeping, maintaining speed, and handling curves."

Dumbaugh (2006) conducted an analysis of roadside safety in urban areas, looking specifically at three treatments:

> . . . widening paved shoulders, widening fixed-object offsets, and providing livable-street treatments. [His] model results indicated that of the three strategies, only the livable-streets variable was consistently associated with reductions in roadside and midblock crashes. Wider shoulders were found to increase roadside and

TABLE 10
EXAMPLE FRAMEWORK FOR SELECTING ONE-WAY TRAVELED WAY WIDTHS

		Metric				U.S. Customary			
		Traveled Way Width (m)				Traveled Way Width (ft)			
	Facility Type	Undivided Highway		Divided Highway		Undivided Highway		Divided Highway	
	Lanes per Direction	One	Two	One	Two	One	Two	One	Two
Traveled Way Edge Conditions	Constraint along neither traveled way edge	3.0[1]	6.0[2,3]	3.3	6.6[3]	10[1]	20[2,3]	11	22[3]
Traveled Way Edge Conditions	Constraint along one traveled way edge	3.3[1]	6.3[2,3]	3.6	6.9[3]	11[1]	21[2,3]	12	23[3]
Traveled Way Edge Conditions	Constraint along both traveled way edges	3.6[1]	6.6[2,3]	3.9	7.2[3]	12[1]	22[2,3]	13	24[3]

Notes:
1. Values apply only when all of the following conditions are met: low truck volumes, all curve radii equal or exceed 555 m (1,820 ft); and anticipated 85th-percentile speeds are less than or equal to 80 km/h (50 mph). If any of the three conditions is not met, add 0.3 m (1 ft) to the base value.
2. Values apply only to roadways carrying moderate truck volumes where all curve radii equal or exceed 555 m (1,820 ft). If either condition is not met, add 0.3 m (1 ft) to the base value.
3. Values shown apply to two-lane, one-way traveled ways. For constricted two-way traveled ways, consider separation of opposing directions using (1) additional traveled way width, (2) channelizing devices, or (3) a traffic barrier.

To use this exhibit, first determine the traveled way edge conditions. "Constraint" refers to the presence of an imposing feature, such as a feature that results in "shying away" at the edge of the traveled way. Temporary barriers are a common constraint feature. Next, identify the type of facility (undivided or divided) approaching the work zone. Using this information and the number of travel lanes through the work zone, determine the base (i.e., unadjusted) value within the appropriate cell. Superscripted numerals indicate the note numbers that should be referenced to determine appropriate adjustments, if any, to the base value.

For traveled ways with edge constraint, the distances indicated are measured to the face of the constraining features (i.e., the offset is included in the tabulated or adjusted dimension).

Values lower that those obtained from this method may be appropriate for very low exposure (i.e., traffic volume, constricted lane segment length, and duration of operation).

Source: Mahoney et al. (2007).

midblock crashes, while unpaved fixed-object offsets had a mixed safety effect [of] decreasing roadside crashes but slightly [increasing] midblock crashes. To understand better the reasons for these findings, the study then examined roadside crash site locations for tree and utility pole crashes. [His conclusion was] that the majority (between 65% and 83%) [of crashes] did not involve random midblock encroachments, as currently assumed, but instead involved objects located behind both driveways and side streets along higher-speed urban arterials. [He stated that], collectively, these findings [suggested] that most urban roadside crashes were not the result of random error but were instead systematically encoded into the design of the roadway. The study concluded by distinguishing between random and systematic driver errors and by discussing strategies for eliminating systematic error while minimizing the consequences of random error.

Lord and Bonneson (2007) examined the safety performance of rural frontage road segments. Their findings suggested that wider lane and shoulder widths are associated with a reduction in segment-related collisions. In addition, the data suggest that the presence of edge marking has a significant impact on the safety performance of rural two-way frontage roads. However, the magnitude of crash reduction resulting from marking presence was significant and believed to overstate the true benefit of such markings. They developed a safety performance function and three CMFs from a statistical model that was estimated through data collected on rural frontage road segments. The variables they

TABLE 11
RECOMMENDED CMFS FOR AVERAGE SHOULDER WIDTH

	Average Shoulder Width (ft)						
Category	0	3	4	5	6	7	8
Undivided	1.22	1.00	0.94	0.87	0.82	0.76	0.71
Divided	1.17	1.00	0.95	0.90	0.85	0.81	0.77

Source: Stamatiadis et al. (2009).
Notes:
[1]CMFs are for all crashes and all severities.
[2]The average shoulder width for undivided highways is the average of the right shoulders; for divided, it is the average of left and right shoulder in the same direction.

found to have significant correlation with crash frequency included lane width, paved shoulder width, and, for two-lane frontage roads, edge marking delineation. Their SPFs and CMFs did not consider crashes that would be attributed to the ramp-frontage road terminal or the frontage road–crossroad intersection. Moreover, they did not consider crashes on the main lanes that may indirectly be related to wrong-way travel down an exit ramp.

The *Highway Safety Manual* (AASHTO 2010) provides CMFs for shoulder width and shoulder type, which are presented in Table 12. The base value of shoulder width and type is a 6-ft paved shoulder.

Operational and Safety Treatments

Rumble Strips

Multiple studies have examined the effects of both shoulder and centerline rumble strips (CLRS). Information and findings for both types are presented in this section. Although not all roadway departure collisions can be attributable to drowsy driving, research shows that a large percentage of them are. Morena (2003) distinguishes between run-off-road and a subset of drift-off-road collisions. Whereas run-off-road crashes can occur for many reasons (loss of control, swerving to avoid another vehicle or object, icy roadway conditions, etc.), drift-off-road crashes are solely attributed to drowsy or inattentive drivers. The FHWA Rumble Strip website estimates that 40% to 60% of single-vehicle crashes on rural freeways are actually drift-off-road crashes. In examining Michigan roadway data, Morena arrived at a much lower percentage of 16%, in part because nearly half (48%) of the run-off-road collisions in that state occurred on snowy or icy roadways and an additional 9% occurred on wet roadways.

Persaud et al. (2003) investigated installation of rumble strips along the centerlines of undivided rural two-lane roads to warn or alert distracted, fatigued, or speeding motorists whose vehicles were susceptible to crossing the centerlines and encroaching into opposing traffic lanes. They analyzed data for approximately 210 mi of treated roads in seven states using an Empirical Bayes before–after methodology. Overall, they found that crashes at treated sites were reduced 14% and injury crashes were reduced by an estimated 15%. All frontal and opposing-direction sideswipe crashes were reduced by an estimated 21%, and those crashes involving injuries were reduced by an estimated 25%. All of the reductions were determined to be statistically significant.

Among the improvements investigated for CMFs in *NCHRP 617* (Harkey et al. 2008) were shoulder and CLRS. The recommendations from that report are shown in Tables 13 and 14.

NCHRP Synthesis 339 (Russell and Rys 2005) summarized the state of the practice on CLRS, examining design practices, installation, configuration, dimensions, and visibility. The synthesis addressed the need for guidance on warrants, benefits, successful practices, and concerns (e.g., external noise and the reduced visibility of centerline striping material). The report also discussed pavement deterioration, ice buildup in the grooves, adverse impact on emergency vehicles, and the effect of CLRS on bicyclists. Particular attention was paid to available before-and-after installation crash data to document the safety aspects of CLRS and the availability of policies, guidelines, warrants, and costs regarding their use and design. The authors did not find reliable evidence of negative effects of CLRS, but they determined that adequate data were not yet available to make definitive conclusions for a number of the issues listed. They noted that there was no standard nationwide design of CLRS and no conclusive studies had been conducted on maintenance issues. They did conclude that there was a definite possibility that CLRS milled over the centerline could increase or accelerate deterioration of the typical centerline pavement joint and they recommended that, at a minimum, CLRS be installed only in good pavement.

In 2006, the Washington State DOT (WSDOT) implemented policy for installing CLRS on undivided highways and invested in funding strategies for those installations. WSDOT subsequently conducted a study (Olson et al. 2011) to evaluate the effectiveness of CLRS under a variety of traffic and geometric conditions, in an effort to develop better guidance on when to use rumble strips to address various collision types. They determined that cross-centerline collisions have been reduced by 44.6% for all injury severi-

TABLE 12
CRASH MODIFICATION FACTORS FOR LANE WIDTH ON ROADWAY SEGMENTS

	AADT (vehicles per day)		
Shoulder Width	<400	400 to 2,000	>2,000
0 ft	1.10	$1.10 + 2.50 \times 10^{-4}$ (AADT – 400)	1.50
2 ft	1.07	$1.07 + 1.43 \times 10^{-4}$ (AADT – 400)	1.30
4 ft	1.02	$1.02 + 8.125 \times 10^{-5}$ (AADT – 400)	1.15
6 ft	1.00	1.00	1.00
8 ft or more	0.98	$0.98 + 6.875 \times 10^{-5}$ (AADT – 400)	0.87

Source: AASHTO (2010).
Note: The collision types for which this CMF is applicable include single-vehicle run-off-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes.

TABLE 13
RECOMMENDED CRASH MODIFICATION FACTOR FOR SHOULDER RUMBLE STRIPS

TREATMENT: Add Shoulder Rumble Strips	CMF Level of Predictive Certainty: Medium-High		
METHODOLOGY: Before-After with Comparison Sites	CRASH TYPE STUDIED AND ESTIMATED EFFECTS		
REFERENCE: Griffith (1999)	All Freeways (Rural and Urban)	Number of Improved Sites	CMF (std. error)
STUDY SITES: • Included 55 treatment sites and 55 matched comparison sites from rural and urban freeways in Illinois. • The treatment sites covered 196 miles of rural freeway and 67 miles of urban freeway. • The treatment sites were not selected on the basis of crash history; thus, there was no selection bias.	All Single-Vehicle Run-Off-Road Crashes	55	0.82 (0.07)
	Injury Single-Vehicle Run-Off-Road Crashes		0.87 (0.12)
	Rural Freeways		
	All Single-Vehicle Run-Off-Road Crashes	29	0.79 (0.10)
	Injury Single-Vehicle Run-Off-Road Crashes		0.93 (0.16)
COMMENTS: • Results for all freeways based on yoked comparison analysis; results for rural freeways based on comparison group method using 29 of the treatment sites. Results could not be developed for urban sites separately. • An analysis of multi-vehicle accidents showed the rumble strips to have no effect on such accidents. • The CMF is not applicable to other road classes (two-lane or multilane).			

Source: Harkey et al. (2008).

TABLE 14
RECOMMENDED CRASH MODIFICATION FACTOR FOR CENTERLINE RUMBLE STRIPS

TREATMENT: Add Centerline Rumble Strips	CMF Level of Predictive Certainty: Medium-High		
METHODOLOGY: Empirical Bayes Before-After	CRASH TYPE STUDIED AND ESTIMATED EFFECTS		
REFERENCE: Persaud et al. (2003)	Crash Type (All Severities)	Number of Improved Sites	CMF (std. error)
STUDY SITES: • Crash and traffic volume data were collected for 98 treatment sites, consisting of 210 miles, where centerline rumble strips had been installed on rural two-lane roads in the states of California, Colorado, Delaware, Maryland, Minnesota, Oregon, and Washington. • The average length of the treatment sites was 2 miles, and the traffic volumes ranged from 5,000 to 22,000 vpd. • The reference group of sites was developed from HSIS data for the states of California, Washington, and Minnesota.[1] Additional data were acquired from Colorado for SPF calibration for the Colorado sites.	All Crashes	98	0.86 (0.05)
	Frontal/Opposing-Direction Sideswipe Crashes		0.79 (0.12)
	Crash Type (Injury Crashes)		
	All Crashes	98	0.85 (0.08)
	Frontal/Opposing-Direction Sideswipe Crashes		0.75 (0.15)
	COMMENTS: • The authors note that the results cover a wide range of geometric conditions, including curved and tangent sections and sections with and without grades. • The results include all rumble strip designs (milled-in, rolled-in, formed, and raised thermo-plastic) and placements (continuous versus intermittent) that were present. • The CMF is not applicable to other road classes (multilane).		

Source: Harkey et al. (2008).
[1]The Highway Safety Information System (HSIS) is a multistate safety database that contains crash, roadway inventory, and traffic volume data for a select group of states and is sponsored by the FHWA.

ties and by 48.6% for fatal and serious injury crashes. They also found that crashes involving asleep or fatigued drivers were reduced by 75.3% (72.6% for fatal and serious injury crashes) where CLRS were installed. Their data showed that on a horizontal curve the rate of fatal and serious injury crashes was almost twice as high for those lane departures to the outside of a curve than to the inside of the curve, but that CLRS were equally effective countermeasures for crashes in both directions, with reductions of about 35%. The researchers recommended that WSDOT's current guidance continue to be implemented to reduce cross-centerline collisions. The researchers also recommended that investment priority be given to locations with AADT less than 8,000, combined lane/shoulder width of 12 to 17 ft, and posted speed of 45 to 55 mph. With consideration of available funding, investment priorities, and site-specific conditions it was the research team's opinion that the installation of CLRS be pursued for all highways that comply with design guidance.

Torbic et al. (2009) conducted NCHRP Project 17-32, the objectives of which were to investigate the safety effectiveness and optimal placement and dimensions of shoulder and CLRS. *NCHRP Report 641,* which documents the project's activities, "provides guidance for the design and application of shoulder and centerline rumble strips as an effective crash reduction measure, while minimizing adverse effects for motorcyclists, bicyclists, and nearby residents." Using the results of previous studies and the research conducted under this project, "researchers developed" safety effectiveness estimates for shoulder rumble strips on rural freeways and rural two-lane roads and for CLRS on rural and urban two-lane roads. Their estimates with associated standard errors (SE) were as follows:

- Urban/Rural Freeways—Rolled shoulder rumble strips:
  - 18% reduction in single-vehicle run-off-road (SVROR) crashes (SE = 7)
  - 13% reduction in SVROR fatal and injury (FI) crashes (SE = 12).
- Rural Freeways—Shoulder rumble strips:
  - 11% reduction in SVROR crashes (SE = 6)
  - 16% reduction in SVROR FI crashes (SE = 8).
- Rural Two-Lane Roads—Shoulder rumble strips:
  - 15% reduction in SVROR crashes (SE = 7)
  - 29% reduction in SVROR FI crashes (SE = 9).
- Urban Two-Lane Roads—CLRS:
  - 40% reduction in total target (head-on and opposite-direction sideswipe) crashes (SE = 17)
  - 64% reduction in FI target crashes (SE = 27).
- Rural Two-Lane Roads—CLRS:
  - 9% reduction in total crashes (SE = 2)
  - 12% reduction in FI crashes (SE = 3)
  - 30% reduction in total target crashes (SE = 5)
  - 44% reduction in FI target crashes (SE = 6).

The NCHRP 17-32 research team added that shoulder rumble strips should be placed as close to the edgeline as possible to maximize safety benefits. They also stated that the safety benefits of CLRS for roadways on horizontal curves and on tangent sections are for practical purposes the same. With regard to rumble strip design, researchers concluded that shoulder rumble strip patterns for freeways and other roadways where bicyclists are not expected be designed to produce sound level differences between 10 to 15 dBA in the passenger compartment; for other roadways, the recommended sound level difference was 6 to 12 dBA. Similarly, they recommended that CLRS patterns be designed to produce sound level differences in the range of 10 to 15 dBA in the passenger compartment, except near residential or urban areas where consideration would be given to designing CLRS to produce sound level differences in the range of 6 to 12 dBA in the passenger compartment.

Treatments for Edge of Roadway

Although the use of curbs is discouraged on high-speed roadways because of their potential for "tripping" a skidding vehicle into a rollover condition, "they are often required because of restricted right-of-way, drainage issues, access control, and other curb functions." Highway agencies have typically tried to reduce problems caused by curbs by offsetting the curb from the travel way as far as possible, using different curb shapes and using a barrier in combination with the curb.

Plaxico et al. (2005) undertook research to develop design guidelines for using curbs and curb–barrier combinations on roadways with operating speeds greater than 60 km/h (37.3 mph). The research team reviewed published literature and conducted computer simulation methods to gain information on the nature of typical designs and crashes of curb systems. Results from computer simulations were used to determine which type of curbs were safe to use on higher-speed roadways and the proper placement of barrier with respect to the curb. They also conducted full-scale crash tests to validate the computer simulations. The results of the study were then synthesized to develop guidelines for the use of curbs and curb–barrier systems. The researchers' recommendations included the following:

- Any combination of a sloping-faced curb that is 150 mm (6 in.) or shorter and a strong-post guardrail can be used where the curb is flush with the face of the guardrail up to an operating speed of 85 km/h (52.8 mph).
- Guardrails installed behind curbs are not to be located closer than 2.5 m (8.2 ft) for any operating speed in excess of 60 km/h (37.3 mph). Upon striking the curb, the vehicle bumper may rise above the critical height of the guardrail for many road departure angles and speeds in this region, making vaulting the barrier likely. A lateral distance of at least 2.5 m (8.2 ft) is needed to allow the vehicle suspension to return to its pre-departure state. Once the suspension and bumper have returned to their normal position, impacts with the barrier would proceed successfully.

- For roadways with operating speeds of 70 km/h (43.5 mph) or less, guardrails may be used with sloping-face curbs no taller than 150 mm (6 in.) as long as the face of the guardrail is located at least 2.5 m (8.2 ft) behind the curb.
- In cases where guardrails are installed behind curbs on roads with operating speeds between 71 and 85 km/h (44.1 and 52.8 mph), a lateral distance of at least 4 m (13.1 ft) is needed to allow the vehicle suspension to return to its pre-departure position. Once the suspension and bumper have returned to their normal position, impacts with the barrier would proceed successfully. At these speeds, guardrails may be used with sloping-face curbs of 100 mm (4 in.) in height or less as long as the face of the guardrail is located at least 4 m (13.1 ft) behind the curb.
- At operating speeds greater than 85 km/h (52.8 mph), guardrails are to only be used with 100-mm (4-in.) or shorter sloping-faced curbs, and they would be placed so that the curb is flush with the face of the guardrail. Operating speeds above 90 km/h (55.9 mph) require that the sloping face of the curb must be 1:3 or flatter and must be no more than 100 mm (4 in.) in height.
- Curbs are to only be used on higher-speed roadways when concerns about drainage make them essential to the proper maintenance of the highway.

NCHRP Report 600C (Campbell et al. 2010) discusses the potential safety ramifications of shoulder edge drop-offs, which typically arise from tire rutting erosion, excessive wear, or resurfacing. Guidelines for treating these locations are offered for purposes of design practices. The report authors cited a previous study by Graham and Glennon (1984), which stated that vertical or near-vertical shoulder drop-off heights in work zones that exceeded the indicated values in Table 15 warrant consideration for drop-off treatment or traffic control. The original source table also contained drop-off height thresholds that were greater than 3 in.; however, these were changed in the NCHRP report to reflect a more conservative assessment of other related driver performance data on driver encounters with drop-offs of various heights (Hallmark et al. 2006).

One potential treatment is a wedge-shaped application of asphalt; when placed between the roadway and the shoulder, the material can help drivers recover from the shoulder to the driving surface. *NCHRP Report 600C* advises that the asphalt material needs to be compacted to increase strength; otherwise the material will break apart over time owing to forces of overrunning vehicles and runoff water. A specific application of this treatment, called "Safety Edge" (shown in Figure 5), is being developed by FHWA, as discussed by Hallmark et al. (2006). An evaluation of this treatment by Graham et al. (2010) indicated small but positive results in crash reduction at 56 of 81 treated sites. Their results indicated that for all two-lane highway study sites in two states, the best estimate of the treatment's effectiveness was a reduction in total crashes of approximately 5.7%. The results were not statistically significant, but they were generally positive.

TABLE 15
VERTICAL DROP-OFF HEIGHT WARRANTING TRAFFIC CONTROL FOR VARIOUS LANE WIDTHS

Speed (mph)	Drop-Off Height (inches) for a Lane Width of			
	12 ft	11 ft	10 ft	9 ft
30	3	3	3	2
35	3	3	2	1
40	3	2	1	1
45	2	1	1	1
≥50	1	1	1	1

Adapted from Graham and Glennon (1984).

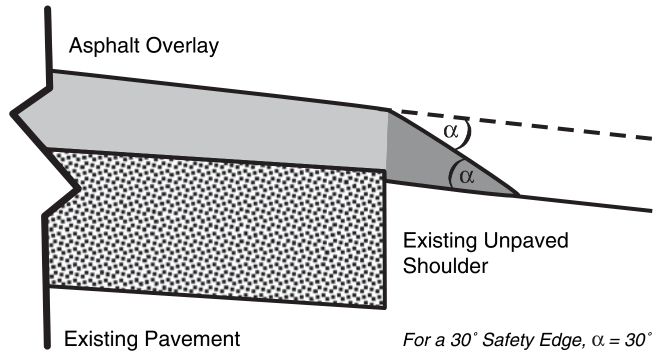


FIGURE 5 Illustration of FHWA's Safety Edge Treatment (Hallmark et al. 2006).

Work Zone Considerations

There are a number of ways in which shoulders may be used under work zone traffic control conditions. The authors of *NCHRP Report 581* (Mahoney et al. 2007) discuss some considerations for designers in the use of shoulders in work zones on high-speed roadways. They state that adoption of a work zone design speed may be appropriate for the evaluation of superelevation and sight distance. Because the shoulders will be part of a permanent high-speed roadway, no horizontal or vertical alignment decisions are generally needed. Temporary work zone features can affect sight distance, and it was recommended that the design be developed and evaluated from that perspective. If the shoulder being used to carry traffic is on a horizontal curve, the magnitude and direction of its cross slope would be compared with the superelevation requirement, and the agency's typical work zone policy for superelevation would be applied. They added that the designer is to also consider the adequacy of the shoulder in terms of structure (ability to carry the vehicle loads) and surface conditions (friction and smoothness), with particular attention to the presence and placement of shoulder rumble strips.

MEDIANS

NCHRP Report 633 (Stamatiadis et al. 2009) presented recommendations for CMFs for shoulder and median width for four-lane roads with 12-ft lanes. The authors made the

TABLE 16
RECOMMENDED CMFs FOR AVERAGE MEDIAN WIDTH ON DIVIDED ROADWAYS

	Average Median Width (ft)							
Category	10	20	30	40	50	60	70	80
Multi-Vehicle	1.00	0.91	0.83	0.75	0.68	0.62	0.57	0.51

Source: Stamatiadis et al. 2009.

assumption that median width had no effect on single-vehicle crashes, so their recommended CMFs for average median width, shown in Table 16, are for multi-vehicle crashes. Recommendations for shoulder width CMFs are provided in the section on shoulder width elsewhere in this chapter.

Tarko et al. (2007) investigated the impact of median designs on crash frequency. They analyzed data collected in eight participating states using negative binomial regression and before-and-after studies, and they examined crash severity using a logit model. The results of their analyses quantified the separate effects of changes in median geometry for single-vehicle, multiple-vehicle same-direction, and multiple-vehicle opposite-direction crashes. They concluded that results were significantly different for the various classes of crash types, indicating that reducing the median width without adding barriers (even if the remaining median width is still reasonably wide) increases the severity of crashes, particularly opposite-direction crashes. Further, they found that reducing the median width and installing concrete barriers eliminated opposite-direction crashes but doubled the frequency of single-vehicle crashes, increased crash severity, and tended to lessen the frequency of same-direction crashes.

ROADSIDE

Horizontal Clearance

The developers of the Roadside Safety Analysis Program included encroachment frequency curves (shown in Figure 6) and adjustment factors to increase encroachment rates on horizontal curves and vertical grades (shown in Figure 7) (Mak and Sicking 2003). The developers found three previous studies on encroachment data: Hutchinson and Kennedy (1966), Cooper (1980), and Calcote et al. (1985). The Hutchinson and Kennedy study involved observation of wheel tracks on medians of rural Illinois Interstate highways in the mid-1960s. Cooper conducted a similar encroachment study in Canada in the late 1970s. This research involved weekly observations of wheel tracks on grass-covered roadsides of rural highways of various functional classes. The data collection periods were during summer months on highways with speed limits between 80 and 100 km/h. Calcote et al. attempted to overcome the major problems with both the Cooper and the Hutchinson and Kennedy studies, but they "still did not offer an effective method to distinguish between controlled and uncontrolled encroachments."

An overwhelming majority of the encroachments recorded involved vehicles "moving slowly off the roadway for some distance and then returning into the traffic stream without any sudden changes in trajectory," which could be caused by "a fatigued or distracted driver drifting off the roadway, or a controlled driver responding to roadway or traffic conditions."

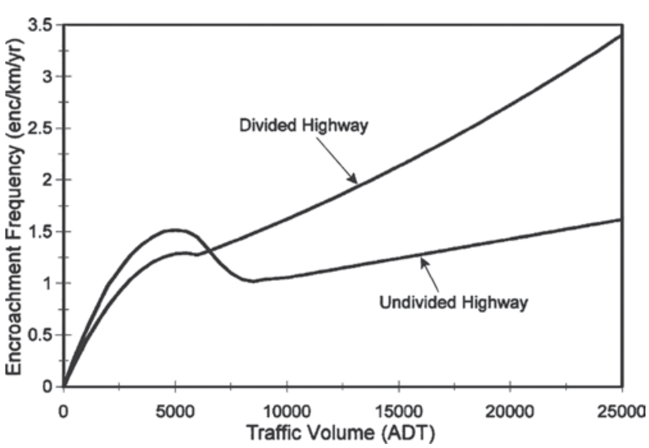


FIGURE 6 Encroachment rates used in Roadside Safety Analysis Program (Mak and Sicking 2003).

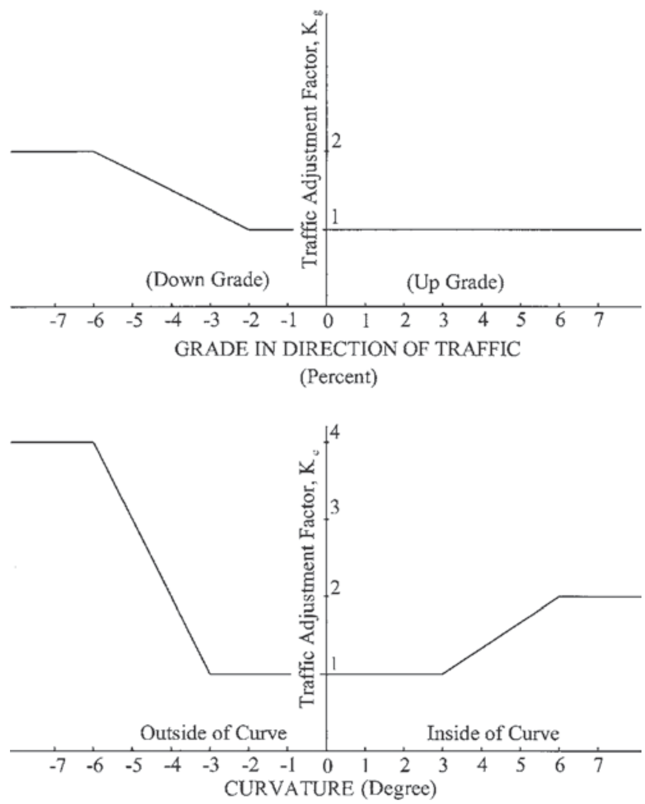


FIGURE 7 Encroachment frequency adjustment factors for curvature (Wright and Robertson 1976).

Roadside Safety Analysis Program developers selected the Cooper encroachment data for use in their encroachment rate–traffic volume relationships, because the Cooper data are more recent, constitute a larger sample size, and are believed to be of better quality than the Hutchinson and Kennedy data. The developers then incorporated adjustment factors based on previous studies (Wright and Robertson 1976; Perchonok et al. 1978) that compared roadway characteristics with fatal single-vehicle run-off-road crashes, with the underlying assumption that differences in roadway characteristics between the fatal crash sites and the comparison sites are correlated with the occurrence of these fatal crashes. They cited studies that showed that crash rates on horizontal curves and vertical grades were significantly higher than those on tangent sections, and they assumed by extension that encroachment rates would also be similarly affected by horizontal curves and vertical grades. The developers also stated their belief that the adjustment factors overstated the effects of curvature on encroachment rates, but represented the best information available at the time of the study.

NCHRP Project 16-04 (Dixon et al. 2008) was initiated to develop design guidelines for safe and aesthetically pleasing roadside treatments in urban areas and a toolbox of effective roadside treatments to balance the safety and mobility needs of pedestrians, bicyclists, and motorists, and accommodate community values. In fulfilling the first of those objectives, researchers recommended the following guidelines for roadside treatments:

- Where possible at curb locations, provide a lateral offset to rigid objects of at least 6 ft from the face of the curb and maintain a minimum lateral offset of 4 ft.
- At lane merge locations, do not place rigid objects in an area that is 10 ft longitudinally from the taper point. This will result in a 20-ft object-free length at the taper point. The lateral offset for this 20-ft section should be consistent with the lane width, typically 12 ft.
- Although many auxiliary lanes, such as bus lanes or bicycle lanes, have low volumes and may be included as part of a clear zone in the urban environment, higher-speed auxiliary lane locations, such as extended length right-turn lanes, are common locations for run-off-road crashes. A lateral offset of 6 ft from the curb face to rigid objects is preferred, and a 4-ft minimum lateral offset should be maintained.
- At locations where a sidewalk buffer is present, such as in Figure 8, rigid objects are not to be located in a buffer area with a width of 3 ft or less. For buffer widths greater than 3 ft, lateral offsets from the curb face to rigid objects are to be maintained with a minimum offset of 4 ft. At these wider buffer locations, other frangible objects can be strategically located to help shield any rigid objects.
- Rigid objects should not be located in the proximity of driveways, and care is to be taken to avoid placing rigid objects on the immediate far side of a driveway. In addition, objects are not to be located within the required sight triangle for a driveway.

FIGURE 8 Example of buffer between sidewalk and street (*Credit*: Marcus Brewer, Texas Transportation Institute).

Safety Treatments

Volume 3 of *NCHRP Report 500* (Neuman et al. 2003a) discusses modifying the clear zone in proximity to trees to reduce crashes.

> This strategy involves any change to the sideslope or roadside clear zone designed to reduce the likelihood of tree crashes by increasing the chances that a [run-off-road] (ROR) vehicle can successfully recover without striking a tree. While both tree removal and shielding strategies modify the roadside, this strategy may be implemented in a variety of ways, such as flattening or grading sideslopes, regrading ditch sections, adding shoulder improvements, or providing protective plantings on the roadside. [The authors state that] the cost to modify the roadside is often considerably higher than tree removal and guardrail installation; however, applying this strategy on specific curves or short tangent sections of roadway may help manage the costs.

The authors of *NCHRP Report 500* add that this strategy has been proven to reduce the severity of ROR crashes and rollover crashes. Although they identified no specific studies that related to only trees, much work has been completed on the benefits of improving the geometry of the roadside to allow vehicles to recover when they encroach on the roadside.

SUMMARY OF KEY FINDINGS

This section summarizes key findings from the research noted in this chapter. This is an annotated summary; conclusions and recommendations are those of the authors of the references cited.

Allocation of Traveled Way Width

- The benefits of 2+1 roads in Europe validated a recommendation for their use in the United States to serve as an intermediate treatment between an alignment with periodic passing lanes and a full four-lane alignment.

Such 2+1 roads are most suitable for level and rolling terrain, with installations to be considered on roadways with traffic flow rates of no more than 1,200 veh/hr in a single direction. The use of a cable barrier as a separator is discouraged, and major intersections should be located in the buffer or transition areas between opposing passing lanes, with the center lane used as a turning lane (Potts and Harwood 2003).

- Passing activity on 2+1 roads was greatest at the beginning of the segments and the greatest benefits of decreased platooning and increased safety occurred within the first 0.9 mi of a passing lane segment (Gattis et al. 2006).
- Most passing on Super 2 passing lanes occurs within the first mile of a passing lane, so additional length may be less useful than additional lanes in a Super 2 corridor, particularly at lower volumes. Designers should avoid intersections with state highways and high-volume county roads within passing lanes, consider terrain and right-of-way in determining alignment and placement of passing lanes, avoid the termination of passing lanes on uphill grades, and discourage passing lane lengths longer than 4 mi (Brewer et al. 2011).
- TWLTLs could be used as a strategy to reduce head-on collisions on two-lane roads (Neuman et al. 2003b).

Lane Width

- Researchers investigating the relationship between lane width and safety on urban and suburban arterials found no general indication that the use of lanes narrower than 12 ft on urban and suburban arterials increased crash frequencies. They suggested that geometric design policies should provide substantial flexibility for use of lane widths narrower than 12 ft (Potts et al. 2007b).
- Lane widths of 11 or 12 ft provide optimal safety benefit for common values of total paved width on rural two-lane roads. Although 12-ft lanes appear to be the optimal design for 26- to 32-ft total paved widths, 11-ft lanes perform equally well or better than 12-ft lanes for 34- to 36-ft total paved widths (Gross et al. 2009).

Road Diet

- Road diet crashes occurring during the period after installation were about 6% lower than that of matched comparison sites. However, controlling for possible differential changes in ADT, study period, and other factors indicated no significant effect of the treatment. Crash severity was virtually the same at road diets and comparison sites. Conversion to a road diet should be made on a case-by-case basis in which traffic flow, vehicle capacity, and safety are all considered (Huang et al. 2002).
- The effects of the road diet on crashes in Iowa, accounting for monthly crash data and estimated volumes for treatment and comparison sites, resulted in a 25.2% reduction in crash frequency per mile and an 18.8% reduction in crash rate (Pawlovich et al. 2006).

Shoulder Width

- For horizontal curves on two-lane nonresidential facilities that have 3 degrees of curvature, the width of the lane plus the paved shoulder should be at least 5.5 m (18 ft) throughout the length of the curve (Staplin et al. 2002).
- Wider lane and shoulder widths are associated with a reduction in segment-related collisions on rural frontage road segments (Lord and Bonneson 2007).

Rumble Strips

- Crashes at approximately 210 mi of undivided rural two-lane roads treated with CLRS were reduced by 14% and injury crashes by an estimated 15%. All frontal and opposing-direction sideswipe crashes were reduced by an estimated 21%, and those crashes involving injuries by an estimated 25%. All of the reductions were determined to be statistically significant (Persaud et al. 2003).
- Crash data on roads treated with CLRS or shoulder rumble strips revealed noticeable crash reductions on all classes of roads (rural and urban two-lane roads and freeways). Shoulder rumble strips should be placed as close to the edgeline as possible to maximize safety benefits. The safety benefits of CLRS for roadways on horizontal curves and on tangent sections are for practical purposes the same (Torbic et al. 2009).

Shoulder Edge Treatments

- Plaxico et al. (2005) made the following recommendations on design guidelines for using curbs on roadways with operating speeds greater than 60 km/h (37.3 mph):
  - Any combination of a sloping-faced curb that is 150 mm (6 in.) or shorter and a strong-post guardrail can be used where the curb is flush with the face of the guardrail up to an operating speed of 85 km/h.
  - Guardrails installed behind curbs are not to be located closer than 2.5 m (8.2 ft) for any operating speed in excess of 60 km/h (37.3 mph).
  - For roadways with operating speeds of 70 km/h (43.5 mph) or less, guardrails may be used with sloping-face curbs no taller than 150 mm (6 in.) as long as the face of the guardrail is located at least 2.5 m (8.2 ft) behind the curb.
  - Where guardrails are installed behind curbs on roads with operating speeds between 71 and 85 km/h (44.1 and 52.8 mph), a lateral distance of at least 4 m (13.1 ft) is needed to allow the vehicle suspension to return to its pre-departure position.
  - At operating speeds greater than 85 km/h (52.8 mph), guardrails are only to be used with 100-mm (4-in.) or shorter sloping-faced curbs, and be placed so that the curb is flush with the face of the guardrail. Operating

speeds above 90 km/h (55.9 mph) require that the sloping face of the curb must be 1:3 or flatter and must be no more than 100 mm (4 in.) in height.

- The "Safety Edge" treatment produced small but positive results in crash reduction at 56 of 81 treated sites. For all two-lane highway study sites in two states, the best estimate of the treatment's effectiveness was a reduction in total crashes of approximately 5.7%. The results were not statistically significant, but they were generally positive (Hallmark et al. 2006).

Roadside

- Where possible at curb locations, provide a lateral offset to rigid objects of at least 6 ft from the face of the curb and maintain a minimum lateral offset of 4 ft (Dixon et al. 2008).
- At lane merge locations, do not place rigid objects in an area that is 10 ft longitudinally from the taper point. The lateral offset for this 20-ft section is to be consistent with the lane width, typically 12 ft (Dixon et al. 2008).
- A lateral offset of 6 ft from the curb face to rigid objects is preferred for higher-speed auxiliary lane locations, such as extended length right-turn lanes, and a 4-ft minimum lateral offset is to be maintained (Dixon et al. 2008).
- At locations where a sidewalk buffer is present, rigid objects are not to be located in a buffer area with a width of 3 ft or less. For buffer widths greater than 3 ft, lateral offsets from the curb face to rigid objects must be maintained with a minimum offset of 4 ft. At these wider buffer locations, other frangible objects can be strategically located to help shield any rigid objects (Dixon et al. 2008).
- Rigid objects are not to be located in the proximity of driveways, and care should be taken to avoid placing rigid objects on the immediate far side of a driveway. In addition, objects should not be located within the required sight triangle for a driveway (Dixon et al. 2008).

CHAPTER FIVE

INTERSECTIONS

OVERVIEW

Design of and warrants for auxiliary lanes were common topics of interest during the past decade. Researchers also revisited effects of and treatments for skewed approaches, channelization, and other intersection configuration elements. Consideration of bicycles and pedestrians at intersections was promoted through a variety of initiatives, particularly related to pedestrians with disabilities. The subject that captured the attention of many, however, was modern roundabouts; the design, installation, and operation of these alternatives to signalization were the subject of much research, and lead to the publication of two FHWA *Guides* during this time. In addition, other innovative intersection designs appeared in research and on highway networks; many of these new intersections are designed with the purpose of improving capacity by changing the manner in which left-turn movements are accommodated. Finally, access management near intersections also garnered a great deal of interest, as researchers looked for ways to minimize impacts of adjacent driveways and side streets.

INTERSECTION CONFIGURATION

Kindler et al. (2004) described the development of an expert system for diagnostic review of at-grade intersections on rural two-lane highways. This system, the Intersection Diagnostic Review Module (IDRM), was developed as a component of the Interactive Highway Safety Design Model to aid designers in assessing the safety consequences of geometric design decisions, particularly for combinations of geometric features. IDRM was developed to allow such problems to be identified and evaluated in an automated and organized fashion. IDRM identifies concerns by "using models of the criticality of specific geometric design situations. These include existing geometric design models—such as sight distance models—as well as newly developed models. IDRM uses 21 specific models to address 15 high-priority issues related to [the] intersection as a whole and [to individual] approach" legs. "IDRM makes no attempt to select a particular treatment as appropriate to the intersection. After further investigation, the IDRM user may select a particular treatment as appropriate [on the basis of the] available evidence and engineering judgment, or the user may conclude that no treatment is necessary and that the project should be built as designed."

ALIGNMENT

FHWA's *Signalized Intersections: Informational Guide* (Rodegerdts et al. 2004) states that "the approach to a signalized intersection should promote awareness of an intersection by providing the required stopping sight distance in advance of the intersection." The document recommends the following guidelines to meet drivers' and cyclists' expectations as they approach intersections:

- "Avoid approach grades to an intersection of greater than 6%. On higher design speed facilities (50 mph and greater), a maximum grade of 3% should be considered.
- Avoid locating intersections along a horizontal curve of the intersecting road.
- Strive for an intersection platform (including sidewalks) with cross slope not exceeding 2%, as needed for accessibility."

Approach curvature is a geometric design treatment that can be used at high-speed intersection approaches to force a reduction in vehicle speed through the introduction of horizontal deflection, as described in *NCHRP Report 613* (Ray et al. 2008). As shown in Figure 9, approach curvature consists of successive curves with progressively smaller radii. Research and applications of approach curvature previously focused on roundabouts. However, the report states that this geometric design treatment has potential to be applied to conventional intersections as well.

The use of approach curvature at downhill approaches was discouraged. The report authors' experience with approach curvature suggested that this geometric treatment can be used in conjunction with reduced speed limit signs or advisory speed signs. The length and curve geometry can be determined from the upstream segment operating speed and the target speed and appropriate design vehicle for the intersection.

EFFECT OF SKEW

Son et al. (2002) developed a methodology for calculating sight distance available to drivers at skewed unsignalized intersections. The methodology considered that the sight distance may vary depending on the driving positions of the drivers and the different lines of sight given to drivers by different

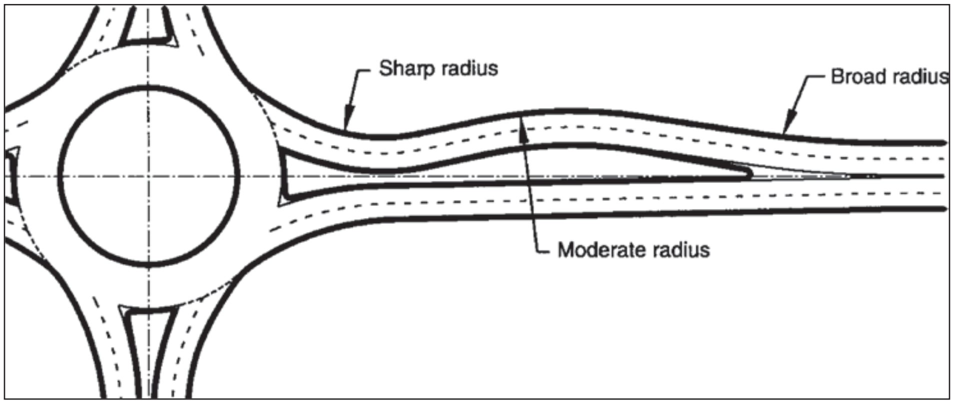


FIGURE 9 Example of approach curvature (Robinson et al. 2000).

types of vehicles with unique sight-line obstructions. They derived equations and nomographs for calculating available sight distance that included the influence of factors such as geometry (intersection angle, lane width, shoulder width, position of stop line), vehicle dimensions, and the driver's field of view. They concluded that findings from their research provided evidence that a skew angle greater than 20 degrees should not be used in design when the design vehicle is a large vehicle or semitrailer, because available sight distances were less than the required stopping sight distance, even with a low value of design speed for intersection angles less than 70 degrees.

The *Highway Design Handbook for Older Drivers and Pedestrians* (Staplin et al. 2002) recommends establishing 15 degrees as a minimum skew angle as a practice to accommodate age-related performance deficits at intersections where right-of-way is restricted; "at skewed intersections where the approach leg to the left intersects the driver's approach leg at an angle of less than 75 degrees, the prohibition of right turn on red (RTOR) is recommended." The *Handbook* cites multiple studies documenting restricted neck movement in older drivers, making detection of and judgments about potential conflicting vehicles on crossing roadways much more difficult.

AUXILIARY LANE DESIGN

Harwood et al. (2002) conducted a study to investigate the safety effectiveness of left- and right-turn lane treatments. The research team collected geometric design, traffic control, traffic volume, and traffic crash data at 280 improved sites in eight states and at 300 similar intersections that were not improved during the study period. The types of improvement projects evaluated included installation of added left-turn lanes, installation of added right-turn lanes, installation of added left- and right-turn lanes as part of the same project, and extension of the length of existing left- or right-turn lanes. Based on the results of the analyses, they concluded the following:

- Adding left-turn lanes is effective in improving safety at signalized and unsignalized intersections. Installing a single left-turn lane on a major-road approach would be expected to reduce total intersection accidents at rural unsignalized intersections by 28% for four-leg intersections and by 44% for three-leg intersections, with corresponding reductions of 27% and 33% at urban unsignalized intersections. At four-leg urban signalized intersections, installation of a left-turn lane on one approach would be expected to reduce accidents by 10%, and installation on both major-road approaches would be expected to increase, but not quite double, the resulting effectiveness measures for total intersection accidents.
- Positive results can also be expected for right-turn lanes, with reductions in total intersection accidents of 14% at rural unsignalized locations and 4% at urban signalized locations for installations on single approaches. "Installation of right-turn lanes on both major-road approaches to four-leg intersections would be expected to increase, but not quite double, the resulting effectiveness measures for total intersection accidents."
- "In general, turn-lane improvements at rural intersections resulted in larger percentage reductions in accident frequency than comparable improvements at urban intersections."
- "Overall, there [was] no indication that any type of turn-lane improvement is either more or less effective for different accident severity levels."

FHWA's *Highway Design Handbook for Older Drivers and Pedestrians* (Staplin et al. 2002) states that "two factors can compromise the ability of older drivers to remain within the boundaries of their assigned lanes during a left turn. One factor is the diminishing ability to share attention (i.e., to assimilate and concurrently process multiple sources of information from the driving environment). The other factor involves the ability to turn the steering wheel sharply enough, given the speed at which they are traveling, to remain within the boundaries of their lanes." Data sources cited by the *Handbook*'s authors indicated that a 12-ft lane width provides the most reasonable tradeoff between the need to accommodate older drivers, as well as larger turning vehicles, without penalizing the older pedestrian in terms of exaggerated crossing distance. The *Handbook's* corresponding recommendation was for a minimum receiving lane width of 12 ft accompanied, wherever practical, by a shoulder of 4 ft minimum width.

The *Handbook* further recommended that, for new or reconstructed facilities, unrestricted sight distance, achieved through positive offset of opposing left-turn lanes, be provided whenever possible. This recommendation was made in anticipation of providing "a margin of safety for older drivers who, as a group, do not position themselves within the intersection before initiating a left turn." Where the provision of unrestricted sight distance is not feasible, positive left-turn lane offsets were recommended to achieve the minimum required sight distances appropriate for major roadway design speed and type of opposing vehicle.

Long (2002) reviewed the characteristics of intervehicle spacing for the purpose of auxiliary lane design. He concluded that the value of 25 ft per vehicle used by the CORSIM modeling software was a severe underestimation for determining queue lengths, as was the 3-ft distance between vehicles. He developed new models for estimating average queue lengths and maximum lengths at a given probability; models based on an intervehicle spacing of 12 ft, a passenger car length of 15 ft, a 65-ft length for combination trucks, and 30 ft for other vehicles.

Kikuchi et al. (2005) developed a method for estimating the needed length of dual left-turn lanes (DLTL). Their procedure first surveyed how drivers choose a lane of the DLTL in the real world and analyzed the relationship between lane use and the volume of left-turn vehicles. Second, the procedure calculated the probability that all arriving left-turn vehicles during the red phase could enter the left-turn lanes (i.e., no queue spillback of vehicles from the DLTL and no blockage of the DLTL by the queue of through vehicles). This probability was presented as a function of the length of the DLTL and the arrival rates of left-turn and through vehicles. The adequate lane length was derived such that the probability of the vehicles entering the DLTL is greater than a threshold value. Third, the recommended adequate length was expressed in number of vehicles, then converted to the actual distance required based on the vehicle mix and preference between the two lanes. Resulting recommended lengths were presented as a function of left-turn and through volumes for practical application.

Lee et al. (2005) developed models to predict lane utilization factors for six types of intersections with downstream lane drops and to assess how low lane utilization affects the observed intersection capacity and level of service. They collected traffic and signal data at 47 sites in North Carolina. On the basis of 15 candidate factors, multiple regression models were developed for predicting the lane utilization factor. They compared field-measured delays with delays estimated by the *HCM* with the use of regression models for lane utilization. They stated that even with the new models for lane utilization, the *HCM* consistently overestimated delay for all types of lane-drop intersections with low lane utilization and suggested that a reassessment of the effect of lane utilization on capacity may be in order. The study also found that the downstream lane length and traffic intensity positively correlated with the lane utilization factor, existence of a TWLTL or midblock left-turn bay increased the lane utilization factor, lane drops resulting from lane usage change had more equal lane volume distribution than the midblock taper lane drop, and that some geometric variables at the approach may also influence lane utilization.

Fitzpatrick et al. (2006) conducted a study to determine variables that affected the speeds of free-flow turning vehicles in an exclusive right-turn lane and explore the safety experience of different right-turn lane designs. Their evaluations found that the variables affecting the turning speed at an exclusive right-turn lane included the type of channelization present (either lane line or raised island), lane length, and corner radius. Variables that affected the turning speed at an exclusive right-turn lane with island design included (1) radius, lane length, and island size at the beginning of the turn, and (2) corner radius, lane length, and turning-roadway width near the middle of the turn. The authors compared this with previous research treatments that had the highest number of crashes were right-turn lanes with raised islands. In their analysis, they found this type of intersection had the second highest number of crashes of the treatments evaluated in this study. In both studies, the "shared through with right lane combination" had the lowest number of crashes. They recommended that these findings be verified through use of a larger, more comprehensive study that includes right-turning volume.

NCHRP Project 3-72 was tasked with developing design guidance related to right-turn lanes on urban and suburban arterials. The research team from Midwest Research Institute discussed results from their research with respect to right-turn deceleration lanes (Potts et al. 2007a). They conducted a computer simulation study of motor vehicles and pedestrians at right-turn lanes to determine their operational effects. They also performed a benefit-cost analysis of right-turn lanes that considered both operational and safety effects. The researchers determined that right-turn maneuvers from a two-lane arterial at an unsignalized intersection or driveway can delay through traffic by 0 to 6 s per through vehicle where no right-turn lane was present. Delays to through traffic owing to right turns in the same situation on a four-lane arterial were substantially lower, in the range from 0 to 1 s per through vehicle. They concluded that pedestrians at unsignalized intersections or driveways can have a substantial impact on delay to through vehicles owing to slowing of right-turning vehicles yielding to pedestrians, but provision of a right-turn lane could reduce pedestrian-related delays to through traffic by as much as

6 s per through vehicle, depending on pedestrian volume. Results from the project's economic analysis procedure developed a method to identify where installation of right-turn lanes at unsignalized intersections and major driveways would be cost-effective, indicating combinations of through-traffic volumes and right-turn volumes for which provision of a right-turn lane would be recommended. The research team stated that their economic analysis procedure can be applied by highway agencies using site-specific values for ADTs, turning volumes, accident frequency, and construction cost for any specific location (or group of similar locations) of interest.

Kikuchi et al. (2007) examined the lengths of turn lanes when a single lane approached a signalized intersection and was divided into three lanes: left-turn, through, and right-turn. Their objective was to determine the appropriate length of each turn lane. From analysis of the vehicle queue pattern at the entrance to the turn lanes, they developed a set of formulas to compute the probabilities of the occurrence of turn-lane overflow and turn-lane blockage. The recommended lane lengths were calculated so that the probabilities that a lane did not overflow and that the entrance of the lane was not blocked were greater than a threshold value of 0.95. Recommended turn-lane lengths, presented in a series of tables, were found to be shorter than those recommended by AASHTO.

In a subsequent study, Kikuchi and Kronprasert (2008) developed analytical and computational processes for determining the length of the right-turn lane at a signalized intersection. They examined the factors that influenced length, reviewed available literature and practices, derived recommended lengths analytically, and developed a set of tables of recommended lane lengths as a function of approach volumes (right-turn, through-traffic, and cross-traffic volumes) and signal timing. Their analysis compared conditions when right-turn-on-red (RTOR) was not permitted and when it was permitted. Based on achieving desired probabilities of turn-lane overflow and turn-lane blockage, they calculated recommended lane lengths based on the number of vehicle spaces and described a procedure to convert that number to actual distance. They compared their guidelines that account for arrival rates of both right-turn and through vehicles with guidelines that only considered right-turn vehicles; as a result, they concluded their proposed lane lengths were different than those in existing guidelines. Their recommended lengths for RTOR conditions were somewhat shorter than non-RTOR conditions when the right-turn arrival rate was greater than the arrival rate for through vehicles.

FHWA's *Highway Design Handbook for Older Drivers and Pedestrians* (Staplin et al. 2002) recommends "raised channelization with sloping curbed medians rather than channelization accomplished through the use of pavement markings, for the following operating conditions:

- Left- and right-turn lane treatments at intersections on all roadways with operating speeds of less than 40 mph.
- Right-turn treatments on roadways with operating speeds equal to or greater than 40 mph."

Where raised channelization is implemented at intersections, they also recommended that median and island curb sides and curb horizontal surfaces be treated with retroreflectorized markings and be maintained at a minimum luminance contrast level of 2.0 with overhead lighting or 3.0 without overhead lighting.

INTERSECTION SIGHT DISTANCE

Where determinations of intersection sight distance (ISD) requirements for any intersection maneuver that is performed by a driver on either a major or a minor road incorporate a PRT component, the FHWA *Highway Design Handbook for Older Drivers and Pedestrians* (Staplin et al. 2002) recommends that a PRT value of no less than 2.5 s be used to accommodate the slower decision times of older drivers. It also recommends that "where determinations of intersection sight-distance requirements for a left-turn maneuver from a major roadway by a stopped passenger car are based on a gap model, a gap of no less than 8.0 s, plus 0.5 s for each additional lane crossed by the turning driver, be used to accommodate the slower decision times of older drivers."

Yan and Radwan (2005) conducted research to develop sight distance geometric models for unprotected left-turning vehicles from the major road to the minor road at signalized intersections; they also sought to evaluate sight improvement effects of two offset methods and analyze the relationship between available sight distance and selected geometric parameters. According to their conclusions, sight distance problems could occur for passenger cars on traditional left-turn lane designs with 14- to 18-ft medians at high design speeds. Using sensitivity analyses, they also developed equations showing a relationship between sight distance and offset value for parallel left-turn lanes and between sight distance and taper angle for taper lanes. Left-turn lane length was also cited as an important variable that affects sight distance.

Easa and Ali (2006) developed an extension of a previous ISD model to consider sight distance for stop-control intersections on three-dimensional alignments. Although the previous model accounted for obstructions inside the horizontal curve and for intersections and major-road vehicles (objects) on the curve, their model was expanded by (1) allowing the object to be anywhere on the horizontal curve or tangent, (2) allowing the horizontal and vertical curves to overlap partially, and (3) considering the case in which the obstruction lies outside the horizontal curve. The obstruction location was formulated through use of a simple variable that takes the value of +1 or −1 for an obstruction, respectively, inside or outside the horizontal curve. They presented design aids for the required minimum lateral clearances (from the minor and major roads) for different radii of horizontal curve and major-road design speeds. They noted that their model considers the vertical obstruction caused by the road surface on crest vertical curves, and recommended further research to explore the case of a sag vertical curve, where the sightline may be obstructed by an overpass.

MODERN ROUNDABOUTS

The increase in the use of modern roundabouts in the United States continued at a high pace during the decade from 2000 to 2010. The need to know more about design, operations, safety, and other aspects of roundabouts in this country prompted a number of research projects. Findings from those projects will be summarized in this section of the report. This section contains subtopics that overlap with headings found elsewhere in this chapter (e.g., design speed and alignment), but the roundabout-specific nature of the information made it appropriate to include here, rather than be distributed throughout other parts of the report.

NCHRP Synthesis 299 had little content on roundabouts, because, to that point, relatively little research had been conducted on them in the United States. A series of projects during the decade led to the publication of two FHWA *Informational Guides* containing recommendations and guidelines for all aspects of roundabout design. As such, a great deal of content was generated on the subject, a sample of which is presented in this section. Many projects were regional or local in nature, however, and specific research reports on many of those projects were not published in a forum that was readily available for this synthesis. However, the FHWA *Informational Guides* summarized much of the existing information and compiled them into the form of nationally distributed research reports as well as guidelines suitable for practitioners. Given the importance of those two *Guides,* a sizeable portion of the research highlighted here is either primarily or secondarily sourced to those two documents.

General Principles

The authors of *NCHRP 672: Roundabouts: An Informational Guide,* Second Edition (Rodegerdts et al. 2010) offered a set of overarching principles to guide the development of designs for all roundabouts. They stated that achieving these principles be the goal of any roundabout design:

- "Provide slow entry speeds and consistent speeds through the roundabout by using deflection.
- Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume balance, and lane continuity.
- Provide smooth channelization that is intuitive to drivers and results in vehicles naturally using the intended lanes.
- Provide adequate accommodation for the design vehicles.
- Design to meet the needs of pedestrians and cyclists.
- Provide appropriate sight distance and visibility for driver recognition of the intersection and conflicting users."

Design Speed

The authors of the first edition of FHWA's *Roundabouts: An Informational Guide* (Robinson et al. 2000) stated that design speed of a roundabout is determined from the smallest radius along the fastest allowable path. In their observations, the smallest radius usually occurred on the path of the circulatory roadway as the vehicle curved to the left around the central island. However, they stated it was "important when designing the roundabout geometry that the radius of the entry path (i.e., as the vehicle curves to the right through entry geometry) not be significantly larger than the circulatory path radius." Recommended maximum entry design speeds from the *Guide* are shown in Table 17.

The design process described in the *Guide* states that

> a vehicle is assumed to be 2 m (6 ft) wide and to maintain a minimum clearance of 0.5 m (2 ft) from a roadway centerline or concrete curb and flush with a painted edge line. Thus the centerline of the vehicle path is drawn with the following distances to the particular geometric features:
>
> - 1.5 m (5 ft) from a concrete curb,
> - 1.5 m (5 ft) from a roadway centerline, and
> - 1.0 m (3 ft) from a painted edge line.

Their desirable radius relationship was that the entry path radius was less than the circulatory path radius, which was less than the exit path radius, ensuring that speeds will be reduced to their lowest level at the roundabout entry. The design speed review process also included the evaluation of the left-turn path radius and the right-turn path radius for speeds consistent with the other three radii. Selection of an appropriate design vehicle, as defined in the *Green Book,* would help to define the necessary radii for a given design speed.

The second edition of *Roundabouts: An Informational Guide* (Rodegerdts et al. 2010) recommends maximum entering design speeds based on a theoretical fastest path of 20 to 25 mph for single-lane roundabouts; 25 to 30 mph is recommended for multilane roundabouts, based on a theoretical fastest path assuming vehicles ignore all lane lines.

TABLE 17
RECOMMENDED MAXIMUM ENTRY DESIGN SPEEDS

Site Category	Recommended Maximum Entry Design Speed, km/h (mph)
Mini-roundabout	25 (15)
Urban Compact	25 (15)
Urban Single Lane	35 (20)
Urban Double Lane	40 (25)
Rural Single Lane	40 (25)
Rural Double Lane	50 (30)

Source: Robinson et al. 2000.

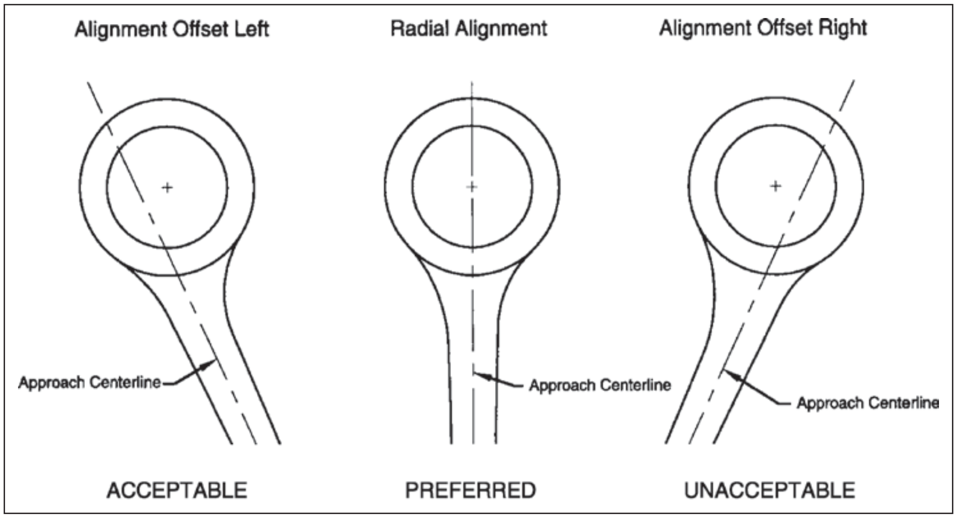


FIGURE 10 Radial alignment of roundabout entries (Robinson et al. 2000).

Alignment

With regard to the alignment of roundabout approaches, the first FHWA *Guide* (Robinson et al. 2000) states that,

> in general, the roundabout is optimally located when the centerlines of all approach legs pass through the center of the inscribed circle. This location usually allows the geometry to be adequately designed so that vehicles will maintain slow speeds through both the entries and the exits. The radial alignment also makes the central island more conspicuous to approaching drivers. If it is not possible to align the legs through the center point, a slight offset to the left (i.e., the centerline passes to the left of the roundabout's center point) is acceptable. It is almost never acceptable for an approach alignment to be offset to the right of the roundabout's center point. This alignment brings the approach in at a more tangential angle and reduces the opportunity to provide sufficient entry curvature.

Examples of all three alignments are shown in Figure 10.

Lane Arrangement

NCHRP Report 672 (Rodegerdts et al. 2010) provides a methodology for conducting an operational analysis of a roundabout, one outcome of which is to determine the required number of entry lanes to serve each approach. The report's authors advise that, for multilane roundabouts, care must be taken to ensure that the design also provides the appropriate number of lanes within the circulatory roadway and on each exit to ensure lane continuity. The primary caution with multilane roundabouts is path overlap, which occurs when the natural path through the roundabout of one traffic stream overlaps the path of another. If the natural path of one lane interferes or overlaps with the natural path of the adjacent lane, the roundabout is not as likely to operate as safely or efficiently as possible. The report advises that a good entry design aligns vehicles into the appropriate lane within the circulatory roadway, and the design of the exits also provides appropriate alignment to allow drivers to intuitively maintain the appropriate lane. The report's authors add that these alignment considerations often compete with the fastest path speed objectives.

Inscribed Circle Diameter

In the first FHWA *Roundabouts Guide* (Robinson et al. 2000), the authors state that the inscribed circle diameter (ICD) in a single-lane roundabout should be a minimum of 30 m (100 ft) to accommodate a WB-15 (WB-50) design vehicle. "Smaller roundabouts can be used for some local street or collector street intersections, where the design vehicle may be a bus or single-unit truck. At double-lane roundabouts, accommodating the design vehicle is usually not a constraint. The size of the roundabout is usually determined either by the need to achieve deflection or by the need to fit the entries and exits around the circumference with reasonable entry and exit radii between them." Thus, the authors recommended that the ICD of a double-lane roundabout generally be a minimum of 45 m (150 ft). The second edition of the FHWA *Guide* modified the ICD recommendations from the first edition; the second edition's typical ICD ranges are shown in Table 18.

Entry Width

According to the first FHWA *Roundabouts Guide* (Robinson et al. 2000),

> determining the entry width and circulatory roadway width involves a trade-off between capacity and safety. The design should provide the minimum width necessary for capacity and accommodation of the design vehicle in order to maintain the highest level of safety. Typical entry widths for single-lane

TABLE 18
TYPICAL INSCRIBED CIRCLE DIAMETER RANGES

Roundabout Configuration	Typical Design Vehicle	Common Inscribed Circle Diameter Range*
Mini-Roundabout	SU-30	45–90 ft
Single-Lane Roundabout	B-40	90–150 ft
	WB-50	105–150 ft
	WB-67	130–180 ft
Multilane Roundabout (2 lanes)	WB-50	150–220 ft
	WB-67	165–220 ft
Multilane Roundabout (3 lanes)	WB-50	200–250 ft
	WB-67	220–300 ft

Source: Rodegerdts et al. (2010).
*Assumes 90-degree angles between entries and no more than four legs. List of possible design vehicles is not all-inclusive.

> entrances range from 4.3 to 4.9 m (14 to 16 ft); however, values higher or lower than this range may be required for site-specific design vehicle and speed requirements for critical vehicle paths.

Where wider entries are required, this can be done in two ways: by adding a full lane upstream of the entrance and maintaining parallel lanes through the entry, or by gradually widening the approach through flaring. When used, the *Guide* states that flare lengths should generally be a minimum of 25 m (80 ft) in urban areas and 40 m (130 ft) in rural areas.

The second edition of the *Guide* (Rodegerdts et al. 2010) revised the previous guidelines to state that typical entry widths for single-lane entrances range from 14 to 18 ft, which are often flared from upstream approach widths. However, values higher or lower than this range may be appropriate for site-specific design vehicle and speed requirements for critical vehicle paths. A 15-ft entry width is a common starting value for a single-lane roundabout. *NCHRP Report 672* also states that care be taken with entry widths greater than 18 ft or for those that exceed the width of the circulatory roadway, as drivers may mistakenly interpret the wide entry to be two lanes when there is only one receiving circulatory lane.

Intersection Sight Distance

Concerning ISD at roundabout approaches, the first edition of the FHWA *Guide* (Robinson et al. 2000) recommended the use of a critical headway of 6.5 s to determine the appropriate length of the conflicting leg of the sight triangle. It further recommended that designers "provide no more than the minimum required intersection sight distance on each approach. Excessive intersection sight distance can lead to higher vehicle speeds that reduce the safety of the intersection for all road users (e.g., vehicles, bicycles, pedestrians)." The authors also stated that landscaping can be effective in restricting sight distance to the minimum requirements.

NCHRP Report 672 (Rodegerdts et al. 2010) advised the use of a critical headway of 5.0 s, based on the critical headway required for passenger cars. The authors added that this value represented an interim methodology pending further research.

Superelevation

Guidelines in the first FHWA *Roundabouts Guide* state that, in general, "a cross-slope of 2% away from the central island should be used for the circulatory roadway. This technique of sloping outward [was] recommended for four main reasons:

- It promotes safety by raising the elevation of the central island and improving its visibility.
- It promotes lower circulating speeds.
- It minimizes breaks in the cross slopes of the entrance and exit lanes.
- It helps drain surface water to the outside of the roundabout."

The outward cross-slope design means vehicles making through and left-turn movements must negotiate the roundabout at negative superelevation; however, the slow speeds through the circulatory roadway were generally expected to negate the effects of the slope on drivers.

Safety

Researchers on NCHRP 3-65 (Rodegerdts et al. 2007) found that crash experience at selected intersections in the United States that had been converted to roundabouts showed an overall reduction in crash frequency; there were selected intersections at which this was not the case (e.g., either no change or a small increase in crash frequency), but in most cases, the crash counts at those locations were too small for increases to be statistically significant. In comparing crash frequency to geometry, researchers listed these findings:

- Eight of the ten sites with the lowest crash frequencies were single-lane roundabouts.
- Twenty-six of the 30 sites with the lowest crash frequencies were single-lane roundabouts.
- Two of the ten sites with the highest crash frequencies were single-lane roundabouts.
- Nine of the 30 sites with the highest crash frequencies were single-lane roundabouts.

- Crash frequency increased as the inscribed circle diameter increased, and as the number of vehicles entering the roundabout increased.
- Crash frequency increased slightly as the number of legs to the roundabout increased.

A review of multilane roundabout characteristics led the research team to believe that most sites were not designed using the natural vehicle path concept, which was likely because the design was completed before the introduction of the concept in the first edition of the FHWA Roundabout *Guide*. Lane widths also appeared to have an effect on crash frequency, particularly lanes that were narrower than those recommended by FHWA.

Analysis of roundabouts in the United States led researchers to conclude that the general principle was true that as the width of an entry increases, the capacity of the entry increases, while the safety of the entry decreases. However, extending the principle beyond the number of lanes to the actual entry width did not appear to have as strong a relationship in the United States as in other countries. Researchers suggested that, although the overall relationship between capacity and entry width appeared to hold true in terms of the aggregate number of lanes on the approach, changes in entry width within a single-lane entry has a much lower-order effect on capacity.

The NCHRP 3-65 research team also reported that the angle between legs of a roundabout appeared to have a direct influence on entering-circulating crashes. As the angle to the next leg decreased, the number of entering-circulating crashes increased, suggesting that roundabouts with more than four legs or with skewed approaches tended to have more entering-circulating crashes. Analysis of U.S. data did not find a significant relationship between the capacity of the entry and the width of the splitter island, nor with the percentage of exiting vehicles. The research team believed that as drivers became more comfortable and efficient in driving roundabouts the effect of the width of the splitter island and/or percentage of exiting vehicles could become more noticeable and recommended further study of the subject in the future.

Researchers suggested that the critical headway estimate of 6.5 s in the first edition of the FHWA *Roundabout Guide* appeared to be somewhat conservative for design purposes for both single-lane and multilane entries. They recommended a lower value of 6.2 s for design purposes, which was approximately one standard deviation above the mean observed critical headway (Rodegerdts et al. 2007).

Isebrands (2009) conducted a review of crashes at 17 intersections on rural high-speed roadways that were converted to roundabouts between 1993 and 2006; "high-speed" was defined as having a posted speed limit of 40 mph or greater. The number of years of before data averaged 4.6, with a minimum of 2.5 years and a maximum of 6.6 years. The after data showed greater variation, with an average of 5.5 years, a minimum of 1.8 years, and a maximum of 12.7 years of data. The results of her analysis showed 52% and 67% reductions in total crashes and crash rate, respectively. Moreover, the findings showed an 84% reduction in injury crashes and an 89% reduction in the injury crash rate. No fatal crashes occurred since the roundabouts were constructed, compared with 11 fatal crashes that were reported in the before period. The number of angle crashes was also reduced by 86%.

Pedestrian Considerations

The first FHWA *Roundabout Guide* (Robinson et al. 2000) discussed considerations for nonmotorized users. The authors provided design dimensions from Pein (1996) for key roundabout design features, which are largely repeated in the second *Roundabout Guide* and are included in Table 19. The *Guide* added that "pedestrian crossing locations must balance pedestrian convenience, pedestrian safety, and roundabout operations." With those issues in mind, the *Guide* recommended

TABLE 19
KEY DIMENSIONS OF NONMOTORIZED ROUNDABOUT DESIGN USERS

User	Dimension (ft)	Affected Roundabout Features
Bicycle		
Length	5.9	Splitter island width at crosswalk
Minimum operating width	4.0	Bicycle lane width
Lateral clearance on each side	2.0 3.3 to obstructions	Shared bicycle–pedestrian path width
Pedestrian (walking)		
Width	1.6	Sidewalk width, crosswalk width
Wheelchair		
Minimum width	2.5	Sidewalk width, crosswalk width
Operating width	3.0	Sidewalk width, crosswalk width
Person pushing stroller		
Length	5.6	Splitter island width at crosswalk
Skater		
Typical operating width	6.0	Sidewalk width

Source: Pein (1996).

that pedestrian crossings be designed with the following characteristics:

- "The pedestrian refuge should be a minimum width of 1.8 m (6 ft) to adequately provide shelter for persons pushing a stroller or walking a bicycle.
- At single-lane roundabouts, the pedestrian crossing should be located one vehicle-length (7.5 m [25 ft]) away from the yield line. At double-lane roundabouts, the pedestrian crossing should be located one, two, or three car lengths (approximately 7.5 m, 15 m, or 22.5 m [25 ft, 50 ft, or 75 ft]) away from the yield line.
- The pedestrian refuge should be designed at street level, rather than elevated to the height of the splitter island. This eliminates the need for ramps within the refuge area, which can be cumbersome for wheelchairs.
- Ramps should be provided on each end of the crosswalk to connect the crosswalk to other crosswalks around the roundabout and to the sidewalk network.
- It is recommended that a detectable warning surface, as recommended in the Americans with Disabilities Act Accessibility Guidelines (ADAAG), be applied to the surface of the refuge within the splitter island."

The second edition of the *Guide* recommended minimum splitter island dimensions, as shown in Figure 11, and the authors encouraged use of standard AASHTO island design for key dimensions, such as offset and nose radii. For sidewalks, authors advised a setback distance of 1.5 m (5 ft), with a minimum of 0.6 m (2 ft).

Research on NCHRP Project 3-65 included the review of pedestrian crossing activity at 10 legs on seven roundabout study sites (Rodegerdts et al. 2007). The researchers determined that the majority of the 769 observed crossing events involved no interaction with a motor vehicle, where interaction is defined as the pedestrian either accepting or rejecting a gap when a vehicle was present. For those pedestrians who did interact with vehicles and ultimately crossed the leg, researchers categorized their behaviors as Normal, Hesitates, or Runs. Three categories of motorist yielding behavior were identified as well:

- Active yield: The motorist slowed or stopped for a crossing pedestrian or a pedestrian waiting on the curb or splitter island to cross. The pedestrian was the only reason the motorist stopped or slowed.
- Passive yield: The motorist yielded to the pedestrian but was already stopped for another reason. This situation occurred most often when there was a queue of vehicles waiting to enter the roundabout or when the vehicle was already stopped for a prior pedestrian crossing event.
- Did not yield: The motorist did not yield to a crossing pedestrian or a pedestrian waiting on the curb or splitter island to cross.

Researchers determined that for pedestrians initiating a crossing on the entry side of one-lane sites, 15% of motorists did not yield to the pedestrian on either the entry or exit side. The remainder of the exit-side vehicles actively yielded. The remainder of the entry-side vehicles included 20% that were classified as passively yielding. For two-lane sites, the percentage of nonyielding vehicles increased to 33% on the entry side and 45% on the exit side. For those vehicles that did yield, 9% and 2% were classified as passive yield for the entry and exit sides, respectively. When crossing began on the exit side, yielding improved for entry-side drivers but declined for exit-side drivers. In all categories, yielding at two-lane sites was lower than at one-lane sites. On average across all sites, approximately 30% of the motorists

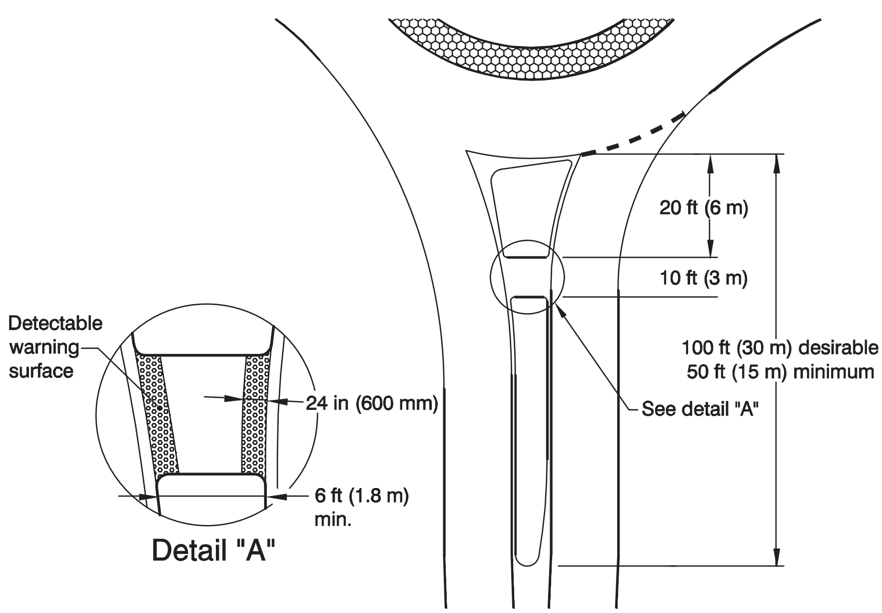


FIGURE 11 Minimum splitter island dimensions (Rodegerdts et al. 2010).

did not yield to pedestrians who were crossing or waiting to cross, although in all but one case the pedestrians were waiting to cross, so there was no imminent risk identified by the research team. Researchers also observed only four conflicts in the 769 crossing events. Comparison with findings from a separate FHWA study (Carter et al. 2005b) indicated that driver yielding at roundabouts was better than at uncontrolled approaches, but not as high as at stop signs or traffic signals. The researchers suggested that design changes could include reductions in exit radii, reductions in lane widths, and/or relocation of crosswalks (Rodegerdts et al. 2007).

Bicycle Considerations

The first FHWA *Roundabout Guide* (Robinson et al. 2000) recommended that the "designer should strive to provide bicyclists the choice of proceeding through the roundabout as either a vehicle or a pedestrian." The *Guide* stated that, "in general, bicyclists are better served by treating them as vehicles; however, the best design provides both options to allow cyclists of varying degrees of skill to choose their more comfortable method of navigating the roundabout."

According to the *Guide,* to "accommodate bicyclists traveling as vehicles, bike lanes should be terminated in advance of the roundabout to encourage cyclists to mix with vehicle traffic." Under this treatment, it was recommended that bike lanes end 100 ft upstream of the yield line to allow for merging with vehicles. This method is most successful at smaller roundabouts with speeds below 20 mph, where bicycle speeds can more closely match vehicle speeds.

To accommodate bicyclists who prefer not to use the circulatory roadway, the *Guide* advised that "a widened sidewalk or a shared bicycle/pedestrian path may be provided physically separated from the circulatory roadway [i.e.,] not as a bike lane within the circulatory roadway. Ramps or other suitable connections [could] then be provided between this sidewalk or path and the bike lanes, shoulders, or road surface on the approaching and departing roadways." The designer was advised to exercise care in locating and designing the bicycle ramps so that they are not misconstrued by pedestrians as unmarked pedestrian crossings, nor should the exits from the roadway onto a shared path allow cyclists to enter the shared path at excessive speeds.

The second edition of the *Roundabouts Guide* (Rodegerdts et al. 2010) advises that, for nonmotorized users, one important consideration during the initial design stage is to maintain or obtain adequate right-of-way outside the circulatory roadway for the sidewalks. All nonmotorized users who are likely to use the sidewalk regularly, including bicyclists in situations where roundabouts are designed to provide bicycle access to sidewalks, should be considered in the design of the sidewalk width. Report authors recommended that bicycle lanes not be provided through the roundabout and be terminated upstream of the entrance line. They recommended designs that encourage bicycle users to merge into the general travel lanes and navigate the roundabout as a vehicle, explaining that the typical vehicle operating speed within the circulatory roadway is in the range of 15 to 25 mph, which is similar to that of a bicycle. Because multilane roundabouts are more challenging for bicyclists, additional design features may be appropriate for those locations.

INNOVATIVE DESIGNS

A number of new, innovative, or otherwise unique intersection designs were topics of considerable attention between 2000 and 2010. An FHWA study by Hughes et al. (2010) examined four alternative intersection designs, reviewing characteristics related to geometric design, access management, traffic control devices, and other features. The four designs included Displaced Left-Turn (DLT), Median U-Turn (MUT), Restricted Crossing U-Turn (RCUT), and Quadrant Roadway (QR) intersections. Findings from that study and others related to the geometric design of those intersection types are summarized here.

Displaced Left-Turn

The main feature of the DLT alternative intersection is the relocation of the left-turn movement on an approach to the other side of the opposing roadway, which consequently eliminates the left-turn phase for this approach at the main intersection (Hughes et al. 2010). Traffic that would normally turn left at the main intersection first crosses the opposing through lanes at a signal-controlled intersection several hundred feet upstream of the main intersection. Left-turning vehicles then travel on a new roadway parallel to the opposing lanes and execute the left-turn maneuver simultaneously with the through traffic at the main intersection. The dashed line in Figure 12 illustrates a typical left-turn maneuver at a DLT intersection. The layout in Figure 12 is for a full version, which has DLT movements on all four approaches; after the eastbound vehicles turn northbound, they must travel through another crossover for southbound left-turning vehicles. This design reflects a shift of the through traffic lanes into the median in an attempt to minimize the need for additional right-of-way. At several locations where DLT intersections have been implemented as a retrofit to an existing conventional at-grade intersection, the existing median has been preserved, and there is no shift in the through lanes. DLT can also be installed at a three-legged intersection with the displacement on the major road in only one direction.

A study by Jagannathan and Bared (2005) investigated the design and operational performance of the DLT, then also known as the crossover displaced left-turn (XDL) or the continuous-flow intersection. The researchers' purpose was to provide a simplified procedure to evaluate the DLT's traffic performance and compare it with conventional intersections. Using microsimulation, they modeled typical geometries

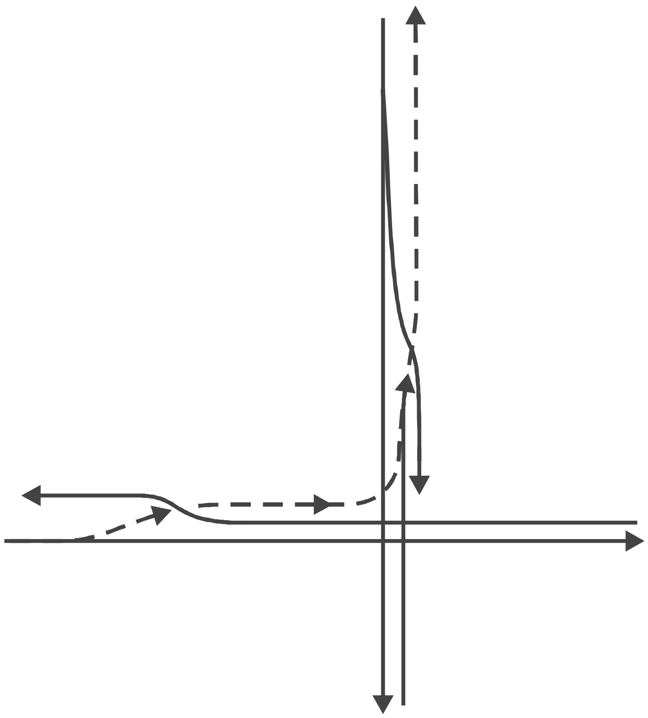


FIGURE 12 Left-turn movement on a typical DLT intersection approach (eastbound to northbound).

over a wide distribution of traffic flow conditions for three different design configurations or cases. They concluded that their comparisons with conventional intersections showed considerable savings in average control delay and average queue length, as well as an increase in intersection capacity. They also concluded that their models provided an accessible tool for the practitioner to assess average delay and average queue length for those configurations.

Simmonite and Chick (2005) conducted a similar study in the United Kingdom to evaluate a displaced right-turn intersection. They concluded that (1) intersection capacity can increase with a footprint similar to a large roundabout and only a small increase in costs, and (2) pedestrians and cyclists can easily be provided for without compromising the capacity using "Walk with Traffic" facilities. They suggested that the concept was an appropriate intersection type for use on the U.K. highway network, providing operational benefits where there were heavy right turns, full provision for non-motorized users, and an expected accident record unlikely to differ from other large signalized intersections.

Hughes et al. (2010) stated that removal of conflict between the left-turn movement and the oncoming traffic at the main intersection is the primary design element in a DLT intersection. The DLT vehicles typically cross the opposing through traffic approximately 300 to 400 ft upstream of the main intersection under the control of another traffic signal. Research referenced in the report indicated that the appropriate upstream distance is dependent on queuing from the main intersection and on costs involved in constructing a left-turn storage area for the crossed-over left-turn movement. Radii of the crossover movements can range from 150 to 200 ft, whereas the radius of the next left-turn movement at the main intersection is dependent on the turning movement of the design vehicle. Lane widths at the crossover reverse curve need to be wider than 12 ft to accommodate larger design vehicles. Consideration could also be given to having wider lane widths (e.g., up to 15 ft) for the receiving crossroad. The angle between the DLT intersection left-turn lanes and the main through lanes is referred to as the crossover angle and is influenced by the median width and the alignment of the mainline lanes; a recommended range of values for this angle is 10 to 15 degrees.

To minimize the footprint of the intersection, Hughes et al. (2010) stated that median widths can be reduced, but they still need to be adequate to accommodate signs. Designers are referred to the *Green Book* for minimum median widths, but caution is advised to also take into account the possibility of installing post-mounted signs in these medians for safe and effective channelization of traffic. Offsets for signs should be in accordance with the *MUTCD.*

Results from an analysis by El Asawey and Sayed (2007) indicated that the capacity of a XDL intersection was higher than that of a conventional intersection by about 90%, and it outperformed conventional and upstream signalized crossover intersections under all of their unbalanced-volume scenarios. They concluded that, for locations where right-of-way is not an issue, the XDL will be recommended for implementation because of its superior performance compared with the other two intersections.

Median U-Turn

The MUT has been used in Michigan and other states as a treatment to balance intersection congestion and safety problems (Hughes et al. 2010). The MUT intersection design involves the elimination of direct left turns from major and/or minor approaches (usually both). Drivers desiring to turn left from the major road onto an intersecting cross street must first travel through the at-grade main intersection and then execute a U-turn at the median opening downstream of the intersection. These drivers then turn right at the cross street. Drivers on the minor street desiring to turn left onto the major road must first turn right at the main intersection, execute a U-turn at the downstream median opening, and proceed back through the main intersection. Figure 13 shows the left-turn movements of a typical MUT geometric design. The optimum directional crossover spacing was recommended to be 660 ft (±100 ft) from the main intersection. Elimination of left-turning traffic from the main intersection simplifies the signal operations at the intersection, which accounts for most of the intended benefits. The MUT intersection is typically a corridor treatment applied at signalized intersections. How-

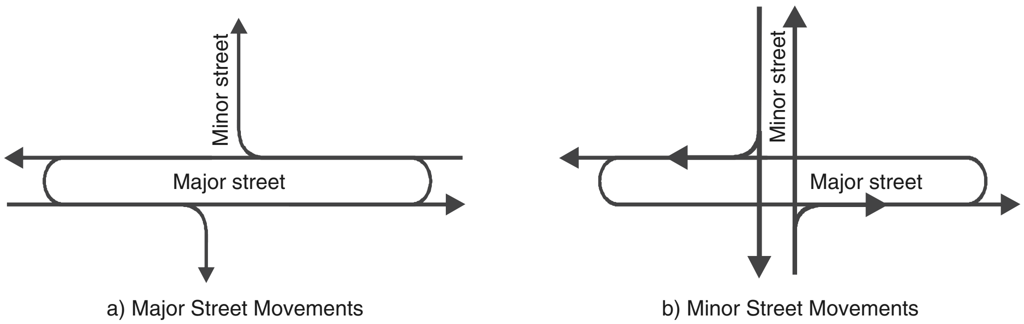


FIGURE 13 MUT left-turn movements (based on Hughes et al. 2010).

ever, the concept has also been used at isolated intersections to alleviate specific traffic operational and safety problems.

The FHWA report states that the MUT intersection performed well on arterials that have sufficient median width to accommodate the U-turn maneuver. Because of Michigan experience with these intersections, the report discussed typical design values from the Michigan DOT. In general, Michigan corridors with MUT intersections have median widths ranging from 60 to 100 ft. This design is used as a corridor treatment in Michigan, although it has also been used for isolated intersections.

At an MUT, the design of the main intersection is similar to the design of a conventional intersection, except that the main intersection is designed for larger volumes of right-turn movements than a conventional intersection serving the same total volumes because the left-turning vehicles become right-turning vehicles. With this in mind, the intersection must be designed with right-turn bays of sufficient width and length to accommodate the volume of turning vehicles. Depending on the right-turn volume, dual right-turn lanes or an exclusive right-turn lane and an adjacent shared-use through and right-turn lane may be needed. Channelized right turns at an MUT intersection are rarely used, because they may require even more right-of-way, present a multistage pedestrian crossing, and create a more difficult driving maneuver for a driver turning right from the minor street and weaving over to use the U-turn crossover. At some MUT intersections (e.g., at partial MUT intersections), left turns from the side road are allowed as well as left-turn bays provided on the minor road approaches. The MUT intersection has secondary intersections at each of the crossover locations. One-way crossovers with deceleration/storage lanes are highly recommended.

MDOT has developed design guidelines for directional median crossovers. In Michigan, the report states, it is customary for drivers of passenger vehicles to queue side-by-side in a 30-ft wide crossover and treat it as if it had two lanes. However, large trucks and other heavy vehicles typically use the entire width of the crossover. MDOT uses striped two-lane crossovers (with two lanes of storage leading up to the crossover) in some places. These crossovers are typically 36 ft wide. The FHWA report refers to the *Green Book* for minimum median widths, and it presents alternatives for locations with restricted right-of-way.

Restricted Crossing U-Turn

Hughes et al. (2010) refer to RCUT intersections as a promising solution for arterials with more dominant flows on the major road. Also referred to as superstreet intersections, they are described as having the potential to move more vehicles efficiently and safely than roadways with comparable traffic volumes that have conventional at-grade intersections with minimal disruptions to adjacent development. The RCUT intersection redirects left-turn and through movements from the side street approaches. Instead of allowing those movements to be made directly through the intersection, as in a conventional design, an RCUT intersection accommodates those movements by requiring drivers to turn right onto the main road and then make a U-turn maneuver at a one-way median opening 400 to 1,000 ft downstream. Figure 14 shows a conceptual diagram of an RCUT intersection. This configuration shown is generally intended for higher-volume major roads in suburban and rural areas, especially at intersections with relatively low through traffic volumes entering from the side road. For this type of intersection, left turns from the main road are similar to conventional intersections, made from left-turn lanes on the main road directly onto the side road. For this type of RCUT intersection design, pedestrians cross the main street in a diagonal fashion, going from one corner to the opposite corner. An RCUT design that does not permit direct left-turns is shown in Figure 15; this design channels all turning traffic to the crossovers on either side of the intersection.

The key difference between an MUT intersection and an RCUT intersection is that an MUT intersection allows through movements from the side street. An RCUT intersection has either no median openings at the intersection or has one-way directional median openings to accommodate traffic making left turns from the main street onto the side street. Similar to the MUT intersection, the median width is a crucial design element for an RCUT intersection. The report states that desirable right-of-way widths needed to accommodate large

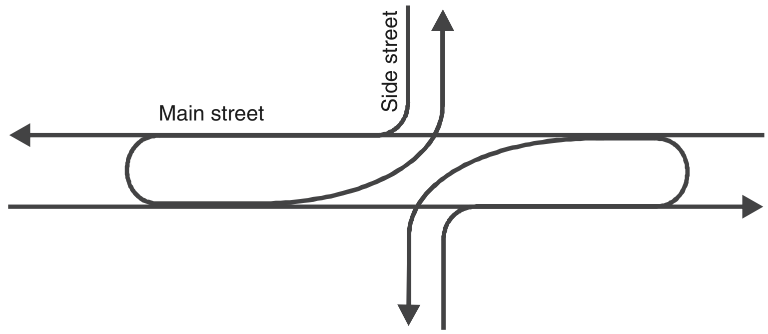


FIGURE 14 Conceptual RCUT configuration with direct left turns from the major road (based on Hughes et al. 2010).

trucks without allowing vehicles to encroach on curbs or shoulders, assuming 12-ft-wide lanes and 10 ft of shoulder, range from approximately 140 ft for four-lane arterials to approximately 165 ft for eight-lane arterials. For this same situation, desirable minimum median widths between 47 and 71 ft are typically needed. As with MUT intersections, designers are referred to the *Green Book* for specific design guidelines for minimum median widths, and much of the guidance in the FHWA report for crossover spacing for MUTs also is applied to RCUT intersections.

The report states that several factors should be considered when selecting the appropriate spacing from a main intersection to a U-turn crossover. Longer spacing between the main intersection and crossovers decreases spillback probabilities, providing more time and space for drivers to maneuver into the proper lane and read and respond to highway signs. Shorter spacing between the main intersection and crossovers translates into shorter driving distances and travel times. AASHTO recommends spacing from 400 to 600 ft for MUT designs based on signal timing, whereas MDOT established 660 ± 100 ft as the standard spacing (Hughes et al. 2010).

Quadrant Roadway

According to FHWA, the primary objective of a QR intersection is to reduce delay at a severely congested intersection of two busy suburban or urban roadways and to reduce overall travel time by removing left-turn movements (Hughes et al. 2010). A QR intersection can reportedly provide other benefits as well, such as improving pedestrian crossing time, and a QR intersection can be among the least costly of the four alternative intersections to construct and maintain.

At a QR intersection, all four left-turn movements at a conventional four-legged intersection are rerouted to use a connector roadway in one quadrant. Figure 16 shows the connector road and how all four of the left-turning movements are rerouted to use it. Left turns from all approaches are prohibited at the main intersection, which consequently allows a simple two-phase signal operation at the main intersection. Each terminus of the connector road is typically signalized. These two secondary signal-controlled intersections usually require three phases.

Key features in the geometric design of a QR intersection are choosing a quadrant in which to locate the connecting roadway; determining the number of connecting roadways; and designing the main intersection, the secondary intersections, and the horizontal alignment and cross section of the connecting road. In choosing a quadrant for the connecting roadway, common considerations are available right-of-way, construction cost, and effect on left-turn movements; for the latter, the quadrant is typically placed so that the movement with the highest volume is least affected by the new design of the intersection. That is, the left turn with the highest demand is the one that receives the most direct path. Discussion of

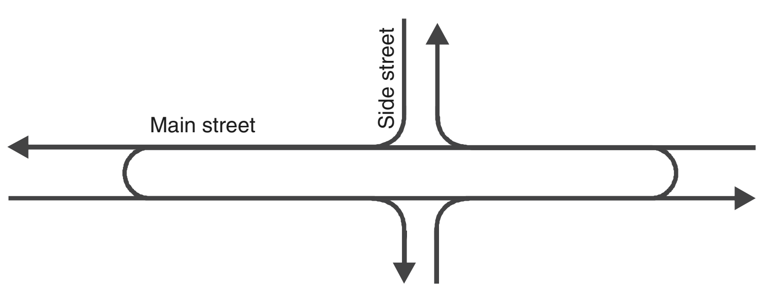
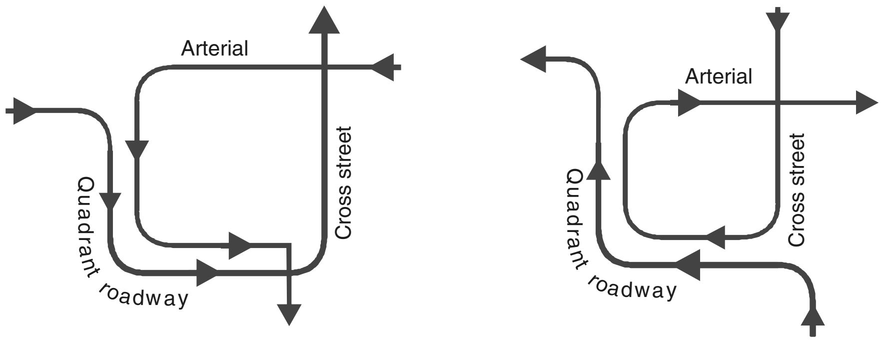


FIGURE 15 Basic RCUT intersection with no direct left turns (based on Hughes et al. 2010).



a) Left Turn Pattern From The Arterial b) Left Turn Pattern From A Cross Street

FIGURE 16 Left-turn movements at a QR intersection (based on Hughes et al. 2010).

QR designs with multiple quadrant connectors is also provided in the FHWA report.

For the main intersection, the design would be similar to that of a conventional intersection with turn prohibitions. Appropriate pavement markings or median designs are to be employed to convey the message to drivers that no left turns or U-turns are allowed. Right-turn lane criteria are the same for a QR intersection as a conventional intersection except for the right turns in the quadrant with the connecting roadway. Right-turn demands do not change at the main intersection in the other three quadrants. Through volumes at the main intersection are higher in all four directions than at a conventional intersection because of rerouted left-turning traffic. Pedestrian crosswalks would normally be provided across all four approaches at the main intersection.

Hughes et al. (2010) state that the distance from the main intersection to the secondary intersections is critical to the success of a QR intersection design. The considerations and trade-offs are similar to those between the main intersection and U-turn crossovers for an MUT or RCUT intersection. The distance needs to be sufficient to provide adequate vehicle storage and prevent spillback from one signal-controlled intersection to the next. It is also necessary to provide enough spacing for adequate signing and to ensure that each set of signal controls is visible. Longer distances lead to higher costs for right-of-way, construction, and maintenance of the connecting road. Longer distances may restrict progression from one signal to the next on the main streets and can translate into more vehicle-hours of travel. Considering all of those factors, a minimum spacing of 500 ft from the center of the main intersection to the center of the secondary intersections is presented as adequate for many situations.

The horizontal alignment of the connecting roadway is key to providing proper access to both roadways as well as any driveway connections. The authors recommend using the relevant geometric design data from the AASHTO *Green Book* for a design speed of 30 mph to determine the appropriate superelevation, radius, and runoff length.

Synchronized Split-Phasing/ Double Crossover Intersection

Bared et al. (2005) studied the operational characteristics of a synchronized split-phasing intersection, also called a double crossover intersection (DXI). An example of a DXI is shown in Figure 17. In this example, eastbound traffic crosses over to the left side at signalized Intersection A (small circle on the left of the figure), whereas the right-turners use the dedicated right lane before reaching A. The crossed traffic will cross over back to the right side at signalized Intersection C (small circle on the right). Westbound traffic also crosses over in a similar way. At Intersection B (large circle in the center of the figure), there is one through lane and one shared (through and left-turn) lane. No dedicated left-turn lanes are provided. Right-turn lanes are required for eastbound and westbound traffic. Merging lanes for the northbound and southbound right-turn movements are required. Radii of crossover movements can range from 150 to 200 ft, and the radius of the left-turn movement at B is 100 ft. Movements can be better understood by following the arrow markings in the figure. The northbound and the southbound traffic are similar to the corresponding movements at a conventional intersection, with one left-turn lane, one through lane, and one shared (through plus right-turn) lane. The length of left-turn lane is 450 ft.

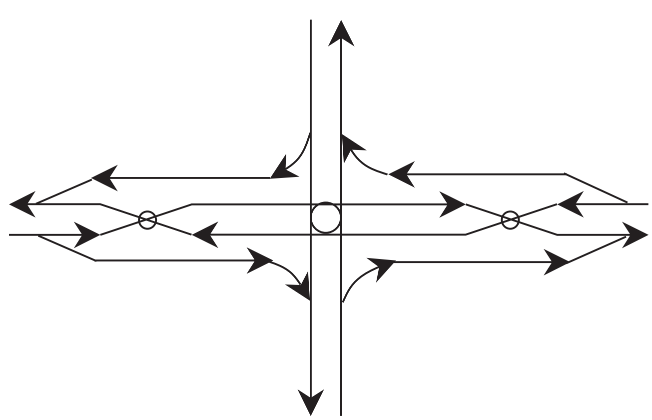


FIGURE 17 Double crossover intersection (Bared et al. 2005).

Results of traffic simulation were presented that showed input flow and throughput for DXI were similar, whereas the throughput was approximately 1,000 veh/hr lower than input flow for a conventional design. For peak volumes, the average delay per vehicle for conventional design was 219 s/veh, compared with 87 s/veh for the DXI. The authors also noted that the numbers of stops, average stop time per vehicle, average queue, and maximum queue length were lower for the DXI than the conventional design. Finally, they concluded that including a pedestrian phase for the DXI produced lower delay than the conventional intersection, and that the left-turn capacity in a DXI was more than twice that of a conventional design.

A variation on the DCI is the Upstream Signalized Crossover (USC) intersection. Sayed et al. (2006) investigated signal optimization strategies for USC intersections and identified selected operational issues. They used microsimulation to model and analyze a USC intersection and, for comparison, a conventional intersection. Their analysis revealed that, for relatively balanced volumes, a USC intersection could significantly reduce average vehicle delays, particularly when the volumes entering the intersection are relatively high. Additionally, the capacity of the USC intersection was found to be approximately 50% greater than that of a conventional intersection with similar geometry under balanced traffic volumes. For highly unbalanced volumes, particularly when the intersection volumes were relatively low, they found that a conventional intersection outperformed the USC intersection. Overall, they concluded, the USC intersection showed considerable potential for situations in which one or more of the following conditions existed: (1) intersection volumes were balanced and near or over the capacity of a conventional intersection; (2) traffic volumes were somewhat unbalanced, but the overall entering volumes were too high to be accommodated with a conventional intersection; or (3) the intersection had heavy left-turn volumes that caused excessive delays.

El Asawey and Sayed (2007) also concluded that the capacity of a simulated USC intersection was approximately 50% higher than that of a conventional intersection. They noticed that with an increase in left-turn percentage from 20% to 30%, there was a relatively constant increase in delay for the USC, between 1 and 4 seconds.

Arterial Interchange

Eyler (2005) discussed a family of interchange designs that were developed for arterial roadways: split-level single-point, left-hand windmill, hybrid (half single-point, half windmill), and partial cloverleaf. Each of the four designs had one consistent requirement, that each at-grade intersection in the interchange was the junction of only one turning movement and one through movement. There was never a location where traffic crossed both directions of an intersecting roadway. Eyler evaluated a selection of design variations using VISSIM modeling, and he determined that the overall capacity was near 75% of a four-lane freeway. He also conducted a generalized cost comparison and found that while the new designs were more expensive than traditional at-grade intersections, the annual travel time savings would offset the construction cost within three years. His conclusion was that these designs merited further consideration as alternatives to both conventional at-grade intersections and typical expansion of arterials to freeways.

Alternatives for Turning Movements at Rural Intersections

The purpose of NCHRP Project 15-30 (Maze et al. 2010) was to investigate alternative safety improvements at rural expressway intersections, to identify their relative effectiveness (if data were available), and to report any experiential information from those agencies that have tried the alternatives. After reviewing existing guidance and literature, the research team conducted case studies to investigate and document the effectiveness of ten treatments. Using that information, researchers recommended improvements to rural median intersection design guidance provided in the *Green Book* and the *MUTCD* for high-speed (50 mph or greater) expressways (divided highways with partial or no access control). The research team recommended that the next update to the *Green Book* include design guidance for rural expressway intersection designs that eliminate or reduce far-side conflict points (e.g., J-turn intersections and offset T-intersections) or those that address the issue of gap selection for minor road drivers (e.g., left-turn median acceleration lanes and offset right-turn lanes). Examples of those designs are shown in Figure 18. Studies of the J-turn and offset-T designs revealed reductions in crashes between 40% and 92%. Definitive results from the turning movement accommodations were unavailable because of a small set of crash data, but the authors were optimistic about the treatments' ability to reduce preventable crashes.

Two-Level Signalized Intersection

Shin et al. (2008) presented an unconventional intersection design (used in China) known as the Two-Level Signalized Intersection (TLSI) that completely separated east-west and north-south traffic. The TLSI, shown in Figure 19, also enabled the use of directional separation and leading, lagging, or overlapping lefts on both upper and lower levels. They described the TLSI as a design consisting of two independently operating intersections that is able to operate signals with flexibility according to changing traffic conditions.

The results from their simulation modeling indicated that, compared with other innovative intersection types, the TLSI had the shortest delay times in most evaluation scenarios as well as the least sensitivity to variations in traffic volume.

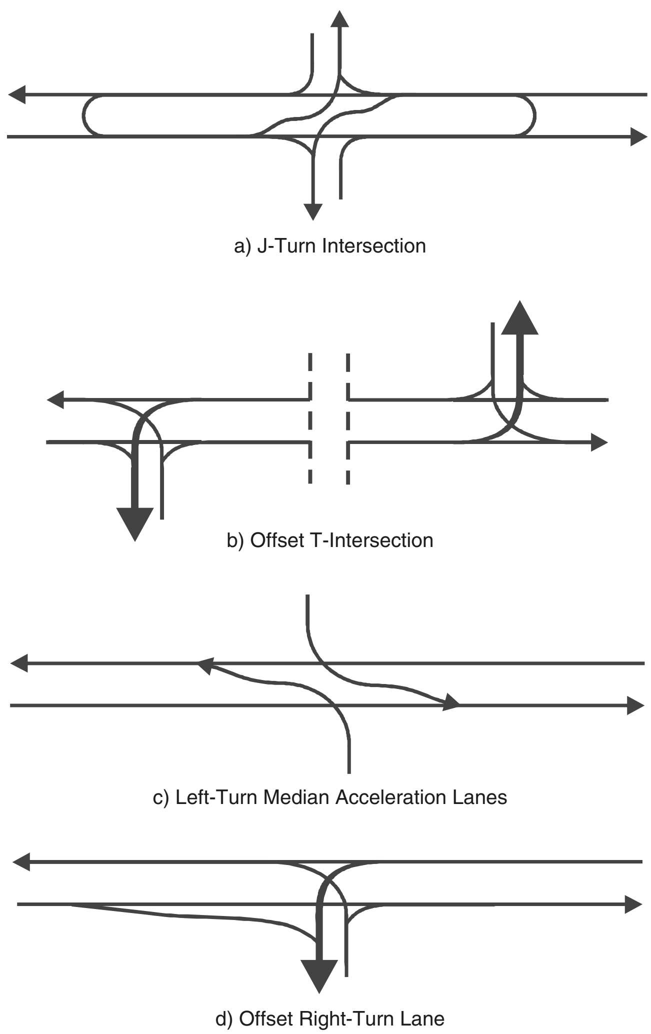


FIGURE 18 Diagrams of median intersection designs on rural expressways (based on Maze et al. 2010).

However, the TLSI showed significant delay when traffic volumes on the major and minor roads are significantly different, and it operated most efficiently when the two crossing roads had similar volumes of traffic.

PEDESTRIAN AND BICYCLE FACILITIES

According to *NCHRP Report 500,* Volume 18 (Raborn et al. 2008), there are several ways to modify the geometry of an intersection to improve bicycle safety, including:

- "Reducing the crossing distance for bicyclists.
- Realigning intersection approaches to reduce or eliminate intersection skew.
- Modifying the geometry to facilitate bicycle movement at interchange on-ramps and off-ramps.
- Providing refuge islands and raised medians."

At path/roadway intersections, an overpass or underpass allows for uninterrupted flow for bicyclists and completely eliminates exposure to vehicular traffic. These grade-separated crossings can improve safety and are desirable at some locations. However, because grade-separated crossings can be quite expensive, may be considered unattractive, can potentially become sites of crime or vandalism, and may even decrease safety if not appropriately located and designed, these types of facilities are primarily used as measures of last resort. The AASHTO *Bicycle Guide* (1999) provides guidance on the design of overpasses and underpasses. This strategy is related to Strategy 9.1 A5—Install Overpasses/Underpasses in *NCHRP Report 500, Volume 10: A Guide for Reducing Collisions Involving Pedestrians* (Zegeer et al. 2004).

FHWA's *Signalized Intersections: Informational Guide* (Rodegerdts et al. 2004) advises that "pedestrian facilities should be provided at all intersections in urban and suburban areas. In general," the authors say, "design of the pedestrian facilities of an intersection with the most challenged users in mind—pedestrians with mobility or visual impairments—should be done, and the resulting design will serve all pedestrians well." The *Guide* adds that the "ADA requires that new and altered facilities constructed by, on behalf of, or for the use of State and local government entities be designed and constructed to be readily accessible to and usable by individuals with disabilities." FHWA's guidelines are based on the premise that "pedestrians are faced with a number of disincentives to walking, including centers and services located far apart, physical barriers and interruptions along pedestrian routes, a perception that routes are unsafe owing to motor vehicle conflicts and crime, and routes that are [aesthetically] unpleasing." FHWA notes "key elements that affect a pedestrian facility that practitioners should incorporate into their design:

- Keep corners free of obstructions to provide enough room for pedestrians waiting to cross.
- Maintain adequate lines of sight between drivers and pedestrians on the intersection corner and in the crosswalk.
- Ensure curb ramps, transit stops (where applicable), push-buttons, etc., are easily accessible and meet ADAAG design standards.
- Clearly indicate the actions pedestrians are expected to take at crossing locations.
- Design corner radii to ensure vehicles do not drive over the pedestrian area yet are able to maintain appropriate turning speeds.
- Ensure crosswalks clearly indicate where crossings should occur and are in desirable locations.
- Provide appropriate intervals for crossings and minimize wait time.
- Limit exposure to conflicting traffic and provide refuges where necessary.
- Ensure the crosswalk is a direct continuation of the pedestrian's travel path.
- Ensure the crossing is free of barriers, obstacles, and hazards."

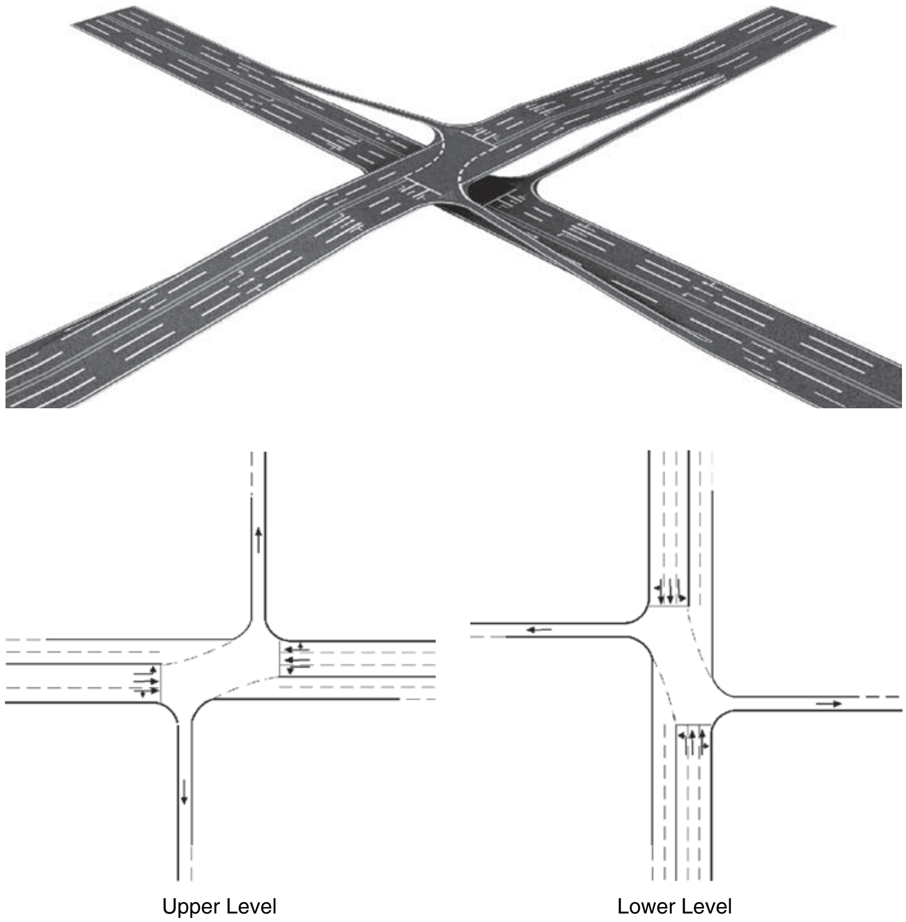


FIGURE 19 Concept design of two-level signalized intersection (Shin et al. 2008).

Where on-street bicycle lanes or off-street bicycle paths enter an intersection, FHWA (Rodegerdts et al. 2004) advises that intersection design should accommodate the needs of cyclists in safely navigating such a large and often complicated intersection. It is recommended that geometric features to be considered include:

- Bike lanes and bike lane transitions between through lanes and right-turn lanes.
- Left-turn bike lanes.
- Median refuges with a width to accommodate a bicycle: 6 ft = poor; 8 ft = satisfactory; 10 ft = good.
- Separate facilities if no safe routes can be provided through the intersection itself.

"Curb ramps provide access between the sidewalk and roadway for people using wheelchairs, strollers, walkers, crutches, handcarts, bicycles, and also for pedestrians with mobility impairments who have trouble stepping up and down high curbs. Curb ramps must be installed at all intersections and midblock locations where pedestrian crossings exist, as mandated by federal legislation," notably the 1990 Americans with Disabilities Act (ADA). According to the Proposed Accessibility Guidelines for Pedestrian Facilities in the Public Right of Way, curb ramps must have a slope of no more than 1:12 (must not exceed 1 in./ft), and a maximum slope on any side flares of 1:10 (United States Access Board 2011). Additional details of curb ramp design are also provided in the Proposed Guidelines.

Channelized turning lanes pose a potential risk to pedestrians, particularly those with disabilities. Researchers on NCHRP Project 3-78 (Schroeder et al. 2011) found anecdotal evidence that a crosswalk located in the middle of a turning lane is preferable to a crosswalk at the upstream or downstream portion of the turn lane. The middle crosswalk establishes a short crossing path roughly perpendicular to the trajectory of turning vehicles (useful for establishing pedestrian alignment), and it physically separates the conflict of turning drivers and pedestrians with the downstream merge point. Based on turning radii and associated design speeds, they posited that this was the likely location where speeds of right-turning vehicles would be lowest.

TRANSIT CONSIDERATIONS

TCRP sponsored a recent project to develop guidance for transit and highway agencies in the operations, planning, and functional designs of at-grade crossings of busways in physically separated rights-of-way by roadways, bike paths, or pedestrian facilities. *TCRP Report 117* (Eccles et al. 2007) documents the activities on that project, and the guidance contained in that report is intended to "provide information that can be applied to enhance safety at busway crossings while maintaining efficient transit and highway operations and minimizing pedestrian delay." General design principles and guidelines included:

- Provide simple intersection designs.
- Provide clear visual cues to make busway intersections conspicuous.
- Maximize driver and pedestrian expectancy.
- Separate conflicting movements.
- Minimize street crossings.
- Incorporate design features that improve safety for vulnerable users.
- Coordinate geometric design features and traffic control devices.

TCRP Report 117 discussed four types of busways found at intersections: median busways, side-aligned busways, separated right-of-way busways, and bus-only ramps. For each busway type, the report contains guidance on safety issues, basic geometry (including placement of bus stops), and traffic control, as well as an example of an intersection that uses each type of busway. Safety issues were generally related to the complexity and/or unfamiliarity of the arrangement of the intersection and the accommodation of pedestrians. Geometry guidelines pertained to channelization and control of turning movements to protect buses and passengers, provision of sufficient right-of-way to include the number of necessary travel lanes, and providing design consistency between busway lanes and the adjacent general-purpose lanes. The traffic control device most frequently recommended was traffic signals; suggestions on timing and phasing were provided to promote optimization of capacity and safety.

ACCESS MANAGEMENT

Recent research has established support for use of access management principles in improving intersection design and safety. The optimal situation is to avoid driveway conflicts before they develop (Neuman et al. 2003b). "This requires coordination with local land use planners and zoning boards in establishing safe development policies and procedures. Avoidance of high-volume driveways near congested or otherwise critical intersections is desirable. Driveway-permitting staff within highway agencies also needs to have an understanding of the safety consequences of driveway requests." Some recent research, findings, and discussion related to access management are contained in this section.

Zhou et al. (2002) studied the operational effectiveness of using right-turn-plus-U-turn (RTUT) as an alternative to DLTs from driveways where raised-curb medians were installed on six-lane highways at eight sites in Florida. After analyzing the data, the authors concluded that U-turns could have better operational performance than DLTs under certain traffic conditions, which they said implied that directional median opening designs would provide more efficient traffic flow than full median openings. They also stated that RTUT would provide better safety with regard to traffic conflicts and fewer effects on through-traffic operations of a major highway; they added that the majority of traffic on the major street in the study was in platoon flow because the signal spacing at study sites was less than 2 mi. Turns could not be made when the platoon was passing the driveway, and stragglers and left-turn-in movements from major roads between the platoons affected the ability to make turns.

Carter et al. (2005a) examined the operational and safety effects of U-turns at signalized intersections. The operational analysis involved measurements of vehicle headways in exclusive left-turn lanes at 14 signalized intersections. Regression analysis of saturation flow data showed a 1.8% saturation flow rate loss in the left-turn lane for every 10% increase in U-turn percentage and an additional 1.5% loss for every 10% U-turns if the U-turning movement was opposed by protected right-turn overlap from the cross street. The safety analysis involved a set of 78 intersections, 54 sites chosen randomly and 24 sites selected on the basis of their reputation as U-turn problem sites. Although the researchers used a group of study sites that was biased toward sites with high U-turn percentages, they found that 65 of the 78 sites did not have any collisions involving U-turns in the 3-year study period. U-turn collisions at the remaining 13 sites ranged from 0.33 to 3.0 collisions per year. Sites with double left-turn lanes, protected right-turn overlap, or high left-turn and conflicting right-turn traffic volumes were found to have a significantly greater number of U-turn collisions. Researchers concluded that, overall, U-turns do not have the large negative effect at signalized intersections that many have assumed, as safety and operational effects were minimal.

In NCHRP Project 17-21, researchers determined state and local agency design practices and policies related to unsignalized median openings for U-turns, such as those shown in Figure 20. After seven categories of midblock and intersection median designs were identified, the research team assessed the designs' effects on safety through field observation and crash data analysis for 115 unsignalized median opening sites with both crash and field data. This knowledge was transferred into design guidelines and a methodology for comparing the expected safety performance of different designs, to enable engineers in setting policy, establishing project-level design, and discussing the impacts of medians with business and property owners. As documented in *NCHRP Report 524* (Potts et al. 2004), researchers made the following conclusions:

- As medians are used more extensively on arterial highways, with direct left-turn access limited to selected

FIGURE 20 Median opening for left-turn lane on a four-lane divided suburban arterial at an unsignalized three-leg intersection (*Credit*: Dan Walker, Texas Transportation Institute).

locations, many arterial highways experience fewer midblock left-turn maneuvers and more U-turn maneuvers at unsignalized median openings.

- Field studies at various median openings in urban arterial corridors found estimated U-turn volumes of no more than 3.2% of the major-road traffic volumes at those locations. At rural median openings, U-turn volumes were found to represent at most 1.4% of the major-road traffic volumes at those locations.
- Accidents related to U-turn and left-turn maneuvers at unsignalized median openings occurred very infrequently. The 103 median opening study sites on urban arterial corridors experienced an annual U-turn plus left-turn crash average of 0.41. Twelve median openings on rural arterial corridors had an annual average crash total of 0.20. Overall, at these median openings, U-turns represented 58% of the median opening movements and left turns represented 42%. Based on these limited crash frequencies, researchers concluded that there was no indication that U-turns at unsignalized median openings constituted a major safety concern.
- For urban arterial corridors, median opening crash rates were substantially lower for midblock median openings than for median openings at three- and four-leg intersections. For example, the crash rate per million median opening movements (U-turn plus left-turn maneuvers) at a directional midblock median opening was typically only about 14% of the median opening crash rate for a directional median opening at a three-leg intersection.
- Crash rates at directional median openings on urban arterial corridors were lower than at traditional median openings, and conventional three-leg median openings had lower crash rates than corresponding four-leg openings.
- Where directional median openings were considered as alternatives to conventional median openings, two or more directional median openings were usually required to serve the same traffic movements as one conventional median opening. Therefore, researchers concluded that design decisions consider the relative safety and operational efficiency of all directional median openings in comparison with the single conventional median opening.
- Analysis of field data found that, for most types of median openings, most observed traffic conflicts involved major-road through vehicles having to brake for vehicles turning from the median opening onto the major road.
- At urban unsignalized intersections, the research found that installation of a left-turn lane on one approach would be expected to reduce accidents by 27% for four-leg intersections and by 33% for three-leg intersections.
- The minimum spacing between median openings then used by highway agencies ranged from 152 to 805 m (500 to 2,640 ft) in rural areas and 91 to 805 m (300 to 2,640 ft) in urban areas. In most cases, highway agencies used spacings between median openings in the upper end of these ranges, but there was no indication that safety problems resulted from occasional use of median opening spacings as short as 91 to 152 m (300 to 500 ft).

Based on these and other findings, supplemented in part by conclusions in *NCHRP Report 348* (Koepke and Levinson 1992), *NCHRP Report 375* (Harwood et al. 1995), and *NCHRP Report 420* (Gluck et al. 1999), the NCHRP 17-21 research team developed and presented a five-step methodology for comparing the expected safety performance of median opening design alternatives to assist in the selection of median opening types and the comparison of alternative median opening arrangements. As part of their conclusions, the research team recommended the following:

- Unsignalized median openings may be used for a broad range of major- and minor-road traffic volumes. However, if the major- and minor-road volumes exceed the traffic volumes given in the *MUTCD* signalization warrants, signalization of the median opening needs to be considered.
- The effects of U-turn and left-turn volumes on median opening crash frequency cannot be separated, because a review of crash data for median openings found that crash report data do not distinguish clearly between crashes involving U-turn maneuvers and those involving left-turn maneuvers.
- For rural unsignalized intersections:
  - They should have medians that are as wide as practical, as long as the median is not so wide that approaching vehicles on the crossroad cannot see both roadways of the divided highway.
  - Where the AASHTO passenger car is used as the design vehicle, a minimum median width of 8 m (25 ft) is recommended.
  - Where a large truck is used as the design vehicle, a median width of 21 to 31 m (70 to 100 ft) generally would be selected. If such a median width cannot be

provided, consideration should be given to providing a loon.

- For suburban unsignalized intersections:
  - Median widths at suburban unsignalized intersections generally should be as narrow as possible while providing sufficient space in the median for the appropriate left-turn treatment.
  - Median widths between 4.2 and 7.2 m (14 and 24 ft) will accommodate left-turn lanes, but are not wide enough to store a crossing or turning vehicle in the median.
  - Medians wider than 7.6 m (25 ft) may be used, but cross-road vehicles making turning and crossing maneuvers may stop on the median roadway.
  - Median widths of more than 15 m (50 ft) generally should be avoided at suburban, unsignalized intersections.
- Median opening lengths at rural divided highway intersections generally should be kept to the minimum possible. Increases in median opening length were found to be correlated with higher rates of undesirable driving behavior. In contrast, researchers found no reason that the median opening in urban and suburban areas should not be as long as necessary.
- Median opening spacing for rural areas typically ranged from 150 to 805 m (500 to 2,640 ft); a minimum median opening spacing of 150 m (500 ft) was recommended in rural areas. Typically, median opening spacing substantially longer than 150 m (500 ft) was considered to be appropriate, unless two public road intersections or major driveways are located relatively close together.
- Median opening spacing for urban areas typically ranged from 90 to 805 m (300 to 2,640 ft); a minimum median opening spacing of 90 m (300 ft) was recommended in urban areas. Researchers stated that, whenever practical, median opening spacing greater than 90 m (300 ft) should be used in urban areas.
- U-turn maneuvers should not be encouraged at locations with limited sight distance. Furthermore, sight distance is an important issue in determining locations where U-turns by larger vehicles should be permitted or encouraged. ISD based on the criteria in the AASHTO *Green Book* for Cases B1, B2, and F should be available to accommodate U-turns and left turns at unsignalized median openings.

Gattis et al. (2010) presented guidelines for driveway spacing near intersections, both signalized and unsignalized. For unsignalized intersections, they stated that spacing should not interfere with safe and relatively unimpeded movement on the through roadway, and driveway spacing practices should provide reasonable access to abutting private property. Other general guidelines included:

- The needed distance between successive connections (both driveways and side streets) increases with higher operating speeds, higher access classifications for the public roadway, and higher driveway volumes.
- A driveway should not be located within the functional area of an intersection or in the influence area of the upstream and downstream driveways.
- Left-turn lane storage requirements should be considered when determining the driveway influence area and can limit how closely driveways can be spaced.
- On roadways that are undivided or have TWLTLs, the alignment of driveways on opposite sides of the road needs to be considered. Driveways on opposite sides of a lower-volume roadway may be aligned across from each other. Alternatively, they should be spaced so that those drivers desiring to travel between the driveways on opposing sides of the roadway need to make a distinct right turn followed by a left turn (or a left followed by a right). A much longer separation is needed on a higher-speed, higher-volume roadway.
- On roadways with restrictive medians, the spacing between right-turn access points on opposite sides of the road can be treated separately.
- Ideally, driveway access for a major development involving left-turn egress movements should be located where effective coordination of traffic signals would be achievable if there is a need to signalize the driveway.
- Driveway connections to public roadways are subject to the same intersection control device analyses as are street intersections. If existing or future volumes warrant installing a traffic signal, and signalized spacing requirements cannot be met, left-turn access should be subject to closure in one or both directions.

For driveways near signalized intersections, Gattis stated that the needed minimum separation distance (i.e., corner clearance) will depend on the function, operation, and design features of the roadway and the characteristics of the access connection, considering the basic principle of locating one connection outside of the functional area of another connection. For a driveway upstream of or approaching a signalized location on a major road, the functional area was defined to include the PRT, maneuver distance, and storage length of the traffic on that approach. The recommended spacing would provide separation between the conflicting movements occurring at the signal and the conflicting movements occurring at the driveway. In addition, this spacing would enable the driveway to operate without being obstructed by the traffic backing up from the signal.

SUMMARY OF KEY FINDINGS

This section summarizes key findings from the research noted in this chapter. This is an annotated summary; conclusions and recommendations are those of the authors of the references cited.

Intersection Alignment

- Avoid approach grades to an intersection of greater than 6%. On higher design speed facilities (50 mph and greater) a maximum grade of 3% should be considered (Rodegerdts et al. 2004).
- Avoid locating intersections along a horizontal curve of the intersecting road (Rodegerdts et al. 2004).
- Strive for an intersection platform (including sidewalks) with a cross slope not exceeding 2%, as needed for accessibility (Rodegerdts et al. 2004).
- Approach curvature can be used as a treatment to force a reduction in vehicle speed through the introduction of horizontal deflection at high-speed intersection approaches, but it is discouraged at downhill approaches (Ray et al. 2008).
- A skew angle greater than 20 degrees should not be used in design when the design vehicle is a large vehicle or semitrailer (Son et al. 2002).
- A minimum skew angle of 15 degrees should be used to accommodate age-related performance deficits at intersections where right-of-way is restricted (Staplin et al. 2002).

Auxiliary Lanes

- Adding left-turn lanes is effective in improving safety at signalized and unsignalized intersections, reducing crashes between 10% and 44% (Harwood et al. 2002).
- Positive results can also be expected for right-turn lanes, with reductions in total intersection crashes between 4% and 14% (Harwood et al. 2002).
- A method was developed to identify where installation of right-turn lanes at unsignalized intersections and major driveways would be cost-effective, indicating combinations of through-traffic volumes and right-turn volumes for which provision of a right-turn lane would be recommended. The economic analysis procedure can be applied by highway agencies using site-specific values for ADTs, turning volumes, crash frequency, and construction cost for any specific location (or group of similar locations) of interest (Potts et al. 2007a).

Modern Roundabouts

- A series of projects during the decade (2000–2010) led to the publication of two FHWA *Informational Guides* containing recommendations and guidelines for all aspects of roundabout design.
- General overarching principles of geometric design of roundabouts (Rodegerdts et al. 2010) included:
  - “Provide slow entry speeds and consistent speeds through the roundabout by using deflection.
  - Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume balance, and lane continuity.
  - Provide smooth channelization that is intuitive to drivers and results in vehicles naturally using the intended lanes.
  - Provide adequate accommodation for the design vehicles.
  - Design to meet the needs of pedestrians and cyclists.
  - Provide appropriate sight distance and visibility for driver recognition of the intersection and conflicting users.”
- Maximum entering design speeds are based on a theoretical fastest path of 20 to 25 mph for single-lane roundabouts and 25 to 30 mph for multilane roundabouts (Rodegerdts et al. 2010).
- Roundabout alignment is described as “optimally located when the centerlines of all approach legs pass through the center of the inscribed circle” (Robinson et al. 2000).
- Common inscribed circle diameters for single-lane roundabouts vary from 90 to 180 ft, depending on design vehicle (Rodegerdts et al. 2010).
- Designers should provide no more than the minimum required ISD on each approach, [because] excessive ISD can lead to higher vehicle speeds that reduce the safety of the intersection for all road users (Robinson et al. 2000).
- Crash experience at selected intersections in the United States indicates an overall reduction in crash frequency at intersections converted to roundabouts (Rodegerdts et al. 2007).
- Pedestrian refuge should be a minimum width of 6 ft to adequately provide shelter for persons pushing a stroller or walking a bicycle (Robinson et al. 2000).
- “At single-lane roundabouts, the pedestrian crossing should be located one vehicle-length (25 ft) away from the yield line. At double-lane roundabouts, the pedestrian crossing should be located one, two, or three car lengths (approximately 25 ft, 50 ft, or 75 ft) away from the yield line” (Robinson et al. 2000).
- “The pedestrian refuge should be designed at street level, rather than elevated to the height of the splitter island. This eliminates the need for ramps within the refuge area, which can be cumbersome for wheelchairs” (Robinson et al. 2000).
- Ramps should be provided on each end of crosswalks to connect the crosswalk to other crosswalks around the roundabout and to the sidewalk network (Robinson et al. 2000).
- A detectable warning surface, as recommended in the ADAAG, should be applied to the surface of the refuge within the splitter island (Robinson et al. 2000).
- Use of standard AASHTO island design for key dimensions, such as offset and nose radii, is encouraged. For sidewalks, a setback distance of 5 ft, with a minimum of 2 ft is advised (Robinson et al. 2000).
- For nonmotorized users such as bicyclists, one important consideration during the initial design stage is to maintain or obtain adequate right-of-way outside the circulatory roadway for the sidewalks. All nonmotorized users who are likely to use the sidewalk regularly, including

bicyclists in situations where roundabouts are designed to provide bicycle access to sidewalks, should be considered in the design of the sidewalk width. Recommended designs for single-lane roundabouts encourage bicycle users to merge into the general travel lanes and navigate the roundabout as a vehicle, explaining that the typical vehicle operating speed within the circulatory roadway is in the range of 15 to 25 mph, which is similar to that of a bicycle (Rodegerdts et al. 2010).

Innovative Intersection Designs

A number of new or innovative intersection designs were considered during the decade; each of the following was described in one or more studies.

- Displaced Left Turns showed considerable savings in average control delay and average queue length, as well as an increase in intersection capacity, in one series of microsimulation analyses (Hughes et al. 2010).
- Median U-turns are typically a corridor treatment applied at signalized intersections, but are also used at isolated intersections to alleviate specific traffic operational and safety problems (Hughes et al. 2010).
- Median width of Restricted Crossing U-Turns is a crucial design element to accommodate large trucks without allowing vehicles to encroach on curbs or shoulders (Hughes et al. 2010).
- Quadrant Roadways should be designed so that the left turn with the highest demand is the one that receives the most direct path (Hughes et al. 2010).
- Double Crossover Intersections are found to have greater throughput than a conventional intersection, along with lower values for number of stops, average stop time per vehicle, average queue, and maximum queue length (Bared et al. 2005).
- Arterial Interchanges have an overall capacity near 75% of a four-lane freeway (Eyler 2005).
- J-Turn and Offset-T designs had reductions in crashes between 40% and 92% (Maze et al. 2010).
- Two-Level Signalized Intersections produced modeled results with the shortest delay times in most evaluation scenarios as well as the least sensitivity to variations in traffic volume compared with other innovative intersection types; however, delay increased when flow was unbalanced between the two crossing roads (Shin et al. 2008).
- The additional right-of-way needed to construct each of these innovative designs was mentioned as a potential drawback by every report and author that addressed the issue of the intersection's footprint.

Pedestrian and Bicycle Facilities at Intersections

- Suggested strategies (Raborn et al. 2008) for modifying intersections to accommodate bicycles and pedestrians included:
  - Reducing the crossing distance for bicyclists,
  - Realigning intersection approaches to reduce or eliminate intersection skew,
  - Modifying the geometry to facilitate bicycle movement at interchange on-ramps and off-ramps,
  - Providing refuge islands and raised medians, and
  - Grade-separated crossings.
- "Pedestrian facilities should be provided at all intersections in urban and suburban areas. In general, design of pedestrian facilities with the most challenged users in mind—pedestrians with mobility or visual impairments—should be done, and the resulting design will serve all pedestrians well. ADA requires that new and altered facilities constructed by, on behalf of, or for the use of State and local government entities be designed and constructed to be readily accessible to and usable by individuals with disabilities" (Rodegerdts et al. 2004).
- Practitioners should incorporate key elements that affect a pedestrian facility into their design (Rodegerdts et al. 2004):
  - "Keep corners free of obstructions to provide enough room for pedestrians waiting to cross.
  - Maintain adequate lines of sight between drivers and pedestrians on the intersection corner and in the crosswalk.
  - Ensure curb ramps, transit stops (where applicable), pushbuttons, etc., are easily accessible and meet ADAAG design standards.
  - Clearly indicate the actions pedestrians are expected to take at crossing locations.
  - Design corner radii to ensure vehicles do not drive over the pedestrian area yet are able to maintain appropriate turning speeds.
  - Ensure crosswalks clearly indicate where crossings should occur and are in desirable locations.
  - Provide appropriate intervals for crossings and minimize wait time.
  - Limit exposure to conflicting traffic, and provide refuges where necessary.
  - Ensure the crosswalk is a direct continuation of the pedestrian's travel path.
  - Ensure the crossing is free of barriers, obstacles, and hazards."

Transit Considerations

- General intersection design principles and guidelines for transit issues (Eccles et al. 2007) include:
  - "Provide simple intersection designs.
  - Provide clear visual cues to make busway intersections conspicuous.
  - Maximize driver and pedestrian expectancy.
  - Separate conflicting movements.
  - Minimize street crossings.
  - Incorporate design features that improve safety for vulnerable users.

  - Coordinate geometric design features and traffic control devices."
- There are four types of busways found at intersections: median busways, side-aligned busways, separated right-of-way busways, and bus-only ramps. Each busway type has unique characteristics that are considerations for guidance on safety issues, basic geometry (including placement of bus stops), and traffic control, along with examples of appropriate intersections for each type of busway (Eccles et al. 2007).

Access Management at Intersections

- Right-turn-plus-U-turn could have better operational performance than direct left turns under certain traffic conditions, implying that directional median opening designs could provide more efficient traffic flow than full median openings (Zhou et al. 2002).
- U-turns at signalized intersections resulted in a 1.8% saturation flow rate loss in the left-turn lane for every 10% increase in U-turn percentage and an additional 1.5% loss for every 10% U-turns if the U-turning movement was opposed by protected right-turn overlap from the cross street (Carter et al. 2005a).
- Recommended practices (Potts et al. 2004) for rural unsignalized intersections include:
  - They should have medians that are as wide as practical, as long as the median is not so wide that approaching vehicles on the crossroad cannot see both roadways of the divided highway.
  - Where the AASHTO passenger car is used as the design vehicle, a minimum median width of 25 ft is recommended.
  - Where a large truck is used as the design vehicle, a median width of 70 to 100 ft generally should be selected. If such a median width cannot be provided, consideration should be given to providing a loon.
- Recommended practices (Potts et al. 2004) for suburban unsignalized intersections include:
  - Median widths at suburban unsignalized intersections generally should be as narrow as possible while providing sufficient space in the median for the appropriate left-turn treatment.
  - Median widths between 14 and 24 ft will accommodate left-turn lanes, but are not wide enough to store a crossing or turning vehicle in the median.
  - Medians wider than 25 ft may be used, but crossroad vehicles making turning and crossing maneuvers may stop on the median roadway.
  - Median widths of more than 50 ft generally should be avoided at suburban, unsignalized intersections.
- Median opening lengths at rural divided highway intersections generally should be kept to the minimum possible. Increases in median opening length are correlated with higher rates of undesirable driving behavior. In contrast, the median opening in urban and suburban areas can be as long as necessary (Potts et al. 2004).

CHAPTER SIX

INTERCHANGES

OVERVIEW

Similar to intersections, new ways to design interchanges received attention during the last 10 years in an attempt to improve capacity while minimizing the cost of constructing or expanding the interchange. Researchers also revisited characteristics of ramp design and ramp terminal design, and they considered the effects of work zones near interchanges.

DESIGN OF RAMPS AND RAMP TERMINALS

Chaudhary and Messer (2002) developed guidelines for designing freeway on-ramps in which ramp metering is envisioned. Specifically, they looked for design issues in which ramp meters use a queue detector to identify and prevent a queue of vehicles from blocking the upstream intersection. They focused on three design elements: safe stopping distance, storage distance, and acceleration distance from meter to merge point. Combining these three elements, they stated that the desired distance between the cross street and freeway merge point be at least 400 m (1,312 ft) for ramps at which metering is envisioned.

Fitzpatrick and Zimmerman (2007) reviewed the *Green Book*'s process for adjusting acceleration and deceleration lengths on graded ramps. They found that the source of the adjustment factors in the 2004 *Green Book* was provided in the 1954 *Policies on Geometric Highway Design* (i.e., the *Blue Book,* AASHO 1954), in which they first appeared as being based on applying “principles of mechanics to rates of speed change for level grades.” Their reviews of that document and others did not reveal a procedure for determining adjustment factors. They posited that a potential source for an adjustment factor for entrance ramps is the calculation of the distance needed to accelerate from one speed to another on different grades by means of vehicle performance equations available in the literature, and thus they reviewed the literature to develop potential acceleration length adjustment factors. They subsequently applied the *Green Book* methodology for calculating SSD on different grades to the equations used to calculate deceleration lengths so as to determine deceleration lengths for different grades. The ratio of the deceleration length on a grade to the deceleration length on a level surface formed the basis for their adjustment factors for deceleration. They recommended that actual performance of vehicles on grades and on a level surface should be measured and compared with the suggested adjustment factors to determine the accuracy of those factors.

Ramp and Interchange Spacing

Under NCHRP Project 03-88, researchers evaluated and summarized design, operations, safety, and signing considerations that influence ramp and interchange spacing decisions (Ray et al. 2011). The *Green Book* contains guidelines on the distance between successive ramp terminals, but they “are acknowledged to be based on operational experience and recommend basing actual spacing on operations and safety procedures derived from applied research.” To provide a better understanding of the impacts of ramp and interchange spacing on safety and operations, researchers collected and analyzed data from a variety of existing freeway ramps and interchanges, focused on relatively simple, single lane, service ramps and interchanges. The team conducted operational and safety assessments of two types of ramp pairs—an entry ramp followed by an exit ramp (EN-EX) and an entry ramp followed by another entry ramp (EN-EN). They then performed simulation modeling, calibrated with field data, of closely spaced pairs of ramps and developed safety performance models.

Based on their findings, the team then developed guidelines to assist practitioners in selecting ramp and interchange spacing values for their particular design context. These guidelines presented substantial discussions on geometric design, traffic operations, safety, and signing, and the role each of these play in determining ramp and interchange spacing needs. The guidelines made a distinction to separately define “ramp spacing” and “interchange spacing” and recommended ramp spacing values be the primary consideration in freeway and interchange planning and design. Guidelines were presented based on four areas of emphasis: geometric design, traffic operations, signing, and safety. Geometric design principles, as well as site-specific features, dictate minimum lengths needed for ramps and other interchange components. Traffic volumes can necessitate increased spacing beyond the dimensions needed purely for geometrics. Safety tradeoffs, which have rarely been quantified until recently, can now be considered in project decision making. Finally, signing and other human factors issues should be taken into account at the earliest in the evaluation process when

TABLE 20
POTENTIAL FEASIBILITY OF SPACING FOR VARIOUS FREEWAY RAMP COMBINATIONS

Combination	Ramp Spacing Dimension (ft)	Feasibility
Diamond Interchange Entrance–Exit	Less than 1,600	Likely not geometrically feasible
	1,600 to 2,600	Potentially geometrically feasible
	Greater than 2,600	Likely geometrically feasible
Partial Cloverleaf Entrance–Exit	Less than 1,600	Likely not geometrically feasible
	1,600 to 1,800	Potentially geometrically feasible
	Greater than 1,800	Likely geometrically feasible
Entrance–Entrance	Less than 1,400	Likely not geometrically feasible
	1,400 to 1,800	Potentially geometrically feasible
	Greater than 1,800	Likely geometrically feasible
Exit–Exit	Less than 900	Likely not geometrically feasible
	900 to 1,100	Potentially geometrically feasible
	Greater than 1,100	Likely geometrically feasible
Exit–Entrance (Braided)	Less than 1,700	Likely not geometrically feasible
	1,700 to 2,300	Potentially geometrically feasible
	Greater than 2,300	Likely geometrically feasible

Source: Ray et al. (2011).

making choices about ramp and interchange spacing. The guidelines were presented as information that can also be incorporated in future editions or updates of relevant manuals and other guidance documents. Among the geometric design guidelines are spacing assessments, as shown in Table 20.

Access Management

The adequate spacing and design of access to crossroads in the vicinity of freeway ramps are critical to the safety and traffic operations of both the freeway and the crossroad. Rakha et al (2008) conducted research for the Virginia DOT to develop a methodology to evaluate the safety impact of different access road spacing standards. The models they developed were used to compute the crash rate associated with alternative section spacing, and the authors concluded that the models satisfied statistical requirements and provided reasonable crash estimates. Their results indicated that the crash rate decreased by 88% when access road spacing increased from 0 to 300 m. An increase in the minimum spacing from 90 m (300 ft) to 180 m (600 ft) resulted in a 50% reduction in the crash rate. The models were used to develop lookup tables that quantified the impact of access road spacing on the expected number of crashes per unit distance. Those tables revealed a decrease in the crash rate as the access road spacing increases. The researchers also attempted to quantify the safety cost of alternative access road spacing using a weighted average crash cost. The weighted average crash cost was computed based on the observed distribution of crashes in Virginia that were fatal, injury, and property damage crashes. Costs of crashes in each severity category were provided by the Virginia DOT, which the researchers used to compute an average weighted crash cost. This average cost was multiplied by the number of crashes per mile to compute the cost associated with different access spacing scenarios. The researchers developed tables containing the cost data that planners, designers, and policymakers could use in determining their choice of intersection and access spacing for specific freeway ramp locations.

Managed Lanes

Fitzpatrick et al. (2003b) conducted an evaluation of managed lane ramp design issues in Texas, with a comparison to then-current practices in national and other states' guidelines. The 2001 *Green Book* (AASHTO 2001) specified a 2,000-ft weaving section for a system-to-service interchange. For a direct-connection ramp between a traffic generator and the managed lane, AASHTO recommended a minimum design speed for direct connection ramps of 40 mph, whereas California's guidelines (Caltrans 2001) called for a minimum of 50 mph. Each state's guidelines that contained specific discussions on the spacing between successive ramps used approximately 900 to 1,000 ft spacing.

They also used computer simulation "to obtain an appreciation of the effects on corridor operations when several pairs of ramps are considered. Speed was the primary measure of effectiveness used to evaluate the effects of different ramp spacings, volume levels, and weaving percentages. The research found that a direct connect ramp between a generator and the managed lane facility should be considered when 400 veh/hr is anticipated to access the managed lanes. If a more conservative approach to preserving freeway performance is desired, then a direct connect ramp should be considered at 275 veh/hr (which reflected the value when the lowest speeds on the simulated corridor for the scenarios examined were at 45 mph or less)." This finding builds on the recommendations made by Venglar et al. (2002) on weaving distances for managed lane cross-freeway maneuvers, shown in Table 21.

TABLE 21
WEAVING DISTANCES FOR MANAGED LANE CROSS-FREEWAY MANEUVERS

Design Year Volume Level	Allow up to 10 mph Mainline Speed Reduction for Managed Lane Weaving	Intermediate Ramp (between freeway entrance/exit and managed lanes entrance/exit)?	Recommended Minimum Weaving Distance Per Lane (ft)
Medium (LOS C or D)	Yes	No	500
		Yes	600
	No	No	700
		Yes	750
High (LOS E or F)	Yes	No	600
		Yes	650
	No	No	900
		Yes	950

Source: Venglar et al. (2002).
Note: The provided weaving distances are appropriate for freeway vehicle mixes with up to 10% heavy vehicles; higher percentages of heavy vehicles will require increasing the per lane weaving distance. The value used should be based on engineering judgment, although a maximum of an additional 250 ft per lane is suggested.

Fitzpatrick et al. (2007) developed guidance materials on intermediate at-grade access to a buffer-separated managed lane. They determined that compliance with access points was better for those with greater lengths (e.g., 1,500 ft), but that over 7% of observed access maneuvers involved vehicles using the managed lane to pass slower-moving vehicles. They also found "that when presented with the opportunity to enter a managed lane that is located very close to an entrance ramp, drivers will attempt to cross multiple lanes to do so." Providing sufficient weaving distance for cross-freeway maneuvers was therefore important to facilitate access to the managed lane. For the design of the at-grade access opening, they recommended the configuration shown in Figure 21.

Toll Facilities

A particular type of managed facility is a tolled facility. Some tolled facilities are separate roadways on unique alignments, whereas others are selected lanes on a concurrent alignment with a general-purpose facility. Each has particular characteristics to consider when designing access points, whether they are at-grade openings or full-fledged interchanges. In ITE's *Freeway and Interchange Geometric Design Handbook* (Leisch et al. 2005), McDonald describes details of geometric design elements for toll plazas. Although a number of the practices listed are influenced by traditional manned toll plazas where tickets and cash are exchanged, there are a variety of examples and principles that are also valid for unmanned electronic toll collection.

Toll plazas typically have more lanes than adjacent sections of a freeway and require sufficient merge and diverge tapers to accommodate the added lanes. Similarly, a toll plaza or toll island on a ramp requires enough lanes to serve the anticipated demand, necessitating the addition and/or reduction of lanes on the ramp proper. Table 22 reproduces the information from the Florida Turnpike cited by McDonald for taper rates at toll plazas with traditional payment collection.

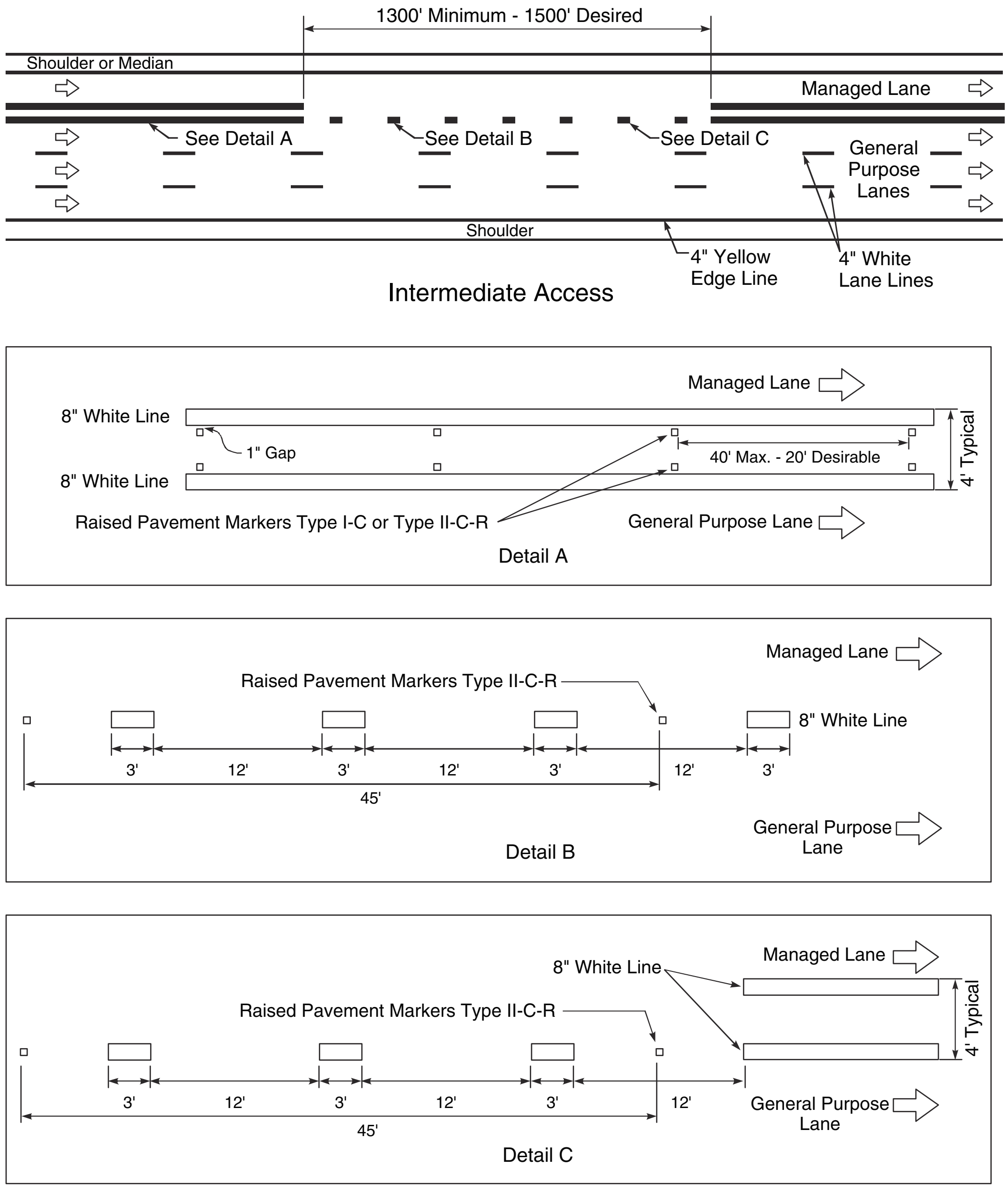
Electronic toll collection methods have improved capacity at toll plazas, but there is still a need to accommodate the anticipated volume of vehicles using the facility. McDonald provides a detailed procedure for estimating the appropriate number of queue lanes and queue length, depending on toll collection method, but he also provides a general rule of thumb from Caltrans to provide 3.5 to 4 toll lanes per approaching freeway lane and a minimum queue storage length between 200 and 250 ft (Leisch et al. 2005).

ALTERNATIVE INTERCHANGE DESIGNS

An FHWA study (Hughes et al. 2010) examined two alternative interchange designs, reviewing characteristics related to geometric design, access management, traffic control devices, and other features. The two designs included Double Crossover Diamond (DCD) and DLT interchanges. Additional studies have also evaluated these interchange designs and others. Findings from those studies related to geometric design are summarized in this section.

Double Crossover Diamond/Diverging Diamond

The DCD interchange, also called a Diverging Diamond interchange (DDI), is a recent interchange design that is being considered as a viable interchange form to improve traffic flow and reduce congestion. Similar to the design of a conventional diamond interchange, the DCD interchange differs in the way that the left and through movements navigate between the ramp terminals. The purpose of this interchange design is to accommodate left-turning movements onto arterials and limited-access highways while eliminating the need for a left-turn bay and signal phase at the signalized ramp terminals. Figure 22 shows the typical movements that are accommodated in a DCD interchange. The highway is connected to the arterial cross street by two on-ramps and two off-ramps in a manner similar to a conventional diamond interchange. However, on the cross street, the traffic moves to the left side of the roadway between the ramp terminals. This allows the vehicles on



Notes:

All pavement marking materials shall meet the required Departmental Material Specifications as specified in plans.

⇨ = Direction of travel.

FIGURE 21 Design of intermediate at-grade access opening for buffer-separated freeway managed lane (Fitzpatrick et al. 2007).

TABLE 22
DESIRABLE TAPER RATES AT TOLL PLAZAS

Plaza Type	Number of Traditional Payment Lanes at Plaza	Desirable Taper Rate
Mainline Plaza	Up to 8 lanes	25:1
	10 to 14 lanes	20:1
	16 or more lanes	15:1
Ramp Toll Plaza	All	20:1

Source: Leisch et al. (2005).

the cross street that need to turn left onto the ramps to continue to the on-ramps without conflicting with the opposing through traffic (Hughes et al. 2010).

The primary design element of a DCD interchange is the relocation of the left-turn and through movements to the opposite side of the road within the bridge structure. The turning radii used at the crossover junction to displace these movements at an existing installation in Springfield, Missouri,

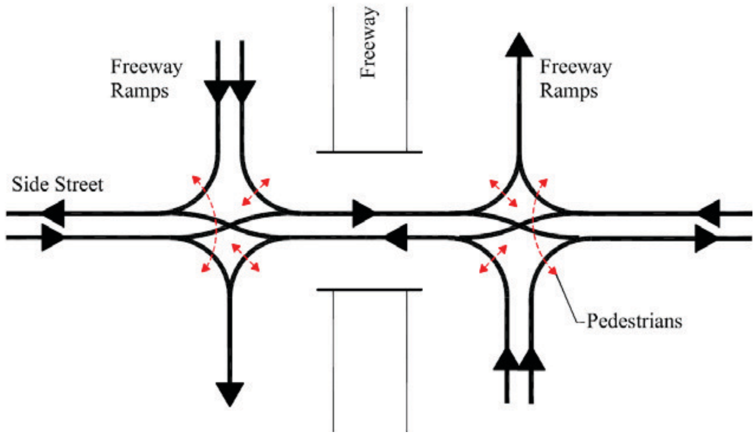


FIGURE 22 Typical DCD interchange configuration (Hughes et al. 2010).

are approximately 300 ft. FHWA advises that consideration should be given to designing radii at crossovers with heavy vehicles in mind. On rural locations where the minor street has high-speed limits, the use of reverse curvature has been suggested. This may result in loon-like flare-outs at the ends of the bridge structure, and additional right-of-way may be required to widen the bridge or the underpass structure.

Median width is also an important design element for a DCD interchange (Hughes et al. 2010). Greater median width is required for the flaring needed for reverse curves. Designers are advised to obtain minimum median widths from the *Green Book* and to take into account the installation of post-mounted signs on medians on the bridge deck for safe and effective channelization of traffic. Appropriate offsets for signs should be in accordance with the *MUTCD.* The report states that driver simulator experiments on the Missouri DCD interchange, which included the use of glare screens, showed no erroneous maneuvers by tested subject drivers. Suggested design practices, based on input from Missouri DOT, include the following:

- The minimum crossing angle of the intersection should be 40 degrees.
- The radius design should accommodate between 25 and 30 mph.
- Superelevation may not be needed because it could detract from any desired traffic calming effect.
- Lane width should be approximately 15 ft.
- Design should accommodate WB-67 trucks.
- Adequate lighting should be provided.
- Nearside signals should be considered.
- DCD interchange designs may only be appropriate where there are high-turning volumes.
- Nearby intersections with long cycle lengths should be avoided.
- Pedestrian crossings at free-turning movements should be evaluated and pedestrian signals may be needed.
- The noses of the median island should extend beyond the off-ramp terminals to improve channelization and prevent erroneous maneuvers.
- Left- and right-turn bays should be designed to allow for separate signal phases.

Bared et al. (2005) used simulation to compare the operational performance of a four-lane DDI with a conventional diamond. They concluded that performances for lower and medium volumes are nearly identical in both designs; however, their results from higher volumes showed that the conventional diamond had lower throughput, higher average delay per vehicle, greater stop time, longer queues, and maximum off-ramp flows as compared with the DDI. Evaluation of a six-lane DDI at three scenarios with very high volume indicated that the left-turn capacity of the DDI was twice that of the conventional diamond.

Displaced Left-Turn

The DLT interchange, also known as the continuous flow interchange, is an innovative interchange design that has several aspects similar to the at-grade DLT intersection and some aspects similar to the DCD interchange. It is a design treatment that has been advocated as promising because it removes the conflict at the main intersection between left-turning and opposing through vehicles (Hughes et al. 2010).

The main feature of the DLT interchange design is the left-turn crossovers that are present on the cross-street approaches. In a DLT intersection, the left-turning traffic is relocated at a location several hundred feet upstream of the first signal-controlled ramp terminal of the diamond interchange, shown at the right side of Figure 23. This left-turning traffic (shown as a dashed line) is crossed over the opposing through lanes. The traffic then travels on a new roadway that is situated between the opposing through lanes and a roadway and that carries the right-turning traffic from the ramp. Drivers

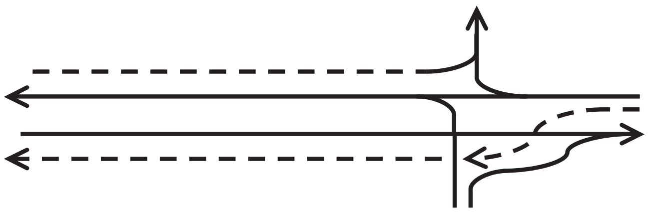


FIGURE 23 Detailed view of movements and paths for half of a DLT interchange.

make the left turn onto the ramp from the new roadway after crossing over the freeway, as shown at the top of Figure 23.

As with a DLT intersection, the differentiating design element of a DLT interchange is the left-turn crossover. The DLT lanes typically cross the opposing through traffic at locations that are approximately 400 to 500 ft upstream of the signal-controlled ramp terminals. Geometrically, the left-turn crossover in a DLT interchange is similar to the design of a left-turn crossover for a DLT intersection. Hughes et al. (2010) cite research into the operation of DLT intersections sponsored by the Maryland State Highway Administration that revealed that the distance between the crossover and the main intersection was dependent on queuing from the main intersection and on costs involved in constructing a left-turn storage area. Radii of the crossover movements range from 150 to 200 ft. The radii of the left-turn movement at the nodes of the interchange are dependent on the turning movement of a design vehicle.

Median width affects the interchange footprint and consequently the right-of-way acquisition. As with DLT intersections, FHWA encourages designers to obtain minimum median widths from the AASHTO *Green Book;* offset recommendations for post-mounted signs should be accounted for in accordance with *MUTCD* when determining median width, though the minimum median width for any type of intersection or interchange is 4 ft (Hughes et al. 2010). The authors further state that a wide median is counterproductive at a DLT interchange for the following reasons:

- Wide medians result in long walking distances for pedestrians at the interchange. In turn, this results in the need for long pedestrian clearance intervals and potentially increased cycle lengths, which is counterproductive to traffic efficiency.
- Wide medians necessitate a wide interchange footprint and consequently higher bridge deck construction costs.

WORK ZONE CONSIDERATIONS

NCHRP Report 581 (Mahoney et al. 2007) discusses a variety of geometric design principles and their applications within work zone traffic control on high-speed highways. In particular, the authors discussed the appropriate design principles for temporary entrance and exit ramps. They stated that a temporary single-lane interchange ramp should have a travel lane of approximately 4.5 m (15 ft), with a 1.8-m (6-ft) right shoulder and minimum 0.6-m (2-ft) left shoulder. However, different cross-sectional arrangements are appropriate when supported by agency experience and in consideration of project-specific factors (e.g., traffic volume, mix, and duration of service).

With respect to entrance ramps, they concluded that the feasibility of maintaining an entrance during construction often hinges on providing an adequate combination of roadway geometry and traffic control to facilitate merging. Figure 24 illustrates a temporary entrance ramp for a median crossover. The authors concluded that the basic principles and issues associated with permanent ramps also pertain to temporary arrangements. Therefore, acceleration lanes in work zones that meet the design criteria for permanent facilities were desirable. However, providing these lane lengths was often not practical. Thus, they provided several "rules of thumb" as options to guide designers:

- Provide at least 90 m [300 ft] of acceleration lane.
- Provide at least 70% of the permanent roadway criteria length.
- Employ traffic control measures (e.g., STOP, YIELD, and other signs) to mitigate less-than-desirable acceleration lane lengths.

The authors also offered similar guidance for exit ramps, stating that it was desirable for exiting traffic to depart the through lanes at mainline speed and not reduce speed while occupying the mainline through lane. When this is not practical, they recommended that the geometry of the ramp be reviewed to determine if the ramp's length, horizontal alignment, and grade allow for gradual deceleration before reaching speed-critical features.

SUMMARY OF KEY FINDINGS

This section summarizes key findings from the research noted in this chapter. This is an annotated summary; conclusions and recommendations are those of the authors of the references cited.

Interchange Ramp Design

- The desired distance between the cross street and freeway merge point is at least 400 m (1,312 ft) for ramps at which metering is envisioned (Chaudhary and Messer 2002).
- The source of the adjustment factors in the 2004 *Green Book* was provided in the 1954 AASHTO *Blue Book,* in which they first appeared as being based on applying "principles of mechanics to rates of speed change for level grades." Further review did not reveal a procedure for determining adjustment factors. A new procedure contains an alternative set of adjustment factors for acceleration length and deceleration length, the latter

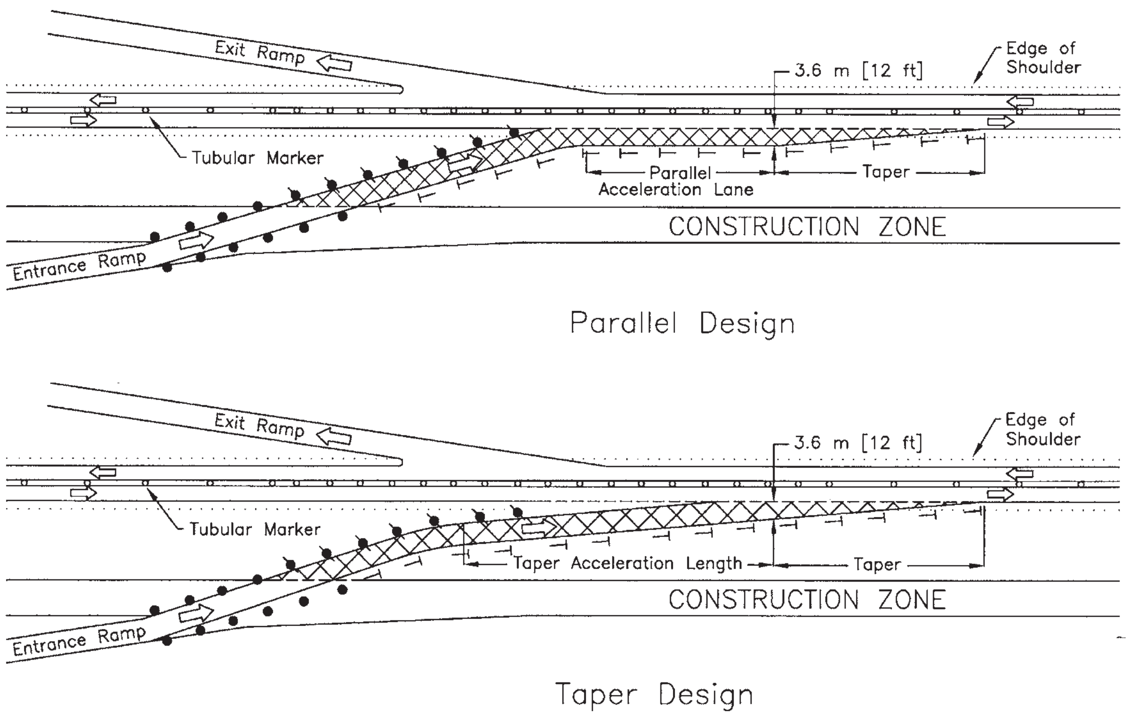


FIGURE 24 Temporary interchange entrance ramp for median crossover (Mahoney et al. 2007).

of which is based on the ratio of the deceleration length on a grade to the deceleration length on a level surface. Actual performance of vehicles on grades and on a level surface should be measured and compared with the suggested adjustment factors to determine the accuracy of those factors (Fitzpatrick and Zimmerman 2007).

Ramp and Interchange Spacing

- Recent guidelines make a distinction to separately define "ramp spacing" and "interchange spacing" and recommend ramp spacing values be the primary consideration in freeway and interchange planning and design (Ray et al. 2011).
- Guidelines are presented based on four areas of emphasis: geometric design, traffic operations, signing, and safety. Geometric design principles, as well as site-specific features, dictate minimum lengths needed for ramps and other interchange components. Traffic volumes can necessitate increased spacing beyond the dimensions needed purely for geometrics. Safety tradeoffs, which until recently have rarely been quantified, can now be considered in project decision making. Finally, signing and other human factors considerations should be taken into account at the earliest in the evaluation process when making choices about ramp and interchange spacing (Ray et al. 2011).
- Spacing assessments indicate that ramp spacing of less than 900 ft is likely not geometrically feasible. That spacing value increases up to 1,600 ft for entrance–exit ramp pairs (Ray et al. 2011).

Alternative Interchange Designs

- Design practices for the DDI (Hughes et al. 2010) include:
  - The minimum crossing angle of intersection should be 40 degrees.
  - The radius design should accommodate between 25 and 30 mph.
  - Superelevation may not be needed because it could detract from any desired traffic calming effect.
  - Lane width should be approximately 15 ft.
  - Design should accommodate WB-67 trucks.
  - Adequate lighting should be provided.
  - Nearside signals should be considered.
  - DCD interchange designs may only be appropriate where there are high-turning volumes.
  - Nearby intersections with high cycle lengths should be avoided.
  - Pedestrians at free-turning movements should be evaluated, and pedestrian signals may be needed.
  - The noses of the median island should extend beyond the off-ramp terminals to improve channelization and prevent erroneous maneuvers.
  - Left- and right-turn bays should be designed to allow for separate signal phases.
- The Displaced Left-Turn interchange has functions similar to a DLT at-grade intersection. DLT lanes typically cross the opposing through traffic at locations that are approximately 400 to 500 ft upstream of the signal-controlled ramp terminals. Minimum median widths are preferred for this design (Hughes et al. 2010).

CHAPTER SEVEN

CONCLUSIONS

BACKGROUND

From 2000 through early 2011 a significant amount of geometric design-related research was conducted on a wide variety of topics and issues. The objective of this study was to identify and summarize a sample of roadway geometric design literature completed and published during that time, particularly research that identified safety, operations, and maintenance impacts. A national literature review represented the vast majority of the effort for this synthesis study.

SUMMARY OF FINDINGS

The body of this report has five primary chapters, in addition to the introductory chapter and this concluding chapter. This section of the report will present a summary of the key findings in the body of the report, categorized by topic. This is an annotated summary of the findings from the research discussed in the body of the report; the recommendations listed are those of the authors of the references cited.

It is important to note that the recommendations included in this list of findings from the literature are those of the authors cited. Before any revisions to AASHTO's *Green Book* were to be made on the basis of these recommendations, they would need to be considered on the basis of the rigor of the research and logic that underlie them. No endorsement of these recommendations is implied by their inclusion in the listing of findings from the literature.

Design Vehicles

- Dimensions of commonly used trucks have changed in recent years, prompting recommendations to revise the dimensions of those vehicles in AASHTO's *A Policy on Geometric Design of Highways and Streets,* commonly known as the *Green Book* (Harwood et al. 2003).
- Along with the changes in dimensions have come changes in performance; however, design guidelines are sufficient to accommodate their performance for many design elements (Harwood et al. 2003a).

Design Speed

- Posted speed limit and anticipated operating speed were frequently associated with the selection of design speed (Fitzpatrick and Carlson 2002).
- Observation of driving behavior revealed that the strongest indicator of operating speed was posted speed limit. Design speed appeared to have minimal impact on operating speeds unless a tight horizontal radius or a low K-value was present (Fitzpatrick et al. 2003a).
- Researchers investigated the possibility of selecting a design speed based more heavily on the context of the environment in which the roadway was located. A primary area of concern, however, was how to define the context to be considered (Garrick and Wang 2005; Wang et al. 2006).

Driver Characteristics

- Designers and traffic engineers must examine the roadway environment for information conflicts that may mislead or confuse road users (Campbell et al. 2008).
- Designers and traffic engineers must also seek road environments that are self-explaining, quickly understood, and easy for users to act on (Campbell et al. 2008).

Stopping Sight Distance

- New values for stopping sight distance (SSD) and new design controls for vertical curves were recommended based on a perception–reaction time of 2.5 s, a 10th percentile deceleration rate of 11.2 ft/s^2, a 10th percentile driver eye height of 3.5 ft, and a 10th percentile object height of 2.0 ft (Fambro et al. 2000).
- Ramp control signals placed on the left side of a curve of a loop on-ramp (even with a radius greater than 300 ft) are more critical for accommodating SSD than those on the right side (Wang 2007).
- The method of selecting SSD values deterministically yielded very conservative estimates of available and required SSD, resulting in a very low probability (0.302%) of hazard (Sarhan and Hassan 2008).

Passing Sight Distance

- An analysis of observed passing maneuvers provided support for the AASHTO passing sight distance (PSD) model, and the model provided reasonable results for the assumptions made. However, the model's assumptions may need to be updated or accommodate more flexibility for speeds higher than 55 mph (Carlson et al. 2005).

- Increased consistency between AASHTO PSD design standards and *Manual on Uniform Traffic Control Devices* (*MUTCD*) pavement marking practices was recommended, specifically accomplished by using the *MUTCD* criteria for marking passing/no-passing zones on two-lane roads in the *Green Book*'s PSD design process. In addition to providing the desired consistency between PSD design and marking practices, two-lane highways could be designed to operate safely with the *MUTCD* criteria (Harwood et al. 2008).

Horizontal Alignment

- Erroneous perceptions by drivers approaching horizontal curves, as influenced by vertical curves, increased as (1) the sight distance increased, (2) the horizontal curve radius increased, and (3) the length of vertical curve per 1% change in grade decreased. Drivers tend to drive faster on horizontal curves in sag combinations and slower on horizontal curves in crest combinations. Designers should establish the profile and predicted operating speed of an alignment based on a three-dimensional model, rather than a traditional two-dimensional model (Bidulka et al. 2002; Hassan et al. 2002).
- For drivers on curves with radii greater than or equal to 350 m (1,146 ft), as the deflection angle increased, speed measures (mean, 85th percentile, and 95th percentile) decreased; as a result, motorists may view a large change in direction as a motivation to slow their speed. In addition, as curve length increased, speed measures increased, suggesting that drivers may become more comfortable at higher speeds because they have more time to adjust their vehicle path to a constant radius. Grade has an influence on the upper-percentage range of vehicle speeds, because the 85th percentile speed decreased as approach grade increased (Schurr et al. 2002).
- A study of driver behavior and errors on a selection of horizontal curves led Lyles and Taylor (2006) to conclude the following:
  - Drivers approaching curves routinely exceeded the posted speed limit as well as the posted advisory speed, where applicable.
  - Drivers had more errors at curves where they had limited or no visibility of the curves when the TCDs were first visible.
  - Drivers made more errors on horizontal curves that were adjacent to vertical curves, particularly crests that obscured a downstream horizontal curve.
  - There were increased errors when curves were combined with other elements, especially intersections.

Vertical Alignment

- Current North American design practices might yield segments of the vertical curve where the driver's view is constrained to a distance shorter than the required SSD. An alternative design procedure is recommended, based on a new model that incorporated longitudinal friction and acceleration, which produced new recommended values for minimum lengths of crest and sag vertical curves (Hassan 2004).
- A weight/power ratio of 102 to 108 kg/kW (170 to 180 lb/hp) would be appropriate for freeways in California and Colorado, and a weight/power ratio of 126 kg/kW (210 lb/hp) would be more appropriate in Pennsylvania, as compared with the 120 kg/kW (200 lb/hp) value recommended in the 2001 *Green Book* (Torbic et al. 2005).
- The upward divergent headlamp angle used in the sag curve design equation should be reduced from 1° to between 0.75° and 0.90° (Hawkins and Gogula 2008).

Allocation of Traveled Way Width

- The benefits of 2+1 roads in Europe validated a recommendation for their use in the United States, to serve as an intermediate treatment between an alignment with periodic passing lanes and a full four-lane alignment. Such 2+1 roads are most suitable for level and rolling terrain, with installations to be considered on roadways with traffic flow rates of no more than 1,200 veh/hr in a single direction. The use of a cable barrier as a separator is discouraged, and major intersections should be located in the buffer or transition areas between opposing passing lanes, with the center lane used as a turning lane (Potts and Harwood 2003).
- Passing activity on 2+1 roads was greatest at the beginning of the segments and the greatest benefits of decreased platooning and increased safety occurred within the first 0.9 mi of a passing lane segment (Gattis et al. 2006).
- Most passing on Super 2 passing lanes occurs within the first mile of a passing lane, so additional length may be less useful than additional lanes in a Super 2 corridor, particularly at lower volumes. Designers should avoid intersections with state highways and high-volume county roads within passing lanes, consider terrain and right-of-way in determining alignment and placement of passing lanes, avoid the termination of passing lanes on uphill grades, and discourage passing lane lengths longer than 4 mi (Brewer et al. 2011).
- Two-way left-turn lanes could be used as a strategy to reduce head-on collisions on two-lane roads (Neuman et al. 2003b).

Lane Width

- There was no general indication that the use of lanes narrower than 12 ft on urban and suburban arterials increased crash frequencies. Geometric design policies should provide substantial flexibility for the use of lane widths narrower than 12 ft (Potts et al. 2007).
- Lane widths of 11 or 12 ft provide optimal safety benefit for common values of total paved width on rural two-lane roads. Although 12-ft lanes appear to be the optimal

design for 26- to 32-ft total paved widths, 11-ft lanes perform equally well or better than 12-ft lanes for 34- to 36-ft total paved widths (Gross et al. 2009).

Road Diet

- The rate of road diet crashes occurring during the period after installation was about 6% lower than that of matched comparison sites. However, controlling for possible differential changes in average daily traffic, study period, and other factors indicated no significant effect of the treatment. Crash severity was virtually the same at road diets and comparison sites. Conversion to a road diet should be made on a case-by-case basis in which traffic flow, vehicle capacity, and safety are all considered (Huang et al. 2002).
- The effects of the road diet on crashes in Iowa, accounting for monthly crash data and estimated volumes for treatment and comparison sites, resulted in a 25.2% reduction in crash frequency per mile and an 18.8% reduction in crash rate (Pawlovich et al. 2006).

Shoulder Width

- For horizontal curves on two-lane nonresidential facilities that have 3 degrees of curvature, the width of the lane plus the paved shoulder should be at least 5.5 m (18 ft) throughout the length of the curve (Staplin et al. 2002).
- Wider lane and shoulder widths are associated with a reduction in segment-related collisions on rural frontage road segments (Lord and Bonneson 2007).

Rumble Strips

- Crashes at approximately 210 mi of undivided rural two-lane roads treated with centerline rumble strips were reduced by 14% and injury crashes were reduced by an estimated 15%. All frontal and opposing-direction sideswipe crashes were reduced by an estimated 21%, and those crashes involving injuries were reduced by an estimated 25%. All of the reductions were determined to be statistically significant (Persaud et al. 2003).
- Crash data on roads treated with centerline rumble strips or shoulder rumble strips revealed noticeable crash reductions on all classes of roads (rural and urban two-lane roads and freeways). Shoulder rumble strips placed as close to the edgeline as possible maximize safety benefits. The safety benefits of centerline rumble strips for roadways on horizontal curves and on tangent sections are for practical purposes the same (Torbic et al. 2009).

Shoulder Edge Treatments

- Plaxico et al. (2005) made the following recommendations on design guidelines for using curbs on roadways with operating speeds greater than 60 km/h (37.3 mph):
  - Any combination of a sloping-faced curb that is 150 mm (6 in.) or shorter and a strong-post guardrail can be used where the curb is flush with the face of the guardrail up to an operating speed of 85 km/h.
  - Guardrails installed behind curbs are best not located closer than 2.5 m (8.2 ft) for any operating speed in excess of 60 km/h (37.3 mph).
  - For roadways with operating speeds of 70 km/h (43.5 mph) or less, guardrails may be used with sloping-face curbs no taller than 150 mm (6 in.) as long as the face of the guardrail is located at least 2.5 m (8.2 ft) behind the curb.
  - Where guardrails are installed behind curbs on roads with operating speeds between 71 and 85 km/h (44.1 and 52.8 mph), a lateral distance of at least 4 m (13.1 ft) is needed to allow the vehicle suspension to return to its pre-departure position.
  - At operating speeds greater than 85 km/h (52.8 mph), guardrails are used with 100-mm (4-in.) or shorter sloping-faced curbs, and are placed so that the curb is flush with the face of the guardrail. Operating speeds above 90 km/h (55.9 mph) require that the sloping face of the curb must be 1:3 or flatter and must be no more than 100 mm (4 in.) in height.
- The "Safety Edge" treatment produced small but positive results in crash reduction at 56 of 81 treated sites. For all two-lane highway study sites in two states, the best estimate of the treatment's effectiveness was a reduction in total crashes of approximately 5.7%. The results were not statistically significant, but they were generally positive (Hallmark et al. 2006).

Roadside

- Where possible at curb locations, provide a lateral offset to rigid objects of at least 6 ft from the face of the curb and maintain a minimum lateral offset of 4 ft (Dixon et al. 2008).
- At lane merge locations, do not place rigid objects in an area that is 10 ft longitudinally from the taper point. The lateral offset for this 20-ft section is consistent with the lane width, typically 12 ft (Dixon et al. 2008).
- A lateral offset of 6 ft from the curb face to rigid objects is preferred for higher-speed auxiliary lane locations, such as extended length right-turn lanes, and a 4-ft minimum lateral offset should be maintained (Dixon et al. 2008).
- At locations where a sidewalk buffer is present, rigid objects are best not located in a buffer area with a width of 3 ft or less. For buffer widths greater than 3 ft, lateral offsets from the curb face to rigid objects are maintained with a minimum offset of 4 ft. At these wider buffer locations, other frangible objects can be strategically located to help shield any rigid objects (Dixon et al. 2008).
- Rigid objects are best not located in the proximity of driveways, and care should be taken to avoid placing

rigid objects on the immediate far side of a driveway. In addition, objects are not to be located within the required sight triangle for a driveway (Dixon et al. 2008).

Intersection Alignment

- Avoid approach grades to an intersection of greater than 6%. On higher design speed facilities (50 mph and greater), a maximum grade of 3% should be considered (Rodegerdts et al. 2004).
- Avoid locating intersections along a horizontal curve of the intersecting road (Rodegerdts et al. 2004).
- Strive for an intersection platform (including sidewalks) with a cross slope not exceeding 2%, as needed for accessibility (Rodegerdts et al. 2004).
- Approach curvature can be used as a treatment to force a reduction in vehicle speed through the introduction of horizontal deflection at high-speed intersection approaches, but it is discouraged at downhill approaches (Ray et al. 2008).
- A skew angle greater than 20 degrees is not recommended in design when the design vehicle is a large vehicle or semitrailer (Son et al. 2002).
- A minimum skew angle of 15 degrees will accommodate age-related performance deficits at intersections where right-of-way is restricted (Staplin et al. 2002).

Auxiliary Lanes

- Adding left-turn lanes is effective in improving safety at signalized and unsignalized intersections, reducing crashes between 10% and 44% (Harwood et al. 2002).
- Positive results can also be expected for right-turn lanes, with reductions in total intersection accidents between 4% and 14% (Harwood et al. 2002).
- A method was developed to identify where installation of right-turn lanes at unsignalized intersections and major driveways would be cost-effective, indicating combinations of through-traffic volumes and right-turn volumes for which provision of a right-turn lane would be recommended. The economic analysis procedure can be applied by highway agencies using site-specific values for average daily traffic, turning volumes, accident frequency, and construction cost for any specific location (or group of similar locations) of interest (Potts et al. 2007).

Modern Roundabouts

- A series of projects during the decade led to the publication of two FHWA *Informational Guides* containing recommendations and guidelines for all aspects of roundabout design.
- General overarching principles of geometric design of roundabouts (Rodegerdts et al. 2010) include:
  - "Provide slow entry speeds and consistent speeds through the roundabout by using deflection.
  - Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume balance, and lane continuity.
  - Provide smooth channelization that is intuitive to drivers and results in vehicles naturally using the intended lanes.
  - Provide adequate accommodation for the design vehicles.
  - Design to meet the needs of pedestrians and cyclists.
  - Provide appropriate sight distance and visibility for driver recognition of the intersection and conflicting users."
- Maximum entering design speeds are based on a theoretical fastest path of 20 to 25 mph for single-lane roundabouts and 25 to 30 mph for multilane roundabouts (Rodegerdts et al. 2010).
- Roundabout alignment is described as "optimally located when the centerlines of all approach legs pass through the center of the inscribed circle" (Robinson et al. 2000).
- Common inscribed circle diameters for single-lane roundabouts vary from 90 to 180 ft, depending on design vehicle (Rodegerdts et al. 2010).
- Designers "should provide no more than the minimum required intersection sight distance on each approach, [because] excessive intersection sight distance can lead to higher vehicle speeds that reduce the safety of the intersection for all road users" (Robinson et al. 2000).
- Crash experience at selected intersections in the United States indicates an overall reduction in crash frequency at intersections converted to roundabouts (Rodegerdts et al. 2007).
- Pedestrian refuge a minimum width of 6 ft will adequately provide shelter for persons pushing a stroller or walking a bicycle (Robinson et al. 2000).
- At single-lane roundabouts, the pedestrian crossing is best located one vehicle-length (25 ft) away from the yield line. At double-lane roundabouts, the pedestrian crossing is best located one, two, or three car lengths (approximately 25 ft, 50 ft, or 75 ft) away from the yield line (Robinson et al. 2000).
- The "pedestrian refuge should be designed at street level, rather than elevated to the height of the splitter island. This eliminates the need for ramps within the refuge area, which can be cumbersome for wheelchairs" (Robinson et al. 2000).
- Ramps may be provided on each end of crosswalks to connect the crosswalk to other crosswalks around the roundabout and to the sidewalk network (Robinson et al. 2000).
- A detectable warning surface, as recommended in the Americans with Disabilities Act Accessibility Guidelines, may be applied to the surface of the refuge within the splitter island (Robinson et al. 2000).
- Use of standard AASHTO island design for key dimensions, such as offset and nose radii, is encouraged. For

sidewalks, a setback distance of 5 ft, with a minimum of 2 ft is advised (Robinson et al. 2000).

- For nonmotorized users such as bicyclists, one important consideration during the initial design stage is to maintain or obtain adequate right-of-way outside the circulatory roadway for the sidewalks. All nonmotorized users who are likely to use the sidewalk regularly, including bicyclists in situations where roundabouts are designed to provide bicycle access to sidewalks, should be considered in the design of the sidewalk width. Recommended designs for single-lane roundabouts encourage bicycle users to merge into the general travel lanes and navigate the roundabout as a vehicle, explaining that the typical vehicle operating speed within the circulatory roadway is in the range of 15 to 25 mph, which is similar to that of a bicycle (Rodegerdts et al. 2010).

Innovative Intersection Designs

A number of new or innovative intersection designs were considered during the decade; each of the following was described in one or more studies.

- Displaced Left Turns showed considerable savings in average control delay and average queue length, as well as an increase in intersection capacity, in one series of microsimulation analyses (Hughes et al. 2010).
- Median U-turns are typically a corridor treatment applied at signalized intersections but are also used at isolated intersections to alleviate specific traffic operational and safety problems (Hughes et al. 2010).
- Median width of Restricted Crossing U-Turns is a crucial design element to accommodate large trucks without allowing vehicles to encroach on curbs or shoulders (Hughes et al. 2010).
- Quadrant Roadways are best designed so that the left turn with the highest demand is the one that receives the most direct path (Hughes et al. 2010).
- Double Crossover Intersections are found to have greater throughput than a conventional intersection, along with lower values for number of stops, average stop time per vehicle, average queue, and maximum queue length (Bared et al. 2005).
- Arterial Interchanges have an overall capacity near 75% of a four-lane freeway (Eyler 2005).
- J-Turn and Offset-T designs had reductions in crashes between 40% and 92% (Maze et al. 2010).
- Two-Level Signalized Intersections produced modeled results with the shortest delay times in most evaluation scenarios as well as the least sensitivity to variations in traffic volume compared with other innovative intersection types; however, delay increased when flow was unbalanced between the two crossing roads (Shin et al. 2008).
- The additional right-of-way needed to construct each of these innovative designs was mentioned as a potential drawback by every report and author that addressed the issue of the intersection's footprint.

Pedestrian and Bicycle Facilities at Intersections

- Suggested strategies (Raborn et al. 2008) for modifying intersections to accommodate bicycles and pedestrians included:
  - Reducing the crossing distance for bicyclists.
  - Realigning intersection approaches to reduce or eliminate intersection skew.
  - Modifying the geometry to facilitate bicycle movement at interchange on-ramps and off-ramps.
  - Providing refuge islands and raised medians.
  - Grade-separated crossings.
- "Pedestrian facilities should be provided at all intersections in urban and suburban areas. In general, design of pedestrian facilities with the most challenged users in mind—pedestrians with mobility or visual impairments—should be done, and the resulting design will serve all pedestrians well. ADA requires that new and altered facilities constructed by, on behalf of, or for the use of State and local government entities be designed and constructed to be readily accessible to and usable by individuals with disabilities" (Rodegerdts et al. 2004).
- Practitioners should incorporate key elements that affect a pedestrian facility into their design (Rodegerdts et al. 2004):
  - "Keep corners free of obstructions to provide enough room for pedestrians waiting to cross.
  - Maintain adequate lines of sight between drivers and pedestrians on the intersection corner and in the crosswalk.
  - Ensure curb ramps, transit stops (where applicable), pushbuttons, etc., are easily accessible and meet ADA Accessibility Guidelines design standards.
  - Clearly indicate the actions pedestrians are expected to take at crossing locations.
  - Design corner radii to ensure vehicles do not drive over the pedestrian area yet are able to maintain appropriate turning speeds.
  - Ensure crosswalks clearly indicate where crossings should occur and are in desirable locations.
  - Provide appropriate intervals for crossings and minimize wait time.
  - Limit exposure to conflicting traffic and provide refuges where necessary.
  - Ensure the crosswalk is a direct continuation of the pedestrian's travel path.
  - Ensure the crossing is free of barriers, obstacles, and hazards."

Transit Considerations

- General intersection design principles and guidelines for transit issues (Eccles et al. 2007) include:
  - "Provide simple intersection designs.
  - Provide clear visual cues to make busway intersections conspicuous.
  - Maximize driver and pedestrian expectancy.
  - Separate conflicting movements.

- Minimize street crossings.
- Incorporate design features that improve safety for vulnerable users.
- Coordinate geometric design features and traffic control devices."

- There are four types of busways found at intersections: median busways, side-aligned busways, separated right-of-way busways, and bus-only ramps. Each busway type has unique characteristics that are considerations for guidance on safety issues, basic geometry (including placement of bus stops), and traffic control, along with examples of appropriate intersections for each type of busway (Eccles et al. 2007).

Access Management at Intersections

- Right-turn-plus-U-turn could have better operational performance than direct left turns under certain traffic conditions, implying that directional median opening designs could provide more efficient traffic flow than full median openings (Zhou et al. 2002).
- U-turns at signalized intersections resulted in a 1.8% saturation flow rate loss in the left-turn lane for every 10% increase in U-turn percentage and an additional 1.5% loss for every 10% U-turns if the U-turning movement was opposed by protected right-turn overlap from the cross street (Carter et al. 2005a).
- Recommended practices (Potts et al. 2004) for rural unsignalized intersections include:
  - Medians that are as wide as practical, as long as the median is not so wide that approaching vehicles on the crossroad cannot see both roadways of the divided highway.
  - Where the AASHTO passenger car is used as the design vehicle, a minimum median width of 25 ft is recommended.
  - Where a large truck is used as the design vehicle, a median width of 70 to 100 ft generally is recommended. If such a median width cannot be provided, consideration should be given to providing a loon.
- Recommended practices (Potts et al. 2004) for suburban unsignalized intersections include:
  - Median widths at suburban unsignalized intersections generally as narrow as possible while providing sufficient space in the median for the appropriate left-turn treatment.
  - Median widths between 14 and 24 ft will accommodate left-turn lanes, but are not wide enough to store a crossing or turning vehicle in the median.
  - Medians wider than 25 ft may be used, but crossroad vehicles making turning and crossing maneuvers may stop on the median roadway.
  - Median widths of more than 50 ft generally should be avoided at suburban, unsignalized intersections.
- Keep median opening lengths at rural divided highway intersections generally to the minimum possible. Increases in median opening length are correlated with higher rates of undesirable driving behavior. In contrast, the median opening in urban and suburban areas can be as long as necessary (Potts et al. 2004).

Interchange Ramp Design

- The desired distance between the cross street and freeway merge point is at least 400 m (1,312 ft) for ramps at which metering is envisioned (Chaudhary and Messer 2002).
- The source of the adjustment factors in the 2004 *Green Book* was provided in the 1954 AASHTO *Blue Book,* in which they first appeared as being based on applying "principles of mechanics to rates of speed change for level grades." Further review did not reveal a procedure for determining adjustment factors. A new procedure contains an alternative set of adjustment factors for acceleration length and deceleration length, the latter of which is based on the ratio of the deceleration length on a grade to the deceleration length on a level surface. Actual performance of vehicles on grades and on a level surface should be measured and compared with the suggested adjustment factors to determine the accuracy of those factors (Fitzpatrick and Zimmerman 2007).

Ramp and Interchange Spacing

- Recent guidelines make a distinction to separately define "ramp spacing" and "interchange spacing" and recommend ramp spacing values be the primary consideration in freeway and interchange planning and design (Ray et al. 2011).
- Guidelines are presented based on four areas of emphasis: geometric design, traffic operations, signing, and safety. Geometric design principles, as well as site-specific features, dictate minimum lengths needed for ramps and other interchange components. Traffic volumes can necessitate increased spacing beyond the dimensions needed purely for geometrics. Safety tradeoffs, which have rarely been quantified until recently, can now be considered in project decision making. Finally, signing and other human factors issues are best taken into account at the earliest in the evaluation process when making choices about ramp and interchange spacing (Ray et al. 2011).
- Spacing assessments indicate that ramp spacing of less than 900 ft is likely not geometrically feasible. That spacing value increases up to 1,600 ft for entrance–exit ramp pairs (Ray et al. 2011).

Alternative Interchange Designs

- Design practices for the Diverging Diamond interchange (Hughes et al. 2010) include:
  - "The minimum crossing angle of intersection should be 40 degrees.
  - The radius design should accommodate between 25 and 30 mph.

  - Superelevation may not be needed because it could detract from any desired traffic calming effect.
  - Lane width should be around 15 ft.
  - Design should accommodate WB-67 trucks.
  - Adequate lighting should be provided.
  - Nearside signals should be considered.
  - Double Crossover Diamond interchange designs may only be appropriate where there are high-turning volumes.
  - Nearby intersections with high cycle lengths should be avoided.
  - Pedestrians at free-turning movements should be evaluated, and pedestrian signals may be needed.
  - The noses of the median island should extend beyond the off-ramp terminals to improve channelization and prevent erroneous maneuvers.
  - Left- and right-turn bays should be designed to allow for separate signal phases."
- The Displaced Left-Turn interchange has functions similar to a DLT at-grade intersection. DLT lanes typically cross the opposing through traffic at locations that are approximately 400 to 500 ft upstream of the signal-controlled ramp terminals. Minimum median widths are preferred for this design (Hughes et al. 2010).

BARRIERS TO WIDESPREAD IMPLEMENTATION

This section discusses some potential barriers to the widespread implementation of the research and findings presented within the report. These potential barriers are presented as observations gleaned through the compilation of the material collected for this research.

- A large number of the sources reviewed for this synthesis produced results and recommendations that incorporated the use of a series of complex equations and/or multiple assumptions to begin the analysis. Such complex methodology may not be conducive to practitioners because the complex equations do not facilitate their use or because the necessary data are not available.
- Similarly, the use of computer-based simulation and modeling has greatly increased as technology improves. However, many designers, particularly those at agencies in smaller jurisdictions, do not have access to such software or expertise to successfully use it to obtain the results described in the research.
- The advent of multiple innovative intersection treatments has led to a wide variety of potential outcomes, and the research to support those outcomes is not yet mature. Practitioners who desire to use one or more of these treatments are cautioned in multiple studies that results are still very preliminary. In addition, these treatments typically require additional right-of-way and construction costs. Although they may be less expensive than a fully grade-separated facility, the cost is still a major factor in determining which treatment to use. The added complexity of the design and the need to "train" drivers how to use the new intersections are also considerations.
- As roadway agencies continue to investigate new ways to use their budgets more efficiently, the cost of any treatment will likely be further scrutinized, whether it is the realignment of a skewed intersection or the addition of rumble strips to a lengthy section of two-lane highway. Treatments that can provide benefits at low costs would appear to become increasingly valuable and desirable in this fiscal environment.

RECOMMENDATIONS FOR FURTHER RESEARCH

During the course of their projects, many researchers identified gaps in knowledge or additional questions that were raised as a result of their findings. Other needs for future research have also been identified based on information that was not found within the literature that was reviewed for this synthesis report. Recommendations for research to fill those needs are summarized here:

- Fitzpatrick et al. (2006) recommended that their findings on safety and operations at exclusive right-turn lanes be verified through use of a larger, more comprehensive study that includes right-turning volume.
- Multiple studies mentioned the lack of data on U-turns at median openings not designed for U-turns and/or suggested this as a valid research topic to examine safety and operational effects of such maneuvers.
- Carter et al. (2005a) discussed several potential research topics for U-turns at signalized intersections. Among them are potential benefits of "U-turn Must Yield" signs; mitigation of the effects of right-turn overlap; a U-turn prediction model based on driveway density, land usage, and other site characteristics; and the effects on capacity and safety of U-turning heavy vehicles.
- *NCHRP Report 672* (Rodegerdts et al. 2010) advised the use of a critical headway of 5.0 s, based on the critical headway required for passenger cars. The authors added that this value represented an interim methodology pending further research.
- With the advent and increasing popularity of electronic toll collection methods, toll plaza design practices are changing to a certain degree. Among the reviewed sources and the practitioners who focus on this area of geometric design, there appears to be a consensus that more recent information on updated practices may be fragmented, scattered, or not yet evaluated; a need exists for at least a compendium of the best knowledge currently available on those measures found to be the most successful in the application of geometric criteria in the design of fixed-barrier manually operated plazas as well as in the removal of the barriers and replacement with electronic open-road tolling gantries.
- Research is needed on intermediate speeds in the range of 40 to 50 mph in urban and suburban areas, and their effects on various cross-sectional design elements. Such

cross-section elements include the allocation of lane and shoulder widths, use of various median types and widths, the provision of bike lanes, parking lanes, use of vertical or sloping curbs and gutters and associated offsets, clear zone widths, traffic barriers, utilities, and interactions of various combinations of these elements.

- Highway designers are under increasing pressure to maximize the use of available right-of-way in freeway corridors to provide safety, mobility, and capacity for growing traffic demand. With right-of-way limitations, increased use of context-sensitive designs, and implementation of managed facilities, designers must maximize the use of freeway cross sections. Although freeway cross-section design guidance suggests that 12-ft lanes with 8- to 10-ft inside and outside shoulders is ideal, there is limited research on how deviations from these ideals individually, or in combination, will affect freeway operations and safety. Highway designers need guidance on the operational and safety impacts for cross-section design trade-offs while trying to balance corridor capacity, project costs, public involvement, and environmental impacts.
- In addition, there is concern over the part-time use of existing shoulders as high-occupancy vehicle, high-occupancy toll, or general-use facilities during peak hour. The trade-offs between operational benefits and safety need to be quantified. Further, the safety implications of violators using the shoulder during the off-peak period need to be quantified. It is unclear whether this changed view of the shoulder as part of the traveled way also transfers to shoulder violations on adjacent facilities. The signing and striping of these shoulders for clear communication of the changed cross section use must also be quantified.
- Despite the many features of the Interactive Highway Safety Design Model, methods to assess design consistency for multi-lane rural highways and urban and suburban arterials are not available. Development of design consistency procedures for these facility types will provide a full suite of mobility and safety assessment tools for use by designers throughout the project development process.
- Typically, ramp terminals and the ramp proper are designed independent of each other and the two components are simply put together in the final design of a ramp. Ramp design practices may consider driver expectations and behaviors over a full range of geometric and traffic conditions that would include the interchange form, ramp type, the area environment (rural vs. urban) and the functional classification of the two interchanging roadways. The issue of an integrated ramp and ramp terminal design is a complex issue in need of basic research.
- Decision sight distance policy is based on a relatively small research study completed for FHWA in 1978 (McGee et al.). Decision sight distance is clearly intended for application at selected locations where greater sight distance than SSD is needed. However, there is little practical guidance to help designers identify situations where decision sight distance is or is not appropriate. And, there is little available information on how decision sight distance criteria are actually being applied by highway engineers and whether the decision sight distance policy is accomplishing its stated objective.
- Traffic calming guidelines often discuss the benefits of designing roadways to improve pedestrian safety. In theory, roadways that are designed with certain characteristics can encourage slower motor vehicle speeds, which cause more motor vehicle drivers to yield to pedestrians crossing the street and result in less severe pedestrian injuries when crashes do occur. Yet, there is a lack of research that quantifies the complexity of relationship between the following three factors: (1) roadway design, (2) motor vehicle speed, and (3) motorist yielding behavior. The effects of roadway design treatments on driver yielding are unknown for many different combinations of traffic speed and roadway conditions. This makes it extremely difficult to craft pedestrian-oriented guidelines that are applicable to the wide range of conditions present in communities throughout the country. More research is needed to quantify how driver yielding behavior is related to travel speed and different roadway characteristics, such as lane widths, pavement conditions, horizontal and vertical shifts, sight distances, lateral clearance, and other factors. This research should be used to create improved guidelines for roadway design and traffic calming practice.

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Abbreviations 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