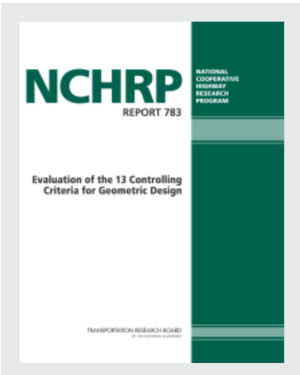


Evaluation of the 13 Controlling Criteria for Geometric Design

DETAILS

94 pages | 8.5 x 11 | PAPERBACK
ISBN 978-0-309-30796-3 | DOI 10.17226/22291

CONTRIBUTORS

Harwood, Douglas W.; Hutton, Jessica M.; Fees, Chris; Bauer, Karin M.; Glen, Alan; Ouren, Heidi; Quincy Engineering; HQE, Inc.

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 783

Evaluation of the 13 Controlling Criteria for Geometric Design

Douglas W. Harwood
Jessica M. Hutton
Chris Fees
Karin M. Bauer
Alan Glen
Heidi Ouren
MRIGlobal
Kansas City, MO

Quincy Engineering
Sacramento, CA

HQE, Inc.
Oakland, CA

Subscriber Categories
Design

Research sponsored by the American Association of State Highway and Transportation Officials
in cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD

WASHINGTON, D.C.
2014
www.TRB.org

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.

The Transportation Research Board of the National Academies was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

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.

NCHRP REPORT 783

Project 17-53
ISSN 0077-5614
ISBN 978-0-309-30796-3
Library of Congress Control Number 2014948497

NOTICE

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.

The Transportation Research Board of the National Academies, the National Research Council, and the sponsors of the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of the report.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
Business Office
500 Fifth Street, NW
Washington, DC 20001

and can be ordered through the Internet at:

http://www.national-academies.org/trb/bookstore

Printed in the United States of America

THE NATIONAL ACADEMIES

Advisers to the Nation on Science, Engineering, and Medicine

The **National Academy of Sciences** is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. Upon the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Ralph J. Cicerone is president of the National Academy of Sciences.

The **National Academy of Engineering** was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. C. D. Mote, Jr., is president of the National Academy of Engineering.

The **Institute of Medicine** was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, upon its own initiative, to identify issues of medical care, research, and education. Dr. Victor J. Dzau is president of the Institute of Medicine.

The **National Research Council** was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both Academies and the Institute of Medicine. Dr. Ralph J. Cicerone and Dr. C. D. Mote, Jr., are chair and vice chair, respectively, of the National Research Council.

The **Transportation Research Board** is one of six major divisions of the National Research Council. The mission of the Transportation Research Board is to provide leadership in transportation innovation and progress through research and information exchange, conducted within a setting that is objective, interdisciplinary, and multimodal. The Board's varied activities annually engage about 7,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation. **www.TRB.org**

www.national-academies.org

COOPERATIVE RESEARCH PROGRAMS

CRP STAFF FOR NCHRP REPORT 783

Christopher W. Jenks, *Director, Cooperative Research Programs*
Christopher Hedges, *Manager, National Cooperative Highway Research Program*
B. Ray Derr, *Senior Program Officer*
Andréa Parker, *Senior Program Assistant*
Eileen P. Delaney, *Director of Publications*
Ellen M. Chafee, *Editor*

NCHRP PROJECT 17-53 PANEL
Field of Traffic—Area of Safety

Terry L. Berends, *Washington State DOT, Wenatchee, WA* (Chair)
Mark A. Marek, *Texas DOT, Austin, TX*
Troy A. Jerman, *Iowa DOT, Atlantic, IA*
In-Kyu "Q" Lim, *Virginia DOT, Richmond, VA*
Paul F. Schneider, *New Jersey DOT, Trenton, NJ*
Xiaoduan Sun, *University of Louisiana at Lafayette, Lafayette, LA*
William C. Taylor, *Michigan State University, East Lansing, MI*
Mark Doctor, *FHWA Liaison*
Bernardo Kleiner, *TRB Liaison*

FOREWORD

By B. Ray Derr
Staff Officer
Transportation Research Board

This report describes the impact of the controlling roadway design criteria on safety and operations for urban and rural roads. This information will be useful to geometric designers and those responsible for reviewing designs, particularly in agencies that are transitioning away from "standards-based design."

In 1985, the FHWA designated 13 specific design elements as controlling criteria for roadway design (see *Mitigation Strategies for Design Exceptions*). The 13 controlling criteria are (1) design speed, (2) lane width, (3) shoulder width, (4) bridge width, (5) structural capacity, (6) horizontal alignment, (7) vertical alignment, (8) grade, (9) stopping sight distance, (10) cross slope, (11) superelevation, (12) vertical clearance, and (13) horizontal clearance. Federally assisted highway construction and reconstruction projects must meet the established design criteria for these elements, or a formal design exception must be prepared and approved. Different procedures apply to rehabilitation projects, but these design elements are still key considerations in design. Since their designation, the 13 controlling criteria and their application have not been reconsidered as new knowledge has been gained about the relationships between geometric design elements and safety and operations.

In NCHRP Project 17-53, MRIGlobal and their subcontractors (Quincy Engineering and HQE, Inc.) investigated what is known about the safety and operational effects of the 13 controlling and other important geometric design criteria. Several small studies were done to augment the information found in the literature. This information was used to assess the sensitivity of safety and operations to design decisions for these criteria for different types of roads. The research also addressed how to reduce confusion related to the definitions of the controlling criteria.

The use of the controlling criteria in design exception processes was also explored, including through interviews with state department of transportation (DOT) personnel. It is expected that the report will be useful to state DOTs in reviewing their design exception policies for non-federal projects.

CONTENTS

SUMMARY

Evaluation of the 13 Controlling Criteria for Geometric Design

The FHWA has, since 1985, designated 13 specific design elements as controlling criteria for roadway design. These are design speed, lane width, shoulder width, bridge width, structural capacity, horizontal alignment, vertical alignment, grade, stopping sight distance, cross slope, superelevation, vertical clearance, and horizontal clearance. Research since 1985, culminating in publication of the most recent *Highway Capacity Manual* in 2010 and the AASHTO *Highway Safety Manual* in 2011, has developed much greater knowledge of the traffic operational and safety effects of the controlling criteria than was available when they were established in 1985.

The objective of the research herein was to describe the impact of the controlling roadway design criteria on safety and operations for various urban and rural roadway types. The research team considered how the current controlling criteria or possible modified criteria (adding, dropping, or combining particular design elements), through the design exception process, influence the provision of flexibility of the design process. The scope of the research was limited to roadway design criteria and, as specified in the NCHRP Project 17-53 statement, did not address intersection design, roadside design, or access control. The NCHRP Project 17-53 scope included new construction and reconstruction projects, but did not include resurfacing, restoration, and rehabilitation projects.

NCHRP Report 783 presents a comprehensive catalog of the known traffic operational and safety effects of the 13 controlling criteria for four roadway types: rural two-lane highways, rural multilane highways, urban and suburban arterials, and freeways. The discussion of each of the 13 controlling criteria addresses current design criteria, particularly those in the AASHTO *Green Book*; traffic operational effects; traffic safety effects; and mitigation strategies that may be considered when a design exception to the controlling criteria is approved.

The report reviews highway agency experience with the design exception process and the specific controlling criteria for which design exceptions are most frequently sought.

The report presents the results of data analyses conducted as part of this research that have provided new knowledge concerning the traffic operational and safety effects of the 13 controlling criteria. The results that were obtained include the following:

- Analysis of traffic crash data for bridges on two-lane rural highways as part of this research revealed no evidence of increased crash frequencies or severities for bridges with roadway widths (lane width plus shoulder width) narrower than the roadway width on the approach roadway.
- Analysis of crash data for rural two-lane highways as part of this research showed no increase of crash frequencies by crash severity level on crest vertical curves as a function of stopping sight distance for a range of stopping sight distance levels above and below the AASHTO stopping sight distance criteria. Crash frequencies increased on a crest vertical

curve only when a horizontal curve, intersection, or driveway hidden from the view of approaching drivers by the crest vertical curve was present.

- Through analysis of traffic speed data collected upstream of and within horizontal curves on rural multilane highways as part of this research a model was developed to predict the reduction in traffic speed on horizontal curves, in comparison to the traffic speed upstream of the curve, as a function of curve radius.
- Through analysis of crash data for rural multilane highways as part of this research models were developed to predict the crash frequency by crash severity level on horizontal curves as a function of curve length and radius.
- Analysis of traffic speed data collected upstream and downstream of lane-width transitions on urban and suburban arterials as part of this research showed no statistically significant effect of lane width on traffic speed.
- Through analysis of traffic speed data collected upstream of and within horizontal curves on urban and suburban arterials as part of this research a model was developed to predict the reduction in traffic speed on horizontal curves, in comparison to the traffic speed upstream of the curve, as a function of curve radius.

Sensitivity analyses were conducted to prioritize the 13 controlling criteria, by roadway type, based on their traffic operational and safety effects. Based on these priorities, specific recommendations on how state departments of transportation may choose to use the controlling criteria on non-Federal-aid projects were developed.

SECTION 1

Introduction

1.1 Background

The FHWA has, since 1985, designated 13 specific design elements as controlling criteria for roadway design (*1*, *2*). These 13 controlling criteria are the following:

- Design speed
- Lane width
- Shoulder width
- Bridge width
- Structural capacity
- Horizontal alignment
- Vertical alignment
- Grade
- Stopping sight distance
- Cross slope
- Superelevation
- Vertical clearance
- Horizontal clearance

Highway construction and reconstruction projects on the National Highway System (NHS) and resurfacing, restoration, and rehabilitation (RRR) projects on NHS freeways must meet the established design criteria for the design elements listed above, or a formal design exception must be prepared and approved. Highway agencies are also encouraged to follow these criteria for non-NHS projects (*2*). The established design criteria for geometric elements on construction and reconstruction projects are those in either the 2001 or 2004 editions of the AASHTO *Policy on Geometric Design of Highways and Streets* (*3, 4*), commonly known as the *Green Book*. The differences between the 2001 and 2004 editions of the *Green Book* are minor except in the area of superelevation. The 2011 edition of the *Green Book* (*5*) has been published by AASHTO, but has not yet been adopted by FHWA in a formal rulemaking for application to the NHS. Different design procedures and criteria apply to RRR projects (*6*), but the 13 controlling criteria are still key elements in the design of such projects.

The *Green Book* presents numerous geometric design elements and dimensional criteria. With the intent of focusing attention on the most important criteria and the understanding that a design exception evaluation for every design element would be impractical, FHWA identified the 13 design criteria listed above as having substantial importance to operational and safety performance (*1, 2, 7*). Although all exceptions from applicable design policies should be documented in some manner, FHWA has established the 13 controlling criteria as requiring formal design exceptions for the types of projects noted above.

But, are the 13 controlling criteria actually the most important design criteria for safety and efficiency? Moreover, is the current design exception process for these controlling criteria effective in achieving the most efficient investment of limited resources to improve safety and operations? Substantial new knowledge about safety and operations has been gained since the 13 controlling criteria were established in 1985. Furthermore, an industry-wide movement away from standards-based design to a more flexible process in which each design is tailored to fit into the context of community and environmental values has been under way since the 1990s (*8*). Software tools such as *SafetyAnalyst* (*9*) and the Interactive Highway Safety Design Model (IHSDM) (*10*) have been created to ensure explicit consideration of operations and safety and cost-effective investment of resources within a more flexible design process. Ultimately, this could lead to a performance-based design process in which the traffic operational and safety effects of design decisions are considered explicitly. The AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT* $\leq$ 400) (*11*) has also created a flexible approach to design appropriate for these very low-volume facilities.

Research is needed to address key questions concerning the 13 controlling criteria and their application in current practice including the following:

- What has been learned about the relationship between the controlling criteria and safety and operations?
- Based on current knowledge, are highway agencies using the appropriate controlling criteria? Should some criteria be dropped or combined? Should others be added?
- Are each of the controlling criteria applicable to all roadway types or should the controlling criteria vary by roadway type?

Answers to these questions are needed to ensure that the design process (including the process for design exceptions) focuses on the design elements that have the most substantial safety and operational impacts. This research provides technical information that could lead to more flexible approaches to design of highway projects, where appropriate. The timing of the research is very appropriate given the publication of the first edition of the AASHTO *Highway Safety Manual* (HSM) (*12*) and an updated edition of the *Highway Capacity Manual* (HCM) (*13*) in 2010. Much of the recent discussion on design flexibility has focused on safety issues because the traffic operational effects of design features are reasonably well established, but knowledge about their safety effects has been growing rapidly. For this reason, the following discussion emphasizes safety effects, but the research reported here definitely included full consideration of both operational and safety effects.

In previous years, when the safety effects of specific design criteria assumed to be important to safety were unknown, requiring compliance with those criteria, except in limited cases, was the most rational approach to providing appropriate levels of crash frequency and severity on a new or reconstructed facility. This approach is referred to in the HSM as achieving nominal safety. However, since the typical or average effects on crash frequency and severity of many geometric design criteria are now documented in the HSM (and others have been quantified in this research), a new paradigm based on quantitative analysis and/or cost-effectiveness analysis has become feasible. This new paradigm is referred in the HSM as substantive safety. Such a process could ensure that, where roadways are upgraded to meet established design criteria, the incremental funds spent in the name of safety actually produce a safety benefit commensurate with their cost. However, an assessment is clearly needed as to whether the state of safety knowledge has reached a point that can fully support such a process. Such an assessment has been performed in this research.

This research has addressed how far toward revised controlling criteria and a revised design exception process the transportation industry can move based on current knowledge and new knowledge obtained during the research.

1.2 Research Objectives and Scope

The objective of this research was to describe the impact of the controlling roadway design criteria on safety and operations for various urban and rural roadway types. The research has considered how the current controlling criteria or possible modified criteria (adding, dropping, or combining particular design elements), through the design exception process, influence the provision of flexibility of the design process. The scope of the research was limited to roadway design criteria and, based on the research scope established by NCHRP Project 17-53, has not addressed intersection design, roadside design, or access control. The project scope includes new construction and reconstruction projects, but not RRR projects.

This research has established priorities among the 13 controlling criteria and suggested directions in which they might evolve, but no specific policy recommendations for changing the 13 controlling criteria have been made. The research has considered how the 13 controlling criteria could be appropriately applied in non-federal projects.

1.3 Organization of *NCHRP Report 783*

The remainder of *NCHRP Report 783* is organized as follows. Section 2 reviews each of the 13 controlling criteria for design, their traffic operational and safety effects, and appropriate mitigation strategies when the controlling criteria cannot be met. Section 3 reviews the design exception practices of highway agencies in using the 13 controlling criteria. Section 4 presents the results of new research, providing expanded knowledge on the traffic operational and safety effects of the 13 controlling criteria. Section 5 suggests potential refinements to the criteria definitions. Section 6 presents priorities for the 13 controlling criteria. Section 7 discusses the interpretation of the research results. Section 8 presents the conclusions and recommendations of the research.

SECTION 2

Design Criteria, Traffic Operational and Safety Effects, and Mitigation Strategies for the 13 Controlling Criteria

This section presents the results of the review of design criteria, traffic operational and safety effects, and mitigation strategies for the 13 controlling criteria. This information concerning each of the controlling criteria is presented in Sections 2.1 through 2.13. The information presented in Section 2 is based primarily on published documentation. The primary sources consulted for each of the 13 controlling criteria are as follows:

- Design criteria are based primarily on the 2004 and 2011 editions of the AASHTO *Green Book* (*4, 5*), unless explicitly stated otherwise. Design criteria for freeways on the Interstate highway system are also presented in AASHTO's *A Policy on Design Standards—Interstate System* (*14*). Published FHWA guidance on the scope and interpretation of the 13 controlling criteria is also presented (*7*).
- Traffic operational effects are based primarily on the 2010 TRB *Highway Capacity Manual* (HCM) (*13*).
- Traffic safety effects are based primarily on the 2010 AASHTO *Highway Safety Manual* (HSM) (*12*).
- Mitigation strategies are based primarily on the FHWA guidance presented in *Mitigation Strategies for Design Exceptions* (*7*) and AASHTO's *A Guide for Achieving Flexibility in Highway Design* (*8*).

In addition, the discussion of the traffic operational and safety effects of the individual design criteria includes all relevant findings of the research conducted in this project, as reported in Section 4. Separate discussions of design criteria, traffic operational effects, and traffic safety effects are presented, where appropriate, for each of four roadway types: rural two-lane highways; rural multilane highways (nonfreeways); urban and suburban arterials (nonfreeways); and freeways. Throughout this report, the term "freeways" applies to both rural and urban freeways except where the terms "rural freeway" or "urban freeway" are used explicitly.

In cases where the primary sources present no information or only limited information on the traffic operational or safety effects of a particular issue, or where there may be concerns about the completeness of the primary sources, results of additional relevant research are presented. For safety effects, many such sources are cited in the FHWA Crash Modification Factors Clearinghouse (CMF Clearinghouse) website (*15*), which includes star ratings to assess the quality of the studies cited. The ratings range from one star (the weakest research) to five stars (the strongest research). Only CMFs included in the HSM or rated three stars or better in the FHWA CMF Clearinghouse website are cited in this section of the report.

Table 1 shows with circular bullets which of the 13 controlling criteria have documented traffic operational and safety effects for each of four roadway types (rural two-lane highways, rural multilane highways, urban and suburban arterials, and freeways). These documented traffic operational and safety effects are presented in Sections 2.1 through 2.13. The traffic operational effects of the 13 controlling criteria are summarized in Section 2.14. The traffic safety effects of the 13 controlling criteria are summarized in Section 2.15. The traffic operational and safety effects include findings from published literature and from research conducted as part of NCHRP Project 17-53, which are reported in Section 4.

2.1 Design Speed

AASHTO defines design speed as (*4*):

> Design speed is a selected speed used to determine the various geometric features of the roadway. The assumed design speed should be a logical one with respect to the topography, anticipated operating speed, the adjacent land use, and the functional classification of the highway.

Design speed is unique among the 13 controlling criteria since it has no direct effect on the design of the roadway, but

Table 1. Summary table for operational and safety effects of the controlling criteria.

	Traffic operational effects				Traffic safety effects			
Design criteria	Rural two-lane highways	Rural multilane highways	Urban and suburban arterials	Freeways	Rural two-lane highways	Rural multilane highways	Urban and suburban arterials	Freeways
Design speed	[a]	[a]	[a]	[a]	[a]	[a]	[a]	[a]
Lane width	•	•	[b]	•	•	•	•	•
Shoulder width	•	•		•	•	•		•
Bridge width					[b]			
Structural capacity	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Horizontal alignment	•	[b]	[b]	•	•	[b]		•
Vertical alignment (sag vertical curve length)								
Grade	•	•		•	•			
Stopping sight distance					[b]			
Cross slope								
Superelevation					•			
Vertical clearance	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Horizontal clearance (lateral offset)	[c]	[c]		[c]	[d]	[d]		[d]

[a] There are no direct operational or safety effects of design speed; however, design speed may influence operations and safety indirectly through the criteria for lane width, horizontal alignment, vertical alignment, and stopping sight distance.
[b] New relationships were developed in this research.
[c] No effect anticipated when full shoulders are present.
[d] There are no known direct effects of lateral offset on safety; however, the influence of lateral offset on safety is known indirectly through the influence of shoulder width.

only an indirect effect. Once a design speed for a project is selected, however, that design speed influences the values (or value ranges) of other controlling criteria, including horizontal alignment, vertical alignment, stopping sight distance, and lane width. Thus, design speed actually serves as a design control rather than a design criterion.

Design speeds should reflect the speeds that drivers expect to travel, which is determined by the physical limitations of the roadway and surrounding traffic rather than by the functional class of the roadway. Specific recommendations for design speeds are provided in several exhibits in the *Green Book* and are based on roadway classification, type of terrain, and volume. Ranges are as follows:

- For local rural roads, design speeds range from 20 mph for low-volume roads in mountainous terrain to 50 mph on high-volume roads in level terrain.
- For rural arterials, the recommended design speed ranges from 40 to 75 mph based on terrain, driver expectancy, and alignment.
- For urban arterials, the design speed should fall between 30 and 60 mph. In more developed areas, such as central business districts, the lower end of that range should be used, while in suburban or developing areas, the higher end of the range may be appropriate.
- For urban freeways, a design speed in the range of 50 to 70 mph should be used with higher speeds being more desirable when alignment and interchange spacing permit. Where lower design speeds are used, speed enforcement may also be needed. For rural freeways, a 70 mph design speed is recommended. Lower design speeds that are consistent with driver expectations are appropriate in mountainous terrain.

Table 2 summarizes the *Green Book* guidance on design speed.

Another aspect of design speed also serves as part of the controlling criteria. *Green Book* Exhibit 10-56 provides guide values for selection of ramp design speeds as a function of the highway design speed. According to the *Green Book*, ramp

Table 2. Ranges for design speed by roadway functional class (*4, 7*).

Roadway functional classification	Terrain	Design speed (mph)	
		Rural	Urban
Freeway	Level	70	50 min
	Rolling	70	50 min
	Mountainous	50 to 60	50 min
Arterial	Level	60 to 75	30 to 60
	Rolling	50 to 60	30 to 60
	Mountainous	40 to 50	30 to 60
Collector	Level	40 to 60	30+
	Rolling	30 to 50	30+
	Mountainous	20 to 40	30+
Local	Level	30 to 50	20 to 30
	Rolling	20 to 40	20 to 30
	Mountainous	20 to 30	20 to 30

design speeds should not be less than the low range presented in Exhibit 10-56, with other specific guidance offered for particular types of ramps (loops as well as direct and semi-direct connections). Some states have adopted design policies requiring the use of middle or higher range values for certain cases, such as system interchanges.

Designers are occasionally confronted with situations in which the appropriate ramp design speed shown in *Green Book* Exhibit 10-56 may not be achievable. Such cases are almost always associated with the inability to achieve minimum radius for the controlling curvature of the exit or entrance ramp. Not meeting the lower (50 percent) range shown in *Green Book* Exhibit 10-56 requires a design exception per FHWA policy. Where the design issue involves curvature, a design exception should be prepared for the non-standard horizontal curve rather than for the use of a lower design speed for the ramp (*7*).

There are no explicit traffic operational effects of design speed. Any traffic operational and safety effects of design speed result from the other design elements that are influenced by design speed. Experience shows that vehicle speeds cannot be reduced merely by reducing the posted speed limit or the design speed. Adjustment of a broad range of design and roadway environment factors is needed to influence vehicle speeds.

In accordance with *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) criteria (*16*), posted speed limits are typically set to approximate the 85th percentile speed of traffic, on the assumption that most drivers select speeds that are reasonable for conditions. Design speed, posted speed, and the roadway environment should all send a clear and consistent message to drivers about the appropriate speed for the roadway.

A 2009 paper by Hauer (*17*) documents the current state of knowledge about the relationship between highway travel speed and safety. Hauer indicates that vehicle travel speeds are affected by the roadway design, speed limits and enforcement, traffic controls, and many other factors. The travel speeds that are chosen by drivers affect the safety performance of the roadway. Although higher speeds will tend to increase the severity of crashes, Hauer states that there is little evidence to support the notion that faster travel speeds necessarily result in a greater likelihood of a crash. However, since higher speeds increase crash severity, higher speeds may increase the likelihood of a reported crash. Hauer also indicates that travel speeds on roadways tend to change over time, and, although this fact is well documented, little is known about why these changes occur.

As indicated by the design speed ranges shown in Table 2, the AASHTO *Green Book* provides substantial flexibility in the choice of an appropriate design speed. As written, AASHTO policy presents little need for design exceptions, because the choice of a design speed is left to the discretion of the designer. FHWA's report, *Mitigation Strategies for Design Exceptions* (*7*), states that the selected design speed should be high enough that an appropriate regulatory speed limit will be less than or equal to it, but this is not a formal FHWA policy.

Mitigation strategies for design speed would typically involve revision of both design elements and the roadway environment to encourage lower vehicle speeds. The FHWA Interactive Highway Safety Design Model (IHSDM) includes a design consistency tool that can be used to evaluate mitigation strategies for design speed (*10*). However, the IHSDM design consistency tool is currently applicable only to rural two-lane highways.

In actual practice, as documented in Section 3 of this report, design exceptions for design speed appear to be seldom requested or approved by highway agencies. Highway agencies generally seek design exceptions for specific design elements that do not meet the criteria for the selected design speed rather than seeking a blanket exception to reduce the design speed. The rare exception is where a highway agency may deem it appropriate to utilize a lower design speed for an entire corridor (or a substantial segment of a corridor) due to topographic or environmental constraints.

2.2 Lane Width

Lane width determines the area where a vehicle can maneuver laterally without encroaching into the path of another vehicle or onto the shoulder. Table 3 summarizes the lane width design criteria in the AASHTO *Green Book*. Separate criteria have also been established for auxiliary lanes, including turn lanes at intersections and center two-way left-turn lanes. Formal design exceptions for lane width are required by FHWA policy for all travel lanes including auxiliary lanes and ramps that do not meet *Green Book* criteria. Some highway agencies have lane width policies that provide less flexibility than the *Green Book* (e.g., specifying the use of 12-ft lanes in nearly all cases). This approach is not required by FHWA policy and may result in more design exceptions than FHWA policy would require. The AASHTO *Green Book* also includes criteria for lane widening on horizontal curves to

Table 3. Ranges for lane width by roadway functional class (*4, 5, 7*).

Functional class	Lane width (ft)	
	Rural	Urban
Freeway	12	12
Ramps (one-lane)	12 to 30[a]	12 to 30[a]
Arterial	11 to 12	10 to 12
Collector	10 to 12	10 to 12
Local	9 to 12	9 to 12

[a] For wider ramps, some of the specified width may be provided by shoulders.

Table 4. Minimum width of traveled way for rural arterials (*4, 5*).

Design speed (mph)	Minimum width of traveled way (ft)[a] for specified design volume			
	Under 400 (veh/day)	400 to 1,500 (veh/day)	1,500 to 2,000 (veh/day)	Over 2,000 (veh/day)
40	22	22	22	24
45	22	22	22	24
50	22	22	24	24
55	22	22	24	24
60	24	24	24	24
65	24	24	24	24
70	24	24	24	24
75	24	24	24	24

[a] On roadways to be reconstructed, an existing 22-ft traveled way may be retained where alignment is satisfactory and there is no crash pattern suggesting the need for widening.
SOURCE: Based on *Green Book* Table 7-3 (abridged).

accommodate truck offtracking; a formal design exception is not required where lane widening is not provided on a horizontal curve (*7*).

2.2.1 Rural Two-Lane Highways

Design Criteria

Chapter 7 (Arterials) of the *Green Book* provides the following guidance for the design of lane widths on rural arterials. The *Green Book* recommends the lane widths shown in Table 4 on rural arterials as a function of design speed and design volume (expressed as an average daily traffic volume, or ADT). Where lane widths narrower than those shown in Table 4 are used, a design exception is required by FHWA policy. In the case that is described in Note a of Table 4, a design exception is not required, although the justification for use of 11-ft lanes should be documented in the project files (*7*).

Traffic Operational Effects

Chapter 15 (Two-Lane Highways) of the HCM provides the estimates shown in Table 5 for reduction in free-flow speed on two-lane highways with lane widths less than 12 ft or shoulder widths less than 6 ft.

The values in Table 5 are used to estimate the actual free-flow speed of traffic on a two-lane highway from the free-flow speed for base conditions, as follows:

$$FFS = BFFS - f_{LS} - f_A \tag{1}$$

where

FFS = free-flow speed (mph)
BFFS = base free-flow speed (mph)
f_{LS} = adjustment for lane shoulder width (mph) from Table 5
f_A = adjustment for access-point density (mph) from HCM Exhibit 15-8

FFS may also be estimated directly from field data. *FFS* is used in estimating the average travel speed (ATS_d), one of the service measures used to determine level of service (LOS) for two-lane highways.

The shoulder-width effects included in f_{LS} are discussed in Section 2.3.1 of this report.

Traffic Safety Effects

Chapter 10 (Rural Two-Lane Highways) of the HSM provides CMFs for lane widths on rural two-lane highways. The

Table 5. HCM adjustment to free-flow speed for lane and shoulder width on two-lane highways (*13*).

Lane width (ft)	Reduction in free-flow speed (mph) Shoulder width (ft)			
	≤ 0 < 2	≤ 2 < 4	≤ 4 < 6	≥ 6
≥ 9 < 10	6.4	4.8	3.5	2.2
≥ 10 < 11	5.3	3.7	2.4	1.1
≥ 11 < 12	4.7	3.0	1.7	0.4
≥ 12	4.2	2.6	1.3	0.0

NOTE: The values in Table 5 are used as f_{LS} in Equation 1.
SOURCE: Based on HCM Exhibit 15-7.

Table 6. CMF for lane width on rural two-lane roadway segments (*12, 18, 19*).

Lane width	Average annual daily traffic (AADT) (veh/day)		
	< 400	400 to 2000	> 2000
9 ft or less	1.05	$1.05 + 2.81 \times 10^{-4}(\text{AADT} - 400)$	1.50
10 ft	1.02	$1.02 + 1.75 \times 10^{-4}(\text{AADT} - 400)$	1.30
11 ft	1.01	$1.01 + 2.5 \times 10^{-5}(\text{AADT} - 400)$	1.05
12 ft or more	1.00	1.00	1.00

NOTE: The collision types related to lane width to which these CMFs apply are single-vehicle run-off-the-road crashes and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes. Standard error of the CMF is unknown. To determine the CMF for changing lane width and/or AADT, divide the "new" condition CMF by the "existing" condition CMF.
SOURCE: Based on HSM Table 10-8.

CMF is calculated using the equations shown in Table 6 based on the lane width and the average annual daily traffic (AADT). A 12-ft lane is considered to be the base condition (CMF = 1.0). The lane-width CMF is illustrated graphically in Figure 1. The lane-width CMF illustrated in Table 6 and Figure 1 applies only to single-vehicle run-off-the-road crashes and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes. The following equation can be used to adjust the lane-width CMF in Table 6 and Figure 1 to CMFs applicable to total crashes:

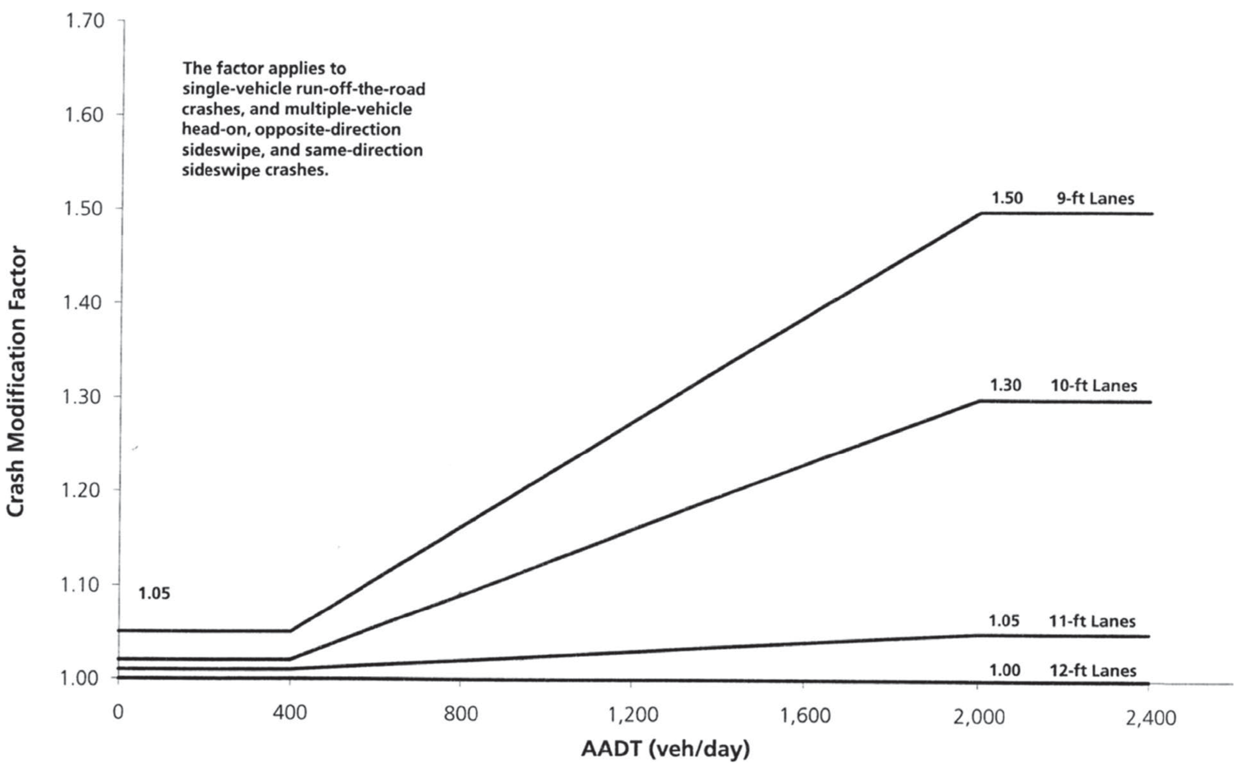
$$\text{CMF} = (\text{CMF}_{ra} - 1.0) \times p_{ra} + 1.0 \qquad (2)$$

where

CMF_{ra} = CMF for the effect of lane width on related crashes (i.e., single-vehicle run-off-the-road crashes and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes), such as the CMF for lane width shown in Table 6

p_{ra} = proportion of total crashes constituted by crash types related to lane and shoulder width

The proportion of related crashes, p_{ra}, (i.e., single-vehicle run-off-the-road crashes and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes) is estimated as 0.574 (i.e., 57.4 percent) based on



SOURCE: Based on HSM Figure 10-7.

Figure 1. CMF for lane width on rural two-lane roadway segments (12).

the default distribution of crash types presented in HSM Table 10-4. This default crash type distribution and, therefore, the value of p_{ra} may be updated from local data as part of the calibration process.

It should be noted that the CMFs for 11- and 12-ft lanes are not very different, which is consistent with both 11- and 12-ft lanes being shown as appropriate over broad ranges of conditions in Table 4.

2.2.2 Rural Multilane Highways

Design Criteria

Table 4 applies to rural multilane arterials as well as to rural two-lane arterials. Where lane widths narrower than those shown in Table 4 are used, a design exception is required by FHWA policy. In the case that is described in Note a of Table 4, a design exception is not required, although the justification for use of 11-ft lanes should be documented in the project files (*7*).

Traffic Operational Effects

Chapter 14 (Multilane Highways) of the HCM provides the estimated reduction in free-flow speed for rural and suburban multilane highways based on lane width as shown in Table 7.

Table 7. HCM adjustment to free-flow speed for average lane width on rural and suburban multilane highways (*13*).

Lane width (ft)	Reduction in free-flow speed (mph)
≥ 12	0.0
≥ 11	1.9
≥ 10	6.6

NOTE: The values in Table 7 are used as f_{LW} in Equation 3.
SOURCE: Based on HCM Exhibit 14-8.

The values in Table 7 are used to estimate the actual *FFS* of traffic on a multilane highway from the *BFFS*, as follows:

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A \tag{3}$$

where

f_{LW} = adjustment for lane width (mph) from Table 7
f_{LC} = adjustment for total lateral clearance (mph) from HCM Exhibit 14-9
f_M = adjustment for median type (mph) from HCM Exhibit 14-10
f_A = adjustment for access-point density (mph) from HCM Exhibit 14-11

FFS may also be estimated directly from field base. *FFS* is used to determine the mean speed of traffic(s) using the relationships show in HCM Exhibits 14-2 and 14-3 and the traffic density (*D*) using HCM Equation 14-5. Density is the service measure used to determine LOS for multilane highways.

Traffic Safety Effects

Chapter 11 (Rural Multilane Highways) of the HSM presents CMFs for lane widths on rural multilane roadways. The CMFs are calculated differently for undivided sections and divided sections, as shown in Tables 8 and 9. The calculation in either case is based on lane width and AADT. These CMFs are illustrated in Figures 2 and 3, respectively.

The CMFs shown in Tables 8 and 9 and Figures 2 and 3 are applicable to single-vehicle run-off-the-road crashes, multiple-vehicle head-on crashes, opposite-direction sideswipe crashes, and same-direction sideswipe crashes. Equation 2 can be used to convert these CMFs to CMFs for total crashes. The default value of p_{ra} in Equation 2 is 0.27 for rural multilane undivided highways and 0.50 for rural multilane divided highways.

Table 8. CMF for lane width on undivided rural multilane roadway segments (*12, 20*).

Lane width	Average annual daily traffic (AADT) (veh/day)		
	< 400	400 to 2000	> 2000
9 ft or less	1.04	$1.04 + 2.13 \times 10^{-4}(AADT - 400)$	1.38
10 ft	1.02	$1.02 + 1.31 \times 10^{-4}(AADT - 400)$	1.23
11 ft	1.01	$1.01 + 1.88 \times 10^{-5}(AADT - 400)$	1.04
12 ft or more	1.00	1.00	1.00

NOTE: The collision types related to lane width to which these CMFs apply are single-vehicle run-off-the-road crashes and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes. Standard error of the CMF is unknown. To determine the CMF for changing lane width and/or AADT, divide the "new" condition CMF by the "existing" condition CMF.
SOURCE: Based on HSM Table 11-11.

Table 9. CMF for lane width on divided rural multilane roadway segment (*12, 20*).

Lane width	Average annual daily traffic (AADT) (vehicles/day)		
	< 400	400 to 2000	> 2000
9 ft or less	1.03	$1.03 + 1.381 \times 10^{-4}(\text{AADT} - 400)$	1.25
10 ft	1.01	$1.01 + 8.75 \times 10^{-4}(\text{AADT} - 400)$	1.15
11 ft	1.01	$1.01 + 1.25 \times 10^{-5}(\text{AADT} - 400)$	1.03
12 ft or more	1.00	1.00	1.00

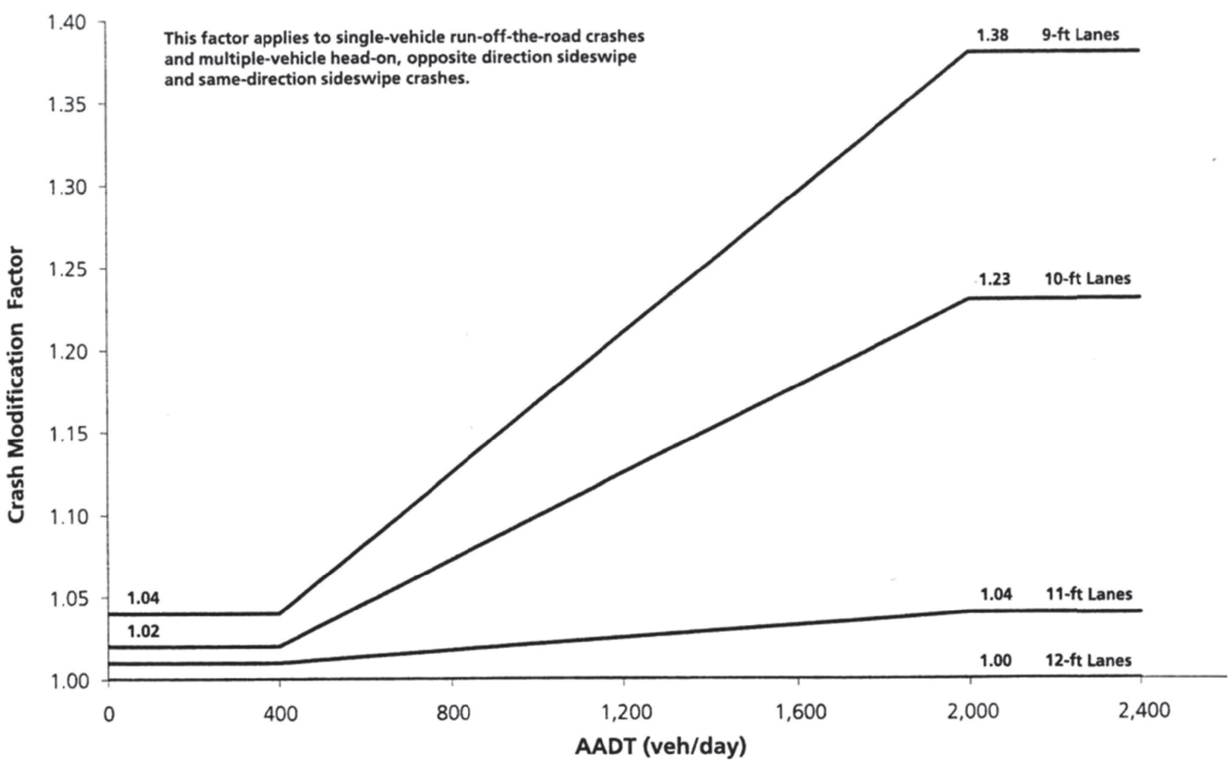
NOTE: The collision types related to lane width to which these CMFs apply are single-vehicle run-off-the-road crashes and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes. Standard error of the CMF is unknown. To determine the CMF for changing lane width and/or AADT, divide the "new" condition CMF by the "existing" condition CMF.
SOURCE: Based on HSM Table 11-16.

2.2.3 Urban and Suburban Arterials

Design Criteria

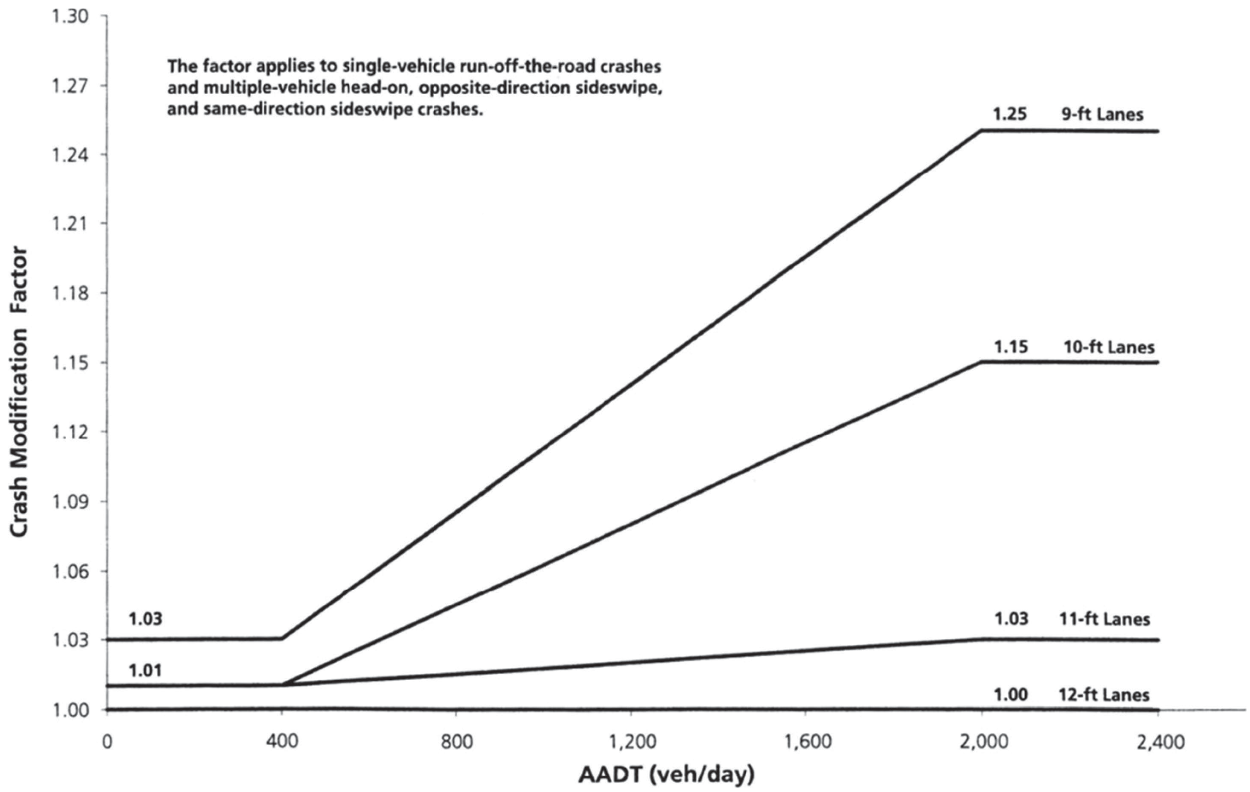
AASHTO policy provides substantial flexibility in the use of 10- to 12-ft lanes on urban arterials. In particular, Chapter 7 of the *Green Book* includes the following guidance:

- Lane widths of 12 ft are most desirable and should be used, where practical, on higher speed, free-flowing, principal arterials.
- Lane widths of 11 ft are used quite extensively for urban arterial street designs. Under interrupted-flow operating conditions at low speeds (45 mph or less), narrower lane widths are normally adequate and have some advantages. For example, narrower lane widths allow more lanes to be provided in some areas with restricted right-of-way and allow shorter pedestrian crossing times because of reduced crossing distances. Arterials with 11-ft lane widths are also more economical to construct. An 11-ft lane width is adequate for through lanes, continuous two-way left-turn lanes, and lanes adjacent to a painted median.



SOURCE: Based on HSM Figure 11-8.

Figure 2. CMF for lane width on undivided segments on rural multilane highways (*12, 20*).



SOURCE: Based on HSM Figure 11-10.

Figure 3. CMF for lane width on divided roadway segments on rural multilane highways (12, 20).

- Lane widths of 10 ft may be used in highly restricted areas having little or no truck traffic. Left-turn and combination lanes used for parking during off-peak hours and for traffic during peak hours may be 10 ft in width.

The *Green Book* also makes reference to the AASHTO bicycle guide (*21*) because use of narrow lane widths may be critical at many locations in reconstruction of existing arterials to provide space for bicycle facilities.

Traffic Operational Effects

Chapter 17 (Urban Street Segments) of the HCM includes a procedure to determine the effect of the features of an urban street segment on free-flow speed. However, lane width is not one of the factors that influences free-flow speed. This suggests that lane width either has no effect on the free-flow speed of an urban street segment or an effect that is very small in comparison to the factors that are in the procedure (see HCM Exhibit 17-5). This zero or negligible effect for lane width in the current HCM contrasts with the HCM 2000 (*22*), which speculated that lane width influenced free-flow speed for urban streets, but did not quantify that effect.

The HCM adjustment for lane width presented in Table 6 is applicable to suburban multilane highways, but not to urban streets. Recent research by Potts et al. (*23, 24*) investigated the effect of lane width on midblock vehicle speeds on urban and suburban arterials based on spot speed measurements at pairs of sites upstream and downstream of lane width transitions. The research of Potts et al. (*23, 24*) found that mean speeds at sites with wider lanes (ranging from 11.9 to 13.3 ft) were approximately 4 mph higher than mean speeds at sites with narrower lanes (ranging from 9.4 to 10.3 ft in width). This finding suggested that lane width has an effect on traffic operations. However, the sample size in the study was relatively small (five pairs of wide- and narrow-lane sites) and was not sufficient to develop a formal relationship between lane width and traffic speed.

A similar evaluation in the NCHRP Project 17-53 research considered a total of 23 additional sites on urban and suburban arterials in the Eastern, Midwest, and Western regions of the United States (see Section 4.1). This evaluation found that lane width had no effect on traffic speeds on urban and suburban arterials. Based on this finding, it appears that the HCM is correct in assuming that lane width has no effect on traffic speeds on urban and suburban arterials.

Chapter 18 (signalized intersections) of the HCM includes an adjustment factor for the effect of lane width on saturation flow rate at signalized intersections (see HCM Exhibit 18-3). However, given that this adjustment is applicable only to signalized intersection approaches and not to midblock sections of arterials, it is not presented in this report, since intersection design criteria are outside the scope of the research.

Traffic Safety Effects

Chapter 12 (Urban and Suburban Arterials) of the HSM does not include a CMF for lane width on urban and suburban arterials. Recent research by Potts et al. (*23, 24*) under NCHRP Project 03-72 found no difference in safety performance for urban and suburban arterials in lane widths ranging from 10 to 12 ft, with only limited exceptions that could represent random effects. Lanes narrower than 12 ft may be a design concern on streets with substantial volumes of bicycles, trucks, and buses.

2.2.4 Freeways

Design Criteria

According to the *Green Book*, freeway lanes should be 12 ft wide. Lane widths of 12 ft are also called for in the AASHTO design standards for the Interstate highway system.

Traffic Operational Effects

Chapter 11 (Basic Freeway Segments) of the HCM presents the estimated reduction in free-flow speed for freeways with lane widths less than 12 ft as shown in Table 10.

The values in Table 10 are used to estimate the actual free-flow speed of traffic on a freeway from the estimated free-flow speed for base conditions, 75.4 mph. This adjustment is made as follows:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\ TRD^{0.84} \quad (4)$$

where

f_{LW} = adjustment for lane width (mph) from Table 10
f_{LC} = adjustment for right-side lateral clearance (mph) from HCM Exhibit 11-9
TRD = total ramp density (ramps/mi)

Table 10. HCM adjustment to free-flow speed for lane width on freeways (*13*).

Lane width (ft)	Reduction in free-flow speed (mph)
12	0.0
11	1.9
10	6.6

NOTE: The values in this table are used as f_{LW} in Equation 4.
SOURCE: Based on HCM Exhibit 11-8.

FFS may also be estimated directly from field data. *FFS* is used to determine the mean speed of traffic (*S*) using the relationships shown in HCM Exhibits 11-2 and 11-3 and the traffic density (*D*) using HCM Equation 11-4. Density is the service measure used to determine LOS for freeways.

Traffic Safety Effects

Results from NCHRP Project 17-45, which developed a proposed HSM safety prediction methodology for freeways, include the following CMF for lane width on freeways where W_e = average lane width for all through lanes (ft) (*25*):

$$CMF = \exp\left(-0.0376\left(W_e - 12\right)\right), \text{ if } W_e < 13 \text{ ft} \quad (5)$$

$$CMF = 0.963, \text{ if } W_e \geq 13 \text{ ft} \quad (6)$$

The base condition for this CMF is a 12-ft lane width, (CMF = 1.0). W_e represents the average lane width for all through lanes on a freeway segment in both directions of travel excluding managed lanes and auxiliary lanes associated with a weaving section. The CMF is applicable to lane widths in the range of 10 to 14 ft. The CMF is intended for application to both multiple- and single-vehicle crashes on rural freeways with four to eight lanes and urban freeways with four to ten lanes.

2.2.5 Mitigation Strategies

Mitigation strategies for lane width are most important on higher speed roadways (speeds above 45 mph). On roadways with speeds of 45 mph or less, there are often good reasons for using narrow lanes as a flexibility measure to obtain other benefits: shorter pedestrian crossing distances, inclusion of turn lanes, medians, bicycle lanes, etc. These other benefits for road users, in and of themselves, constitute mitigation for the use of narrower lanes. The best use of available cross-section width should be determined on a case-by-case basis.

The mitigation strategies where narrower lanes are used on higher speed facilities include (*7*):

- Provide warning of lane width reduction
- Improve ability of drivers to stay within their travel lane through use of enhanced pavement markings, delineations, lighting, shoulder rumble strips, painted edge line rumble strips, and/or centerline rumble strips
- Improve ability to recover if driver leaves the lane (paved or partially paved shoulders, safety edge treatment)
- Reduce crash severity if the driver leaves the roadway (clear recovery area, traversable slopes, breakaway safety hardware, and barriers where appropriate)
- Provide pull-off areas where shoulder width is limited

Table 11. Ranges for minimum shoulder width by roadway functional class (*4, 5, 7*).

Functional class	Shoulder width (ft)	
	Rural	Urban
Freeway	4 to 12	4 to 12
Ramps (one-lane)	1 to 10	1 to 10
Arterial	2 to 8	2 to 8
Collector	2 to 8	2 to 8
Local	2 to 8	—

NOTE: Ranges shown include both right and left shoulder widths for ramps and divided highways.

Table 12. Minimum width of usable shoulder for rural arterials (*4, 5*).

Minimum width of usable shoulder (ft) for specified design volume			
Under 400 veh/day	400 to 1,500 veh/day	1,500 to 2,000 veh/day	Over 2,000 veh/day
4	6	6	8

NOTE: Usable shoulders on arterials should be paved; however, where volumes are low or a narrow section is needed to reduce construction impacts, the paved shoulder may be reduced to 2 ft.
SOURCE: Based on *Green Book* Table 7-3 (abridged).

2.3 Shoulder Width

Shoulder width affects both capacity and safety on roadways. A wide shoulder increases capacity by reducing lateral friction between traffic and roadside objects and thereby increasing driver comfort. Shoulders can reduce the likelihood of crashes in several ways, including providing a location for emergency stops and broken-down vehicles outside the traveled way, providing a space for drivers of errant vehicles to make steering corrections before leaving the roadway, and providing space for evasive maneuvers. Shoulders also provide space for enforcement activities, maintenance activities, and bicycle accommodations. Table 11 summarizes the range of minimum shoulder widths for travel lanes and ramps presented in the *Green Book*.

2.3.1 Rural Two-Lane Highways

Design Criteria

The shoulder widths presented in Table 12 are recommended in the *Green Book*, as a function of AADT. The usable shoulder-width values in Table 12 require a design exception if they are not met. Usable shoulder width is measured from the edge of the traveled way to the point of intersection of the shoulder slope and mild slope (for example, 1V:4H or flatter) or to the beginning of rounding to slopes steeper than 1V:4H (*7*).

Traffic Operational Effects

Chapter 15 (Two-Lane Highways) of the HCM presents the estimated reductions in free-flow speed for two-lane highways with lane widths less than 12 ft or shoulder widths less than 6 ft, as shown in Table 5. The values shown in Table 5 are used as f_{LS} in Equation 1 to estimate the free-flow speed on two-lane highways (see Section 2.2.1).

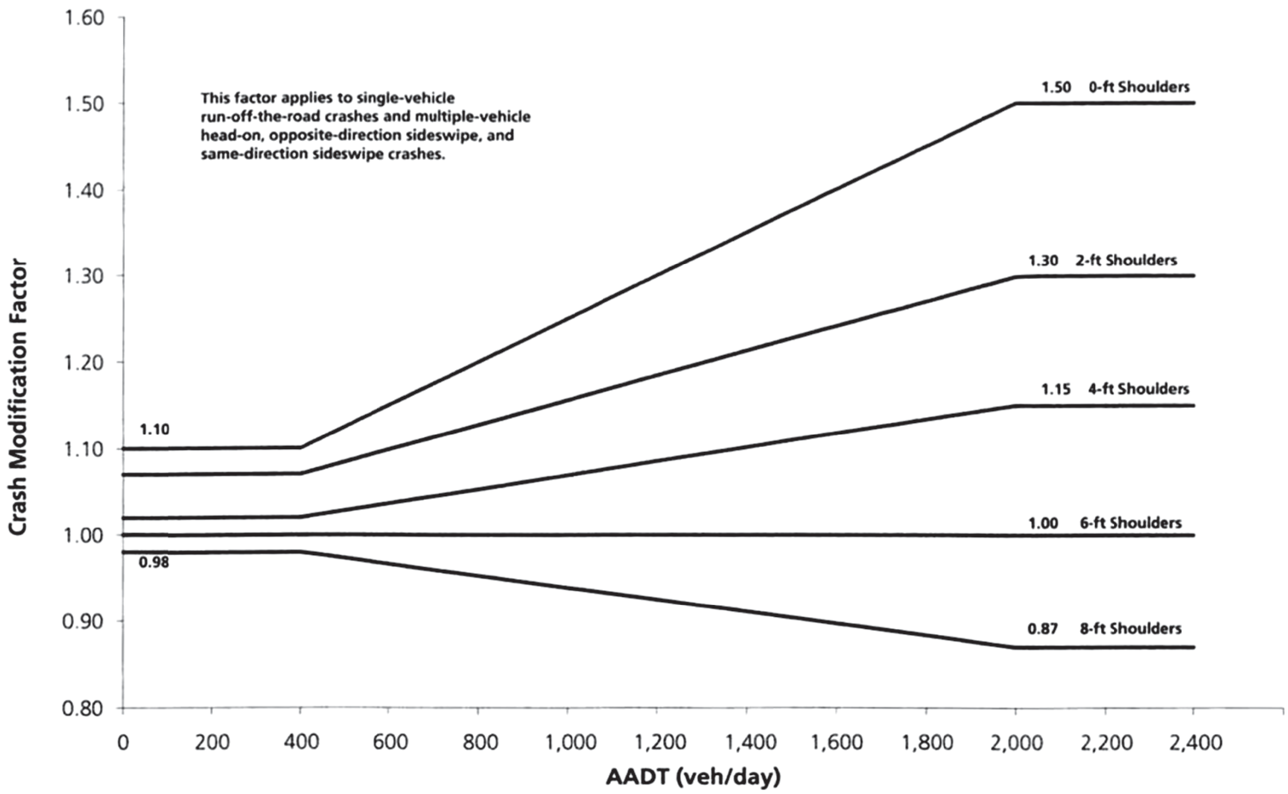
Traffic Safety Effects

Chapter 10 (Rural Two-Lane Highways) of the HSM provides CMFs for paved shoulders on rural two-lane roadways for specific crash types related to lane encroachment. The value of CMF_{wra} for shoulder width is calculated using the equations shown in Table 13 based on the shoulder width and the traffic volume (AADT). A 6-ft shoulder is considered to be the base condition (CMF = 1.0). Wider shoulders have CMFs less than 1.0, and narrower shoulders have CMFs

Table 13. CMFs for shoulder width on rural two-lane roadway segments (CMF_{wra}) (*12, 18*).

Shoulder width	Average annual daily traffic (AADT) (veh/day)		
	< 400	400 to 2000	> 2000
0 ft	1.10	$1.10 + 2.5 \times 10^{-4}(AADT - 400)$	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

NOTE: The collision types related to lane width to which these CMFs apply include single-vehicle run-off-the-road crashes and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes. Standard error of the CMF is unknown. To determine the CMF for changing paved shoulder width and/or AADT, divide the "new" condition CMF by the "existing" condition CMF.
SOURCE: Based on HSM Table 10-9. The values from Table 13 are used as CMF_{wra} in Equation 7.



SOURCE: Based on HSM Figure 10-8.

Figure 4. CMF for shoulder width on roadway segments for two-lane highway (12, 18).

greater than 1.0. The shoulder-width CMF for rural two-lane highways is illustrated in Figure 4.

The base condition for shoulder type is paved (CMF = 1.0). Table 14 presents values for CMF_{tra}, which adjusts for the safety effects of gravel, turf, and composite shoulders as a function of shoulder width.

A combined CMF for shoulder width and type is computed as

$$CMF = (CMF_{wra} \times CMF_{tra} - 1.0) \times p_{ra} + 1.0 \quad (7)$$

where

CMF_{wra} = crash modification factor for shoulder width from the equations in Table 13

CMF_{tra} = crash modification factor for shoulder type from Table 14

If the shoulder types and/or widths for the two directions of a roadway segment differ, the CMFs are determined separately for the shoulder type and width in each direction of travel and the resulting CMFs are then averaged.

The CMFs for shoulder width and type shown above apply only to the collision types that are most likely to be affected by shoulder width and type: single-vehicle run-off-the-road crashes, multiple-vehicle head-on crashes, opposite-direction sideswipe crashes, and same-direction sideswipe crashes. The CMFs expressed on this basis are, therefore, adjusted to total crashes using Equation 7. The HSM default value for p_{ra} for two-lane highways in Equation 7 is 0.574.

Table 14. CMFs for shoulder types and shoulder width on roadway segments (CMF_{tra}) (*12, 18*).

	Shoulder width (ft)						
Shoulder type	0	1	2	3	4	6	8
Paved	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Gravel	1.00	1.00	1.01	1.01	1.01	1.02	1.02
Composite	1.00	1.01	1.02	1.02	1.03	1.04	1.06
Turf	1.00	1.01	1.03	1.04	1.05	1.08	1.11

NOTE: The values for composite shoulders in this table represent a shoulder for which 50 percent of the shoulder width is paved and 50 percent of the shoulder width is turf.
SOURCE: Based on HSM Table 10-10.

Table 15. Recommended shoulder widths for rural multilane divided arterials (*4, 5*).

Number of lanes in single direction	Recommended right (outside) shoulder width (ft)	Recommended left (inside) shoulder width (ft)
2 lanes	8	4
3 or more lanes	8	8

SOURCE: Adapted from *Green Book* Chapter 7.

2.3.2 Rural Multilane Highways

Design Criteria

The *Green Book* states that the design criteria for shoulder width on rural two-lane highways presented in Table 12 are generally applicable to rural undivided multilane arterials, as well. For rural divided multilane arterials, the shoulder widths presented in Table 15 are recommended.

Traffic Operational Effects

Chapter 14 (Multilane Highways) of the HSM estimates free-flow speed based on the total lateral clearance, defined as the sum of the lateral clearance on the left side of the roadway (maximum of 6 ft) and the right side of the roadway (maximum of 6 ft). Lateral clearance is defined as the distance from the edge of the travel lane to the nearest obstruction. Thus, roadways with wide shoulders inherently have larger lateral clearance values than roadways with narrow shoulders. Total lateral clearance for multilane highways is generally interpreted as equivalent to the sum of the left (inside) and right (outside) shoulder widths, since some objects (e.g., guardrail) may be located immediately outside the shoulders. The free-flow speed reduction values are shown in Table 16; these values are used in Equation 3 (see Section 2.2.2).

In addition, Chapter 14 of the HCM predicts a free-flow speed reduction of 1.6 mph for an undivided roadway relative to a divided highway or a highway with a two-way left-turn lane. This value is used in f_M in Equation 3, where applicable.

Traffic Safety Effects

Chapter 11 (Rural Multilane Highways) of the HSM presents CMFs for paved shoulders on rural multilane roadways. CMFs are calculated differently for undivided and divided roadways. CMFs for undivided sections of multilane highways are calculated using the same equations as two-lane highways, as shown in Table 13 (see also Figure 4). The base condition for this CMF is a 6-ft shoulder (CMF = 1.0). As for rural two-lane highways, this CMF is adjusted to total crashes using Equation 7. The HSM default value for p_{ra} for rural multilane undivided highways used in Equation 7 is 0.27.

CMFs for divided sections of multilane highways are presented in Table 17. The base condition (CMF = 1.0) is an 8-ft shoulder. This CMF applies to total crashes and is not adjusted using a p_{ra} value.

2.3.3 Urban and Suburban Arterials

Design Criteria

Chapter 7 of the *Green Book* states that shoulders are desirable on any highway, but high right-of-way costs in urban areas may often preclude their use. When sufficient right-of-way is available, the design criteria previously presented for rural highways apply. Shoulders are not required by the *Green Book* for urban areas, and many such roadways are built using curbed cross sections, rather than shoulders.

Traffic Operational Effects

Chapter 17 (Urban Street Segments) of the HCM includes a procedure to determine the effect of the features of an urban street segment on free-flow speed. However, shoulder width

Table 16. Adjustment to free-flow speed for lateral clearance on rural and suburban multilane highways (*13*).

Four-lane highways		Six-lane highways	
Total lateral clearance (ft)	Reduction in free-flow speed (mph)	Total lateral clearance (ft)	Reduction in free-flow speed (mph)
12	0.0	12	0.0
10	0.4	10	0.4
8	0.9	8	0.9
6	1.3	6	1.3
4	1.8	4	1.7
2	3.6	2	2.8
0	5.4	0	3.9

NOTE: The values for reduction in free-flow speed presented in this table are used as f_{LW} in Equation 3.
SOURCE: Based on HCM Exhibit 14-9.

Table 17. CMFs for paved right (outside) shoulder width on multilane divided highway segments (*12, 26*).

Average paved shoulder width				
0 ft	2 ft	4 ft	6 ft	8 ft or more
1.18	1.13	1.09	1.04	1.00

SOURCE: Based on HSM Table 11-17.

is not one of the factors that influences free-flow speed. This suggests that shoulder width either has no effect on the free-flow speed on an urban street segment or an effect that is very small in comparison to the factors that are in the procedure (see HCM Exhibit 17-5). This contrasts with the HCM 2000 (*22*) which speculated that shoulder width influenced free-flow speed for urban streets, but did not quantify that effect.

Traffic Safety Effects

The HSM does not provide a CMF for shoulder width on urban and suburban arterials.

2.3.4 Freeways

Design Criteria

Chapter 8 of the *Green Book* recommends the shoulder widths for freeways shown in Table 18.

The AASHTO policy on design standards for the Interstate highway system (*14*) requires a right (outside) shoulder with 10 ft of paved width. Where truck traffic exceeds a directional design hour volume (DDHV) of 250, a paved shoulder width of 12 ft should be considered. On a four-lane section, the paved width of the left (inside) shoulder is required to be at least 4 ft. On sections with six or more lanes, a left (inside) shoulder with a 10-ft width should be provided. Where truck traffic exceeds 250 DDHV, a paved width of 12 ft should be considered for the left (inside) shoulder. On four- to six-lane freeways in mountainous terrain, 8-ft paved right (outside) shoulders and 4-ft paved left (inside) shoulders may be used. On sections with eight or more lanes in mountainous terrain, a minimum paved shoulder width of 8 ft should be used on both sides of the roadway.

Traffic Operational Effects

Chapter 11 (Basic Freeway Segments) of the HCM estimates free-flow speed based on the lateral clearance on the right side of the roadway. Lateral clearance is measured from the edge of the travel lane to the edge of the paved shoulder. If the right-side lateral clearance is greater than or equal to 6 ft, no reduction in free-flow speed is made. The amount of free-flow speed reduction increases as the right-side lateral clearance decreases. Left-side lateral clearance is assumed to be greater than or equal to 2 ft for all cases. The free-flow speed reductions for right shoulder lateral clearance (generally interpreted as equivalent to right [outside] shoulder width) are shown in Table 19. The values in Table 19 are used as f_{LC} in Equation 4 to determine free-flow speed (see Section 2.2.4).

Traffic Safety Effects

Results from NCHRP Project 17-45, which developed a proposed HSM safety prediction methodology for freeways, include CMFs for both right (outside) shoulder width and left (inside) shoulder width on freeways (*25*). The CMF for right (outside) shoulder width (where W_s = average right [outside] shoulder width for both directions of travel combined [ft]) is the following:

- For fatal-and-injury single-vehicle crashes on tangent sections,

$$CMF = \exp\left(-0.0647\left(W_s - 10\right)\right) \tag{8}$$

- For fatal-and-injury single-vehicle crashes on horizontal curves,

$$CMF = \exp\left(-0.097\left(W_s - 10\right)\right) \tag{9}$$

- For property-damage-only single-vehicle crashes on tangent sections,

$$CMF = 1.0 \tag{10}$$

Table 18. Recommended shoulder widths for freeways (*4, 5*).

Side of roadway	DDHV for truck traffic (veh/h)	Total number of freeway lanes	Recommended shoulder width (ft)
Right shoulder	≤250	All	10
Right shoulder	>250	All	12
Left shoulder	≤250	Less than 6	4
Left shoulder	≤250	6 or more	10
Left shoulder	>250	All	12

SOURCE: Adapted from Chapter 8 of the AASHTO *Green Book*.

Table 19. Adjustments for free-flow speed right-side lateral clearance on freeways (*13*).

Right Shoulder Lateral Clearance (ft)	Reduction in free-flow speed (mph) Number of lanes in one direction			
	2 lanes	3 lanes	4 lanes	≥5 lanes
≥ 6	0.0	0.0	0.0	0.0
5	0.6	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

NOTE: The values in this table are used as f_{LC} in Equation 4.
SOURCE: Based on HCM Exhibit 11-9.

- For property-damage-only single-vehicle crashes on horizontal curves,

$$CMF = \exp\left(-0.0840\left(W_s - 10\right)\right) \qquad (11)$$

The base condition for this CMF is a 10-ft shoulder width (CMF = 1.0). The CMF is applicable to shoulders in the range of 4 to 14 ft. This CMF applies only to single-vehicle crashes; right (outside) shoulder width does not appear to have any effect on multiple-vehicle crashes.

The CMF for left (inside) shoulder width (where W_{is} = average inside shoulder width for both directions of travel combined [ft]) is the following:

- For fatal-and-injury crashes,

$$CMF = \exp\left(-0.0172\left(W_{is} - 6\right)\right) \qquad (12)$$

- For property-damage-only crashes,

$$CMF = \exp\left(-0.0153\left(W_{is} - 6\right)\right) \qquad (13)$$

The base condition for this CMF is a 6-ft shoulder width. The CMF is applicable to left (inside) shoulders in the range of 2 to 12 ft. The CMF applies to both multiple- and single-vehicle crashes.

2.3.5 Mitigation Strategies

All the mitigation strategies for lane width presented in Section 2.2.5 also apply to shoulder width, with the obvious exception that adding paved or partially paved shoulders does not apply because the lack of a full shoulder is the condition to be mitigated.

2.4 Bridge Width

Bridge width is the total width of all lanes and shoulders on a bridge, measured between the points on the bridge rail, curb, or other vertical elements that project farthest onto the roadway. A bridge width that meets design criteria maintains the minimum acceptable lane and shoulder width for the particular design condition as defined by area, functional class, design speed, and traffic volume. FHWA policy requires a design exception when a bridge is proposed to be constructed or retained with narrower lanes, shoulders, or both (*7*). Chapter 7 (Arterials) of the *Green Book* includes specific guidance on bridge widths that may remain in place on reconstruction projects (see Sections 2.4.1 and 2.4.2).

Potential concerns associated with narrow bridges are twofold. Narrow bridges that are relatively short represent a discontinuity that may affect driver behavior. The narrowed cross section can make some drivers uncomfortable and cause them to dramatically reduce speed, increasing the risk of rear-end crashes and degrading operations on high-speed, high-volume facilities. The bridge rail may be close enough to the travel lanes to cause drivers to move toward the centerline or into adjacent lanes. In narrow bridges, the bridge railing itself is closer to the edge of pavement and thus represents a roadside hazard. Even when properly designed and delineated, there is an increased risk of a roadside collision with the bridge railing or bridge end being closer to the edge of traveled way.

A second set of concerns is evident for narrow bridges that are longer (say, greater than 500 ft in length). The safety and operational concerns at narrow bridges are similar to those on roads with narrow shoulders. There may be inadequate space for storage of disabled vehicles, enforcement activities, emergency response, and maintenance work. The lack of shoulder width on the bridge may make it impossible to avoid a crash or object on the roadway ahead. In addition, options are limited for non-motorized users such as bicyclists, forcing them onto the traveled lanes or close to the bridge rail.

Narrow bridges on horizontal curves can have limited horizontal stopping sight distance past the bridge rail. Operations can be degraded, particularly on long bridges on high-speed roadways, because of speed reductions as drivers enter the narrowed cross section as well as decreased driver comfort on the bridge.

2.4.1 Rural Two-Lane Highways

Design Criteria

The minimum lane widths and shoulder widths shown in Tables 4 and 12, based on *Green Book* Exhibit 7-3, serve as the recommended minimum bridge widths for rural two-lane arterials. The combined minimum widths (lane width plus shoulder width) range from 30 ft (for a design speed of 40 mph and ADT less than 400 veh/day) to 40 ft (for a design speed of 75 mph and an ADT above 2,000 veh/day). On long bridges, defined as bridges with lengths of more than 200 ft, the offset to the parapet, rail, or barrier should be at least 4 ft from the edge of the traveled way or both sides of the roadway. Chapter 7 of the *Green Book* indicates that bridges with widths equal to the width of the traveled way plus 2 ft of clearance on each side may remain in place in reconstruction projects on arterials.

Traffic Operational Effects

Chapter 15 (Two-Lane Highways) of the HCM provides estimates for free-flow speeds on rural two-lane highways based on lane width and shoulder width. Bridges wide enough to accommodate 12-ft lanes and 6-ft shoulders will not reduce the free-flow speed below the base free-flow speed of the roadway; bridges of lesser widths will result in reduced free-flow speeds. Sections 2.2.1 and 2.3.1 of this report present more detailed information. The actual reduction in free-flow speed may be even greater than suggested in the HCM, particularly for long bridges, because the lateral obstruction is generally presented for the entire length of the bridge.

Traffic Safety Effects

The effects of lane and shoulder widths on safety for rural two-lane highways have been documented in Sections 2.2.1 and 2.3.1 of this report. While the design criteria for bridge width are based on the lane and shoulder width design criteria, it seems likely that safety might be more sensitive to bridge width than the lane and shoulder width, because every bridge has lateral obstructions (i.e., bridge rail or curb) at the outside edge of the shoulder.

Turner (*27*) conducted research to predict crash rates as a function of bridge width, but the results appear potentially biased because only bridges that had experienced at least one crash were studied. A recent study by Bigelow et al. (*28*) in the FHWA CMF Clearinghouse provides a CMF for changing bridge width (bridge minus roadway width) from X to Y. The CMF is

$$\mathrm{CMF} = 100 * (1 - \exp(-0.116(\mathrm{Y} - \mathrm{X}))) \tag{14}$$

where

X = bridge width before improvement (ft)
Y = bridge width after improvement (ft)

This is applicable to all crash types and severities. However, this CMF applies only to low-volume roads with AADT less than or equal to 400 veh/day and speed limits greater than or equal to 45 mph.

Research conducted under NCHRP Project 17-53 (see Section 4.3) included analysis of the crash history of 624 bridges on rural two-lane highways in California and 337 bridges on rural two-lane highways in Washington and found no statistically significant effect of differences between roadway width on the approach roadway and on the bridge on crash frequency.

2.4.2 Rural Multilane Highways

Design Criteria

Design criteria for bridge widths on rural multilane highways are based on the lane and shoulder-width design criteria presented in Sections 2.2.2 and 2.3.2. Those design criteria in Chapter 7 of the *Green Book* recommend 12-ft lane widths for rural divided multilane arterials. For long bridges over 200 ft in length, the *Green Book* states that 4-ft right and left shoulders are acceptable. For shorter bridges, the normal recommendation of an 8-ft right shoulder applies. Chapter 7 of the *Green Book* indicates that bridges with widths equal to the width of the traveled way plus 2 ft of clearance on each side may remain in place in reconstruction projects.

Traffic Operational Effects

Chapter 14 (Multilane Highways) of the HCM provides estimates for free-flow speeds on multilane highways based on lane width and lateral clearance. Bridges wide enough to accommodate 12-ft lanes and at least 6 ft of lateral clearance on both the left and right sides of the road will not reduce the free-flow speed below the base level; bridges of lesser widths will result in reduced free-flow speed levels. Sections 2.2.2 and 2.3.2 of this report present more detailed information. The actual reduction in free-flow speed may be even greater than suggested in the HCM, particularly for long bridges, because the lateral obstruction is generally present for the entire length of the bridge.

Traffic Safety Effects

See discussion in Section 2.4.1 of this report.

2.4.3 Urban and Suburban Arterials

Design Criteria

Chapter 7 of the *Green Book* states that the minimum clear width for new bridges should be the same as the minimum curb-to-curb distance of the roadway for general conditions. For bridges that exceed 200 ft in length, the offsets to parapets, rails, or barriers may be reduced to 4 ft where shoulders or parking lanes are provided on the arterial.

Traffic Operational Effects

According to the "Limitations of the Methodology" discussion in Chapter 17 (Urban Streets) of the HCM, the HCM urban streets methodology does not directly account for capacity constraints such as a narrow bridge between intersections.

Traffic Safety Effects

See discussion in Section 2.4.1 of this report.

2.4.4 Freeways

Design Criteria

Minimum widths for lanes and shoulders on freeways are presented in Chapter 8 of the *Green Book* and have been summarized in Sections 2.2.4 and 2.3.4 of this report. A total bridge width for a freeway would depend on these minimum width values. As a general example, the following widths are recommended for a two-way viaduct freeway with ramps:

- Median width: 10 to 22 ft
- Lane width: 12 ft
- Right shoulder width: 10 ft
- Left shoulder width: 4 to 10 ft
- Parapet width: 2 ft
- Clearance between structure and building line: 15 ft

Traffic Operational Effects

Chapter 11 (Basic Freeway Segments) of the HCM provides estimates for free-flow speeds on freeways based on lane width and lateral clearance. Bridges wide enough to accommodate 12-ft lanes, at least 6 ft of right-side lateral clearance, and at least 2 ft of left-side lateral clearance will not reduce the free-flow speed below the base value; bridges of lesser widths will result in reduced free-flow speed values. Sections 2.2.4 and 2.3.4 of this report present more detailed information. The actual reduction in free-flow speed may be even greater than suggested in the HCM, particularly for long bridges, because the lateral obstruction is generally presented for the entire length of the bridge.

Traffic Safety Effects

See discussion in Section 2.4.1 of this report.

2.4.5 Mitigation Strategies

Strategies for mitigating narrow bridge widths are directed primarily at improving a driver's ability to see or to anticipate the narrowed cross section of the bridge, the bridge rail, and the lane lines. Typical mitigation strategies include the following (*7*):

- Advance signing
- Improved delineation (pavement makings, lane delineation, roadside reflectors, high-visibility bridge rail)
- Bridge lighting
- Skid-resistant pavement
- Anti-icing systems
- Crashworthy bridge rail and approach guardrail
- Emergency pull-off areas
- Surveillance (for long, high-volume bridges)

2.5 Structural Capacity

Structural capacity has no effect on traffic operations, and its effect on safety is related only to the probability of a structural failure, not to the likelihood of traffic crashes. For this reason, structural capacity is not reviewed here and will not be addressed in this research.

2.6 Horizontal Alignment

Horizontal alignment involves design of the horizontal curves and tangents along a roadway section. In the context of the controlling criteria for design, horizontal alignment addresses only horizontal curves, not tangent sections, and the horizontal alignment criterion addresses only curve radius. Superelevation of horizontal curves is addressed by a separate controlling criterion. While the length of a horizontal curve and the length of tangent preceding a horizontal curve may influence traffic operations and safety and should be considered as part of the design process, they are not part of the controlling criteria and do not require design exceptions.

Chapter 3 of the *Green Book* provides guidance for selecting minimum radii for horizontal curves based on design speed, the maximum superelevation rate (e_{max}), and the maximum side friction factor (f_{max}), which sets an upper limit on lateral acceleration based on driver comfort. This methodology is applicable to each of the road types discussed below, although additional guidance is provided for each road type individually as well. Table 20 presents design criteria for minimum curve radius for three selected maximum superelevation rates.

Table 20. Design criteria for minimum curve radius for three selected maximum superelevation rates (*4, 5*).

Design speed (mph)	Maximum *e* (%)	Maximum *f*	Total (*e*/100 + *f*)	Calculated minimum radius (ft)	Rounded minimum radius (ft)
10	6.0	0.38	0.44	15.2	15
15	6.0	0.32	0.38	39.5	39
20	6.0	0.27	0.33	80.8	81
25	6.0	0.23	0.29	143.7	144
30	6.0	0.20	0.26	230.8	231
35	6.0	0.18	0.24	340.3	340
40	6.0	0.16	0.22	484.8	485
45	6.0	0.15	0.21	642.9	643
50	6.0	0.14	0.20	833.3	833
55	6.0	0.13	0.19	1,061.4	1,060
60	6.0	0.12	0.18	1,333.3	1,330
65	6.0	0.11	0.17	1,656.6	1,660
70	6.0	1.10	0.16	2,041.7	2,040
75	6.0	0.09	0.15	2,500.0	2,500
80	6.0	0.08	0.14	3,047.6	3,050
10	8.0	0.38	0.46	14.5	14
15	8.0	0.32	0.40	37.5	38
20	8.0	0.27	0.35	76.2	76
25	8.0	0.23	0.31	134.4	134
30	8.0	0.20	0.28	214.3	214
35	8.0	0.18	0.26	314.1	314
40	8.0	0.16	0.24	444.4	444
45	8.0	0.15	0.23	587.0	587
50	8.0	0.14	0.22	757.6	758
55	8.0	0.13	0.21	960.3	960
60	8.0	0.12	0.20	1,200.0	1,200
65	8.0	0.11	1.09	1,482.5	1,480
70	8.0	1.10	0.18	1,847.8	1,810
75	8.0	0.09	0.7	2,205.9	2,210
80	8.0	0.08	1.16	2,666.7	2,670
10	12.0	0.38	0.50	13.3	13
15	12.0	0.32	0.44	34.1	34
20	12.0	0.27	0.39	68.4	68
25	12.0	0.23	3.35	119.0	119
30	12.0	0.20	0.32	187.5	188
35	12.0	0.18	0.30	272.2	272
40	12.0	0.16	0.28	381.0	381
45	12.0	0.15	0.27	500.0	500
50	12.0	0.14	0.26	641.0	641
55	12.0	0.13	0.25	806.7	807
60	12.0	0.12	0.24	1,000.0	1,000
65	12.0	0.11	0.23	1,224.6	1,220
70	12.0	0.10	0.22	1,484.8	1,480
75	12.0	0.099	0.24	1,785.7	1,790
80	12.0	0.08	0.20	2,133.3	2,130

SOURCE: Based on *Green Book* Table 3-7 (abridged).

2.6.1 Rural Two-Lane Highways

Design Criteria

The design criteria for minimum curve radius presented in Table 20 apply to rural two-lane highways.

Traffic Operational Effects

Chapter 15 (Two-lane Highways) of the HCM uses free-flow speed in the determination of LOS. The chapter states that the base free-flow speed is the speed that would be expected on the basis of the facility's horizontal and vertical alignment, if standard lane and shoulder widths were present and there were no roadside access points. However, the HSM provides no methodology to determine the effect of horizontal curvature on base free-flow speed.

The IHSDM design consistency module (*10, 29*) includes a series of models for predicting the reduction in vehicle speed on horizontal curves from the design speed or tangent speed. These models are presented in Table 21. It should be noted

Table 21. IHSDM speed prediction equations for passenger vehicles[a] (*10, 29*).

AC EQ#[b]	Alignment condition	Equation[c]	# of sites	R^2	MSE
1.	Horizontal curve on grade: –9% ≤ G < –4%	$V_{85} = 102.10 - \frac{3077.13}{R}$	21	0.58	51.95
2.	Horizontal curve on grade: –4% ≤ G < 0%	$V_{85} = 105.98 - \frac{3707.90}{R}$	25	0.76	28.46
3.	Horizontal curve on grade: –0% ≤ G < 4%	$V_{85} = 104.82 - \frac{3574.51}{R}$	25	0.76	24.34
4.	Horizontal curve on grade: –4% ≤ G < 9%	$V_{85} = 96.61 - \frac{2752.19}{R}$	23	0.53	52.54
5.	Horizontal curve combined with sag vertical curve	$V_{85} = 105.32 - \frac{3438.19}{R}$	25	0.92	10.47
6.	Horizontal curve combined with non-limited sight distance crest vertical curve	[d]	13	n/a	n/a
7.	Horizontal curve combined with limited-sight-distance crest vertical curve (i.e., K ≤ 43 m/%)	$V_{85} = 103.24 - \frac{3576.51}{R}$	22	0.74	20.06
8.	Sag vertical curve on horizontal tangent	V_{85} = assumed desired speed	7	n/a	n/a
9.	Vertical crest with non-limited-sight-distance (i.e., K > 43 m/%) on horizontal tangent	V_{85} = assumed desired speed	6	n/a	n/a
10.	Vertical crest with limited sight distance (i.e., K ≤ 43 m/%) on horizontal tangent	$V_{85} = 105.08 - \frac{149.69}{K}$	9	0.60	31.10

[a] Check the speeds predicted from Equations 1 or 2 in this table (for the downgrade) and Equations 3 or 4 in this table (for the upgrade) and use the lowest speed. This will ensure that the speed predicted along the combined curve will not be better than if just the horizontal curve was present (i.e., that the inclusion of a limited-sight-distance crest vertical curve will result in a higher speed).
[b] AC EQ# = Alignment condition equation number; MSE = mean squared error.
[c] Where: V_{85} = 85th percentile speed of passenger cars (km/h); K = rate of vertical curvature; R = radius of curvature (m); G = grade (%)
[d] Use lowest speed of the speeds predicted from Equations 1 or 2 in this table (for the downgrade) and Equations 3 or 4 in this table (for the upgrade).

that Table 21, as it appears in the original research, uses metric units for speed and curve radius.

Traffic Safety Effects

Chapter 10 (Rural Two-Lane Highways) of the HSM provides a CMF for horizontal curves on rural two-lane roads which is computed as shown in Equation 15:

$$CMF = \frac{(1.55 \times L_c)\left(\frac{80.2}{R}\right) - (0.012 \times S)}{(1.55 \times L_c)} \tag{15}$$

where

L_c = Length of horizontal curve including length of spiral transitions, if present (mi)
R = Radius of curvature (ft)
S = 1 if spiral transition curve is present: 0 if spiral transition curve is not present

The base condition (CMF = 1.0) is a tangent segment with no curvature. This CMF applies to total crashes and is based on research by Zegeer et al. (*30*).

An alternative CMF that incorporates the effects of both horizontal curvature and grade on straight grades (i.e., grades with constant percent grade) has been developed by Bauer and Harwood (*31*) in an FHWA study for consideration for a future edition of the HSM:

- For fatal-and injury-crashes,

$$CMF_{SG,FI} = \begin{cases} \exp\left[\begin{array}{l} 0.044\,G + 0.19\ln\left(2 \times \frac{5730}{R}\right) \\ + 4.52\left(\frac{1}{R}\right)\left(\frac{1}{L_C}\right) \end{array}\right] & \textit{for horizontal curves} \\ \exp[0.044\,G] & \textit{for tangents on nonlevel grades} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \tag{16}$$

- For property-damage-only crashes,

$$CMF_{SG,PDO} = \begin{cases} \exp\left[\begin{array}{l} 0.040\,G + 0.13\ln\left(2\times\dfrac{5730}{R}\right) \\ +\,3.80\left(\dfrac{1}{R}\right)\left(\dfrac{1}{L_C}\right)\end{array}\right] & \textit{for horizontal curves} \\ \exp[0.040\,G] & \textit{for tangents on nonlevel grades} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (17)$$

where

G = absolute value of percent grade

2.6.2 Rural Multilane Highways

Design Criteria

The design criteria for minimum curve radius presented in Table 20 apply to rural multilane highways.

Traffic Operational Effects

Chapter 14 (Multilane Highways) of the HCM uses free-flow speed in the determination of LOS. The chapter states that the base free-flow speed is the speed that would be expected on the basis of the facility's horizontal and vertical alignment, if standard lane and shoulder widths were present and there were no roadside access points. However, the HCM provides no methodology to determine the effect of horizontal curvature on base free-flow speed.

Research conducted under NCHRP Project 17-53 (see Section 4.4) quantified the effect of horizontal curve radius on traffic speed for rural multilane highways as follows:

$$Speed_{curve} = Speed_{approach} - \frac{3136}{R} \quad (18)$$

where

$Speed_{curve}$ = Speed of traffic on horizontal curve (mph)
$Speed_{approach}$ = Speed of traffic on tangent approaching curve (mph)
R = Radius of curvature (ft)

Traffic Safety Effects

Chapter 11 (Rural Multilane Highways) of the HSM does not include any CMFs for horizontal curves on rural multilane highways. Thus, the safety effect of horizontal curves on rural multilane highways has not been documented. There are several CMFs for horizontal curve radius in the FHWA CMF Clearinghouse, but none of these is specifically applicable to rural multilane highways.

Research conducted under NCHRP Project 17-53 (see Section 4.4) developed the following CMFs for the effect of horizontal curvature on rural four-lane divided highways:

- For fatal-and-injury crashes,

$$CMF = \exp\left(-0.87L_c + 0.22\ln\left(2\times\frac{5730}{R}\right)\right) \quad (19)$$

- For property-damage-only crashes,

$$CMF = \exp\left(-0.95L_c + 0.26\ln\left(2\times\frac{5730}{R}\right)\right) \quad (20)$$

No comparable CMFs are available for rural four-lane undivided highways.

2.6.3 Urban and Suburban Arterials

Design Criteria

The design criteria for minimum curve radius presented in Table 20 apply to urban and suburban arterials. On low-speed urban streets, with design speeds of 45 mph or less, minimum radii sharper than those shown in Table 20 can be used (see *Green Book* Exhibit 3-16).

Traffic Operational Effects

Chapter 17 (Urban Street Segments) of the HSM includes a method for estimating the free-flow speed for an urban street section. The factors considered include speed limit, median type, curb presence, and access-point density. There is no effect of horizontal alignment in the procedure. In essence, the procedure assumes that the effect of curvature on speed is minimal.

Research conducted under NCHRP Project 17-53 (see Section 4.4) quantified the effect of horizontal curve radius on traffic speed urban and suburban arterials as follows:

$$Speed_{curve} = Speed_{approach} - \frac{2203}{R} \quad (21)$$

Traffic Safety Effects

Chapter 12 (Urban and Suburban Arterials) of the HSM does not include any CMFs for the effect of horizontal curves on urban and suburban arterials. Recent research by Hauer et al. (*32*) observed on-road crash frequencies for horizontal curves on urban four-lane undivided arterials to be lower than tangent sections in the same corridors; the opposite was found to be the case for run-off-road crashes. Since on-road crashes are predominant on urban arterials, Hauer et al.

concluded that the role of horizontal curvature in safety for this type of road may need reconsideration. There are several CMFs for horizontal curve radius in the FHWA CMF Clearinghouse, but none of these is specifically applicable to urban and suburban arterials.

2.6.4 Freeways

Design Criteria

The design criteria for minimum curve radius presented in Table 20 apply to freeways.

Traffic Operational Effects

Chapter 11 (Multilane Highways) of the HCM uses free-flow speed in the determination of LOS. The chapter states that the base free-flow speed is the speed that would be expected on the basis of the facility's horizontal and vertical alignment, if standard lane and shoulder widths were present and there were no roadside access points. However, no methodology to determine the effect of horizontal curvature on base free-flow speed is provided in the HCM.

Traffic Safety Effects

Results from NCHRP Project 17-45, which developed a proposed HSM safety prediction methodology for freeways, includes a CMF for the safety effect of horizontal curves on safety (*25*). The CMFs for horizontal curves (where R = radius of curvature [ft])are the following:

- For fatal-and-injury multiple-vehicle crashes,

$$CMF = 1.0 + 0.0172 \times \left(\frac{5730}{R}\right)^2 \tag{22}$$

- For property-damage-only multiple-vehicle crashes,

$$CMF = 1.0 + 0.0340 \times \left(\frac{5730}{R}\right)^2 \tag{23}$$

- For fatal-and-injury single-vehicle crashes,

$$CMF = 1.0 + 0.0719 \times \left(\frac{5730}{R}\right)^2 \tag{24}$$

- For property-damage-only single-vehicle crashes,

$$CMF = 1.0 + 0.0626 \times \left(\frac{5730}{R}\right)^2 \tag{25}$$

2.6.5 Mitigation Strategies

Mitigation strategies for horizontal curves with sharper radii than established design criteria include the following (*7*):

- Advance warning with signing and pavement markings
- Dynamic message signs
- Delineation (chevrons, post-mounted delineators, reflectors on barriers)
- Roadway widening
- Skid-resistant pavement
- Lighting
- Shoulder, painted edgeline, or centerline rumble strips
- Paved or partially paved shoulders
- Safety edge treatment
- Roadside improvements (clear recovery area, traversable slopes, breakaway safety hardware, barrier where appropriate)

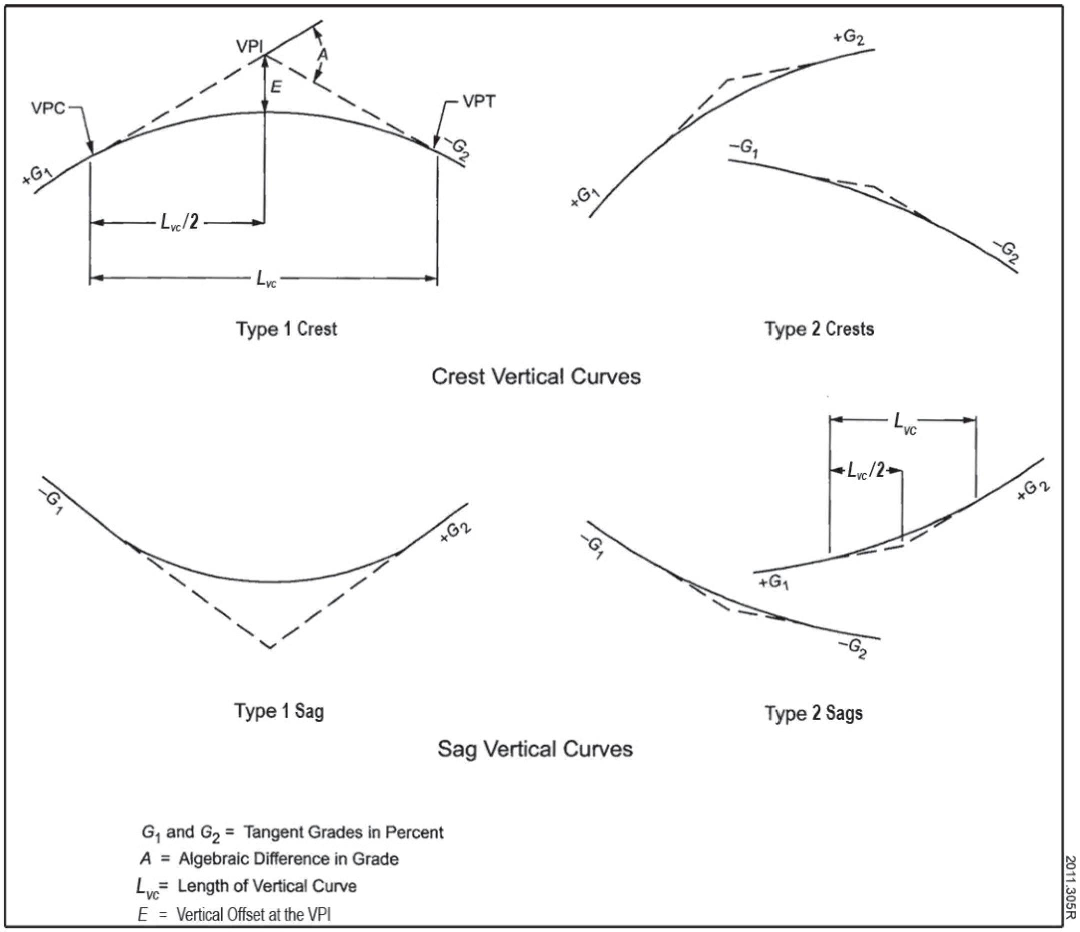
2.7 Vertical Alignment

Vertical alignment generally consists of two elements: grades and vertical curves. Both of these elements are considered in the controlling criteria. Grade is treated as a separate controlling criterion (see Section 2.8). Two types of vertical curves are considered in vertical alignment design: crest vertical curves and sag vertical curves. Both crest and sag vertical curves have two types, known as Type 1 and Type 2, as illustrated in Figure 5. The *Green Book* design criteria for crest vertical curve lengths are illustrated in Figure 6. Crest vertical curve length is selected primarily to achieve minimum stopping sight distance on the vertical curve. Stopping sight distance is treated as a separate controlling criterion (see Section 2.9). Thus, the only element of vertical alignment not dealt with by a separate controlling criterion is sag vertical curve length. Sag vertical curve length is normally selected so that the curve does not restrict the length of roadway illuminated by vehicle headlights, which would reduce stopping sight distance at night. Figure 7 presents the *Green Book* design criteria for sag vertical curve length. The parameter, K, in Figures 6 and 7 is the ratio of the algebraic difference in grade, A, to the length of the vertical curve. Recent research on sag vertical curves is documented in *NCHRP Web-Only Document 198: Sag Vertical Curve Design Criteria for Headlight Sight Distance.*

2.7.1 Rural Two-Lane Highways

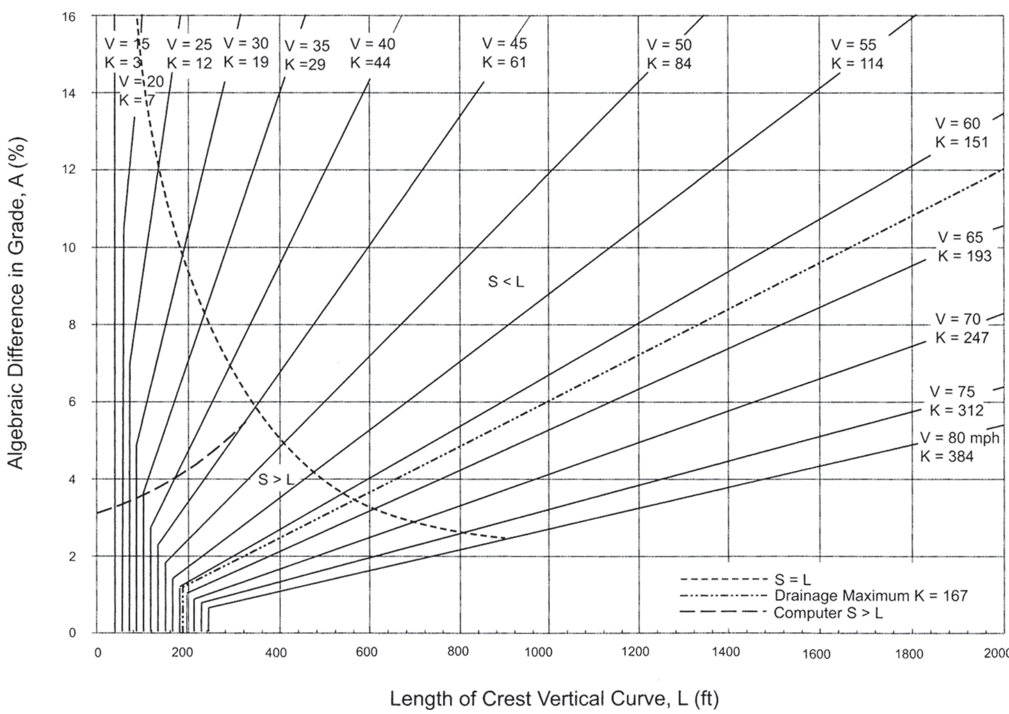
Design Criteria

The design criteria for crest and sag vertical curves, presented in Figures 6 and 7, respectively, are applicable to rural two-lane highways.



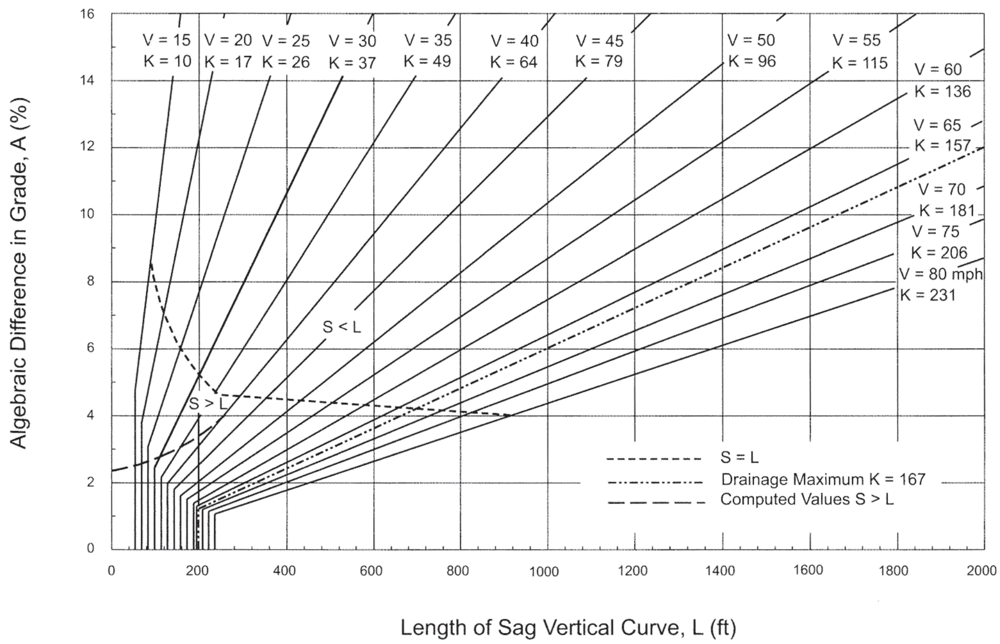
SOURCE: Based on *Green Book* Figure 3-41.

***Figure 5. Types of vertical curves* (4, 5).**



SOURCE: Based on *Green Book* Figure 3-43.

***Figure 6. Design controls for crest vertical curves—open road conditions* (4, 5).**



SOURCE: Based on *Green Book* Figure 3-44.

Figure 7. Design controls for sag vertical curves—open road conditions* (4, 5*).

Traffic Operational Effects

Chapter 15 (Two-Lane Highways) of the HCM provides a methodology for adjusting the LOS boundaries on rural two-lane highways to account for vertical alignment, considering general terrain classes or specific grades, as well as the percentages in the traffic flow of two types of heavy vehicles (trucks and recreational vehicles). Since these vertical alignment effects are primarily a function of grade, they are discussed in Section 2.8 of this report. Crest vertical curve effects are addressed in Section 2.9 of this report. There are no known quantifiable operational effects of sag vertical curve length; it is likely that any such effects are minimal as long as the ride comfort criteria in *Green Book* Equation 3-51 are met.

Traffic Safety Effects

Chapter 10 (Rural Two-Lane Highways) of the HSM includes a factor for the effect of grade on safety; this effect is discussed in Section 2.8 of this report. Chapter 10 (HSM) does not include any effect of crest or sag vertical curves on safety. The effect of crest vertical curves on safety is likely related to stopping sight distance and is discussed in Section 2.9 of this report. There is no known effect of sag vertical curve length on safety. Sag vertical curve length is essentially irrelevant to safety under daytime conditions, because the driver can see beyond the sag vertical curve unless a horizontal curve is present. At night, drivers at speeds of 50 mph or more generally outdrive their headlights. This is generally true whatever the vertical alignment, so there is no special risk on sag vertical curves. Furthermore, as discussed in Section 2.9, the object most likely to be struck by a driver in a limited-sight-distance situation is another vehicle on the roadway ahead. The taillights of such vehicles and the dispersion of light from their headlights should make such vehicles clearly visible at night, even beyond the limits of the sag vertical curve unless a horizontal curve is also present. Thus, it seems unlikely that sag vertical curve length would have much effect on safety. An important exception occurs when an overpass that might block the driver's view of the road ahead is located on a sag vertical curve. This situation is addressed explicitly in *Green Book* Chapter 3. It should also be noted that overpass structures on rural two-lane highways are not common.

Recent research for FHWA by Bauer and Harwood (*31*) completed since the publication of the first edition of the HSM, developed the following CMFs for Type 1 crest vertical curves (L_{VC} = length of vertical curve):

- For fatal-and injury-crashes,

$$CMF_{C1,FI} = \begin{cases} \exp\left[0.0088\left(\dfrac{5730}{R}\right)\dfrac{L_{VC}}{K}\right] & \textit{for horizontal curves} \\ 1.0 & \textit{for tangents at Type 1 crests} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (26)$$

- For property-damage-only crashes,

$$CMF_{C1,PDO} = \begin{cases} \exp\left[0.0046\left(\frac{5730}{R}\right)\frac{L_{VC}}{K}\right] & \textit{for horizontal curves} \\ 1.0 & \textit{for tangents at Type 1 crests} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (27)$$

The equivalent CMFs for Type 2 crest vertical curves are the following:

- For fatal-and injury-crashes,

$$CMF_{C2,FI} = \begin{cases} \exp\left[0.20 \ln\left(2 \times \frac{5730}{R}\right)\right] & \textit{for horizontal curves} \\ 1.0 & \textit{for tangents at Type 2 crests} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (28)$$

- For property-damage-only crashes,

$$CMF_{C2,PDO} = \begin{cases} \exp\left[0.10 \ln\left(2 \times \frac{5730}{R}\right)\right] & \textit{for horizontal curves} \\ 1.0 & \textit{for tangents at Type 2 crests} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (29)$$

Bauer and Harwood (*31*) also developed the following CMFs for Type 1 sag vertical curves:

- For fatal-and injury-crashes,

$$CMF_{S1,FI} = \begin{cases} \exp\left[10.51\frac{1}{K} + 0.011\left(\frac{5730}{R}\right)\frac{L_{VC}}{K}\right] & \textit{for horizontal curves} \\ \exp\left[10.51\frac{1}{K}\right] & \textit{for tangents at Type 1 sags} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (30)$$

- For property-damage-only crashes,

$$CMF_{S1,PDO} = \begin{cases} \exp\left[8.62\frac{1}{K} + 0.010\left(\frac{5730}{R}\right)\frac{L_{VC}}{K}\right] & \textit{for horizontal curves} \\ \exp\left[8.62\frac{1}{K}\right] & \textit{for tangents at Type 1 sags} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (31)$$

The equivalent CMFs for Type 2 sag vertical curves are the following:

- For fatal-and injury-crashes,

$$CMF_{S2,FI} = \begin{cases} \exp\left[0.188 \ln\left(2 \times \frac{5730}{R}\right)\right] & \textit{for horizontal curves} \\ 1.0 & \textit{for tangents at Type 2 sags} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (32)$$

- For property-damage-only crashes,

$$CMF_{S2,PDO} = \begin{cases} \exp\left[0.022\left(\frac{5730}{R}\right)A\right] & \textit{for horizontal curves} \\ 1.0 & \textit{for tangents at Type 2 sags} \\ 1.0 & \textit{for level tangents (base condition)} \end{cases} \quad (33)$$

2.7.2 Rural Multilane Highways

Design Criteria

The design criteria for crest and sag vertical curves, presented in Figures 6 and 7, respectively, are applicable to rural multilane highways.

Traffic Operational Effects

Chapter 14 (Multilane Highways) of the HCM provides a methodology for adjusting the LOS boundaries on a multilane highway to account for vertical alignment considering general terrain classes or specific grades, as well as the percentages in the traffic flow of two types of heavy vehicles (trucks and recreational vehicles). Since these vertical alignment effects are primarily a function of grade, they are discussed in Section 2.8. Crest vertical curve effects are addressed in Section 2.9 of this report. There are no known quantifiable operational effects of sag vertical curve length; it is likely that such effects are minimal, as long as the ride comfort criteria in *Green Book* Equation 3-51 are met.

Traffic Safety Effects

Chapter 11 (Rural Multilane Highways) of the HSM does not include any factors to account for the effects of grade, crest vertical curve length, or sag vertical curve length on safety. Based on the reasoning presented in Section 2.7.1, sag vertical curve length in particular seems unlikely to have much influence on safety except where an overpass is located on a sag vertical curve.

2.7.3 Urban and Suburban Arterials

Design Criteria

The design criteria for crest and sag vertical curves, presented in Figures 6 and 7, respectively, are applicable to urban and suburban arterials.

Traffic Operational Effects

Chapter 17 (Urban Street Segments) of the HCM recommends that free-flow speeds for urban street segments be measured in the field or estimated based on the street's functional and design categories. No specific quantitative procedures are provided.

Traffic Safety Effects

Chapter 12 (Urban and Suburban Arterials) of the HSM does not include any factors to account for the effects of grade, crest vertical curve length, or sag vertical curve length on safety. Crest vertical curve effects are addressed in Section 2.9 of this report. There are no known quantifiable operational effects of sag vertical curve length; it is likely that such effects are minimal, as long as the ride comfort criteria in *Green Book* Equation 3-51 are met.

2.7.4 Freeways

Design Criteria

The design criteria for crest and sag vertical curve length, presented in Figures 6 and 7, respectively, are applicable to freeways.

Traffic Operational Effects

Chapter 11 (Basic Freeway Segments) of the HCM provides a methodology for adjusting the LOS boundaries on a freeway to account for vertical alignment considering general terrain classes or specific grades, as well as the percentages in the traffic flow of two types of heavy vehicles (trucks and recreational vehicles). Since these vertical alignment effects are primarily a function of grade, they are discussed in Section 2.8. Crest vertical curve effects are addressed in Section 2.9 of this report. There are no known quantifiable operational effects of sag vertical curve length; it is likely that such effects are minimal.

Traffic Safety Effects

The HSM safety prediction methodology for freeways developed in NCHRP Project 17-45 does not include any safety effects for grades, crest vertical curve length, or sag vertical curve length (*25*).

2.7.5 Mitigation Strategies

Most design exceptions for vertical alignment are related to grades and crest vertical curves. Appropriate mitigation strategies for grades and crest vertical curves are discussed in Sections 2.8 and 2.9, respectively. Sag vertical curve lengths that do not meet established criteria do not often need design exceptions (*7*). Mitigation of sag vertical curve lengths that do not meet established criteria is unlikely to be needed unless there is a specific crash pattern of rear-end crashes or an overpass is present on the sag vertical curve. If mitigation is needed, the provision of lighting is an obvious strategy.

2.8 Grade

Grade is the rate of change of vertical elevation along a roadway. The controlling criterion for grade includes both maximum and minimum grades. Maximum grades are established for specific roadway types and functional classes (see below). A design exception is needed where steeper grades are to be provided or retained.

Chapter 3 of the *Green Book* provides general guidance for selecting acceptable grades for roadways. Generally, a maximum grade of 5 percent is appropriate for a design speed of 70 mph, while maximum grades of 7 to 12 percent are appropriate for design speeds of 30 to 50 mph.

Green Book Exhibits 3-55 and 3-56 (not shown here) estimate running speeds of typical heavy trucks based on the percent grade and the length of the roadway section at that grade. These exhibits or the Truck Speed Performance Model (TSPM) developed by Harwood et al. (*33*) can be used to

Table 22. Maximum grade for rural arterials (*4, 5*).

Type of terrain	Maximum grade (%) for specified design speed								
	40 mph	45 mph	50 mph	55 mph	60 mph	65 mph	70 mph	75 mph	80 mph
Level	5	5	4	4	3	3	3	3	3
Rolling	6	6	5	5	4	4	4	4	4
Mountainous	8	7	7	6	6	5	5	5	5

SOURCE: Based on AASHTO *Green Book* Table 7-2.

establish critical lengths of grade that would produce a differential of 15 mph or more between the minimum speed of trucks and the average speed of traffic. Depending on traffic and truck volumes, locations with critical length of grade may warrant the addition of truck climbing lanes. However, the truck climbing lane criteria are not part of the controlling criterion for grade and do not require design exceptions. In fact, quite the opposite is true—the critical length of grade criteria merely suggest locations where truck climbing lanes might be considered.

2.8.1 Rural Two-Lane Highways

Design Criteria

Chapter 7 of the *Green Book* provides additional guidance for maximum grade selection for rural arterials, including rural two-lane highways. Table 22 shows the recommended maximum grades for rural arterials based on terrain type and design speed.

Traffic Operational Effects

Chapter 15 (Two-Lane Highways) of the HCM provides a methodology for adjusting demand flow rates for two-lane highways based on grade. Two adjustment factors in Chapter 15 (HCM) are affected by grade: the grade adjustment factor (f_g) and the heavy vehicle adjustment factor (f_{HV}). Separate adjustments are made in the computations for the two service measures for two-lane highways: average travel speed and percent time spent following.

Table 23. Grade adjustment factor (f_g) to determine speeds on two-way and directional segments for two-lane highways (*13*).

One-direction demand flow rate (veh/h)	Type of terrain	
	Level terrain and specific downgrades	Rolling terrain
≤ 100	1.00	0.67
200	1.00	0.75
300	1.00	0.83
400	1.00	0.90
500	1.00	0.95
600	1.00	0.97
700	1.00	0.98
800	1.00	0.99
≥ 900	1.00	1.00

SOURCE: Based on HCM Exhibit 15-9.

Average Travel Speeds. The grade adjustment factor, f_g, accounts for vehicles traveling more slowly on grades than they would on a level roadway. A smaller value of f_g will result in a higher demand flow rate. Table 23 presents values of f_g for various flow rates for level or rolling terrain. For segments with mountainous terrain, or on any segment with a grade steeper than 3 percent over a distance of 0.6 mi or more, the procedure for calculating f_g relies on more extensive criteria partially illustrated in Table 24.

Table 24. Grade adjustment factor for estimating travel speed on specific upgrades for two-lane highways (*13*).

Grade (%)	Grade length (mi)	Grade adjustment factor, f_g								
		Directional demand flow rate v_{vph} (veh/h)								
		≤ 100	200	300	400	500	600	700	800	≥ 900
≥ 3 < 3.5	0.25	0.78	0.84	0.87	0.91	1.00	1.00	1.00	1.00	1.00
	0.50	0.75	0.83	0.86	0.90	1.00	1.00	1.00	1.00	1.00
	0.75	0.73	0.81	0.85	0.89	1.00	1.00	1.00	1.00	1.00
	1.00	0.73	0.79	0.83	0.88	1.00	1.00	1.00	1.00	1.00
	1.50	0.73	0.79	0.83	0.87	0.99	0.99	1.00	1.00	1.00
	2.00	0.73	0.79	0.82	0.86	0.98	0.98	0.99	1.00	1.00
	3.00	0.73	0.78	0.82	0.85	0.95	0.96	0.96	0.97	0.98
	≥ 4.00	0.73	0.78	0.81	0.85	0.94	0.94	0.95	0.95	0.96

SOURCE: Based on HCM Exhibit 15-10.

Table 25. Passenger-car equivalents for trucks (E_T) and recreational vehicles (RVs) (E_R) to determine speeds on directional segments for two-lane highways (*13*).

Vehicle type	Directional demand flow rate, V_{vph} (veh/h)	Passenger-car equivalents for level terrain and specific downgrades	Passenger-car equivalents for rolling terrain
Trucks, E_T	≤ 100	1.9	2.7
	200	1.5	2.3
	300	1.4	2.1
	400	1.3	2.0
	500	1.2	1.8
	600	1.1	1.7
	700	1.1	1.6
	800	1.1	1.4
	≥900	1.0	1.3
RV_s, E_R	All flows	1.0	1.1

SOURCE: Based on HCM Exhibit 15-11.

The heavy vehicle adjustment factor, f_{HV}, accounts for heavy vehicles traveling more slowly on grades than passenger cars. A larger value of the passenger-car equivalence factors for heavy vehicles, E_T or E_R, results in a higher demand flow rate. Table 25 presents passenger-car equivalence factors for trucks (E_T) and recreational vehicles (E_R). For segments with mountainous terrain, or on any segment with a grade steeper than 3 percent over a distance of 0.6 mi or more, the procedures for calculating f_{HV} rely on the more extensive criteria in Tables 26 and 27.

The demand flow rate in the analysis direction of travel for use in the average travel speed determination is computed as:

$$v_d = \frac{V_d}{PHF \times f_g \times f_{HV}} \tag{34}$$

where

v_d = demand flow rate for analysis direction (pc/L)
PHF = peak hour factor

Table 26. Passenger-car equivalents for trucks for estimating travel speed on specific upgrades for two-lane highways (*13*).

Grade (%)	Grade length (mi)	Passenger-car equivalent for trucks, E_T — Directional demand flow rate v_{vph} (veh/h)								
		≤ 100	200	300	400	500	600	700	800	≥ 900
≥ 3 < 3.5	0.25	2.6	2.4	2.3	2.2	1.8	1.8	1.7	1.3	1.1
	0.50	3.7	3.4	3.3	3.2	2.7	2.6	2.6	2.3	2.0
	0.75	4.6	4.4	4.3	4.2	3.7	3.6	3.4	2.4	1.9
	1.00	5.2	5.0	4.9	4.9	4.4	4.2	4.1	3.0	1.6
	1.50	6.2	6.0	5.9	5.8	5.3	5.0	4.8	3.6	2.9
	2.00	7.3	6.9	6.7	6.5	5.7	5.5	5.3	4.1	3.5
	3.00	8.4	8.0	7.7	7.5	6.5	6.2	6.0	4.6	3.9
	≥ 4.00	9.4	8.8	8.6	8.3	7.2	6.9	6.6	4.8	3.7

SOURCE: Based on HCM Exhibit 15-12.

Table 27. Passenger-car equivalents for RVs for estimating travel speed on specific upgrades for two-lane highways (*13*).

Grade (%)	Grade length (mi)	Passenger-car equivalent for RV_s, E_R — Directional demand flow rate v_{vph} (veh/h)								
		≤ 100	200	300	400	500	600	700	800	≥ 900
≥ 3 < 3.5	≤ 0.25	1.1	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0
	> 0.25 ≤ 0.75	1.2	1.2	1.1	1.1	1.0	1.0	1.0	1.0	1.0
	> 0.75 ≤ 1.25	1.3	1.2	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	> 1.25 ≤ 2.25	1.4	1.3	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	> 2.25	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0

SOURCE: Based on HCM Exhibit 15-13.

f_g = grade adjustment factor from Table 23 or 24
f_{HV} = heavy vehicle adjustment factor from HCM Equations 15-4 or 15-5, which utilize data from Tables 25 through 27

The demand flow rate in the opposing direction is determined in a manner entirely analogous to Equation 34. The service measure average travel speed, which is one of two measures used to determine LOS, is then determined with HCM Equation 15-6.

Percent Time Spent Following. The demand flow rates are determined slightly differently when used for percent time spent following rather than average travel speed as the service measure. Similar to the methodology for speed calculations, two adjustment factors are affected by grade: the grade adjustment factor (f_g), and the heavy vehicle adjustment factor (f_{HV}). Demand flow rate for the analysis and opposing directions is determined using Equation 34. However, for these calculations, Tables 28 through 31 are used instead of Tables 23 through 27 to determine the values of f_g and f_{HV}.

Table 28. Grade adjustment factor (f_g) to determine percent time spent following on directional segments for two-lane highways (*13*).

Directional demand flow rate (veh/h)	Level terrain and specific downgrades	Rolling terrain
≤ 100	1.00	0.73
200	1.00	0.80
300	1.00	0.85
400	1.00	0.90
500	1.00	0.96
600	1.00	0.97
700	1.00	0.99
800	1.00	1.00
≥900	1.00	1.00

SOURCE: Based on HCM Exhibit 15-16.

Traffic Safety Effects

Chapter 10 (Rural Two-Lane Highways) of the HSM presents the CMF for grade on two-lane highways as shown in Table 32. Table 32 presents the CMF by terrain categories.

Table 29. Grade adjustment factor (f_g) for estimating percent time spent following on specific upgrades for two-lane highways (*13*).

Grade (%)	Grade length (mi)	Grade adjustment factor, f_g								
		Directional demand flow rate v_{vph} (veh/h)								
		≤ 100	200	300	400	500	600	700	800	≥ 900
≥ 3 < 3.5	0.25	1.00	0.99	0.97	2.2	1.8	1.8	1.7	1.3	1.1
	0.50	1.00	0.99	0.98	3.2	2.7	2.6	2.6	2.3	2.0
	0.75	1.00	0.99	0.98	4.2	3.7	3.6	3.4	2.4	1.9
	1.00	1.00	0.99	0.98	4.9	4.4	4.2	4.1	3.0	1.6
	1.50	1.00	0.99	0.98	5.8	5.3	5.0	4.8	3.6	2.9
	2.00	1.00	0.99	0.98	6.5	5.7	5.5	5.3	4.1	3.5
	3.00	1.00	1.00	0.99	7.5	6.5	6.2	6.0	4.6	3.9
	≥ 4.00	1.00	1.00	1.00	8.3	7.2	6.9	6.6	4.8	3.7

SOURCE: Based on HCM Exhibit 15-17.

Table 30. Passenger-car equivalents for trucks (E_T) and RVs (E_R) for estimating percent time spent following on directional segments for two-lane highways (*13*).

Vehicle type	Directional demand flow rate (veh/h)	Passenger-car equivalents for level and specific downgrades	Passenger-car equivalents for rolling terrain
Trucks, E_T	≤ 100	1.1	1.9
	200	1.1	1.8
	300	1.1	1.7
	400	1.1	1.6
	500	1.0	1.4
	600	1.0	1.2
	700	1.0	1.0
	800	1.0	1.0
	≥ 900	1.0	1.0
RVs, E_R	All	1.0	1.0

SOURCE: Based on HCM Exhibit 15-18.

Table 31. Passenger-car equivalents for trucks for estimating percent time spent following on specific upgrades for two-lane highways (*13*).

Grade (%)	Grade length (mi)	Passenger-car equivalent for trucks E_T								
		Directional demand flow rate v_{vph} (veh/h)								
		≤ 100	200	300	400	500	600	700	800	≥ 900
≥ 3 < 3.5	≤ 2.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	3.00	1.5	1.3	1.3	1.2	1.0	1.0	1.0	1.0	1.0
	≥ 4.00	1.6	1.4	1.3	1.3	1.0	1.0	1.0	1.0	1.0
≥ 3 < 4.5	≤ 1.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.50	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	2.00	1.6	1.3	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	3.00	1.8	1.4	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	≥ 4.00	2.1	1.9	1.8	1.7	1.4	1.4	1.4	1.4	1.4

SOURCE: Based on HCM Exhibit 15-19.

Table 32. CMF for grade of roadway segments (*12*).

Level grade (≤ 3%)	Moderate terrain (3% < grade ≤ 6%)	Steep terrain (> 6%)
1.00	1.10	1.16

SOURCE: Based on HSM Table 10-11.

The underlying research (*34, 35*) presents the CMF as a continuous function rather than a step function, as follows:

$$CMF = (1.0 + 0.016\,G) \qquad (35)$$

where

G = absolute value of percent grade. In other words, the CMF increases by 0.016 for each percent grade.

2.8.2 Rural Multilane Highways

Design Criteria

The maximum grade criteria presented in Table 22 also apply to rural multilane highways.

Traffic Operational Effects

Chapter 14 (Multilane Highways) of the HCM presents a methodology for determining the effect of grades on operations of multilane highways. The procedure is similar to the procedure described above for two-lane highways. The multilane highway methodology is much simpler—the only factor that is used in determining the LOS boundaries is the f_{HV} factor.

The heavy vehicles adjustment factor, f_{HV}, adjusts the demand flow rate to account for the fact that heavy vehicles generally travel more slowly on grades than passenger cars. A larger value of E_T (or E_R) results in a higher demand flow rate. Table 33 presents passenger equivalence factors for trucks and buses (E_T) and RVs (E_R). For segments with a grade between 2 and 3 percent for more than 0.5 mi or with a grade steeper than 3 percent for more than 0.25 mi, the procedures for calculating E_T and E_R rely on the more extensive Tables 34, 35, and 36. The value of f_{HV} is determined with HCM Equation 14-4, the demand flow rate is determined with HCM Equation 14-3, and density, the service measure for multilane highways, is determined with HCM Equation 14-5.

Traffic Safety Effects

Chapter 11 of the HSM does not include a CMF for grade on rural multilane highways.

2.8.3 Urban and Suburban Arterials

Design Criteria

Table 37 presents recommended maximum grades for urban arterials. The *Green Book* states that when these cannot be attained, climbing lanes should be considered; in this

Table 33. Passenger-car equivalents for heavy vehicles in general terrain segments on multilane highways (*13*).

Passenger-car equivalent	Type of terrain		
	Level	Rolling	Mountainous
E_T (trucks and buses)	1.5	2.5	4.5
E_R (RVs)	1.2	2.0	4.0

SOURCE: Based on HCM Exhibit 14-12.

Table 34. Passenger-car equivalents for trucks and buses on upgrades on multilane highways (*13*).

Upgrade (%)	Length (mi)	E_T Percentage of trucks and buses 2%	4%	5%	6%	8%	10%	15%	20%	25%
≤ 2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
> 2 to 3	0.00 to 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.25 to 0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.50 to 0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.75 to 1.00	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	> 1.00 to 1.50	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 1.50	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0

SOURCE: Based on HCM Exhibit 14-13 (abridged).

Table 35. Passenger-car equivalents for RVs on upgrades on multilane highways (*13*).

Upgrade (%)	Length (mi)	E_R Percentage of RVs 2%	4%	5%	6%	8%	10%	15%	20%	25%
≤ 2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
> 2 to 3	0.00 to 0.50	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.50	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
> 3 to 4	0.00 to 0.25	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.25 to 0.50	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5

SOURCE: Based on HCM Exhibit 14-14 (abridged).

Table 36. Passenger-car equivalents for trucks (E_T) on specific downgrades on rural and suburban multilane highways (*13*).

Percent downgrade	Length of grade (mi)	Proportion of trucks and buses 5%	10%	15%	20%
< 4	All	1.5	1.5	1.5	1.5
4 to 5	≤ 4	1.5	1.5	1.5	1.5
	> 4	2.0	2.0	2.0	1.5
> 5 to 6	≤ 4	1.5	1.5	1.5	1.5
	> 4	5.5	4.0	4.0	3.0
> 6	≤ 4	1.5	1.5	1.5	1.5
	> 4	7.5	6.0	5.5	4.5

SOURCE: Based on HCM Exhibit 14-15.

case, the use of a climbing lane would be considered a mitigation strategy and not part of the controlling criterion.

Traffic Operational Effects

According to Chapter 17 (Urban Street Segments) of the HCM, one of the first steps in determining the LOS for an urban street is determining the free-flow speed of traffic on the road segment. The steeper the upgrade of a roadway segment, the slower the free-flow speed will be. Chapter 17 (HCM) recommends that the free-flow speed be measured if possible; otherwise it must be estimated based on the street's functional and design categories. No methodology is provided for estimating the effect of grade on free-flow speed for an urban street.

Table 37. Maximum grades for urban arterials (*13*).

Type of terrain	Maximum grade (%) for specified design speed 30 mph	35 mph	40 mph	45 mph	50 mph	55 mph	60 mph
Level	8	7	7	6	6	5	5
Rolling	9	8	8	7	7	6	6
Mountainous	11	10	10	9	9	8	8

SOURCE: Based on *Green Book* Table 7-4.

Table 38. Maximum grades for rural and urban freeways (*4, 5*).

Type of terrain	Maximum grade (%) for specified design speed						
	50 mph	55 mph	60 mph	65 mph	70 mph	75 mph	80 mph
Level	4	4	3	3	3	3	3
Rolling	5	5	4	4	4	4	4
Mountainous	6	6	6	5	5	-	-

SOURCE: Based on *Green Book* Table 8-1.

Traffic Safety Effects

Chapter 12 (Urban and Suburban Arterials) of the HSM does not include a CMF for grade on urban and suburban arterials.

2.8.4 Freeways

Design Criteria

Chapter 8 of the *Green Book* provides the following specific guidance for urban freeways. Grades on urban freeways should generally be comparable to those in rural areas. Steeper grades can be tolerated in urban areas, but because interchanges may be closely spaced in urban areas, flatter grades are desirable when practical. Table 38 provides recommended maximum grades for rural and urban freeways.

Traffic Operational Effects

Chapter 11 (Basic Freeway Segments) of the HCM provides a methodology for determining the effect of grades on operations of freeways. The procedure is very similar to the procedure described above for multilane highways.

The heavy vehicles adjustment factor, f_{HV}, adjusts the demand volume to account for the tendency of heavy vehicles to travel more slowly on grades than passenger cars. Table 39 provides passenger-car equivalence factors for trucks and buses (E_T) and RVs (E_R). For any segment with a grade between 2 and 3 percent for more than 0.5 mi or with a grade steeper than 3 percent for more than 0.25 mi, the procedures for calculating E_T and E_R rely on the more extensive Tables 40, 41, and 42. A larger value of E_T or E_R results in a larger demand flow rate. The value of f_{HV} is determined with HCM Equation 11-3, the demand flow rate is determined with HCM Equation 11-2, and the service measure for multilane highways is determined with HCM Equation 11-4.

Traffic Safety Effects

The HSM safety prediction methodology for freeways developed in NCHRP Project 17-45 does not include any safety effects for grades on freeways (*25*).

2.8.5 Mitigation Strategies

The strategies for mitigating steep grades include the following (*7*):

- Providing drivers with advance warning signs for steep grades
- Providing climbing lanes and downgrade lanes
- Providing emergency escape ramps for trucks

Table 39. Passenger-car equivalents on extended freeway segments (*13*).

Passenger-car equivalent	Type of terrain		
	Level	Rolling	Mountainous
E_T (trucks and buses)	1.5	2.5	4.5
E_R (RVs)	1.2	2.0	4.0

SOURCE: Based on HCM Exhibit 11-10.

Table 40. Passenger-car equivalents for trucks and buses on upgrades for specific grades on freeways (*13*).

Upgrade (%)	Length (mi)	E_T Percentage of trucks and buses								
		2%	4%	5%	6%	8%	10%	15%	20%	25%
< 2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
≥ 2 to 3	0.00 to 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.25 to 0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.50 to 0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.75 to 1.00	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5

SOURCE: Based on HCM Exhibit 11-11 (abridged).

Table 41. Passenger-car equivalents for RVs on upgrades for specific grade segments on freeways (*13*).

Upgrade (%)	Length (mi)	E_R								
		Percentage of RVs								
		2%	4%	5%	6%	8%	10%	15%	20%	25%
≤ 2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
> 2 to 3	0.00 to 0.50	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.50	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
> 3 to 4	0.00 to 0.25	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.25 to 0.50	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5

SOURCE: Based on HCM Exhibit 11-12 (abridged).

Table 42. Passenger-car equivalents for trucks and buses on downgrades on specific grade segments on freeways (*13*).

Downgrade (%)	Length (mi)	E_T			
		Percentage of trucks			
		5%	10%	15%	20%
< 4	All	1.5	1.5	1.5	1.5
4 to 5	≤ 4	1.5	1.5	1.5	1.5
4 to 5	> 4	2.0	2.0	2.0	1.5
> 5 to 6	≤ 4	1.5	1.5	1.5	1.5

SOURCE: Based on HCM Exhibit 11-13 (abridged).

- Reducing the frequency or severity of lane-departure crashes (enhanced pavement markings; delineation; shoulder, painted edgeline, or centerline rumble strips; paved or partially paved shoulders; safety edge treatment; clear recovery area; traversable slopes; breakaway safety hardware; and barrier where appropriate).

The strategies for mitigating flat grades include the following (*7*):

- Adjusting the gutter profile
- Providing special drainage systems

2.9 Stopping Sight Distance

Stopping sight distance is the distance required for a driver to perceive or recognize a need to stop, react to that perception, and then decelerate to a stop. Horizontal and vertical curves limit available sight distance for drivers, requiring a careful analysis of stopping sight distance during the design process. Sight distance needs are based on the design speed of the roadway and the grade of the roadway, since cars traveling downhill require a greater distance to stop than cars traveling uphill or on the level. The minimum stopping sight distance is calculated using equations provided in the *Green Book* based on design speed and grade and assumed values of perception-reaction time and deceleration rate. Table 43 provides minimum stopping sight distances for various roadway design speeds and grades. The stopping sight distance criteria shown in Table 43 apply to all roadway types, including ramps and turning roadways. A design exception is required where stopping sight distances less than those shown in Table 43 are provided or retained.

Stopping sight distance generally provides drivers with enough distance to make a hurried stop, but these distances may not be adequate for a driver to interpret complex information or make a complex decision. In some cases, a maneuver other than a quick stop would be preferable, but would require more time for the driver to make that decision. For these reasons, the *Green Book* also provides decision sight distance guidelines for several different avoidance maneuver conditions that each assumes a different perception and reaction time. The decision sight distance criteria are presented in *Green Book* Table 3-3 (not shown here). Decision sight distance is not part

Table 43. Design criteria for stopping sight distance (*4, 5*).

Design speed (mph)	Stopping sight distance (ft)						
	Level	Downgrade			Upgrade		
	0%	3%	6%	9%	3%	6%	9%
15	80	80	82	85	75	74	73
20	115	116	120	126	109	107	104
25	155	158	165	173	147	143	140
30	200	205	215	227	200	184	179
35	250	257	271	287	237	229	222
40	305	315	333	354	289	278	269
45	360	378	400	427	344	331	320
50	425	446	474	507	405	388	375
55	495	520	553	593	469	450	433
60	570	598	638	686	538	515	495
65	645	682	728	785	612	584	561
70	730	771	825	891	690	658	631
75	820	866	927	1003	772	736	704
80	910	965	1035	1121	859	817	782

SOURCE: Based on AASHTO *Green Book* Tables 3-1 and 3-2.

of the controlling criteria; no design exceptions are required for decision sight distances less than the *Green Book* criteria.

The HCM does not include any effect of stopping sight distance on LOS for any roadway type. *Green Book* criteria for stopping sight distance assume that vehicles on a crest vertical curve, or in a region of restricted horizontal sight distance, are traveling at the design speed. There does not appear to be any basis on which to presume that limited stopping sight distance, especially marginal limitations, affects vehicle speeds or other traffic operational performance measures.

Research by Fambro et al. (*36*) found very few collisions on highways with objects smaller than another vehicle, even in areas of limited stopping sight distance. This led to the change in stopping sight distance from a 6-in. object to a 2-ft object (equivalent to the height of vehicle taillights) that was made in the 2001 edition of the *Green Book* (*3*). Thus, available research suggests that at most places on the highway with limited stopping sight distance there is unlikely to be anything in the roadway that a driver might strike. Safety is unlikely to be affected by limited stopping sight distance in such cases. However, when the limited sight distance restricts the driver's view of a location where other vehicles may be slowing or stopping (e.g., intersections, driveways, horizontal curves, entrance or exit ramps, or locations with daily congestion), improving limited sight distance may be very important to safety.

Neither the HSM nor the FHWA CMF Clearinghouse includes any CMFs indicating an effect of stopping sight distance on safety. Research conducted under NCHRP Project 17-53 (see Section 4.7) investigated the relationship between stopping sight distance and crash frequency. The research team compared the crash frequencies for crest vertical curves on rural two-lane highways with stopping sight distance less than AASHTO stopping sight distance criteria to crest vertical curves with stopping sight distance equal to or more than AASHTO stopping sight distance criteria. A statistical analysis found no differences in crash frequency (either for total crashes or fatal-and-injury crashes) between the crest vertical curves with differing stopping sight distance values, but there was a statistically significant difference in crash frequency (for both total crashes and fatal-and-injury crashes) between sites with and without horizontal curves, intersections, or driveways hidden by the presence of the crest vertical curve. The observed effect on crash frequency of the presence of a hidden horizontal curve, intersection, or driveway was 0.36 crashes per mi per year for total crashes and 0.48 crashes per mi per year for fatal-and-injury crashes.

Mitigation strategies for limited stopping sight distance include the following (*7*):

- Signing for crest vertical curves
- Lighting for intersections, sag vertical curves, or merge/diverge areas
- Lower height barriers to reduce sight distance limitations due to presence of the barrier
- Adjustment of lane placement within the roadway cross section on horizontal curves
- Selection of cross-sectional elements to manage speed
- Wider shoulders and wider clear zones
- Static or dynamic warning of intersections or entering traffic
- Repositioning, adding, or enhancing intersection signs

2.10 Cross Slope

The controlling criterion for cross slope addresses the traverse slope of the pavement surface on tangent sections or on horizontal curves where superelevation is not used. Superelevation on horizontal curves is addressed in Section 2.11.

The cross-slope design criterion is important because cross slope facilitates runoff of water from rain, snow, or ice from the pavement surface. In general, the steeper the cross slope, the more efficiently water flows to the edge of the lanes and off the roadway. Flat cross slopes can lead to water ponding on the lanes, especially where a curb is used. At the same time, a steep cross slope can affect steering and can make vehicles more susceptible to cross winds; drivers may tend toward the lower edge of the traveled way, and lateral skidding can become more likely when braking on wet or icy pavement. On roadways with a center crown, vehicles making passing maneuvers experience double the change in cross slope as they move over the crown, reversing the direction of lateral acceleration, and potentially causing trucks to sway from side to side. For these reasons, a balance must be struck between a steeper cross slope that efficiently moves water to the edge of the roadway and a shallow cross slope that is imperceptible to drivers during lane changes. The *Green Book* recommends a normal cross slope of 1.5 to 2 percent, although when two or more lanes are inclined in the same direction, each successive lane may be given a greater cross slope by 0.5 to 1.0 percent, not to exceed 4 percent in the outermost lanes. In areas of intense rainfall, a slope of 2.5 percent may be used. The National Transportation Safety Board (NTSB) has asked FHWA and AASHTO to investigate the appropriateness of design criteria for cross-slope breaks at the outside edge of the traveled way on horizontal curves for current passenger cars and trucks, especially trucks with high centers of gravity (*37*). The research underlying the current 8-percent design criterion for cross-slope breaks was completed in 1982 using an older vehicle dynamics simulation model (HVOSM) that simulated cross-slope break traversals by a 1971 Dodge Coronet passenger car (*38*). Research for a current passenger car and larger trucks, including trucks with high centers of gravity, would clearly be desirable.

Neither the HCM nor the HSM shows any qualitative effect of cross-slope or cross-slope breaks on traffic operations or safety. There are also no safety effects found in the FHWA CMF Clearinghouse.

The primary concern for locations with insufficient cross slope is inadequate drainage and ponding of water on the travel lanes. Mitigation strategies for inadequate cross slope include the following (*7*):

- SLIPPERY WHEN WET signing
- Grooved, textured, or open-graded pavements to improve surface friction
- Slope inside lanes toward the median and outside lanes toward the outside of the roadway (on multilane divided facilities)

Mitigation strategies for large pavement/shoulder cross slope breaks include the following:

- Adjustment of the high-side shoulder cross slope, including sloping the shoulder toward the traveled way
- Rounding of the cross-slope break (feasible for hot-mix asphalt pavements)

2.11 Superelevation

The *Green Book* provides equations and tables for determining the appropriate superelevation rate for specific horizontal curves based on the design speed, curve radius, and assumed maximum values of superelevation rate and friction demand. Maximum superelevation rates (e_{max}) are selected by highway agency policies; *Green Book* Chapter 3 permits highway agencies to choose e_{max} in the range of 4 to 12 percent. Where snow and ice are factors, the *Green Book* recommends that superelevation should not exceed 8 percent. For lower speed urban arterials, the *Green Book* recommends that little or no superelevation be used. *Green Book* Chapter 8 recommends that superelevation should not exceed 6 percent on freeways with viaducts where snow and ice are factors.

Neither the HCM nor any other available source indicates that superelevation has a quantifiable effect on traffic operations. It seems unlikely that minor variations in superelevation from the AASHTO design values would have much effect on traffic operations.

HSM Chapter 10 (Rural Two-Lane Highways) presents a CMF for superelevation on rural two-lane highways that is shown in the following equations:

$$\mathrm{CMF} = 1.00 \text{ for } \mathrm{SV} < 0.01 \tag{36}$$

$$\mathrm{CMF} = 1.00 + 6 \times (\mathrm{SV} - 0.01) \text{ for } 0.01 \le \mathrm{SV} < 0.02 \tag{37}$$

$$\mathrm{CMF} = 1.06 + 3 \times (\mathrm{SV} - 0.02) \text{ for } \mathrm{SV} \ge 0.02 \tag{38}$$

where

CMF = crash modification factor for the effect of superelevation variance on total crashes
SV = superelevation variance (ft/ft), which represents the superelevation rate contained in the *Green Book* minus the actual superelevation of the curve

The CMF applies to total roadway segment crashes for roadway segments located on horizontal curves. No CMFs are available and no trends are known for the safety effects of superelevation on roadway types other than rural two-lane highways.

The mitigation strategies for superelevation lower than *Green Book* criteria are the same as those described for horizontal alignment in Section 2.6.5 of this report.

2.12 Vertical Clearance

In general, vertical clearance does not affect operations on the roadway other than for those vehicles that are taller than the available vertical clearance allows for. When overpasses or other structures do not allow for taller vehicles to pass underneath, these vehicles use an alternate route, potentially increasing travel time. Guidance for vertical clearance is provided in the *Green Book* as follows:

- For rural arterials, the recommended minimum vertical clearance is 16 ft
- The preferred vertical clearance on urban arterials is 16 ft; however, when existing structures offer at least 14 ft of clearance, these structures may be retained as long as an alternate route with 16 ft of clearance is provided
- The recommended minimum vertical clearance on freeways is 16 ft; however, in highly developed areas, where replacement of structures would be costly, a minimum clearance of 14 ft is permitted, provided an alternate route with 16 ft of clearance is available. Sign trusses and pedestrian overpasses should be built with a minimum clearance of 17 ft.

There are no operational or safety effects of insufficient vertical clearance except for increased travel times for vehicles taller than the available vertical clearance.

Vertical clearance guidelines do not directly impact safety for the majority of vehicles, although in cases where the recommended vertical clearance is not provided, advanced warning and alternate route designation become important mitigation strategies for avoiding possible crashes involving tall vehicles. Vertical clearance crashes can have severe impacts on operations by damaging overpasses or other structures that result in extended road closures.

Special attention is given to vertical clearance on Interstate freeways to maintain the integrity of the system for national defense purposes. On rural Interstate freeways, vertical clearance at structures of at least 16 ft is maintained. In urban areas, 16 ft of clearance is maintained for at least one Interstate routing through the urban area, with other urban Interstate routes having vertical clearance of at least 14 ft. The 16-ft vertical clearance for Interstate freeways in rural areas and for the single routing in urban areas applies to the entire roadway width, including the usable shoulder width and the ramps and collector-distributor roadways at Interstate-to-Interstate interchanges.

2.13 Horizontal Clearance/ Lateral Offset

The controlling criterion known in current FHWA policy as horizontal clearance has been renamed lateral offset in the 2011 edition of the *Green Book* (*5*) to avoid confusion about the definition of this criterion. Lateral offset deals with the distance from the edge of the traveled way, face of curb, shoulder, or other designated point to a vertical roadside element or obstruction (*7*). Lateral offset can be thought of as an operational offset; vertical roadside elements are offset (1) so that they do not affect a driver's speed or lane position and (2) so that adequate clearance to vertical roadside elements is provided for overhangs or mirrors of trucks and buses and for opening curbside doors where on-street parking is provided.

Lateral offset as a controlling criterion is primarily of interest for roads with curb-and-gutter sections, such as urban and suburban arterials. For roads without curbs, the minimum shoulder widths generally take care of providing a minimum lateral offset from the traveled way.

Design criteria in the 2004 *Green Book* (*4*) specify a minimum lateral offset of 1.5 ft to address operational concerns for all roadway conditions and classifications. The 2011 *Green Book* (*5*) does not state an explicit lateral offset, but makes reference to the AASHTO *Roadside Design Guide* (RDG) (*39*). The 2006 edition of the RDG (*39*), as well as previous editions, incorporated the same 1.5-ft lateral offset as the 2004 *Green Book* (*4*). The 2011 edition of the RDG (*40*) encourages wider lateral offsets, particularly on urban and suburban arterials (see Section 2.13.3 below).

A design exception is required when the specified minimum lateral offset is not provided. It is important to note that the controlling criterion for lateral offset does not include the provision of clear recovery zones. Lateral offset is an operational criterion and, as explicitly stated by FHWA policy, does not address clear-zone width (*2*).

2.13.1 Rural Two-Lane Highways

Design Criteria

Relatively few rural two-lane highways have curb-and-gutter sections, so the minimum shoulder-width criteria generally provide the minimum lateral offset needed for operational reasons.

Traffic Operational Effects

Chapter 15 (Two-Lane Highways) of the HCM provides guidance for estimating the free-flow speed for two-lane highways. Although the LOS boundaries are not directly adjusted for lateral clearance, Table 5 provides an adjustment to free-flow speed based on lane and shoulder widths. As shown in Table 5, a 6-ft shoulder on a rural two-lane highway provides sufficient lateral clearance that there is no effect on vehicle speeds.

Traffic Safety Effects

Chapter 10 (Rural Two-Lane Highways) of the HSM does not contain any CMF for lateral offset. However, the CMF for shoulder width on two-lane highway segments presented in Table 13 and Figure 4 implicitly reflects, at least in part, the safety effects of lateral offset.

2.13.2 Rural Multilane Highways

Design Criteria

Relatively few rural multilane highways have curb-and-gutter sections, so the minimum shoulder-width criteria generally provide the minimum lateral offset needed for operational reasons.

Traffic Operational Effects

Chapter 14 (Multilane Highways) of the HCM provides guidance for estimating free-flow speed for multilane highways. Although the LOS boundaries are not directly adjusted for lateral clearance, Table 16 provides an adjustment to free-flow speed based on the sum of the lateral clearance on the left side of the roadway (maximum of 6 ft) and the right side of the roadway (maximum 6 ft).

Traffic Safety Effects

Chapter 11 (Rural Multilane Highways) of the HSM does not contain any CMF for lateral offset. However, the CMF for shoulder width in Table 13 and Figure 4 for undivided roadways and in Table 17 for divided roadways implicitly reflects, at least in part, the safety effects of lateral offset.

2.13.3 Urban and Suburban Arterials

Design Criteria

The design criterion for lateral offset on urban and suburban arterials in the 2006 RDG (*39*), and previous editions, is 1.5 ft. The 2011 RDG (*40*), which is referred to explicitly in Chapter 7 (Arterials) of the 2011 *Green Book* (*5*), states that a lateral offset of 3 ft from the face of the curb to obstructions should be provided at intersections and driveway openings, while a minimum lateral offset of 1.5 ft should be used elsewhere. However, the new RDG also presents a targeted design approach for high-risk urban roadside corridors:

- For locations with vertical curbs, provide a 6-ft offset from the face of curb to obstacles on the outside of curves, because obstacles on the outside of curves are hit more often, and provide a 4-ft offset elsewhere
- For locations without a vertical curb, 12-ft offsets to obstacles on the outside of curves and 8-ft offsets on tangent sections are recommended as reasonable goals where the clear-zone widths in RDG Chapter 3 cannot be achieved.

Traffic Operational Effects

Chapter 17 (Urban Street Segments) of the HCM includes a procedure for estimating free-flow speeds, but neither lateral offset nor shoulder width is considered as part of that procedure.

Traffic Safety Effects

Chapter 12 (Urban and Suburban Arterials) of the HSM does not include a CMF for either lateral offset or shoulder width. There is currently no quantifiable safety effect for these design elements.

2.13.4 Freeways

Design Criteria

Lateral offset is not generally relevant on freeways because minimum shoulder widths should always provide the minimum lateral offset from the traveled way.

Traffic Operational Effects

Chapter 11 (Basic Freeway Segments) of the HCM includes criteria for estimating the effect of shoulder width on free-flow speed (see Table 19).

Traffic Safety Effects

There are no CMFs for lateral offset on freeways, as freeway shoulders are usually wide enough to provide the minimum lateral offset. The results of NCHRP Project 17-45 include a CMF for right (outside) clearance (*25*). This is essentially a CMF for clear-zone width on freeways, which incorporates an adjustment for right (outside) shoulder width. The NCHRP Project 17-45 methodology also includes CMFs for right (outside) roadside barriers on freeways. Neither of these CMFs appears applicable to lateral offset on freeways because the shoulder-width CMFs from NCHRP Project 17-45, presented in Equations 8 through 11, should account for the effect of lateral offset on safety.

2.13.5 Mitigation Strategies

The primary mitigation strategy for lateral obstructions within the minimum lateral offset that cannot practically be removed is to delineate such obstacles with reflectors or reflective sheeting so that they become more visible, particularly at night (*7*).

2.14 Summary of Traffic Operational Effects

Table 44 summarizes which traffic operational effects for the 13 controlling criteria have been quantified and where in this report the information concerning each of those known effects can be found.

2.15 Summary of Traffic Safety Effects

Table 45 summarizes which traffic safety effects for the 13 controlling criteria have been quantified and where in this report the information covering each of those known effects can be found.

Table 44. Summary of traffic operational effects of the 13 controlling criteria for design.

Design criterion	Roadway type	Traffic operational effects
Design speed	All	No direct effects.[a]
Lane width	Rural two-lane highways	See Table 5 (based on HCM Exhibit 15-7) and Equation 1.
	Rural multilane highways	See Table 7 (based on HCM Exhibit 14-8) and Equation 3.
	Urban and suburban arterials	No quantified effects.
	Freeways	See Table 10 (based on HCM Exhibit 11-8) and Equation 4.
Shoulder width	Rural two-lane highways	See Table 5 (based on HCM Exhibit 15-7) and Equation 1.
	Rural multilane highways	See Table 16 (based on HCM Exhibit 14-9) and Equation 3.
	Urban and suburban arterials	No quantified effects.
	Freeways	See Table 19 (based on HCM Exhibit 11-9) and Equation 4.
Bridge width	Rural two-lane highways	Bridge roadway widths less than the approach roadway width do not appear to increase crash frequency or severity.
	Rural multilane highways	No quantified effects directly applicable to bridge width; related effects for lane and shoulder width are known (see Sections 2.2.2 and 2.3.2).
	Urban and suburban arterials	No quantified effects.
	Freeways	No quantified effects directly applicable to bridge width; related effects for lane and shoulder width are known (see Sections 2.2.4 and 2.3.4).
Structural capacity	All	No relationship to traffic operations; controlling criterion is based on risk of structural failure.
Horizontal alignment	Rural two-lane highways	See Table 21.
	Rural multilane highways	See Equation 18.
	Urban and suburban arterials	See Equation 21.
	Freeways	No quantified effects.
Vertical alignment (sag vertical curves)	Rural two-lane highways	No quantified effects.
	Rural multilane highways	No quantified effects.
	Urban and suburban arterials	No quantified effects.
	Freeways	No quantified effects.
Grade	Rural two-lane highways	See Tables 23 through 31 (based on HCM Exhibits 15-9, 15-10, 15-11, 15-12, 15-13, 15-16, 15-17, 15-18, 15-19) and Equation 34.
	Rural multilane highways	See Tables 33 through 36 (based on HCM Exhibits 14-12, 14-13, 14-14, 14-15) and HCM Equation 14-4.
	Urban and suburban arterials	No quantified effects.
	Freeways	See Tables 39 through 42 (based on HCM Exhibits 11-10, 11-11, 11-12, 11-13) and HCM Equations 11-2, 11-3, and 11-4.
Stopping sight distance	All	No quantified effects.
Cross slope	All	No quantified effects.
Superelevation	All	No quantified effects.
Vertical clearance	All	No quantified effects.
Horizontal clearance/lateral offset	Rural two-lane highways	Effect discussed in shoulder-width section (see Section 2.3.1).
	Rural multilane highways	Effect discussed in shoulder-width section (see Section 2.3.2).
	Urban and suburban arterials	No quantified effects.
	Freeways	Effect discussed in shoulder-width section (see Section 2.3.4)

[a] For indirect effects, see lane width, horizontal alignment, vertical alignment, and stopping sight distance.

Table 45. Summary of traffic safety effects for the 13 controlling criteria for design.

Design criterion	Roadway type	Traffic safety effects
Design speed	All roadway types	No direct effects.[a]
Lane width	Rural two-lane highways	See Equation 2 and Table 6 (based on HSM Equation10-11 and Table 10-8).
	Rural multilane highways	For undivided sections, see Equation 2 and Table 8 (based on HSM Equation 11-13 and Table 11-11); for divided sections, see Equation 2 and Table 9 (based on HSM Equation 11-16 and Table 11-16).
	Urban and suburban arterials	Lane width does not appear to affect crash frequency or severity. Lanes narrower than 12 ft may not be desirable on streets where substantial volumes of bicycles, trucks, or buses are present.
	Rural freeways	See Equations 5 and 6.
Shoulder width	Rural two-lane highways	See Equation 7 and Tables 13 and 14 (based on HSM Equation10-12 and Table 10-9 and 10-10).
	Rural multilane highways	For undivided sections, see Equation 7 and Tables 13 and 14 (based on HSM Equation10-12 and Table 10-9 and 10-10); for divided sections, see Table 17 (based on HSM Table 11-17).
	Urban and suburban arterials	No quantified effects.
	Freeways	See Equations 8 through 13.
Bridge width	Rural two-lane highways	No quantified effects directly applicable to bridge width; related effects for lane and shoulder width are known (see Sections 2.2.1 and 2.3.1).
	Rural multilane highways	No quantified effects directly applicable to bridge width; related effects for lane and shoulder width are known (see Sections 2.2.2 and 2.3.2).
	Urban and suburban arterials	No quantified effects.
	Freeways	No quantified effects directly applicable to bridge width; related effects for lane and shoulder width are known (see Sections 2.2.4 and 2.3.4).
Structural capacity	All roadway types	No relationship to traffic safety; controlling criterion is based on risk of structural failure.
Horizontal alignment	Rural two-lane highways	See Equation 15 (based on HSM Equation 10-13); potential updated effects are presented in Equations 16 and 17.
	Rural multilane highways	See Equations19 and 20.
	Urban and suburban arterials	No quantified effects.
	Freeways	See Equations 22 through 25.
Vertical alignment (sag vertical curves)	Rural two-lane highways	See Equations 30 through 33.
	Rural multilane highways	No quantified effects.
	Urban and suburban arterials	No quantified effects.
	Freeways	No quantified effects.
Grade	Rural two-lane highways	See Table 32 (based on HSM Table 10-11) and Equation 35; potential updated effects are presented in Equations 16 and 17.
	Rural multilane highways	No quantified effects.
	Urban and suburban arterials	No quantified effects.
	Freeways	No quantified effects.
Stopping sight distance	Rural two-lane highways	No effect on safety unless a hidden horizontal curve, intersection, or driveway is present.
	Rural multiline highways	No quantified effects.
	Urban and suburban arterials	No quantified effects.
	Freeways	No quantified effects.
Cross slope	All roadway types	No quantified effects.
Superelevation	Rural two-lane highways	See Equations 36 through 38 (based on HSM Equations 10-14 through 10-16).
	Rural multilane highways	No quantified effects.
	Urban and suburban arterials	No quantified effects.
	Freeways	No quantified effects.
Vertical clearance	All roadway types	No quantified effects.
Horizontal clearance	Rural two-lane highways	Only known effects are based on shoulder width (See Section 2.3.1).
	Rural multilane highways	Only known effects are based on shoulder width (See Section 2.3.2).
	Urban and suburban arterials	No quantified effects.
	Freeways	Only known effects are based on shoulder width (See Section 2.3.4).

[a] For indirect effects, see lane width, horizontal alignment, vertical alignment, and stopping sight distance.

SECTION 3

Design Exception Practices

This section of the report reviews current practices of highway agencies with respect to design exceptions. Although all exceptions from applicable design policies should be documented in some manner, as specified in the policies of individual highway agencies, formal design exceptions are required by federal policy only for projects on the NHS for which agencies seek exceptions to the 13 controlling criteria. Internal state agency policies may require formal design exceptions for additional design criteria or for non-NHS projects. To distinguish between these types of design exceptions, one state agency—the Georgia Department of Transportation—refers to exceptions from federal policies as *design exceptions* and exceptions from state policies as *design variances*. Virginia uses the term *design waiver*, rather than *design variance*, for deviations from state criteria. The information presented here is based both on published literature and on interviews with experienced designers.

3.1 Published Reviews of Design Exception Practices

This section presents a summary of three published reviews of design exception practices—Mason and Mahoney (*41*), McGee et al. (*42*), and Stamatiadis et al. (*43*)—to complement the results of interviews with highway agency staff presented in the next portion of this section.

3.1.1 Mason and Mahoney

NCHRP Synthesis 316: Design Exception Practices, by Mason and Mahoney (*41*), reviewed the design exception practices of state highway agencies, including the conduct of a survey on design exceptions to the controlling criteria to which 45 of the 50 states and the District of Columbia responded. Table 46 summarizes the controlling criteria that respondents indicated frequently required design exceptions.

Mason and Mahoney found that the annual number of design exceptions prepared by state highway agencies ranges from 1 to approximately 500. Some of this variation is attributed to the basic characteristics of the states, their road systems, and their capital construction programs. The factors that an agency uses to determine if a design exception is needed are an additional source of variation in the numbers of design exceptions. These factors are

- Project location/road system (considered by 28 percent of highway agencies)
- Project funding source (13 percent)
- Project scope/type (65 percent)
- Supplemental agency design criteria (i.e., in addition to the 13 controlling criteria) (33 percent)
- Agency design criteria values higher than AASHTO criteria (44 percent)
- Use of established RRR criteria (87 percent)

Six highway agencies noted that developing design exception documentation is time and cost intensive; limited resources (agency personnel, funds, and time) may discourage the use of design exceptions to achieve design flexibility for these agencies. Five agencies asked that FHWA clarify the controlling criteria and provide better guidelines; this request has been addressed by the subsequent FHWA publication, *Mitigation Strategies for Design Exceptions* (*7*). The following policy changes were requested by the individual state highway agencies interviewed:

- Eliminate design speed as a controlling criterion
- Revise the process for resurfacing (i.e., RRR projects)
- Do not require design exceptions for existing features that do not meet current policy

NCHRP Synthesis 316 recommended that the relationship of the controlling criteria to traffic operations and safety

Table 46. Controlling criteria identified by state highway agencies as commonly requiring design exceptions (*41*).

Design element	Responses from state highway agencies	
	Number of responses	Percentage of responses
Horizontal alignment	25	54
Shoulder width	24	52
Vertical alignment	20	43
Stopping sight distance	18	39
Lane width	12	26
Design speed	12	26
Superelevation	9	20
Bridge width	8	17
Grade	7	15
Horizontal clearance/lateral offset	7	15
Vertical clearance	3	7
Cross slope	1	2
Structural capacity	0	0

should be reviewed to determine which relationships are strongest (*41*). This recommendation is addressed in this report. *NCHRP Synthesis 316* also recommended that future research develop improved guidance for evaluating the safety implications of design exceptions. The publication of the first edition of the HSM (*12*) provides a tool that can be utilized for this purpose. However, specific guidance on how the HSM should be used in such analyses would be desirable.

3.1.2 McGee et al.

McGee et al. (*42*) conducted a survey of state highway agencies concerning their practices for conducting safety analyses in connection with design projects. Responses were received from 37 of the 50 states. A summary of the responses from state highway agencies related to the types of design elements frequently requiring design exceptions is presented in Table 47.

3.1.3 Stamatiadis et al.

Stamatiadis et al. (*43*) prepared a paper reporting on their investigation of the design exception practices of the Kentucky Transportation Cabinet. They summarized past experience with design exceptions and found that design exceptions were requested for an average of 40 projects per year, often with more than one individual design exception per project. Over the 8-year period from 1993 to 2000, a total of 562 individual design exceptions, or 70 design exceptions per year, were requested. This experience is summarized in Table 48. The majority of the projects involved bridge replacement, with the next most frequent being reconstruction for roadway widening or construction of turn lanes.

Table 47. Design elements identified by state highway agencies as frequently requiring design exceptions (*42*).

Design element	Responses from state highway agencies	
	Number of responses	Percentage of responses
Shoulder width	21	57
Vertical alignment/curvature	12	32
Lane width	11	30
Horizontal alignment/curvature	11	30
Stopping sight distance[a]	7	19
Bridge width	6	16
Maximum grade	6	16
Clear zone[b]	5	14
Sideslope[b]	5	14
Lateral clearance	4	11
Superelevation	4	11
Reduced design speed	3	8
Existing bridge rail[b]	2	5
Cross slope	2	5
Vertical clearance	2	5

[a] Includes only alignment-related design exceptions.
[b] Not one of the 13 controlling criteria.

As indicated in Table 48, the most common design exception was for design speed lower than the posted speed limit. This contrasts with the experience reported by Mason and Mahoney (*41*), by McGee et al. (*42*), and by the interviews with highway agencies presented in Section 3.2, which indicated that design exceptions were generally not sought for design speed for an entire project, but rather for individual

Table 48. Number of design exceptions requested in Kentucky (1993 to 2000) (*43*).

Design element	Number of design exceptions	Percentage of design exceptions
Design speed	191	34.0
Horizontal alignment/curvature	67	11.9
Stopping sight distance	65	11.6
Shoulder width	63	11.2
Ditch width[a]	43	7.7
Roadway width/lane width	42	7.5
Bridge width	35	6.2
Number of lanes[a]	16	2.8
Maximum grade	15	2.7
Superelevation	12	2.1
Acceleration lane[a]	4	0.7
Clear zone/border[b]	3	0.5
Earth cut/fill slope[a]	2	0.4
Bridge railing[a]	1	0.2
Tie down[a]	1	0.2
Access spacing[a]	1	0.2
Guardrail end treatment[a]	1	0.2
TOTAL	562	

[a] Not one of the 13 controlling criteria.
[b] Clear zone is not one of the 13 controlling criteria, but lateral offset (border) is a controlling criterion.

design elements (curve radius, superelevation, vertical curves, or lane width) where the designs specified in the controlling criteria could not be provided. Other common design exceptions were for stopping sight distance, curve radius, or shoulder width.

Stamatiadis et al. conducted a crash analysis of 65 projects with design exceptions for which before and after data could be obtained. For 59 of the 65 projects, the crash rate following the project was (1) lower than the average crash rate for similar sites or (2) lower than the crash rate at the same site before the project. However, this finding was based on a naïve before-after study that did not incorporate any compensation for the effects of regression to the mean. Relatively short before- and after-study periods were used because both periods, including an allowance for the "during construction" period, had to be fitted into the 6 years (1995 to 2000) for which data were available. A closer examination of the six projects for which crashes increased found that the patterns of increased crashes were not related to the design elements covered by the design exception. The study concluded that the design exceptions being requested were reasonable, and there was no reason for the Kentucky Transportation Cabinet to change its design exception process.

3.2 Interviews with State Highway Agency Staff

Interviews with state highway agency staff were conducted at highway agency offices, on the telephone, and at two national meetings:

- AASHTO Technical Committee on Geometric Design, Irvine, California, July 25 through 27, 2011
- Highway Safety Manual Peer Exchange, convened as part of NCHRP Project 17-50, Irvine, California, August 10 and 11, 2011

An in-depth interview was conducted with highway agency design engineers in California. Brief interviews were conducted with engineers from the following states: Georgia, Kansas, Louisiana, Michigan, Minnesota, Missouri, Oregon, Tennessee, and Virginia.

There were several common themes in the interview results for all states:

- All agencies interviewed have a formal design exception process and a standard format for written documentation/justification of design exceptions.
- Design exceptions for design speed are seldom requested or granted. Where the design speed cannot be fully attained, it is more common to approve design exceptions for individual design elements than to change the overall design speed for the project.
- Highway agencies are just beginning to consider how the HSM might be used in analysis of design exceptions. FHWA plans to address design exception analysis in a forthcoming *HSM Applications Guide*, but details of these plans are not yet known.
- No highway agencies routinely do before-after evaluations of projects that included design, but some states are considering doing this in the future.

The results of these interviews are summarized below.

3.2.1 California Department of Transportation (Caltrans)

Interviews were conducted on July 15, 2011, with project development coordinators and geometric design reviewers in the Caltrans Division of Design.

Caltrans Highway Design Manual and Exception Process

Caltrans publishes the *Highway Design Manual* (HDM). The HDM is a design manual for internal and external use and establishes uniform policies and procedures for the design of and designs on California highways. The manual consists of standards as well as techniques and reference materials. (It should be noted that Caltrans uses the word *standards*, in referring to the controlling criteria, while most other states avoid this term.) The manual is ever changing and expanding as the Division of Design releases new or amended standards and techniques. The standards contained in the manual fall into three categories: mandatory standards, advisory standards, and permissive standards. "Mandatory design standards are those considered the most essential to achieve overall design objectives." (HDM 82.1[2]) "Advisory design standards are important also, but allow greater flexibility in application . . . " (HDM 82.1[3]) Permissive standards are all other standards or recommendations contained in the manual and have no requirement of application. The FHWA 13 controlling criteria are all designated as mandatory standards, except for the final standards of structural capacity, which are covered by Caltrans' bridge design manuals.

While the standards contained in the HDM are meant to promote uniform and safe design, for various reasons, it is sometimes difficult during project design to fully meet the standard. In cases where a mandatory or advisory standard can't be met (and the project is under the jurisdiction of Caltrans), a formal process of requesting an exception to the design standards must be followed. The Caltrans Division of Design is responsible for overseeing this process and for the approval or denial of all requests for exceptions to design standards. Per the Caltrans *Project Development and Procedures*

Manual, “The purpose of the design exception process is to create a record that documents the engineering decisions leading to the approval of each exception from a design standard.”

The Relationship between Caltrans Policy and the 13 Controlling Criteria

While the official design manual for the California highway system is the HDM, there are many references within the HDM to FHWA/AASHTO documents that can be used as supplementary information. Specifically, the *Green Book*, and the RDG are commonly referred to in the HDM.

Caltrans’ designers and geometric reviewers are currently utilizing the HSM, but not in an official capacity. The HSM contains CMFs that represent percentages of crash reduction that could be expected upon implementation of a given countermeasure. Many project development coordinators and geometric reviewers find CRFs helpful when reviewing exception requests. Caltrans will likely be releasing an official policy on the validity and usage of the HSM and, specifically, CMFs. In fact, Caltrans is currently conducting research of its own to validate or invalidate CMFs based on California-specific crash data.

The 13 controlling criteria adopted by FHWA are meant to apply to projects on the NHS. Many, if not most, of the mandatory and advisory standards found within the California HDM are based on the 13 controlling criteria. Basing its standards on the 13 controlling criteria means that Caltrans has taken standards that apply to the NHS and extended them to all California state highway facilities. Furthermore, as many local agencies default to the HDM for their design standards, the 13 controlling criteria get carried further into the design of local roads.

Often, Caltrans’ geometric reviewers consider the 13 controlling criteria when reviewing requests for exceptions to the mandatory/advisory standards, and some even go beyond the 13 controlling criteria back to the research data that support the criteria. Unfortunately, there are some highway engineers who are unaware of the existence of the 13 controlling criteria and, therefore, are unaware of the origins of the standards within the HDM.

All too often, exceptions are requested due to political pressures and budget shortfalls and not enough consideration is given to the implications of implementing a lesser standard.

The Caltrans Division of Design staff expressed great interest in the research conducted under NCHRP Project 17-53 and its potential ability to reaffirm and revalidate not only the 13 controlling criteria, but also standards in the HDM. This research could also serve to educate some that may be unaware of the origins of Caltrans’ standards.

Though the majority of the mandatory/advisory standards found within the HDM can be traced back to 12 of the 13 controlling criteria, over the years many new standards have been added that do not relate directly to the FHWA criteria. Caltrans’ position is that some key issues of transportation design are not covered by the 13 controlling criteria and that the criteria could perhaps be expanded to include these issues.

Caltrans staff members stated that the most commonly mentioned issue unrepresented by the 13 controlling criteria is access control. In the HDM, the term access control refers to how a public authority controls (whether partially or fully) access from adjoining lands to the highway system. At a time when high levels of development are surrounding aging highway systems, access control has become a significant topic.

The HDM also contains mandatory/advisory standards on pavement design, which the 13 controlling criteria do not address. The additional pavement design standards are based on a reasoning that longer pavement design life correlates to a higher level of worker safety and a lower level of interruption to the traveling public. As evolving pavement designs provide for longer pavement life, the reduced frequency of maintenance means that fewer workers are in possibly unsafe environments. Less maintenance, of course, also means less traffic control and less traffic interruption. Caltrans does not believe it is necessary to include the pavement standards in the FHWA criteria.

Caltrans also noted that the clear-zone concept is not included in the 13 controlling criteria. The clear-zone concept refers to the idea that having areas adjacent to the roadway be clear of any fixed objects is desirable for public safety. Studies show that on higher speed facilities a clear width of as much as 30 ft should be provided to allow the majority of errant vehicles to recover. This width makes a significant impact on a project, particularly when right-of-way is constrained. With such an impact and no mandatory standard in place, the clear zone provided is often below recommendations. The clear-zone concept is closely related to horizontal clearance, one of the 13 controlling criteria. The horizontal clearance criteria could possibly be expanded to include provisions for clear zone. Side slopes also are not mentioned in the 13 controlling criteria, but are certainly closely related to public safety and are part of the clear zone concept considerations.

While the Caltrans list of mandatory and advisory standards is growing, the list of most commonly requested design exceptions remains short and surprisingly does not vary substantially by route designation.

For conventional highways, the most common design exception request is for lane and shoulder widths. Many agencies have the view that wider facilities equal higher speeds. While it is increasingly perceived that providing less width for the traveling public will result in lower speeds, Caltrans believes there is actually no direct research to validate this perception. In rural areas, route consistency is often a motivating factor for the request. If a rural corridor has a limited lane and/or shoulder width for its entire length, it may not make sense to upgrade a small section. Another factor in rural areas is environmental impacts. Most projects today require a “visual assessment” as part of the environmental studies and, in many rural areas, additional paved width (particularly full shoulders) is not considered desirable. On urban routes, dense development often means making existing widths work for a project design. This is usually done by reallocating the paved width to accommodate more lanes and/or bicycle facilities.

For freeways, the most commonly requested exception is also for lane and shoulder widths, but it is usually the inside shoulder width and lane width for which Caltrans reviewers receive exception requests, as attempts are made to squeeze more capacity out of existing freeways.

There are those in the industry who see some of the controlling criteria as interrelated and perhaps needing clarification. For example, design speed and horizontal alignment, as well as sight distance and vertical alignment (sag vertical curves) are closely related. However, the reviewers at Caltrans are sufficiently experienced that they do not find these interrelations to be an issue. As the reviewers see it, a design speed that makes sense for a corridor should be set, and then any needed exceptions would be written for the alignment at spot locations. Rarely does Caltrans approve, or even receive, requests for exception to design speed for an entire project. Exceptions for sight distance in sag vertical curves rarely go unapproved, as this situation is easily mitigated with lighting.

While some of the 13 controlling criteria continually receive design exceptions, there are also some criteria for which Caltrans rarely to never approves exceptions. Exceptions to the standard for vertical clearance to falsework are rarely approved as reviewers feel it is too important, and there are often collisions at structures during construction. Similarly, exceptions to structural capacity are never considered or required. There is a general consensus that structural capacity does not belong in the 13 controlling criteria and should be handled independently.

3.2.2 Georgia Department of Transportation

An interview with Brent Story of the Georgia Department of Transportation (DOT) was conducted on July 26, 2011. The Georgia DOT approves approximately 120 design exceptions and design variances per year; as noted above, the term *design variance*, rather than design exception, is used in circumstances in which the federal policy on design exceptions does not apply. Georgia considers intersection skew angle as part of the controlling criteria for horizontal alignment, even though federal policy does not appear to require this. Georgia also treats intersection sight distance as a state controlling criterion; intersection sight triangles are documented graphically on plan and profile sheets in an early design stage to confirm the availability of intersection sight distance.

The most common design exceptions and design variances in Georgia are for inside shoulder widths on multilane roadways and for lateral offset on local roads.

In training design and design review staff, Georgia emphasizes that a design exception or design variance is merely a mechanism to document a deviation from a criterion, but it should not be a roadblock to design flexibility. Thus, design exceptions and design variances are not discouraged where they make engineering sense.

3.2.3 Kansas Department of Transportation

An interview with James Brewer of the Kansas DOT was conducted on July 25, 2011. The Kansas DOT approves only about three or four design exceptions per year. The Kansas DOT does its best to avoid the need for design exceptions. This is possible, perhaps, because of flatter terrain and fewer constraints than other states (at least in rural areas). The number of design exceptions might increase in the future if Kansas were to adopt a *practical design* philosophy like some other states, but they have not so far seen a need for this.

The most common design exceptions have been for cross-section width, particularly, bridge width. It does not make sense to reconstruct or replace bridges that may be within a few inches of meeting established bridge width criteria. Very few exceptions are needed for horizontal or vertical alignment.

3.2.4 Louisiana Department of Transportation and Development

Terri Monaghan from the Louisiana Department of Transportation and Development (DOTD) safety staff made a presentation and was interviewed at the HSM peer exchange on August 11, 2011. The Louisiana DOTD has adopted a policy that all statements about safety in design documents, including Stage 0 (Feasibility) documents, Stage 1 (Environmental) documents, and design exceptions, must be quantified using HSM principles or other safety analyses, as applicable. Thus, the DOTD is now requiring HSM analyses as part of the design exception process.

3.2.5 Michigan Department of Transportation

Brian Chomas of the Michigan DOT was interviewed on July 27, 2011. The Michigan DOT approves approximately 600 design exceptions per year. The most common design exceptions are for shoulder width, K values for vehicle curves (i.e., stopping sight distance), and acceleration/deceleration lengths for freeway ramps. (The latter is a state criterion, rather than one of the 13 controlling criteria.)

With respect to design speed, the FHWA Michigan Division Office allows design speeds equal to posted speeds, while the Michigan DOT prefers to use a design speed equal to the posted speed plus 5 mph.

The Michigan DOT does not yet utilize the HSM in justifying design exceptions, but every design exception includes a crash analysis.

3.2.6 Minnesota Department of Transportation

An interview with James Rosenow of the Minnesota DOT was conducted on July 27, 2011, and, as a follow-up, Mr. Rosenow provided a copy of the state policy on design standards and exceptions and the data shown in Table 49. The table shows the number of design exceptions requested by the Minnesota DOT during the years 2004 to 2010, inclusive.

To keep the preparation of design exceptions from becoming too burdensome, the Minnesota DOT has developed sets of standard language that can be used, where appropriate, in documentation of design exceptions for particular design elements. This approach helps to keep documentation more uniform and to prevent the burden of preparing design exception documentation from becoming a barrier to the use of appropriate design exceptions.

Table 49. Number of design exceptions requested by the Minnesota DOT (2004 to 2010).

Design element	Number of design exceptions	Percentage of design exceptions
Shoulder width[a]	78	24.9
Bridge shoulder width[b]	46	14.7
Horizontal alignment	36	11.5
Sag vertical curves	26	8.3
Normal cross slope	25	8.0
Lane width	22	7.0
Vertical clearance	22	7.0
Stopping sight distance[c]	17	5.4
Ramp length[d]	16	5.1
Grades	6	1.9
Type of bridge railing[d]	5	1.6
Lateral offset	4	1.3
Design speed	3	1.0
Crest vertical curves	3	1.0
Superelevation	3	1.0
Structural capacity	1	0.3
TOTAL	313	

[a] Forty-one (41) design exceptions for right shoulder and 37 for left shoulder.
[b] Twenty-three (23) design exceptions for right bridge shoulder and 23 for left bridge shoulder.
[c] Horizontal stopping sight distance only.
[d] Not one of the 13 controlling criteria.

3.2.7 Missouri Department of Transportation

Jonathan Nelson from the Missouri DOT (MoDOT) safety staff made a presentation and was interviewed at the HSM peer exchange on August 11, 2011. MoDOT has made extensive use of the *practical design* philosophy; design exceptions are used to document and justify deviations from the controlling criteria. In June 2011, MoDOT adopted a new policy in their *Engineering Policy Guide* that addresses safety analysis for design exceptions, incorporating requirements for use of both crash analysis and HSM analysis, as follows:

> If the design exception request involves any features that are safety related, then sufficient accident data and history is attached to the request to support the reasons for justification. A summary report of the accident information is acceptable if the volume of the data is excessive. Examples of safety related features are included in, but not limited to, the following list: lane width, shoulder width, shoulder type, rumble strips, turn lanes, bridge width, bridge approach rail, horizontal alignment, vertical alignment, grade horizontal clearance, vertical clearance, guardrail, etc. Any other items that may be perceived as a safety concern will also follow these requirements.
>
> In addition, if the design exception request involves safety related features that are adequately addressed in the AASHTO *Highway Safety Manual*, then documentation of the exception should include a safety analysis as described in the manual. In general, this safety analysis should compare the expected number of crashes for the facility with the design exception to the expected number of crashes of the facility without the design exception. Currently, not all safety related features are explicitly addressed in the *Highway Safety Manual.* A list of features currently addressed by the manual include: lane width, shoulder width, shoulder type, center line rumble strips, horizontal alignment (length, radius), grade roadside hazard rating, fixed objects, driveway density, median width sideslope, lighting, intersection skew angle and turn lanes. Not all features in the manual are addressed for each facility type.

Since this policy is very new, there is little experience with it to date.

3.2.8 Oregon Department of Transportation

Kent Belleque of the Oregon DOT was interviewed on July 27, 2011, and again on August 17, 2011. The Oregon DOT approves approximately 200 design exception elements per year (including RRR projects); multiple exceptions may be approved for a given project. The most common exceptions are for shoulder width, lane width, and clear zones. (It should be noted that the latter item, clear zones, is not one of the 13 controlling criteria; however, Oregon has established the clear-zone guidelines of the AASHTO *Roadside Design Guide* (*39*, *40*) as an internal policy for which design exceptions are required.) Oregon has also added a 6-ft sidewalk width to the controlling criteria and seeks exceptions, where needed, to utilize the minimum width of 5 ft in the U.S. Access Board's proposed Public Rights of Way Guidelines (PROWAG). Oregon uses a centralized review committee to evaluate each proposed design exception prior to requesting approval from the Chief Engineer. Legislation enacted at the request of the trucking industry requires review of design exceptions that would restrict trucks by a Motor Carrier Committee.

Oregon uses the practical design philosophy and has adopted a *Practical Design Guide*. Oregon does not yet use the HSM in justifying design exceptions.

3.2.9 Tennessee Department of Transportation

An interview with Jeff Jones of the Tennessee DOT was conducted on July 26, 2011. The Tennessee DOT approves approximately 10 to 12 design exceptions per year. Design exceptions might be used more often in Tennessee, but the onerous nature of the design exception process has become a barrier to the use of design exceptions. For example, FHWA is requiring design exception reports to include the cost of complying with the controlling criteria, even when this cost is very expensive to estimate and would be impractical to build.

The most common design exceptions in Tennessee are for shoulder widths on existing roads originally built with design criteria lower than current criteria, including bridge widths, and vertical alignment, particularly crest vertical curves.

3.2.10 Virginia Department of Transportation

Interviews were conducted with Bart Thrasher of the Virginia DOT (VDOT)on July 26, 2011, and with Theoron Knouse of VDOT on August 11, 2011. VDOT approves approximately 20 to 40 design exceptions per year, including both NHS and non-NHS facilities. The most common design exceptions are for shoulder width and horizontal alignment. (It should be kept in mind that VDOT maintains much of the local rural highway system that would be under county jurisdiction in other states.) Virginia has begun using the HSM in design exception analyses. A VDOT representative stated that use of the HSM has modified the mitigation strategy for nearly every project to which it has been applied.

SECTION 4

Expanded Traffic Operational and Safety Knowledge Concerning the 13 Controlling Criteria

This section of the report presents the results of research performed as part of NCHRP Project 17-53 to expand knowledge of the traffic operational and safety effects of the 13 controlling criteria. These research results have also been incorporated in the summary of traffic operational and safety effects presented in Section 2 of this report.

4.1 Operational Effects of Lane Width on Urban and Suburban Arterials

Field studies were conducted to determine the effect of lane width on traffic speeds on urban and suburban arterials using a field study procedure similar to that used by Potts et al. (*23*, *24*). The research team identified lane-width transitions on urban and suburban arterials in three geographic regions of the United States, as shown in Table 50, and collected speed data upstream and downstream of these lane-width transitions for comparison. The wider and narrower road sections were on the same roadway, with essentially the same traffic volume, and were generally located within 2 mi of one another. The driver populations were essentially identical between the measurement locations on wider and narrower roadways. In some cases, a signalized intersection was located between the two measurement locations, but both measurement locations were located far enough from the signal that the presence of the signal would not influence traffic speeds.

During field data collection, speeds of approximately 120 unimpeded vehicles were measured at each upstream and downstream location. Vehicle speeds were measured with lidar-based Kustom ProLaser speed guns. The measurement sites were sufficiently far removed from the lane-width transition that no substantial accelerations or decelerations due to the change in lane width were taking place. Following data collection, four sites in the East region of the United States were excluded from the analysis because field measurements of lane width indicated that the "wide" section of roadway had lane widths less than 11 ft. The research team determined that these should be classified as narrow lanes and that the change in lane width on these sections was actually from narrow to narrower, rather than from wide to narrow.

The posted speed limit at the sites used in the analysis ranged from 35 to 45 mph.

Lane width and speed statistics based on the 19 pairs of sites on adjacent wide and narrow roadway segments are presented in Table 51, according to geographic region. Average lane width for both narrow and wide sites and the average of their paired differences were calculated across all sites within a geographic region. The statistics for each region were then averaged to obtain overall lane width statistics. Speed statistics were calculated in a similar way and are shown in the right-hand half of Table 51.

The pairwise speed differences in mean speeds between narrow and wide roadway segments were analyzed by means of an analysis of variance (ANOVA) mixed model considering the following factors: lane width (narrow or wide), change sequence (narrow to wide or wide to narrow), and region. The 19 sites were treated as a random blocking factor. All two-way interactions and the three-way interactions were considered as well. The ANOVA results showed that of all the factors and interactions included in the model, only region had a statistically significant effect ($p = 0.0001$) on the speed change due to lane-width change (this is also evident in the last column of Table 51). A final model to estimate the effect of lane width on speed was evaluated after taking into account the regional effect. Lane width was not statistically significant ($p = 0.97$). The estimated difference in mean speed between wide and narrow lanes across all sites in the three regions is 0.2 mph with a 95-percent confidence interval of –0.96 to 0.99 mph.

Thus, the analysis of the speed data collected in this research on pairs of sites on wide and narrow roadway segments shows no statistically significant effect of urban and suburban arterial lane width on traffic speed. In fact, the average speed difference between wide and narrow lanes across all three regions

Table 50. Number of sites by geographic region for the evaluation of the operational effect of lane width on urban and suburban arterials.

Region	Site locations	Number of sites[a]
Midwest	Kansas City, Missouri Columbia, Missouri St. Louis, Missouri	5
East	Raleigh, North Carolina Greensboro, North Carolina Wilson, North Carolina	9[b]
West	Phoenix, Arizona	9

[a] A site is defined as a specific lane-width transition location in one direction of travel. Therefore, if the location was suitable for data collection in both directions of travel, that location provided two study sites.
[b] Four sites determined to be unsuitable for analysis were excluded.

Table 51. Mean speed difference for pairs of wide and narrow roadway segments on urban and suburban arterials.

Region	Number of sites	Average lane width (ft)		Average lane-width difference (ft) (wide - narrow)	Number of speed measurements		Average speed (mph)		Average difference in mean speed (mph) (wide - narrow)
		Narrow	Wide		Narrow	Wide	Narrow	Wide	
East	5	9.5	12.0	2.5	599	600	41.5	40.6	–0.9
Midwest	5	9.0	11.5	2.5	600	600	38.9	38.6	–0.3
West	9	10.0	12.0	2.0	1,080	1,080	45.1	45.8	0.7
All	19	9.5	11.8	2.3	2,279	2,280	41.8	41.7	–0.1

is nearly zero. By contrast, previous research conducted by Potts et al. (*23*, *24*) under NCHRP Project 03-72 (which included only five site pairs) found a statistically significant average speed difference of 4 mph between arterials with narrow and wide lanes. No explanation for the difference in results between the research conducted under NCHRP Project 17-53 and previous research has been found, but the reported results of the NCHRP Project 17-53 research are more credible because they are based on a larger sample size and a broad geographical distribution of sites.

4.2 Safety Effects of Lane Width on Urban and Suburban Arterials

A further review of available information on the safety effect of lane width was conducted. The HSM does not include any CMF for lane width on urban and suburban arterials. Research conducted by Potts et al. (*23*, *24*) in NCHRP Project 03-72 found that under a broad range of conditions, lane width on urban and suburban arterials has little or no effect on safety. Analysis of geometric design, traffic volume, and accident data collected by Potts et al. found that, with limited exceptions, there is no consistent, statistically significant relationship between lane width and safety for midblock sections of urban and suburban arterials. In general, there is no indication that the use of 10- or 11-ft lanes, rather than 12-ft lanes, for arterial midblock segments leads to increases in crash frequency. There are situations in which use of narrower lanes may provide benefits in traffic operations, pedestrian safety, and/or reduced interference with surrounding development and may provide space for geometric features that enhance safety such as medians or turn lanes. The analysis results indicate that narrow lanes can generally be used to obtain these benefits without compromising safety.

Several caveats in the preceding results should be noted. First, the data from one of the two states included in the analysis showed an increase in crash rates for four-lane undivided arterials with lane widths of 10 ft or less, while the data from another state showed an increase in crash rates for four-lane divided arterials with lane widths of 9 ft or less. While the results from each state were not confirmed in data from the other state, the findings indicate that lane widths of 10 ft or less on four-lane undivided arterials and lane widths of 9 ft or less on four-lane divided arterials should be used cautiously unless local experience indicates otherwise. Second, lane widths less than 12 ft should be used cautiously where substantial volumes of bicyclists share the road with motor vehicles, unless an alternative facility for bicycles such as a wider curb lane or paved shoulder is provided. Third, lane widths less than 12 ft should be used cautiously on streets with substantial truck and bus volumes; in particular, mirror overhang from heavy vehicles is an issue where roadside objects are close to the road.

Based on the available information, a CMF of 1.0 for a lane width of 10 ft or more on urban and suburban arterials seems reasonable, accompanied with design guidance emphasizing the importance of bicycle and heavy vehicle considerations.

4.3 Safety Effects of Bridge Width on Rural Two-Lane Highways

Existing guidance suggests that the known safety effects for lane and shoulder width (i.e., the safety effects of lane and shoulder width specified in the HSM) should be applied to bridge width. However, it seems likely that the safety effects of bridge width would be different from the effects of lane and shoulder width on an open roadway section given that, on a bridge, there is always a roadside obstacle (the bridge rail) at the outside edge of the shoulder or just beyond any sidewalk present. Given the high expense associated with widening or replacing a bridge, designers need better information about the safety effects of bridge width.

The most appropriate measures to represent the "narrowness" of a bridge are the total lane-plus-shoulder width on the bridge (i.e., curb-to-curb or rail-to-rail width) and the difference between the total lane-plus-shoulder width on the bridge and the total lane-plus-shoulder width on the bridge approach. This latter measure is referred to as the bridge-width difference.

Using available data from state highway agencies, the National Bridge Inventory (NBI), and the FHWA Highway Safety Information System (HSIS), the research team assembled databases for bridges on two-lane highways in California and Washington. The analysis included all bridges on two-lane rural highways except the following:

- Bridges longer than 200 ft
- One-lane bridges
- Bridges with at least one approach that included an intersection within 500 ft of the bridge
- Bridges with approach widths that are not equal to each other (i.e., the approach on one side of the bridge has a different width than the approach on the other side of the bridge)

Summary statistics for the California bridge data included in the analysis, categorized by bridge-width difference, are shown in Table 52. The table also provides the average AADT for the bridges in each bridge-width difference category, the number of crashes that occurred during a 5-year period (2004 to 2008), and the average crash rate for the bridges in each category, presented in terms of crashes per million veh-mi of travel (MVMT).

The data in Table 52 indicate that bridges that are only slightly narrower than their approaches experience about the same crash frequency as bridges for which the approach and bridge width are the same. Bridges with bridge-width difference in the range from 2 to 10 ft experience more crashes than bridges with the same approach and bridge widths, while bridges with bridge-width differences greater than 10 ft experience fewer crashes than bridges with the same approach and bridge widths. Total and fatal-and-injury crash frequencies per mile per year were each analyzed using a negative binomial regression model that included AADT and bridge-width difference. The analyses showed that while AADT is highly significant ($p < 0.0001$), the safety effect of bridge-width difference is not statistically significant either for total crashes ($p = 0.14$) or for fatal-and-injury crashes ($p = 0.20$).

Summary statistics for the Washington bridge data included in the analysis, categorized by bridge-width difference, are shown in Table 53. The table also provides the average AADT for the bridges in each bridge-width difference category, the number of crashes that occurred during a 5-year period (2004 to 2008), and the average crash rate for the bridges in each category, presented in terms of crashes per MVMT.

The differences in crash rates between the bridge-width difference categories for Washington bridges have a pattern that is different from that found for California bridges. However, as in the case of the California bridges, the safety effect of bridge-width difference for Washington bridges is not statistically significant either for total crashes ($p = 0.79$) or for fatal-and-injury crashes ($p = 0.84$).

In summary, there is no evidence of a statistically significant effect of bridge-width difference on crash frequency for either total crashes or for fatal-and-injury crashes in either the California or the Washington data, after accounting for the effect of AADT. In fact, for several of the bridge-width difference categories, bridges with widths narrower than the approach width appear to have fewer crashes than bridges with widths equal to the approach widths, although such

Table 52. Summary statistics for selected California bridges on rural two-lane highways.

Bridge-width difference (ft)	Number of bridges	Number of crashes (2004–2008)		Average AADT	MVMT (2004–2008)	Crash rate (per MVMT)	
		Total	Fatal and injury			Total	Fatal and injury
0	458	546	227	5,394	494	1.11	0.46
> 0 to ≤ 2	50	37	17	4,172	41	0.90	0.41
> 2 to ≤ 5	42	65	24	5,431	45	1.44	0.53
> 5 to ≤ 10	48	63	26	5,130	50	1.26	0.52
> 10	26	19	5	5,945	31	0.61	0.16

Table 53. Summary statistics for selected Washington bridges on rural two-lane highways.

Bridge-width difference (ft)	Number of bridges	Number of crashes (2004–2008)		Average AADT	MVMT (2004–2008)	Crash rate (per MVMT)	
		Total	Fatal and injury			Total	Fatal and injury
0	122	164	66	2,992	76	2.16	0.87
> 0 to ≤ 2	74	84	32	2,906	43	1.95	0.74
> 2 to ≤ 5	48	63	19	2,912	29	2.17	0.66
> 5 to ≤ 10	70	92	37	3,572	53	1.74	0.70
> 10	23	42	15	3,748	19	2.21	0.79

observed differences are not statistically significant. Thus, there is no evidence that there would, in general, be any documentable safety benefit from widening a rural two-lane highway bridge with a roadway narrower than its approach.

Additional analyses were conducted to examine possible effects of bridge width and bridge length on the safety effect of bridge-width difference, but no statistically significant effects were found.

4.4 Operational Effects of Horizontal Alignment on Rural Multilane Divided Highways and Urban and Suburban Arterials

Field data were collected to investigate the operational effect of horizontal curve radius for rural multilane highways and urban and suburban multilane highways. The number of rural and suburban sites where speed data were collected in each of three regions of the country (the Midwest, the Eastern United States, and the Western United States) is shown in Table 54. Data are available for a total of 28 rural sites and 31 suburban sites.

Data collection efforts were similar to those described in Section 4.1 of this report. Speeds of approximately 120 unimpeded vehicles were measured at a position upstream of the curve and a position close to the middle of the curve for each direction of travel. Vehicle speeds were measured with lidar-based Kustom ProLaser speed guns. The upstream measurement sites were sufficiently far from the curve entrance that vehicles would not yet have begun to slow for the curve.

Descriptive statistics are presented in Table 55 for rural multilane highways and in Table 56 for urban and suburban arterials.

Models were developed to predict mean vehicle speed on a horizontal curve as a function of the mean vehicle speed on the tangent approach to the curve and the curve radius separately for rural multilane highways and for urban and suburban arterials. The pairwise speed differences in mean speeds between tangent approach and curve were analyzed using an ANOVA mixed model without intercept, considering the following factors: inverse of radius, region (East, Midwest, or West), and presence of median (divided or undivided). The sites were treated as a random blocking factor. The interaction between region and presence of median was not included due to missing combinations. The ANOVA results showed that neither region nor presence of median was statistically significant for either roadway type (p-values ranged from 0.24 to 0.80). The final models included only one statistically significant variable, the inverse of radius ($p < 0.0001$). The final mean speed model for curves on rural multilane highways is shown in Equation 39; and the model for curves on suburban arterials is shown in Equation 40:

- For rural multilane highways,

$$\text{Speed}_{\text{curve}} = \text{Speed}_{\text{approach}} - \frac{3{,}136}{R} \tag{39}$$

- For urban and suburban arterials,

$$\text{Speed}_{\text{curve}} = \text{Speed}_{\text{approach}} - \frac{2{,}303}{R} \tag{40}$$

Table 54. Number of rural and suburban data collection sites for vehicle speeds on horizontal curves by region.

Region	Site locations	Number of rural curve sites[a]	Number of suburban curve sites[a]
Midwest	Kansas City metropolitan area, Missouri/Kansas	9	10
East	North Carolina and Virginia	9	11
West	California and Nevada	10	10

[a] A site is defined as one direction of travel along a curve. Therefore, if the curve was suitable for data collection in both directions of travel, that location provided two study sites.

Table 55. Mean speeds on tangents and curves for rural multilane highways.

Site group	Mean curve radius (ft)	Speed limit range (mph)	Mean total lane width (ft)	Number of curves	Mean tangent speed (mph)	Mean curve speed (mph)	Mean speed difference (mph)
Divided routes							
East	2,179	55 to 70	23.2	9	62.2	61.0	1.2
Midwest	2,368	65 to 70	23.3	9	66.4	65.6	0.8
West	2,060	60 to 65	22.9	8	65.6	63.4	2.2
Undivided routes							
West	1,435	55	22.7	2	54.0	50.5	3.5

Table 56. Mean speeds on tangents and curves for urban and suburban arterials.

Site group	Mean curve radius (ft)	Speed limit range (mph)	Mean total lane width (ft)	Number of curves	Mean tangent speed (mph)	Mean curve speed (mph)	Mean speed difference (mph)
Divided routes							
East	1,448	35 to 55	23.9	7	50.6	49.3	1.3
West	935	30 to 50	21.2	5	43.5	41.7	1.8
Undivided routes							
East	841	35 to 45	20.1	3	46.1	42.6	3.5
Midwest	674	35 to 40	23.4	10	40.0	36.8	3.2
West	1,032	35 to 50	23.5	5	44.2	41.0	3.2

where

$Speed_{curve}$ = Speed of vehicle on horizontal curve (mph)
$Speed_{approach}$ = Speed of vehicle on tangent approaching curve (mph)
R = Curve radius (ft)

For the rural multilane highway model in Equation 39, the coefficient 3,136 has a standard error of 602.7; for the urban and suburban arterial model in Equation 40, the coefficient 2,303 has a standard error of 268.7.

Figure 8 compares the two models. For both models, as the curve radius decreases, the difference between the tangent

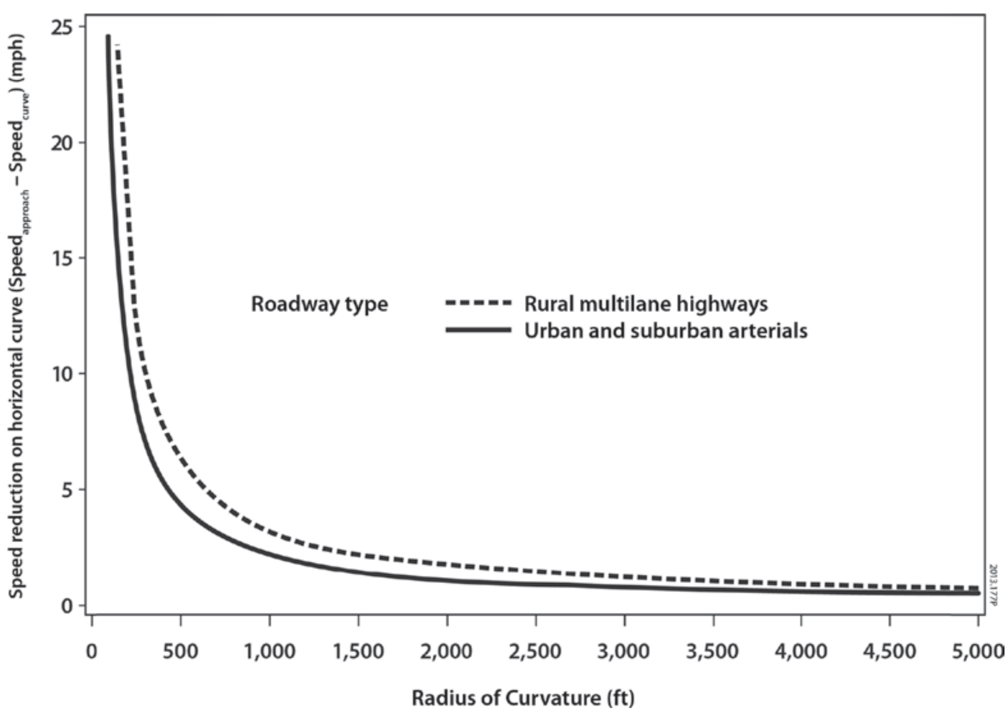


Figure 8. Comparison of models for speed reduction on horizontal curves on rural multilane highways and urban and suburban arterials.

speed and curve speed increases. In other words, drivers have to reduce their speed to navigate the curve more for sharper curves than for flatter curves. This result is consistent with expectations and with previous modeling for rural two-lane highways. In addition, these models tell us that curves on suburban arterials must be sharper than curves on rural multilane highways to cause a reduction in speed of a given magnitude. This makes engineering sense given that tangent speeds are lower on suburban arterials than on rural multilane highways.

4.5 Safety Effects of Horizontal Alignment on Rural Freeways and Rural Multilane Highways

An analysis of horizontal alignment data for rural multilane highways and rural freeways in Washington was conducted to develop relationships between horizontal curve radius and length and crash frequency and severity. This analysis used an approach similar to recent analyses conducted for FHWA by Bauer and Harwood (*31*).

Of the 6,944 mi of roadway in the Washington HSIS database, 212.1 mi (3.1 percent) are on rural multilane highways and 466.3 mi (6.7 percent) on rural freeways. Of these, 182.5 mi of rural multilane highways and 432.7 mi of rural freeways were used for analysis. Rural multilane highways and rural freeways with passing or climbing lanes and segments with missing or obviously incorrect alignment data (e.g., overlapping curves) were excluded from the study. Of the 182.5 mi of rural multilane highways, rural four-lane undivided highways represented only 6.1 mi and were therefore excluded from analysis. Thus safety effects of horizontal alignment were studied for rural four-lane divided highways and rural freeways only.

4.5.1 Descriptive Statistics for Roadway, Exposure, and Crash Data

Descriptive statistics for the rural roadway sections available for analysis including roadway length (miles), exposure (MVMT in the 6-year period from 2003 to 2008), crash frequencies, and crash rates per MVMT for each horizontal alignment are shown separately for rural four-lane divided highways and rural freeways in Table 57.

Prior to statistical modeling, the parameters of interest were assessed for extreme values (both high and low); this was done using a combination of plots of crash rates per MVMT versus selected parameters and distributions of the individual parameters. The following rules were implemented:

- Roadway segments less than 0.01 mi in length were excluded from analysis (such short segments are unlikely to be useful analysis sections.)
- Horizontal curves with a curve radius exceeding 11,460 ft were included in the analysis but their radius was set at 11,460 ft.
- Horizontal curves with a radius less than 100 ft were included in the analysis but their radius was set at 100 ft, based on guidance in HSM Chapter 10.

Table 57. Descriptive statistics by horizontal alignment type in available data from the Washington HSIS database (2003 to 2008).

Horizontal alignment	Rural four-lane divided highways	Rural freeways
ROADWAY LENGTH (MI)		
Tangent	122.4	306.5
Curve	54.0	126.2
Total	176.4	432.7
EXPOSURE (MVMT)		
Tangent	3,648	17,534
Curve	1,588	6,825
Total	5,236	24,359
FATAL-AND-INJURY CRASH FREQUENCIES IN 6 YEARS		
Tangent	865	2,717
Curve	353	1,321
Total	1,218	4,038
PROPERTY-DAMAGE-ONLY CRASH FREQUENCIES IN 6 YEARS		
Tangent	1,403	5,419
Curve	621	2,405
Total	2,024	7,824
TOTAL CRASH FREQUENCIES IN 6 YEARS		
Tangent	2,268	8,136
Curve	974	3,726
Total	3,242	11,862
FATAL-AND-INJURY CRASH RATE PER MVMT		
Tangent	0.237	0.155
Curve	0.222	0.194
PROPERTY-DAMAGE-ONLY CRASH RATE PER MVMT		
Tangent	0.385	0.309
Curve	0.391	0.352
TOTAL CRASH RATE PER MVMT		
Tangent	0.622	0.464
Curve	0.614	0.546

4.5.2 Models Developed for Horizontal Curves on Rural Four-Lane Divided Highways and Rural Freeways

Separate models were developed for fatal-and-injury and property-damage-only crashes. Tangents served as the base condition in all models. The parameters considered in each model included the following:

- AADT (averaged across all 6 years)
- Segment length (offset—used to estimate crashes per mile)
- Horizontal curve length
- Horizontal curve radius

The final crash prediction models for fatal-and-injury and property-damage-only crashes for horizontal curves on rural four-lane divided roadways and rural freeways are the following:

$$N_{FI} = \exp\begin{bmatrix} b_0 + b_1 \ln(AADT) + b_2 L_C \times I_{HC} \\ + b_3 \ln\left(2 \times \frac{5730}{R}\right) \times I_{HC} \end{bmatrix} \quad (41)$$

$$N_{PDO} = \exp\begin{bmatrix} b_0 + b_1 \ln(AADT) + b_2 L_C \times I_{HC} \\ + b_3 \ln\left(2 \times \frac{5730}{R}\right) \times I_{HC} \end{bmatrix} \quad (42)$$

where

N_{FI} = fatal-and-injury crashes/mi/yr
N_{PDO} = property-damage-only crashes/mi/yr
AADT = veh/day
R = curve radius (ft); missing for tangents
I_{HC} = horizontal curve indicator: 1 for horizontal curves; 0 otherwise
L_C = horizontal curve length (mi); not applicable for tangents
ln = natural logarithm function
$b_0, \ldots, b_3$ = regression coefficients

The regression results, including the coefficient estimate, dispersion parameter, standard error, confidence limit, chi-square statistic, and significance level for all statistically significant parameters, are shown as follows:

- Fatal-and-injury crashes on rural four-lane divided highways in Table 58
- Property-damage-only crashes on rural four-lane divided highways in Table 59
- Fatal-and-injury crashes on rural freeways in Table 60
- Property-damage-only crashes on rural freeways in Table 61

Table 58. Fatal-and-injury crash modeling results for horizontal curves on rural four-lane divided highways.

Parameter	Estimate	Standard error	95% Lower confidence limit	95% Upper confidence limit	Chi-square	Significance level
Intercept	–4.19	0.93	–6.02	–2.37	n/a	n/a
ln(AADT)	0.47	0.10	0.28	0.66	22.48	< .001
Horizontal curve length (mi)	–0.87	0.27	–1.39	–0.35	10.20	0.001
ln(2x5730/R)	0.22	0.08	0.07	0.37	7.82	0.005
Dispersion	0.52	0.07	0.40	0.67	n/a	n/a

Table 59. Property-damage-only crash modeling results for horizontal curves on rural four-lane divided highways.

Parameter	Estimate	Standard error	95% Lower confidence limit	95% Upper confidence limit	Chi-square	Significance level
Intercept	–5.75	0.81	–7.33	–4.16	n/a	n/a
ln(AADT)	0.69	0.08	0.52	0.85	61.23	< .001
Horizontal curve length (mi)	–0.95	0.23	–1.41	–0.50	15.90	< .001
ln(2x5730/R)	0.26	0.07	0.13	0.39	15.47	< .001
Dispersion	0.44	0.05	0.35	0.56	n/a	n/a

Table 60. Fatal-and-injury crash modeling results for horizontal curves on rural freeways.

Parameter	Estimate	Standard error	95% Lower confidence limit	95% Upper confidence limit	Chi-Square	Significance level
Intercept	–7.56	0.42	–8.39	–6.74	n/a	n/a
ln(AADT)	0.79	0.04	0.71	0.87	295.13	< .001
Horizontal curve length (mi)	–0.40	0.14	–0.68	–0.11	7.55	0.006
ln(2x5730/R)	0.35	0.04	0.27	0.44	62.31	< .001
Dispersion	0.16	0.02	0.13	0.21	n/a	n/a

Table 61. Property-damage-only crash modeling results for horizontal curves on rural freeways.

Parameter	Estimate	Standard error	95% Lower confidence limit	95% Upper confidence limit	Chi-square	Significance level
Intercept	–8.32	0.41	–9.13	–7.52	n/a	n/a
ln(AADT)	0.93	0.04	0.85	1.01	425.91	< .001
Horizontal curve length (mi)	–0.43	0.13	–0.69	–0.16	10.01	0.002
ln(2x5730/R)	0.28	0.04	0.20	0.35	47.78	< .001
Dispersion	0.22	0.02	0.19	0.27	n/a	n/a

These results imply the following CMFs for horizontal curvature:

- For fatal-and-injury crashes on rural four-lane divided highways,

$$CMF = \exp\left[-0.87L_C + 0.22\ln\left(2\times\frac{5730}{R}\right)\right] \quad (43)$$

- For property-damage-only crashes on rural four-lane divided highways,

$$CMF = \exp\left[-0.95L_C + 0.26\ln\left(2\times\frac{5730}{R}\right)\right] \quad (44)$$

- For fatal-and-injury crashes on rural freeways,

$$CMF = \exp\left[-0.40L_C + 0.35\ln\left(2\times\frac{5730}{R}\right)\right] \quad (45)$$

- For property-damage-only crashes on freeways,

$$CMF = \exp\left[-0.43L_C + 0.28\ln\left(2\times\frac{5730}{R}\right)\right] \quad (46)$$

Equations 45 and 46 generally provide lower CMF values than Equations 22 through 25. However, because Equations 22 through 25 were developed in a more comprehensive analysis and are in the process of being approved for use in the AASHTO *Highway Safety Manual* (*12*), Equations 22 through 25 are recommended for use in preference to Equations 45 and 46.

4.6 Safety Effect of Vertical Alignment on Rural Multilane Divided Highways

An analysis of vertical alignment data for rural multilane highways in Washington was conducted to develop relationships between vertical alignment design parameters and crash frequency and severity for straight grades and vertical curves. The analysis used an approach similar to recent analyses conducted for FHWA by Bauer and Harwood (*31*). The same database that was used in estimating the safety effect of horizontal alignment on rural multilane highways in Section 4.5 was also used for this analysis. Crash data were analyzed separately for straight grades and each of the four types of vertical alignment shown in Figure 5 (see Section 2.7).

4.6.1 Descriptive Statistics for Roadway, Exposure, and Crash Data

As discussed previously, only rural four-lane divided highways were considered. Descriptive statistics for the rural roadway sections available for analysis including roadway length (miles), exposure (MVMT in the 6-year period from 2003 to 2008), crash frequencies, and crash rates per MVMT for each vertical alignment are shown in Table 62.

Table 62. Descriptive statistics by vertical alignment type in available data from the Washington HSIS database for rural four-lane divided highways (2003 to 2008).

			6-year crash frequencies			Crash rate (per MVMT)		
Vertical alignment	Roadway length (mi)	Exposure (MVMT)	Fatal and injury	Property damage only	Total	Fatal and injury	Property damage only	Total
Straight grade	107.6	3,217	740	1,273	2,013	0.230	0.396	0.626
Type 1 Crest	19.9	637	125	229	354	0.196	0.360	0.556
Type 2 Crest	20.1	521	97	162	259	0.186	0.311	0.497
Type 1 Sag	12.8	393	108	177	285	0.275	0.451	0.726
Type 2 Sag	16.0	469	148	183	331	0.316	0.390	0.706
Total	**176.4**	**5,237**	**1,218**	**2,024**	**3,242**	**n/a**	**n/a**	**n/a**

Prior to statistical modeling, the parameters of interest were assessed for extreme values (both high and low); this was done using a combination of plots of crash rates per MVMT versus selected parameters and distributions of the individual parameters. The following rules were implemented:

- Roadway segments less than 0.01 mi in length were excluded from analysis (such short segments are unlikely to be useful analysis sections)
- For Type 1 crest and Type 1 sag vertical curves, segments where both initial (G_1) and final (G_2) grades were, in absolute value, less than 1 percent were excluded (such minor vertical curves are very close to being level)
- For Type 2 crest and Type 2 sag vertical curves, segments where A, the algebraic difference between G_1 and G_2 (equivalent to abs $[G_1 - G_2]$),was less than 1 percent were excluded (such minor vertical curves are very close to being straight grades)
- All records with K exceeding 2,500 were excluded (these are typically long vertical curves with small grade changes and could be classified as straight grades).

Key design parameters for vertical curves include the following:

- Algebraic difference in grade
- Length of curve
- Ratio of algebraic difference in grade and length of curve (K), which represents the sharpness of the vertical curve

4.6.2 Models Developed for Vertical Curves on Rural Four-Lane Divided Highways

The parameters considered in each model may include the following:

- AADT (averaged across all 6 years)
- Segment length (offset—used to estimate crashes per mile)
- Absolute value of percent grade (straight-grade models only)
- Vertical curve length
- A, the algebraic difference between the initial and final grades
- K, a measure of the sharpness of vertical curvature;

$$K = \frac{\text{Vertical curve length}}{A}$$

Separate models were developed for straight, crest, and sag vertical curves and for fatal-and-injury and property-damage-only crashes. Level roadway segments (i.e., abs(grade) < 1 percent) served as the base condition in all models.

Straight-Grade Models on Rural Four-Lane Divided Highways

The final crash prediction models for fatal-and-injury and property-damage-only crashes for straight grades on rural four-lane divided roadways are the following:

$$N_{FI} = \exp[b_0 + b_1 \ln(AADT) + b_2 L_{VC}] \quad (47)$$

$$N_{PDO} = \exp[b_0 + b_1 \ln(AADT) + b_2 L_{VC} + b_3 \text{Grade}] \quad (48)$$

where

N_{FI} = fatal-and-injury crashes/mi/yr
N_{PDO} = property-damage-only crashes/mi/yr
AADT = veh/day
L_{VC} = vertical curve length (mi)
Grade = absolute value of percent grade for non-level grades; 0 for level grades
ln = natural logarithm function
$b_0, \ldots, b_3$ = regression coefficients

Note that grade was not significant in the fatal-and-injury crash model ($p = 0.22$) and was therefore excluded from the model in Equation 47.

The regression results, including the coefficient estimate, dispersion parameter, standard error, confidence limit, chi-square-statistic, and significance level for all statistically significant parameters are shown as follows:

- Fatal-and-injury crashes in Table 63
- Property-damage-only crashes in Table 64

Table 63. Fatal-and-injury crash modeling results for straight grades on rural four-lane divided highways.

Parameter	Estimate	Standard error	95% Lower confidence limit	95% Upper confidence limit	Chi-square	Significance level
Intercept	–7.47	1.10	–9.63	–5.31	n/a	n/a
ln(AADT)	0.81	0.12	0.59	1.04	46.81	< .001
Vertical curve length (mi)	–0.34	0.17	–0.68	–0.01	3.67	0.056
Dispersion	0.60	0.10	0.44	0.83	n/a	n/a

Table 64. Property-damage-only crash modeling results for straight grades on rural four-lane divided highways.

Parameter	Estimate	Standard error	95% Lower confidence limit	95% Upper confidence limit	Chi-square	Significance level
Intercept	–7.77	0.92	–9.57	–5.97	n/a	n/a
ln(AADT)	0.90	0.10	0.71	1.08	80.42	<.001
Vertical curve length (mi)	–0.38	0.15	–0.67	–0.08	5.53	0.019
Abs(Percent grade)	0.11	0.03	0.04	0.18	9.87	0.002
Dispersion	0.47	0.07	0.36	0.62	n/a	n/a

It is disappointing that the percent-grade variable was statistically significant in the model for property-damage-only crashes, but not for fatal-and-injury crashes. With this inconsistency, it does not appear that the models presented in Tables 61 and 63 can be used to represent the effect of percent grade on crashes for rural multilane divided highways.

Vertical Curve Models for Rural Four-Lane Divided Highways

Models including either algebraic difference in grade (A) and vertical curve length (L_{VC}), or simply their ratio, K, in addition to ln(AADT) showed that neither A nor K were statistically significant at the 10-percent significance level, across all vertical curve and crash types. As a result, the final models included ln(AADT) and vertical curve length only.

The final crash prediction models for fatal-and-injury and property-damage-only crashes at Type 1 crest, Type 1 sag, Type 2 crest, and Type 2 sag vertical curves on rural four-lane divided highways are the following:

$$N_{FI} = \exp[b_0 + b_1 \ln(AADT) + b_2 L_{VC}] \quad (49)$$

$$N_{PDO} = \exp[b_0 + b_1 \ln(AADT) + b_2 L_{VC}] \quad (50)$$

where

N_{FI} = fatal-and-injury crashes/mi/yr
N_{PDO} = Property-damage-only crashes/mi/yr
AADT = veh/day
L_{VC} = vertical curve length (mi)
ln = natural logarithm function
$b_0, \ldots, b_2$ = regression coefficients

The regression results, including the coefficient estimate, dispersion parameter, standard error, confidence limit, chi-square statistic, and significance level for all statistically significant parameters, are shown in Table 65. The table is organized by crash type within vertical grade type. As neither A nor K was statistically significant in these models, it does not appear that they can be used to predict the effects of vertical curve design on crash frequency.

4.7 Safety Effects of Stopping Sight Distance at Crest Vertical Curves on Rural Two-Lane Highways

The effect of stopping sight distance on crash frequency and severity for rural two-lane highways was evaluated by comparing the safety performance of vertical curves with stopping sight distance less than AASHTO design criteria to vertical curves with stopping sight distance greater than AASHTO design criteria. The research team reviewed vertical profile data for Type 1 crest vertical curves on rural two-lane highways in Washington using data available from HSIS. Type 1 crest vertical curves are hillcrests with an upgrade on the approach to the crest and a downgrade on the departure roadway (see Figure 5 in Section 2.7). For each Type 1 crest vertical curve, the AASHTO stopping sight distance was calculated and compared to the actual stopping sight distance available at each curve to categorize each curve as either greater or less than AASHTO SSD criteria. Each vertical curve was reviewed in videolog data (and a sample was reviewed in the field) to verify the accuracy of the curve length and algebraic difference in grade data used to compute stopping sight distance. The research team also reviewed videolog data for each curve to identify whether there were horizontal curves, intersections, or driveways within or near the vertical curve. In addition to the presence of these features, it was noted whether the feature was hidden from the view of an approaching driver by the presence of the crest vertical curve. Crash data were obtained for each crest vertical curve and for an additional 0.1 mi of roadway at each end of the vertical curve.

Observed crash rates per MVMT are shown in Table 66 for Type 1 crest vertical curves with and without horizontal curves, intersections, or driveways present. Table 66 also includes basic site descriptives (number of sites and site length), 5-year crash frequencies, average AADT, and exposure. Table 67 has a similar format but addresses crash rates for vertical curves with hidden horizontal curves, intersections, or driveways (i.e., curves, intersections, or driveways that are not visible to an approaching driver because of the presence of the crest vertical curve). Crash rates in the top

Table 65. Fatal-and-injury and property-damage-only crash modeling results for vertical curves on rural four-lane divided highways.

Parameter	Estimate	Standard error	95% Lower confidence limit	95% Upper confidence limit	Chi-square	Significance level
Fatal-and-Injury Crashes per Mile per Year—Type 1 Crest Vertical Curves and Level Roadways						
Intercept	–6.96	1.40	–9.70	–4.21	n/a	n/a
ln(AADT)	0.76	0.15	0.48	1.05	25.35	<.001
Vertical curve length (mi)	–0.47	0.18	–0.82	–0.12	6.22	0.013
Dispersion	0.68	0.12	0.48	0.97	n/a	n/a
Property-Damage-Only Crashes per Mile per Year—Type 1 Crest Vertical Curves and Level Roadways						
Intercept	–9.68	1.16	–12.0	–7.41	n/a	n/a
ln(AADT)	1.11	0.12	0.87	1.34	74.36	<.001
Vertical curve length (mi)	–0.46	0.16	–0.77	–0.15	7.29	0.007
Dispersion	0.47	0.08	0.34	0.64	n/a	n/a
Fatal-and-Injury Crashes per Mile per Year—Type 1 Sag Vertical Curves and Level Roadways						
Intercept	–9.02	1.42	–11.8	–6.23	n/a	n/a
ln(AADT)	0.99	0.15	0.70	1.28	40.74	<.001
Vertical curve length (mi)	–0.54	0.18	–0.88	–0.19	7.91	0.005
Dispersion	0.66	0.12	0.46	0.94	n/a	n/a
Property-Damage-Only Crashes per Mile per Year—Type 1 Sag Vertical Curves and Level Roadways						
Intercept	–10.8	1.19	–13.1	–8.49	n/a	n/a
ln(AADT)	1.23	0.12	0.98	1.47	86.15	<.001
Vertical curve length (mi)	–0.47	0.16	–0.79	–0.16	7.56	0.006
Dispersion	0.48	0.08	0.34	0.66	n/a	n/a
Fatal-and-Injury Crashes per Mile per Year—Type 2 Crest Vertical Curves and Level Roadways						
Intercept	–7.18	1.41	–9.94	–4.42	n/a	n/a
ln(AADT)	0.78	0.15	0.49	1.07	26.12	<.001
Vertical curve length (mi)	–0.38	0.18	–0.74	–0.02	3.79	0.052
Dispersion	0.67	0.12	0.46	0.96	n/a	n/a
Property-Damage-Only Crashes per Mile per Year—Type 2 Crest Vertical Curves and Level Roadways						
Intercept	–8.96	1.13	–11.2	–6.75	n/a	n/a
ln(AADT)	1.03	0.12	0.80	1.26	66.04	<.001
Vertical curve length (mi)	–0.47	0.16	–0.77	–0.16	8.04	0.005
Dispersion	0.44	0.08	0.31	0.63	n/a	n/a
Fatal-and-Injury Crashes per Mile per Year—Type 2 Sag Vertical Curves and Level Roadways						
Intercept	–8.26	1.49	–11.2	–5.34	n/a	n/a
ln(AADT)	0.91	0.16	0.60	1.22	31.28	< .001
Vertical curve length (mi)	–0.53	0.19	–0.89	–0.16	6.80	0.009
Dispersion	0.77	0.14	0.55	1.09	n/a	n/a
Property-Damage-Only Crashes per Mile per Year—Type 2 Sag Vertical Curves and Level Roadways						
Intercept	–10.8	1.23	–13.2	–8.37	n/a	n/a
ln(AADT)	1.22	0.13	0.97	1.47	80.19	< .001
Vertical curve length (mi)	–0.43	0.16	–0.76	–0.11	6.18	0.013
Dispersion	0.49	0.08	0.35	0.67	n/a	n/a

Table 66. Crash summary for Type 1 crest vertical curves by stopping sight distance category and presence of a horizontal curve, intersection, or driveway within or near the vertical curve.

Feature present	Number of sites	Combined length (mi)	Number of crashes			Average AADT	Exposure (MVMT)	Crash rate (per MVMT)		
			FI[a]	PDO[b]	Total			FI	PDO	Total
Stopping sight distance above AASHTO criteria										
None	52	18.8	30	64	94	3,376	113	0.27	0.57	0.83
Intersection only	17	7.0	12	10	22	1,894	26	0.46	0.38	0.84
Curve only	61	23.3	48	80	128	2,338	105	0.46	0.76	1.22
Driveway only	24	8.2	20	33	53	3,868	62	0.32	0.53	0.85
Either curve or intersection or driveway	162	60.9	178	244	422	3,147	360	0.49	0.68	1.17
All cases combined	214	79.7	208	308	516	3,203	473	0.44	0.65	1.09
Stopping sight distance below AASHTO criteria										
None	58	16.1	33	29	62	1,651	52	0.64	0.56	1.20
Intersection only	13	3.5	4	16	20	1,956	12	0.33	1.31	1.64
Curve only	83	23.6	49	67	116	1,810	80	0.61	0.83	1.44
Driveway only	18	5.1	12	17	29	2,747	25	0.47	0.67	1.14
Either curve or intersection or driveway	180	51.5	124	186	310	2,285	218	0.57	0.85	1.42
All cases combined	238	67.6	157	215	372	2,131	270	0.58	0.80	1.38

[a] FI = fatal and injury.
[b] PDO = property damage only.

Table 67. Crash summary for Type 1 crest vertical curves by stopping sight distance category and presence of a hidden horizontal curve, intersection, or driveway within or near the vertical curve.

Feature present	Number of sites	Combined length (mi)	Number of crashes			Average AADT	Exposure (MVMT)	Crash rate (per MVMT)		
			FI[a]	PDO[b]	Total			FI	PDO	Total
Stopping sight distance above AASHTO criteria										
No features	52	18.8	30	64	94	3,376	113	0.27	0.57	0.83
No hidden features	194	72.5	189	282	471	3,338	449	0.42	0.63	1.05
Hidden intersection only	3	1.1	6	5	11	2,870	6	1.04	0.86	1.83
Hidden curve only	16	5.8	11	21	32	1,766	18	0.62	1.18	1.78
Hidden driveway only	0	--	--	--	--	--	--	--	--	--
Either hidden curve or hidden intersection or hidden driveway	20	7.1	19	26	45	1,887	24	0.79	1.08	1.88
All cases combined	214	79.7	208	308	516	3,203	473	0.44	0.65	1.09
Stopping sight distance below AASHTO criteria										
No features	58	16.1	33	29	62	1,651	52	0.64	0.56	1.20
No hidden features	185	52.7	114	162	276	2,117	208	0.55	0.78	1.33
Hidden intersection only	1	0.3	3	8	11	14,111	9	0.34	0.91	1.22
Hidden curve only	37	10.2	26	27	53	1,570	30	0.86	0.89	1.75
Hidden driveway only	10	2.9	9	15	24	3,358	18	0.50	0.84	1.34
Either hidden curve or hidden intersection or hidden driveway	53	14.8	43	53	96	2,181	62	0.69	0.86	1.55
All cases combined	238	67.6	157	215	372	2,131	270	0.58	0.80	1.38

[a] FI = fatal and injury.
[b] PDO = property damage only.

half of each table are for Type 1 crest vertical curves with stopping sight distance greater than the AASHTO stopping sight distance design criteria, and crash rates in the lower half of each table are for Type 1 crest vertical curves that have stopping sight distance less than the AASHTO design criteria.

The data shown in Table 66 indicate that crash rates for all crest vertical curves with stopping sight distance less than AASHTO stopping sight distance criteria are, on average, 27 percent higher than those for all curves with stopping sight distance greater than AASHTO stopping sight distance criteria for vertical curves (1.38 versus 1.09 crashes per million vehicle-miles of travel). Table 67 indicates that for crest vertical curves with stopping sight distance at or above AASHTO criteria but with a horizontal curve, intersection, or driveway present, the crash risk is 41 percent higher than for the base condition with no horizontal curve, intersection, or driveway present (1.17 versus 0.83 crashes per million vehicle-miles of travel). Table 67 shows that, if an intersection, horizontal curve, or driveway is present and the stopping sight distance is less than AASHTO stopping sight distance criteria, the crash rate for a crest vertical curve is 71 percent higher than the base condition (1.42 versus 0.83 crashes per million vehicle-miles of travel). Table 67 also shows that, if the intersection, horizontal curve, or driveway is not visible to opposing drivers until they reach the crest and the stopping sight distance is less than the AASHTO stopping sight distance criteria, the crash rate for a crest vertical curve is 87 percent higher than the base condition with stopping sight distance above AASHTO criteria with no horizontal curves, intersections, or driveways present (1.55 versus 0.83 crashes per million vehicle-miles of travel). These results suggest an influence of stopping sight distance and the presence of features such as horizontal curves, intersections, and driveways on crash risk.

To evaluate the effect of stopping sight distance and other features of crest vertical curves further, the crash frequencies per mile per year for crest vertical curves were analyzed with a negative binomial (NB) regression model that included AADT and the factor indicating whether the stopping sight distance is at, above, or below AASHTO criteria. It should be noted that the length of each study site was calculated as the length of the crest vertical curve plus an additional 0.1 mi at each end of the vertical curve. Prior to analysis, crash frequencies were adjusted for the effect of lane width, shoulder width, and shoulder type based on the CMFs used in HSM Chapter 10 (*12*). Two additional NB regression models were considered: one including a factor that indicates whether a horizontal curve, intersection, or driveway is present, and one indicating whether a horizontal curve, intersection, or driveway hidden from the view of approaching drivers is present. All regression models were developed separately for fatal-and-injury and property-damage-only crashes. The results of these analyses are discussed next.

The first analysis, including only AADT and the AASHTO criteria, showed statistically significant differences in crash frequency between crest vertical curves with stopping sight distance less than the AASHTO stopping sight distance criteria and crest vertical curves with stopping sight distance above the AASHTO stopping sight distance criteria. The observed differences were statistically significant for total crashes ($p = 0.07$) and for fatal-and-injury crashes ($p = 0.07$). The estimated differences were in the expected direction with 22 percent more total crashes per mi per year and 45 percent more fatal-and-injury crashes per mi per year on the crest vertical curves with less stopping sight distance than AASHTO criteria in comparison to the crest vertical curves with stopping sight distance at or above AASHTO criteria.

The second analysis, however, yielded different results when the presence of horizontal curves, intersections, or driveways hidden by the crest vertical curve was taken into account. In this analysis, the difference between crest vertical curves with limited stopping sight distance and crest vertical curves with stopping sight distance above AASHTO criteria was no longer statistically significant ($p = 0.17$ for both total crashes and fatal-and-injury crashes), while the variable indicating the presence of hidden horizontal curves, intersections, or driveways was highly statistically significant ($p = 0.01$ for total crashes and $p = 0.02$ for fatal-and-injury crashes). The observed effect on crash frequency of the presence of a hidden horizontal curve, intersection, or driveway was a 43 percent increase in crashes per mi per year for total crashes and a 62 percent increase in crashes per mi per year for fatal-and-injury crashes. Further investigation established that the effect on crashes of the difference between crest vertical curves with limited stopping sight distance and crest vertical curves with stopping sight distance at or above AASHTO criteria remained statistically significant at the 10-percent significance level ($p = 0.09$ for total crashes and $p = 0.07$ for fatal-and-injury crashes) when the presence of a horizontal curve, intersection, or driveway was considered, but became not statistically significant only when the presence of a *hidden* horizontal curve, intersection, or driveway was considered.

This finding implies that the presence of a crest vertical curve with stopping sight distance below AASHTO stopping sight distance criteria does not of itself increase crash frequency, but does so only when combined with the presence of a horizontal curve, intersection, or driveway hidden by the crest vertical curve. Therefore, there would appear to be little or no benefit from improving a crest vertical curve with limited stopping sight distance on a rural two-lane highway unless there is a horizontal curve, intersection, or driveway present that cannot be seen by an approaching driver because of the presence of the crest vertical curve.

SECTION 5

Refinement of Definitions for the 13 Controlling Criteria

This section of the report addresses refinement of the definitions for the 13 controlling criteria to reduce overlaps and confusion in how they are applied in a typical design exception process. The discussion also addresses potential modifications to the list of controlling criteria.

5.1 Refinement of Criteria Definitions

Specific pairs or sets of design criteria from among the 13 controlling criteria that are closely related to one another and/or need to be carefully distinguished from one another include the following:

- Design speed, horizontal alignment, vertical alignment, stopping sight distance, and lane width
- Lane width and shoulder width
- Shoulder width and horizontal clearance/lateral offset
- Bridge width, lane width, and shoulder width
- Horizontal alignment, superelevation, and cross slope
- Grade, vertical alignment, and stopping sight distance
- Horizontal and vertical alignment
- Horizontal clearance (lateral offset) and clear-zone width

Each of these pairs or sets of design criteria are discussed below.

5.1.1 Design Speed, Horizontal Alignment, Vertical Alignment, Stopping Sight Distance, and Lane Width

Design speed differs from the other controlling criteria because it is really a design control rather than a design criterion. When a highway agency chooses a design speed for a project, this does not directly affect the design of the project, but it does have an important indirect effect through the role of design speed in determining the criteria for lane width, horizontal alignment, vertical alignment, and stopping sight distance (see Sections 2.2, 2.6, 2.7, and 2.9 of this report).

Given the role of design speed as a design control, rather than a design criterion, it might be desirable to distinguish it from the other controlling criteria in some way.

The FHWA policy on design speed (*2*) states that the purpose of this controlling criterion is "to assure that drivers operating at the legal speed limit can do so without unwittingly exceeding the safe design speed." The use of the words "safe design speed" in this statement appears to presume knowledge that the highway engineering profession does not, in fact, have. There is no research that demonstrates that vehicle operations at the design speed are safe or that vehicle operations at speeds above the design speed are unsafe. Indeed, design policies are generally developed with substantial (though usually unquantified) margins of safety. There have been some attempts to quantify such margins of safety. For example, Harwood et al. (*33*) have demonstrated that current AASHTO policies for horizontal curve design provide substantial margins of safety against skidding and rollover for vehicles traveling at, and even above, the design speed. The requirement to use design criteria for a design speed equal to at least the posted or legal speed, or develop a formal design exception, seems a reasonable administrative control given our lack of knowledge in the area, but the use of the term "safe design speed" seems to imply a level of certainty that is not supported by research. It would certainly be desirable for researchers to provide better information to designers about the current lack of knowledge concerning the relationship of design speed to safety and to conduct research to better quantify such effects.

A clear weakness of the first edition of the HSM (*12*) is the absence of factors to quantify explicitly the effect of traffic speed on safety. The HSM does not include such information because such effects are largely undocumented. As discussed in Section 2.1 of this report, Hauer (*17*) indicates that crash severity increases with vehicle speed, but there is little evidence to support the notion that faster travel speeds necessarily result in greater likelihood of a crash. These issues need to be sorted out in future HSM editions based on valid research.

Section 3 of this report indicates that, as a practical matter, few highway agencies seek design exceptions for design speed per se. Rather, design speed is usually left as equal to (or above) the posted or regulatory speed limit and design exceptions are sought, as needed, for the individual design criteria that are affected by design speed. The FHWA *Mitigation Strategies* guide (*7*) states that documenting a design exception for design speed should involve analysis of every individual design element that does not meet criteria appropriate for a design speed equal to the posted or legal speed limit.

In summary, consideration might be given to the following:

- Treating design speed in some different way than the other controlling criteria
- Discouraging or prohibiting design exceptions for design speed, and focusing the design exception process on individual design elements that do not meet established criteria for the design speed. This appears consistent with FHWA guidance and current highway agency practice
- Conducting research to better quantify the relationship of traffic safety and speed

5.1.2 Lane Width and Shoulder Width

Lane width and shoulder width are closely related design criteria, since they are adjacent elements of the roadway cross section. In new construction, or in the absence of design constraints (such as existing development), lane and shoulder width can be determined independently. However, reconstruction projects often have substantial design constraints, which limit the total cross-section width. In such a constrained situation, any increase in lane width may decrease shoulder with, and vice versa. And, there is often a need to consider reducing both lane and shoulder widths to provide space for other features that clearly enhance safety such as median treatments, left- and right-turn lanes, bicycle lanes, parking lanes, and shorter pedestrian crossing distances.

Currently *Green Book* policies present lane- and shoulder-width criteria independently. The lane- and shoulder-width effects on safety in the HSM are presented as independent effects, although most researchers presume that lane and shoulder widths have effects that interact in ways that we do not yet fully understand. The HCM addresses the operational effects of lane and shoulder widths with a combined table (see Table 5) for two-lane highways, but addresses their effects separately for other facility types.

There has been some research to look at combined effects of lane- and shoulder-width, including recent research on combined lane- and shoulder-width effects on safety for two-lane highways for FHWA by Gross et al. (*44*). However, there does not yet appear to be any work sufficiently complete and comprehensive to serve as a basis for policy on combined lane- and shoulder-width criteria.

The time may come when lane and shoulder width may be dealt with by a combined set of controlling criteria for design or a combined set of operational and safety analysis procedures for use in the design process. For example, at some future time, design criteria might be established for total roadway width, with guidelines for optimally allocating space to lane width, shoulder width, and other cross-section features. However, there is not yet sufficient knowledge to achieve this goal. For the present, there appears to be a need to work toward the development of such combined operational and safety effectiveness measures for lane and shoulder width, while improving our design policies to emphasize even further the need for flexibility in allocating space to lane width, shoulder width, and other features.

In summary, consideration might be given to the following:

- More strongly encouraging flexibility in allocating available cross-section width to lane width, shoulder width, and other features that enhance safety, particularly in reconstruction projects, by either:
  - Simplifying the design exception approval process for lane and shoulder width where space is provided in the cross-section width for other features that enhance safety such as median treatments, left- and right-turn lanes, bicycle lanes, parking lanes, and shorter pedestrian crossing distances, or
  - Revising the design criteria for roadway cross sections to address lane width, shoulder width, and these other features in a combined, flexible design criterion
- Encouraging individual agencies to develop design manuals and internal design and design exception policies to take advantage of the flexibility that is already available in the *Green Book* (*4*) and *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)* (*11*)
- Encouraging research to better understand the interactions among lane width, shoulder width, and other cross-section design features

5.1.3 Shoulder Width and Horizontal Clearance/Lateral Offset

The controlling criteria for design include separate criteria for shoulder width and horizontal clearance/lateral offset (see Sections 2.3 and 2.13). However, these two criteria are closely related.

Usable shoulder width is defined as the width of the paved or unpaved area from the edge of the traveled way to the point of intersection of the shoulder slope and mild roadside slope (for example, 1V:4H or flatter) or to the beginning of rounding for slopes steeper than 1V:4H. The horizontal clearance, being renamed lateral offset in the 2011 edition of the *Green Book* (*5*), is defined as the distance immediately outside the traveled way, face of curb, shoulder, or other designated point to a vertical roadside element or obstruction. Given these definitions, the minimum shoulder widths, where provided, should ensure that the required lateral offset is available. Thus, lateral offset really becomes directly applicable to traffic operations and safety only where no shoulder is present, such as where the roadway is designed with a curb-and-gutter section.

Since shoulder width and lateral offset are so closely related, their definitions as controlling criteria might be better coordinated, as follows:

- For roadways with open cross sections, minimum shoulder widths are needed in accordance with *Green Book* criteria. Where the minimum shoulder widths are not provided or retained, a design exception that addresses traffic operational and safety effects is needed. Where the minimum shoulder width is not provided, it is still desirable that the minimum lateral offset be provided, and a design exception for lateral offset would also be needed.
- For roadways with curb-and-gutter sections, minimum lateral offsets from the face of curb should be provided, and a design exception is needed where the minimum lateral offset is not provided.
- Design exceptions for lateral offset need to address operational or safety considerations only where the available lateral offset is less than 1.5 ft from the edge of the traveled way. Where the design exception is related to lack of a 1.5-ft lateral offset from the face of a curb or from the edge of a shoulder, but there is a 1.5-ft offset available from the edge of the traveled way, the design exception should address mitigation strategies, but need not address traffic operations and safety. Otherwise, the controlling criterion would be interpreted as a clear-zone criterion, which it is not.

The change in the 2011 RDG (*40*) to presenting a lateral offset of 4 to 8 ft rather than the previous 1.5 ft needs to be carefully considered in relation to the controlling criterion for lateral offset. This issue is addressed more fully in Section 5.1.8.

5.1.4 Bridge Width, Lane Width, and Shoulder Width

It is clearly desirable at bridges to carry the full lane and shoulder width available on bridge approaches across the bridge. However, this is not always practical, especially in reconstruction projects involving existing bridges. Section 2.4 of this report discuss the provisions in AASHTO policy that allow bridges with less than the full approach lane and shoulder widths to be constructed or retained without the need for a design exception.

The research reported in Section 4.3 found no statistically significant effect of bridge-width difference on crash frequency and severity for bridges on rural two-lane highways. Thus, there is no indication of any safety concern at rural two-lane highway locations where the bridge width is narrower than the approach roadway width. This implies that existing bridges in good structural condition on rural two-lane highways can remain in place in reconstruction projects. No similar results are available for bridge widths on other roadway types.

5.1.5 Horizontal Alignment, Superelevation, and Cross Slope

FHWA policy includes controlling criteria for horizontal alignment, superelevation, and cross slope. These controlling criteria actually incorporate four separate design elements:

- Horizontal curve radius
- Superelevation
- Cross slope
- Cross-slope breaks between the pavement and shoulder at the outside of superelevated horizontal curves

The relationships of these four design elements to the controlling criteria are not always understood because (1) three of these four design elements (all but normal cross slope) are related to horizontal alignment design; (2) three of these four design elements (all but horizontal curve radius) are cross-section elements rather than alignment elements; and (3) three of these four design elements (all but horizontal curve radius) address cross-slope issues. These criteria overlap and relate to one another in such a complex manner that not everyone reading the list of the 13 controlling criteria immediately understands that all four of the design elements are included in the controlling criteria.

The definition of the controlling criterion for horizontal alignment is not self-evident because it does not include all aspects of horizontal alignment design. It essentially includes only horizontal curve radius, because horizontal curve length and transition design details are not part of the controlling criteria, and superelevation and pavement/shoulder slope breaks are addressed by separate controlling criteria. Still another controlling criterion, stopping sight distance, is closely related to horizontal alignment, because roadside obstructions on the inside of horizontal curves can limit stopping sight distance.

It would be helpful if the definitions of the controlling criteria were realigned so that

- There are separate controlling criteria for horizontal curve radius, superelevation, and cross slope (including normal cross slope and pavement/shoulder cross-slope breaks), or

- There is one controlling criterion for horizontal alignment that includes horizontal curve radius, superelevation, and pavement/shoulder slope breaks and a separate controlling criterion for normal cross slope

Of these alternatives, the former approach appears preferable because the latter preserves the term "horizontal alignment" as a controlling criterion, even though not all elements of horizontal alignment are included in that criterion.

5.1.6 Grade, Vertical Alignment, and Stopping Sight Distance

FHWA policy includes separate controlling criteria for grade, vertical alignment, and stopping sight distance. These controlling criteria are closely related and all, in one way or another, are included within the term "vertical alignment." Grade is clearly an element of vertical alignment, and crest vertical curves, another vertical alignment element, are features that commonly limit stopping sight distance. The controlling criterion for vertical alignment is potentially confusing because grade and stopping sight distance are part of, or closely related to, vertical alignment but are treated as separate controlling criteria. In fact, the only aspect of vertical alignment design considered part of the controlling criteria, but not included explicitly within grade or stopping sight distance, is sag vertical curve length.

The controlling criteria might be clearer if the term "vertical alignment" were eliminated and separate criteria were established for the following:

- Grade
- Stopping sight distance (which includes consideration of crest vertical curve length, sight obstructions on the inside of horizontal curves, and overpass structures)
- Sag vertical curve length

5.1.7 Horizontal and Vertical Alignment

Discussions in Sections 5.1.5 and 5.1.6 of this report have suggested eliminating the controlling criteria named "horizontal alignment" and "vertical alignment" and focusing instead on separate controlling criteria for selected critical elements of horizontal and vertical alignment design.

5.1.8 Horizontal Clearance (Lateral Offset) and Clear-Zone Width

The 2004 *Green Book* (*4*) and earlier editions have specified in Chapter 4 (Cross Section Elements) a horizontal clearance of 1.5 ft from the face of the curb to roadside objects for curbed sections on urban arterials, collectors, and local streets. The purpose of this horizontal clearance is to provide an "operational offset" with sufficient space so that passengers would have space to open doors of parked vehicles, even where a roadside object was present, and so that mirrors of turning vehicles would not strike roadside objects. This horizontal clearance is one of FHWA's 13 controlling criteria.

The 2004 *Green Book* states that, in addition to the "operational offset," an additional clear-zone distance commensurate with prevailing traffic volumes and vehicle speeds should be provided where practical. FHWA guidance also states that horizontal clearance is an operational offset and is not intended as a clear-zone criterion.

FHWA policy (*2*) states explicitly that clear-zone width or other roadside design features intended to reduce the severity of run-off-road collisions are not part of the controlling criteria for design. Roadside features are designed based on guidance in the RDG (*39*), and roadside design, other than hardware testing addressed by the *Manual for Assessing Safety Hardware* (MASH) (*45*), is normally based on benefit-cost analysis using the Roadside Safety Analysis Program (RSAP) (*46,47,48*) rather than on standards.

The 2011 edition of the *Green Book* (*5*), in Section 4.6.2, has renamed horizontal clearance as lateral offset. The 2011 edition no longer presents the previous 1.5-ft criterion for horizontal clearance/lateral offset, but rather refers to the 2011 RDG (*40*) for lateral offset criteria. Revised language in Chapter 10 of the 2011 RDG states the following criteria for lateral offset:

- Lateral offset of 1.5 ft from the face of the curb should be provided where curb is used, with 3 ft lateral offset at intersections.
- In high-risk urban roadside corridors, a recommended goal is to achieve 6-ft lateral offset to roadside objects on the outside of horizontal curves and at other locations and 4-ft lateral offset to roadside objects elsewhere. The accompanying text states that lateral offsets of 12 ft on the outside of horizontal curves and 8 ft at tangent locations are reasonable goals where the clear-zone widths in RDG Chapter 3 cannot be achieved.
- Lateral offset of 12 ft is suggested where feasible at merge points on curbed roadways in high-risk urban roadside corridors.
- Lateral offset of 10 to 15 ft is suggested along the roadway within 10 to 15 ft immediately downstream of driveways.
- Lateral offset should be 6 ft at intersection curb returns in high-risk urban roadside corridors with a minimum value of 3 ft.

The rationale presented in the RDG for the 4- and 6-ft lateral offset criteria for high-risk urban roadside corridors is based on the likelihood of vehicles running off the road and striking roadside objects. Thus, the 4- and 6-ft lateral offset criteria are not intended as operational offsets, but rather appear to represent “mini-clear-zone” criteria. The 4-ft lateral offsets along the inside of horizontal curves and downstream of driveways are specifically shown in the RDG as providing appropriate sight distance for through or turning vehicles.

These new RDG guidelines are based on research by Dixon et al. (*49*), who found that approximately 80 percent of roadside crashes in an urban environment involved an offset from the curb face less than or equal to 4 ft, and more than 90 percent of urban roadside crashes involved lateral offsets less than or equal to 6 ft. The research by Dixon et al. (*49*) specifically discusses minimizing roadside objects in the border area of urban arterials between the traveled way and the shoulder to reduce run-off-road crashes. The RDG does not define “high-risk urban roadside corridors” explicitly, but related language defines critical urban roadside locations as being best determined by identifying locations with a history of over-representation of roadside crashes. Other relevant factors mentioned are operating speed, functional purpose, and other specific road features.

Several issues arise with these changes:

- FHWA policy identifies horizontal clearance as one of the 13 controlling criteria, but there are no longer any corresponding AASHTO criteria for horizontal clearance. Instead, the *Green Book* now uses the term lateral offset instead of horizontal clearance.
- The current *Green Book* has no numerical criteria for lateral offset. The 1.5-ft criterion that appeared in previous editions has been replaced by a reference to the RDG.
- The RDG still includes the 1.5-ft lateral offset criterion that has historically been used as FHWA’s controlling criterion for horizontal clearance, but the RDG also uses the same term, lateral offset, to refer to “mini-clear-zone criteria” for high-risk urban roadside corridors. The dual use of the term lateral offset needs clarification because FHWA documentation states explicitly that the controlling criterion for horizontal clearance is not intended to include clear-zone considerations. Clarification of the language in the RDG is needed so that the RDG criteria for lateral offsets greater than 1.5 ft are not mistakenly considered as part of the controlling criterion for horizontal clearance.

Potential courses of action toward resolving this issue that may be considered include the following:

- Horizontal clearance could be dropped from the list of controlling criteria, as recommended in Sections 7 and 8 of this report. This would resolve the potential uncertainty about whether the RDG language on lateral offsets larger than 1.5 ft is part of the controlling criterion for horizontal clearance.
- If horizontal clearance is retained as a controlling criterion, its name could be charged to lateral offset for consistency with the *Green Book* and the RDG, with the accompanying clarification that this controlling criterion applies only to the 1.5-ft operational offset and not to wider lateral offsets presented in the RDG intended to reduce fixed-object collisions for vehicles that run off the road.
- The RDG could be changed to use different terms for the 1.5-ft offset intended as an operational offset and the wider offsets intended to reduce collisions with vehicles that run off the road.

5.2 Suggested Renaming of the 13 Controlling Criteria

If all of the current controlling criteria are retained, it is suggested that they be renamed to minimize any potential confusion over which design elements are, or are not, included as part of the controlling criteria. The rationale for this renaming of the controlling criteria has been presented in Section 5.1. The suggested names are the following:

- Design speed
- Lane width
- Shoulder width
- Bridge width
- Structural capacity
- Horizontal curve radius
- Superelevation
- Grade
- Stopping sight distance
- Sag vertical curve length
- Cross slope
- Vertical clearance
- Lateral offset

If these suggested names are used, it would be useful if the accompanying documentation made it clear that the stopping sight distance criterion includes stopping sight distance as limited by any roadway or roadside feature—including crest vertical curves, sight obstructions on the inside of horizontal curves, and overpass structures. Thus, the controlling criterion for stopping sight distance directly influences the minimum crest vertical curve length for any given algebraic difference in grade and the offset to roadside sight obstructions for any curve radius on horizontal curves.

The potential need to add other controlling criteria is addressed in Section 5.3. The potential need to drop specific controlling criteria is addressed in Sections 6 and 7.

5.3 Other Potential Controlling Criteria for Design

Consideration was given in the research to whether additional design elements might be considered in the future as controlling criteria for design. It is suggested that factors for adding a design element as a controlling criterion would include the following:

- Known traffic operational and safety effects
- Traffic operational and safety effects large enough to warrant inclusion as a controlling criterion for design
- Within the scope of this research and the existing controlling criteria (which exclude design elements related to intersection design, roadside design, and access control)

The research team identified three design elements with potential traffic operational or safety effects that appear to be important enough to merit consideration for inclusion as controlling criteria for design. These were

- Intersection sight distance
- Decision sight distance
- Spacing between crossroad ramp terminals at interchanges and the nearest access point

Ultimately, however, the research team's assessment concluded that none of these three design criteria currently meet the tests set out by the factors above.

Intersection sight distance (particularly the provision of clear sight triangles with dimensions sufficient to provide clear sight lines for drivers) is presumed by most researchers and practitioners to be important to safety, but no CMFs or safety effectiveness measures are available for intersection sight distance. In fact, research is currently being conducted under NCHRP Project 17-59 to develop such CMFs. Therefore, it cannot be stated at this time that the safety effects of intersection sight distance are known. Intersection sight distance is determined by both roadway alignment design characteristics and intersection design characteristics, so it might also be ruled out as being an intersection design element.

Decision sight distance, while having obvious implications for safety, does not have a documented CMF or a defined relationship to safety. The current design criteria for decision sight distance involve a great deal of judgment in their application; two experienced designers applying the current criteria might reach different conclusions as to whether enhanced decision sight distance is needed in advance of a particular decision point. To serve as a controlling criterion for design, revised criteria for decision sight distance would need to be applied more objectively, and there would need to be a demonstrated relationship of the revised decision sight distance criteria to safety. Without more definitive design criteria and research quantifying the traffic operational and safety effects of those criteria, designation of decision sight distance as a controlling criterion for geometric design does not appear appropriate.

Spacing between crossroad ramp terminals at interchanges and the nearest access point has an obvious, but unquantified, relationship to traffic operations and safety. While there are no quantified traffic operational or safety relationships, there is ample anecdotal evidence from many locations that traffic operational or safety concerns may arise where access points are located close to interchange ramp terminals. However, in the absence of quantitative relationships, and because this is both an access control and an intersection design issue, spacing between crossroad ramp terminals and the nearest access point does not appear appropriate as a controlling criterion for design.

All other ideas for potential controlling criteria considered by the research team were rejected as being clearly related to intersection design, roadside design, or access control.

SECTION 6

Prioritization of the 13 Controlling Criteria

This section of the report applies the existing traffic operational and safety relationships from Section 2 and the new relationships from the research presented in Section 4 to prioritize the 13 controlling criteria for specific roadway types. The discussion begins by presenting the ranking of the 13 controlling criteria from a recent survey of highway agencies, followed by presentation of the priorities developed in this research.

6.1 Ranking of the 13 Controlling Criteria from *NCHRP Synthesis 417*

NCHRP Synthesis 417: Geometric Design Practices for Resurfacing, Restoration, and Rehabilitation (*50*) included the results of a survey in which state highway agencies were asked to rank the importance of the 13 controlling criteria for RRR projects from 1 (most important) to 13 (least important). The results of the survey are summarized in Table 68. While the survey did not specifically mention ranking the 13 controlling criteria based on their perceived relationship to safety, it is difficult to imagine that perceived safety effects (and possibly perceived traffic operational effects) were not a key element in consideration of this question by the respondents. The survey did not specify a particular roadway type to be considered by respondents in making these rankings, but given that the survey concerned RRR projects, it is likely that rural two-lane highways, and perhaps rural multilane highways or urban and suburban arterials, were key considerations for the respondents. It is also likely that freeways were not a key consideration.

The average ranks for the design criteria in Table 68 show some clear distinctions in importance among the 13 controlling criteria, with lane width and shoulder width being rated as having the highest importance and grade being rated as having the least importance. However, it is also evident that there is a broad range of opinion on the relative importance of the 13 controlling criteria. Every design criteria was ranked first or second in importance by at least one state and last or next to last by at least one state. Some difference in assessments between states is to be expected because different states face difference circumstances with respect to terrain, climate, and level of development. For example, it was noted that some states with high rainfall levels rated cross slope as particularly important. Despite such differing circumstances, the survey results in Table 68 reflect a greater lack of consensus among highway agencies than seems desirable given the current state of knowledge about traffic operations and safety. The sensitivity analysis and prioritization results presented later in this chapter are intended to reduce the uncertainty in prioritizing the 13 controlling criteria by setting priorities based on known traffic operational and safety relationships.

The inclusion of design speed, structural capacity, and vertical clearance in Table 68 may contribute to some of the uncertainty about the rankings. For example, design speed by itself does not directly affect traffic operations or safety, but has an indirect traffic operational or safety effect because choosing a design speed influences the design criteria for lane width, horizontal alignment, vertical alignment, and stopping sight distance. Structural capacity has no direct effect on traffic operations or safety, but, obviously, a structural failure has traffic operational and safety effects. It is easy to see why some respondents to the survey might have rated structural capacity as having critical importance—structural failures can be catastrophic. Nonetheless, it is just as easy to see why other respondents may have rated it as having little importance—structural capacity has no direct influence on traffic operations or safety. There are similar concerns with vertical clearance. To eliminate these concerns, Table 69 presents the results of the *NCHRP Synthesis 417* survey for the 10 remaining controlling criteria when design speed, structural capacity, and vertical clearance are omitted.

Table 69 indicates that the 10 controlling criteria of greatest interest are ranked in the same order as the complete survey results shown in Table 68, with lane width and shoulder

Table 68. Ranking of the 13 controlling criteria from *NCHRP Synthesis 417* (*50*).

Design criterion	Average rank	Highest rank	Lowest rank
Lane width	3.8	1	12
Shoulder width	4.6	1	13
Design speed	4.6	1	13
Stopping sight distance	5.8	1	13
Horizontal alignment	6.4	1	13
Structural capacity	7.0	1	13
Superelevation	7.1	1	13
Bridge width	7.4	2	12
Vertical alignment	7.7	2	13
Cross slope	8.1	1	13
Horizontal clearance/lateral offset	9.3	1	13
Vertical clearance	9.3	1	13
Grade	9.9	2	13

NOTE: Based on ranking of the importance of each design criterion by 46 highway agencies on a scale from 1 (most important) to 13 (least important)

Table 69. Ranking of the 13 controlling criteria omitting design speed, structural capacity, and vertical clearance (adapted from *NCHRP Synthesis 417* [*50*]).

Design criterion	Average rank	Highest rank	Lowest rank
Lane width	2.8	1	9
Shoulder width	3.7	1	10
Stopping sight distance	4.4	1	10
Horizontal alignment	4.9	1	10
Superelevation	5.7	1	10
Bridge width	5.8	1	10
Vertical alignment	6.2	1	10
Cross slope	6.3	1	10
Horizontal clearance/lateral offset	7.2	1	10
Grade	8.0	1	10

NOTE: Based on ranking of the importance of each design criterion by 46 highway agencies on a scale from 1 (most important) to 10 (least important); adapted from results presented in Table 68 by omitting rankings for design speed, structural capacity, and vertical clearance.

width ranked as having the greatest importance and horizontal clearance and grade ranked as having the least importance.

6.2 Sensitivity Analysis of Key Controlling Criteria for Specific Roadway Types

The research included a sensitivity analysis for both traffic operations and safety of the 10 key controlling criteria (excluding design speed, structural capacity, and vertical clearance for the reasons cited earlier). The sensitivity analyses varied each of the controlling criteria over a specified range of interest to determine its traffic operational and safety effects. Whenever possible, the traffic operational and safety effects were determined with established relationships from the HCM (*13*), the HSM (*12*), and other previously published sources. Where no previously published sources are available, new relationships developed in Section 4 were used or engineering judgments were made by the research team based on the current state of knowledge. The roadway types considered in the sensitivity analysis were rural two-lane highways, rural multilane highways, urban and suburban arterials, and rural freeways.

A key issue in planning this sensitivity analysis was deciding what variations in each of the controlling criteria to assess. Since each of the controlling criteria have different magnitudes and different units of measure, and some measures are unitless (e.g., cross slope and percent grade), a sensitivity analysis could not be performed simply by varying each criterion by one unit. Therefore, to conduct this assessment, the research team identified a typical range of design values in current use for each of the controlling criteria on each roadway type and then assessed the traffic operational and safety effect of reducing each of the controlling criteria, in turn, from the upper end of that range to the midpoint of the range.

The range of interest for each of the controlling criteria was defined as follows:

- The upper end of the range of interest was set equal to an ideal or, at least, typical high value for each of the controlling criteria; for example, the upper end of the range of interest for lane width was set equal to 12 ft.

- The lower end of the range of interest, representing restrictive design, was set equal to the lowest value generally used in practice or a typical low value for each of the controlling criteria; for example, the lower end of the range of interest for lane width was set equal to 9 ft.

Using the example of lane width, the sensitivity of traffic operations to lane width and the sensitivity of safety to lane width were each determined by reducing lane width from the upper end value of 12 ft to the midpoint value of 10.5 ft.

For each roadway type, a typical roadway section was specified as the base condition for sensitivity analyses. For example, the base condition for rural two-lane highway analyses was a roadway section 5 mi in length with two intersections, five driveways, and one horizontal curve per mile. Three scenarios for sensitivity analyses were developed considering a range of AADT levels and variations of one to three selected key variables. For example, the sensitivity analysis scenarios for rural two-lane highways included a roadway with AADT of 2,000 veh/day and gravel shoulders, a roadway with AADT of 5,000 veh/day and paved shoulders, and a roadway with AADT of 10,000 veh/day and paved shoulders. All other characteristics of the base condition remained unchanged in all analyses except for the values of the controlling criteria and the three scenarios described above.

All 10 relevant controlling criteria (i.e., the 13 controlling criteria excluding design speed, structural capacity, and vertical clearance) were considered in each sensitivity analysis. For stopping sight distance, two scenarios were considered in each sensitivity analysis: scenarios with and without the presence of horizontal curves, intersections, or driveways hidden from the view of approaching drivers by limited sight distance.

The sensitivity analysis for each roadway type is presented below.

6.2.1 Rural Two-Lane Highways

Table 70 presents the plan for the sensitivity analysis of rural two-lane highways. The table shows, for rural two-lane highways, the range over which the controlling criteria were varied (ideal or typical value to midpoint of the range of interest), the site characteristics that varied among the three scenarios considered, and the values of the base condition characteristics that remained constant in all of the scenarios.

Table 71 identifies the estimation methods that were used to quantify the traffic operational and safety relationships for the scenarios considered for rural two-lane highways.

The results of the traffic operational and safety sensitivity analyses for rural two-lane highways are presented in Tables 72 and 73, respectively. Table 72 presents the traffic operational effect of each design criterion in terms of the reduction in average travel speed due to the change in the value of that criterion, as described above. Table 73 presents comparable values for the safety effect of each design criterion in terms of the change in fatal-and-injury crash frequency per mile per year. Tables 72 and 73 also present a rank order (from 1 to 11) for the magnitude of the traffic operational and safety effects of each design criterion, so the design criteria with the largest effects are ranked highest.

Table 72 shows that three of the design criteria have quantitative, nonzero effects on traffic speeds on rural two-lane highways. These three design criteria are, in descending order of the magnitude of the effect on traffic speed: shoulder width, lane width, and horizontal curve radius. There is no evidence that any of the remaining design criteria have effects on traffic speed sufficiently large to be quantified. It is possible that some of these design criteria have effects on traffic speed that are too small to be quantified, and it is likely that some design criteria, in fact, have no effect on traffic speed.

The design criteria shown in Table 72 as having no quantified effect on traffic speed have been ranked in descending rank order based on literature review results and judgment. Grade, for example, clearly affects vehicle speeds to some extent (especially for heavy vehicles), but such effects are quantified in HCM procedures only if the terrain category changes or if an individual grade is over 0.5 mi in length. Bridge width has no quantified effect on average travel speed; the lane- and shoulder-width effects suggest that a bridge that is narrower than the approach roadway may slow traffic, but such effects on the average travel speed over an extended roadway section are likely to be small because of the limited length of a bridge in relation to the section length as a whole. Other design criteria likely have zero or essentially zero effect on traffic speed. In particular, lateral offset likely has no effect on traffic speed on rural two-lane highways because the presence of shoulders in the sensitivity analysis scenarios ensures that appropriate lateral offset should always be present.

Table 73 shows that six of the design criteria have quantitative, nonzero effects on fatal-and-injury crash frequencies on rural two-lane highways. These six design criteria are, in descending order of the magnitude of the effect on crash frequency are shoulder width, lane width, grade, horizontal curve radius, superelevation, and stopping sight distance (in the situation where a hidden curve, intersection, or driveway is present). There is no evidence that any of the remaining design criteria have effects on crash frequency sufficiently large to be quantified. It is possible that some of these design criteria have effects on crash frequency that are too small to be quantified, and it is likely that some design criteria, in fact, have no effect on crash frequency.

The design criteria shown in Table 73 as having no quantified effect on fatal-and-injury crash frequency have been ranked in descending rank order based on literature review results and judgment. For example, bridge width has no

Table 70. Base case and analysis scenarios for rural two-lane highways.

Levels for Specific Controlling Criteria

Design criterion	Typical or ideal design	Restrictive design	Midpoint between typical and restrictive design	Comment
Lane width (ft)	12	9	10.5	Range from ideal to most restrictive lane width
Shoulder width (ft)	6	0	3	Range from ideal to most restrictive shoulder width
Bridge width difference (ft)	0	10	5	Range from ideal 36-ft bridge width to 26-ft bridge width
Horizontal curve radius (ft)	3000	1000	2000	Range from well above minimum radius to slightly below minimum
Sag vertical curve length (ft)	above AASHTO criteria	below AASHTO criteria	below AASHTO criteria	Range from above AASHTO minimum to below AASHTO minimum for A=6%
Grade (%)	3	6	4.5	Range from steepest rural arterial grade in level terrain to steepest grade in mountainous terrain
Stopping sight distance category	meets AASHTO criteria	below AASHTO criteria	below AASHTO criteria	Presence of a hidden curve, intersection, or driveway is addressed in a separate analysis
Cross slope (%)	2	0	1	Range from normal cross slope to no cross slope
Superelevation (%)	5	2	3.5	Range from normal superelevation for 3,000-ft radius curve with emax=8% to normal cross slope
Lateral offset (ft)	1.5	0	0.75	Range from AASHTO minimum lateral offset to no lateral offset

Input Parameters That Vary by Scenario

Input parameter	Scenario #1	Scenario #2	Scenario #3
AADT for major road (veh/day)	2000	5000	10000
Shoulder type	gravel	paved	paved

Input Parameters Held Constant for All Scenarios

Input parameter	Value for all scenarios
Analysis section length (mi)	5
Design speed (mph)	60
Base free-flow speed (mph)	60
e_{max} (%)	8
Roadside rating	3
Intersections per mi	2
AADT for minor road (veh/day)	1000
Intersection type	4-leg, minor road stop
Major-road left-turn lanes per intersection	2
Major-road right-turn lanes per intersection	2
Presence of skew angle at intersections	not present
Presence of lighting at intersections	not present
Driveways per mi	5
No. of curves per mi	1
Length of curve (mi)	0.2
Distance between crests and sags (mi)	0.5
Presence of spiral transitions on curves	none present
Presence of centerline rumble strips	none present
Presence of passing lanes	none present
Presence of two-way left-turn lanes	none present
Presence of lighting between intersections	none present
Use of automated speed enforcement	not used
Design hour factor (K)	0.1
Directional factor (D)	0.5
Peak-hour factor (PHF)	0.95
Percent trucks in traffic flow	5
Percent RVs in traffic flow	2
Percent no-passing zones	80
Highway class	Class I
Calibration factor for roadway segments	1
Calibration factor for intersections	1

Table 71. Estimation methods for traffic operational and safety effects for rural two-lane highways.

Controlling criterion	Traffic operational effect	Traffic safety effect
Lane width	Table 5 and Equation 1	Table 6 and Equation 2
Shoulder width	Table 5 and Equation 1	Tables 13 and 14 and Equation 7
Bridge width	Table 5 and Equation 1[a]	No known effect based on Section 4.3
Horizontal curve radius	Table 21	Equation 15
Sag vertical curve length	No known effect based on Section 2.7	Equations 30 through 33
Grade	Tables 23 through 31 and Equation 34	Table 32 and Equation 35
Stopping sight distance	No known effect based on Section 2.9	Effect based on Section 4.7 if a hidden feature is present
Cross slope	No known effect based on Section 2.10	No known effect based on Section 2.10
Superelevation	No known effect based on Section 2.11	Equations 36 through 38
Lateral offset	Not applicable where shoulders are present	Not applicable where shoulders are present

[a] No additional effect beyond the effect of a narrower lane or shoulder, if present on bridge.

Table 72. Traffic operational effects of the controlling criteria for selected scenarios on rural two-lane highways.

Design criterion	Traffic operational effect: Change in average travel speed (mph)[a] in comparison to base condition			Rank order of traffic operational effect[b]			
	Scenario[c] No. 1	Scenario No. 2	Scenario No. 3	Scenario No. 1	Scenario No. 2	Scenario No. 3	Combined
Lane width	–0.75	–0.75	–0.75	2	2	2	2
Shoulder width	–1.14	–1.22	–1.22	1	1	1	1
Bridge width	0.00	0.00	0.00	5	5	5	5
Horizontal curve radius	–0.24	–0.24	–0.24	3	3	3	3
Sag vertical curve length	0.00	0.00	0.00	9	9	9	9
Grade	0.00	0.00	0.00	4	4	4	4
Stopping sight distance[d]	0.00	0.00	0.00	8	8	8	8
Stopping sight distance[e]	0.00	0.00	0.00	7	7	7	7
Cross slope	0.00	0.00	0.00	10	10	10	10
Superelevation	0.00	0.00	0.00	6	6	6	6
Lateral offset	0.00	0.00	0.00	11	11	11	11

[a] Traffic operational effects are in comparison to average travel speed of 52.7 mph for the base condition in Scenario #1, 49.8 mph for base condition in Scenario #2, and 47.4 mph for base condition in Scenario #3.
[b] Estimated based on literature review results and judgment for situations with zero effects.
[c] Scenarios No. 1 through 3 are defined in Table 70; methods for estimating traffic operational effects are defined in Table 71.
[d] With no hidden features.
[e] With hidden curve, intersection, or driveway.

Table 73. Traffic safety effects of the controlling criteria in comparison to base condition for selected scenarios on rural two-lane highways.

Design criterion	Traffic safety effect: Percent change in fatal-and-injury crashes/mi/year[a] in comparison to base condition			Rank order of traffic safety effect[b]			
	Scenario[c] #1	Scenario #2	Scenario #3	Scenario #1	Scenario #2	Scenario #3	Combined
Lane width	4.27	5.15	5.84	2	2	2	2
Shoulder width	5.24	6.62	7.51	1	1	1	1
Bridge width	0.00	0.00	0.00	7	7	7	7
Horizontal curve radius	0.88	1.06	1.20	4	4	4	4
Sag vertical curve length	0.00	0.00	0.00	9	9	9	9
Grade	0.97	1.17	1.33	3	3	3	3
Stopping sight distance[d]	0.00	0.00	0.00	10	10	10	10
Stopping sight distance[e]	0.03	0.03	0.02	6	6	6	6
Cross slope	0.00	0.00	0.00	8	8	8	8
Superelevation	0.66	0.80	0.91	5	5	5	5
Lateral offset	0.00	0.00	0.00	11	11	11	11

[a] Traffic safety effects are in comparison to crash frequency of 0.71 fatal-and-injury crashes/mi/year for the base condition in Scenario #1, 1.46 fatal-and-injury crashes/mi/year for base condition in Scenario #2, and 2.58 fatal-and-injury crashes/mi/year for base condition in Scenario #3.
[b] Estimated based on literature review results and judgment for situations with zero effects.
[c] Scenarios #1 through #3 are defined in Table 70; methods for estimating traffic safety effects are defined in Table 71.
[d] With no hidden features.
[e] With hidden curve, intersection, or driveway.

quantified effect on crash frequency; the lane- and shoulder-width effects suggest that a bridge that is narrower than the approach roadway might increase crash risk, but such effects on crash frequency are likely to be very small over an extended roadway section because of the limited length of a bridge in relation to the section length as a whole. Other design criteria likely have zero or essentially zero effect on crash frequency. In particular, lateral offset likely has no effect on crash frequency on rural two-lane highways because the presence of shoulders in the sensitivity analysis scenarios ensures that appropriate lateral offset should always be present.

6.2.2 Rural Multilane Highways

Table 74 presents the plan for the sensitivity analysis of rural multilane highways. The table shows, for rural multilane highways, the range over which the controlling criteria were varied (ideal or typical value to midpoint of the range of interest), the site characteristics that varied among the three scenarios considered, and the values of the base condition characteristics that remained constant in all of the scenarios.

Table 75 identifies the estimation methods that were used to quantify the traffic operational and safety relationships for the scenarios considered for rural multilane highways.

The results of the traffic operational and safety sensitivity analyses for rural multilane highways are presented in Tables 76 and 77, respectively. Tables 76 and 77 also present a rank order (from 1 to 11) for the magnitude of the traffic operational and safety effects of each design criterion, so the design criteria with the largest effects are ranked highest.

Table 76 shows that only two of the design criteria have quantitative, nonzero effects on traffic speeds on rural multilane highways. These two design criteria are, in descending order of the magnitude of the effect on traffic speed: lane width and horizontal curve radius. There is no evidence that any of the remaining design criteria have effects on traffic speed sufficiently large to be quantified. It is possible that some of these design criteria have effects on traffic speed that are too small to be quantified. For example, shoulder width likely has at least a small effect on traffic speed. In addition, grade clearly has some effect on traffic speed, although that effect is minimal unless the terrain category changes or an individual grade is more than 0.5 mi in length. It is also likely that some design criteria, in fact, have no effect on traffic speed.

Table 77 shows that three of the design criteria have quantitative, nonzero effects on fatal-and-injury crash frequencies on rural multilane highways. These three design criteria are, in descending order of the magnitude of the effect on crash frequency: shoulder width, lane width, and stopping sight distance (in the situation where a hidden curve, intersection, or driveway is present). The stopping sight distance effect is based on an analogy to the documented effect for rural two-lane highways. There is no evidence that any of the remaining design criteria have effects on crash frequency sufficiently

Table 74. Base case and analysis scenarios for rural multilane highways.

Levels for Specific Controlling Criteria

Design criterion	Typical or ideal design	Restrictive design	Midpoint between typical and restrictive design	Comment
Lane width (ft)	12	9	10.5	Range from ideal to most restrictive lane width
Outside shoulder width (ft)	6	0	3	Range from ideal to most restrictive shoulder width
Bridge width difference (ft)	0	10	5	Range from ideal 36-ft bridge width to 26-ft bridge width
Horizontal curve radius (ft)	3000	1000	2000	Range from well above minimum radius to slightly below minimum
Sag vertical curve length (ft)	above AASHTO criteria	below AASHTO criteria	below AASHTO criteria	Range from above AASHTO minimum to below AASHTO minimum for A=6%
Grade (%)	3	6	4.5	Range from steepest rural arterial grade in level terrain to steepest grade in mountainous terrain
Stopping sight distance	meets AASHTO criteria	below AASHTO criteria	below AASHTO criteria	Presence of a hidden curve, intersection, or driveway is addressed in a separate analysis
Cross slope (%)	2	0	1	Range from normal cross slope to no cross slope
Superelevation (%)	5	2	3.5	Range from normal superelevation for 3,000-ft radius curve with e_{max}=8% to normal cross slope
Lateral offset (ft)	1.5	0	0.75	Range from AASHTO minimum lateral offset to no lateral offset

Input Parameters That Vary by Scenario

Input parameter	Scenario #1	Scenario #2	Scenario #3
AADT for major road (veh/day)	10000	20000	30000
Divided/undivided	undivided	divided	divided
Median width (ft)	N/A	20	40
Presence of median barrier	N/A	Yes	Yes
Outside shoulder type	gravel	paved	paved

Input Parameters Held Constant for All Scenarios

Input parameter	Value for all scenarios
Analysis section length (mi)	5
Design speed (mph)	60
Base free-flow speed (mph)	60
e_{max} (%)	8
Roadside slopes	1V:4H
Intersections per mi	2
AADT for minor road (veh/day)	1000
Intersection type	4-leg, minor road stop
Major-road left-turn lanes per intersection	2
Major-road right-turn lanes per intersection	2
Presence of skew angle at intersections	not present
Presence of lighting at intersections	not present
No. of curves per mi	1
Length of curve (mi)	0.2
Distance between crests and sags (mi)	0.5
Presence of lighting between intersections	not present
Use of automated speed enforcement	not used
Design hour factor (K)	0.1
Directional factor (D)	0.5
Peak-hour factor (PHF)	0.95
Driver population factor (f_p)	1
Percent trucks in traffic flow	5
Percent RVs in traffic flow	2

Table 75. Estimation methods for traffic operational and safety effects for rural multilane highways.

Controlling criterion	Traffic operational effect	Traffic safety effect
Lane width	Table 7 and Equation 3	Table 8 or 9 and Equation 2
Shoulder width	Table 16 and Equation 3	Tables 13 and 14 and Equation 7 or Table 17
Bridge width	Table 16 and Equation 3[a]	No known effect based on Section 2.4
Horizontal curve radius	Equation 18	Equations 19 and 20
Sag vertical curve length	No known effect based on Section 2.7	No known effect based on Section 2.7
Grade	Tables 33 through 36 and HCM Equation 14-4	No known effect based on Section 2.8
Stopping sight distance	No known effect based on Section 2.9	Effect estimated as equivalent to the rural two-lane highway effect in Section 4.7 if a hidden feature is present
Cross slope	No known effect based on Section 2.10	No known effect based on Section 2.10
Superelevation	No known effect based on Section 2.11	No known effect based on Section 2.11
Lateral offset	Not applicable where shoulders are present	Not applicable where shoulders are present

[a] No additional effect beyond the effect of a narrower lane or shoulder, if present on bridge

Table 76. Traffic operational effects of the controlling criteria for selected scenarios on rural multilane highways.

Design criterion	Traffic operational effect: Change in average travel speed (mph)[a] in comparison to base condition			Rank order of traffic operational effect[b]			
	Scenario #1	Scenario #2	Scenario #3	Scenario #1	Scenario #2	Scenario #3	Combined
Lane width	–2.50	–7.50	–7.50	1	1	1	1
Shoulder width	0.00	0.00	0.00	3	3	3	3
Bridge width	0.00	0.00	0.00	5	5	5	5
Horizontal curve radius	–0.10	–0.10	–0.10	2	2	2	2
Sag vertical curve length	0.00	0.00	0.00	9	9	9	9
Grade	0.00	0.00	0.00	4	4	4	4
Stopping sight distance[c]	0.00	0.00	0.00	8	8	8	8
Stopping sight distance[d]	0.00	0.00	0.00	7	7	7	7
Cross slope	0.00	0.00	0.00	10	10	10	10
Superelevation	0.00	0.00	0.00	6	6	6	6
Lateral offset	0.00	0.00	0.00	11	11	11	11

[a] Traffic operational effects are in comparison to average travel speed of 54.8 mph for the base condition in Scenario #1, 59.8 mph for base condition in Scenario #2, and 59.8 mph for base condition in Scenario #3.
[b] Estimated based on literature review results and judgment for situations with zero effects.
[c] With no hidden features.
[d] With hidden curve, intersection, or driveway.

NOTE: Scenarios #1 through #3 are defined in Table 74; methods for estimating traffic operational effects are defined in Table 75.

Table 77. Traffic safety effects of the controlling criteria in comparison to base condition for selected scenarios on rural multilane highways.

Design criterion	Traffic safety effect: Percent change in fatal-and-injury crashes/mi/year[a] in comparison to base condition			Rank order of traffic safety effect[b]			
	Scenario #1	Scenario #2	Scenario #3	Scenario #1	Scenario #2	Scenario #3	Combined
Lane width	2.82	2.80	2.81	2	2	2	2
Shoulder width	4.52	3.78	3.79	1	1	1	1
Bridge width	0.00	0.00	0.00	7	7	7	7
Horizontal curve radius	0.00	0.00	0.00	6	6	6	6
Sag vertical curve length	0.00	0.00	0.00	9	9	9	9
Grade	0.00	0.00	0.00	5	5	5	5
Stopping sight distance[c]	0.00	0.00	0.00	10	10	10	10
Stopping sight distance[d]	0.03	0.03	0.02	3	3	3	3
Cross slope	0.00	0.00	0.00	8	8	8	8
Superelevation	0.00	0.00	0.00	4	4	4	4
Lateral offset	0.00	0.00	0.00	11	11	11	11

[a] Traffic safety effects are in comparison to crash frequency of 2.83 fatal-and-injury crashes/mi/year for the base condition in Scenario #1, 3.13 fatal-and-injury crashes/mi/year for base condition in Scenario #2, and 4.49 fatal-and-injury crashes/mi/year for base condition in Scenario #3.
[b] Estimated based on literature review results and judgment for situations with zero effects.
[c] With no hidden features.
[d] With hidden curve, intersection, or driveway.

NOTE: Scenarios #1 through #3 are defined in Table 74; methods for estimating traffic safety effects are defined in Table 75.

large to be quantified. It is possible that some of these design criteria have effects on crash frequency that are too small to be quantified, and it is likely that some design criteria, in fact, have no effect on crash frequency. A rank order for the remaining design criteria has been established based on literature review results and judgment.

6.2.3 Rural Freeways

The sensitivity analysis for freeways was based on rural freeways, rather than urban freeways, because it was expected that traffic operational and safety effects would be more critical on higher speed roadways. The results may also be applicable to urban freeways, although urban freeways present their own unique issues with higher volumes and more frequent entrance and exit ramps than rural freeways.

Table 78 presents the plan for the sensitivity analysis of rural freeways. The table shows, for rural freeways, the range over which the controlling criteria were varied (ideal or typical value to midpoint of the range of interest), the site characteristics that varied among the three scenarios considered, and the values of the base condition characteristics that remained constant in all of the scenarios.

Table 79 identifies the estimation methods that were used to quantify the traffic operational and safety relationships for the scenarios considered for rural multilane highways.

The results of the traffic operational and safety sensitivity analyses for rural multilane highways are presented in Tables 80 and 81, respectively. Tables 80 and 81 also present a rank order (from 1 to 11) for the magnitude of the traffic operational and safety effects of each design criterion, so the design criteria with the largest effects are ranked highest.

The only variable shown in Table 80 with a quantitative, nonzero effect on average travel speeds on rural freeways is lane width. The likely range in variation of shoulder widths on rural freeways was too small to have any quantifiable effect. It is possible that some of these design criteria have effects on traffic speed that are too small to be quantified. For example, grade clearly has some effect on traffic speed, although that effect is minimal unless the terrain category changes or an individual grade is more than 0.5 mi in length. It is also likely that some design criteria, in fact, have no effect on traffic speed.

Table 81 shows that four of the design criteria have quantitative, nonzero effects on fatal-and-injury crash frequencies on rural freeways. These four design criteria are, in descending order of the magnitude of the effect on crash frequency: shoulder width, lane width, horizontal curve radius, and stopping sight distance (in the situation where a hidden curve or ramp is present). The stopping sight distance effect is based on an analogy to the documented effect for rural two-lane highways. There is no evidence that any of the remaining design criteria have effects on crash frequency sufficiently large to be quantified. It is possible that some of these design criteria have effects on crash frequency that are too small to be quantified, and it is likely that some design criteria, in

Table 78. Base case and analysis scenarios for rural freeways.

Levels for Specific Controlling Criteria

Design criterion	Typical or ideal design	Restrictive design	Midpoint between typical and restrictive design	Comment
Lane width (ft)	12	9	10.5	Range from ideal to most restrictive lane width
Outside shoulder width (ft)	10	6	8	Range from ideal to most restrictive typical shoulder width
Bridge width difference (ft)	0	10	5	Range from ideal 36-ft bridge width to 26-ft bridge width
Horizontal curve radius (ft)	4500	2000	3750	Range from well above minimum radius to slightly below minimum
Sag vertical curve length (ft)	above AASHTO criteria	below AASHTO criteria	below AASHTO criteria	Range from above AASHTO minimum to below AASHTO minimum for A=6%
Grade (%)	3	6	4.5	Range from steepest rural arterial grade in level terrain to steepest grade in mountainous terrain
Stopping sight distance	meets AASHTO criteria	below AASHTO criteria	below AASHTO criteria	Presence of a hidden curve or ramp is addressed in a separate analysis
Cross slope (%)	2	0	1	Range from normal cross slope to no cross slope
Superelevation (%)	5	2	3.5	Range from normal superelevation for 3,000-ft radius curve with emax=8% to normal cross slope
Lateral offset (ft)	N/A	N/A	N/A	Not applicable to freeways

Input Parameters That Vary by Scenario

Input parameter	Scenario #1	Scenario #2	Scenario #3
AADT for major road (veh/day)	20000	40000	60000
Proportion of AADT under congested conditions	0	0	0.2
Median width (ft)	40	60	60

Input Parameters Held Constant for All Scenarios

Input parameter	Value for all scenarios
Analysis section length (mi)	5
Design speed (mph)	75
Base free-flow speed (mph)	75
e_{max} (%)	8
Number of lanes per direction of travel	2
Inside shoulder width (ft)	4
Left-side clearance (ft)	6
Interchanges per mi	0.4
Entrance ramps per mi	0.4
Exit ramps per mi	0.4
Intersections per mi	2
Ramp AADT (veh/day)	1000
No. of curves per mi	1
Length of curve (mi)	0.2
Distance between crests and sags (mi)	0.5
Design hour factor (K)	0.1
Directional factor (D)	0.5
Peak-hour factor (PHF)	0.95
Percent trucks in traffic flow	10
Percent RVs in traffic flow	2
fp	1
Roadside slopes	1V:6H
Clear zone width (ft)	30
Proportion of length with barrier beyond outside shoulder	0.05
Distance from edge of outside shoulder to barrier face (ft)	10
Proportion of length with barrier beyond inside shoulder	0.05
Distance from edge of inside shoulder to barrier face (ft)	10
Proportion of segment length with rumble strip on outside shoulder	1
Proportion of segment length with rumble strip on inside shoulder	1
L_{en}, ramp entrance length (i.e., speed-change lane length) (mi)	0.5
Left-side ramp indicator for entranance ramps	0
Length of entrance-ramp speed-change lanes adjacent to through lanes	0.2
Number of lanes on entrance-ramp speed-change lane adjacent to freeway	1
Proportion of entrance-ramp speed-change lane length on curve	0
Proportion of entrance-ramp speed-change lane length with barrier present	0
L_{ex}, ramp exit length (i.e., speed-change lane length) (mi)	0.5
Left-side ramp indicator for entrance ramps	0
Length of exit-ramp speed-change lanes adjacent to through lanes (mi)	0.2
Number of lanes on exit-ramp speed-change lane adjacent to freeways	1
Proportion of exit-ramp speed-change lane length on curve	0
Proportion of exit-ramp speed-change lane length with barrier present	
Presence of weaving section in analysis section	not present

Table 79. Estimation methods for traffic operational and safety effects for rural freeways.

Controlling criterion	Traffic operational effect	Traffic safety effect
Lane width	Table 10 and Equation 4	Equations 5 and 6
Shoulder width	Table 19 and Equation 4	Equations 8 through 13
Bridge width	Table 19 and Equation 4[a]	No known effect based on Section 2.4
Horizontal curve radius	No known effect based on Section 2.6	Equations 22 through 25
Sag vertical curve length	No known effect based on Section 2.7	No known effect based on Section 2.7
Grade	Tables 39 through 42 and HCM Equations 11-2 through 11-4	No known effect based on Section 2.8
Stopping sight distance	No known effect based on Section 2.9	Effect estimated as equivalent to the rural two-lane highway effect in Section 4.7 if a hidden feature is present
Cross slope	No known effect based on Section 2.10	No known effect based on Section 2.10
Superelevation	No known effect based on Section 2.11	No known effect based on Section 2.11
Lateral offset	Not applicable where shoulders are present	Not applicable where shoulders are present

[a] No additional effect beyond the effect of a narrower lane or shoulder, if present on bridge

Table 80. Traffic operational effects of the controlling criteria for selected scenarios on rural freeways.

	Traffic operational effect: Change in average travel speed (mph)[a] in comparison to base condition			Rank order of traffic operational effect[b]			
Design criterion	Scenario #1	Scenario #2	Scenario #3	Scenario #1	Scenario #2	Scenario #3	Combined
Lane width	–7.50	–6.91	–3.40	1	1	1	1
Shoulder width	0.00	0.00	0.00	2	2	2	2
Bridge width	0.00	0.00	0.00	5	5	5	5
Horizontal curve radius	0.00	0.00	0.00	3	3	3	3
Sag vertical curve length	0.00	0.00	0.00	9	9	9	9
Grade	0.00	0.00	0.00	4	4	4	4
Stopping sight distance[c]	0.00	0.00	0.00	8	8	8	8
Stopping sight distance[d]	0.00	0.00	0.00	7	7	7	7
Cross slope	0.00	0.00	0.00	10	10	10	10
Superelevation	0.00	0.00	0.00	6	6	6	6
Lateral offset	0.00	0.00	0.00	11	11	11	11

[a] Traffic operational effects are in comparison to average travel speed of 75.0 mph for the base condition in Scenario #1, 74.4 mph for base condition in Scenario #2, and 67.1 mph for base condition in Scenario #3.
[b] Estimated based on literature review results and judgment for situations with zero effects.
[c] With no hidden features.
[d] With hidden curve or ramp junction.

NOTE: Scenarios #1 through #3 are defined in Table 78; methods for estimating traffic operational effects are defined in Table 79.

Table 81. Traffic safety effects of the controlling criteria in comparison to base condition for selected scenarios on rural freeways.

Design criterion	Traffic safety effect: Percent change in fatal-and-injury crashes/mi/year[a] in comparison to base condition			Rank order of traffic safety effect[b]			
	Scenario #1	Scenario #2	Scenario #3	Scenario #1	Scenario #2	Scenario #3	Combined
Lane width	5.74	5.73	5.73	2	2	2	2
Shoulder width	9.74	8.39	7.05	1	1	1	1
Bridge width	0.00	0.00	0.00	7	7	7	7
Horizontal curve radius	0.75	0.68	0.60	3	3	3	3
Sag vertical curve length	0.00	0.00	0.00	9	9	9	9
Grade	0.00	0.00	0.00	6	6	6	6
Stopping sight distance[c]	0.00	0.00	0.00	10	10	10	10
Stopping sight distance[d]	0.03	0.03	0.02	4	4	4	4
Cross slope	0.00	0.00	0.00	8	8	8	8
Superelevation	0.00	0.00	0.00	5	5	5	5
Lateral offset	0.00	0.00	0.00	11	11	11	11

[a] Traffic safety effects are in comparison to crash frequency of 0.9 fatal-and-injury crashes/mi/year for the base condition in Scenario #1, 1.7 fatal-and-injury crashes/mi/year for base condition in Scenario #2, and 2.6 fatal-and-injury crashes/mi/year for base condition in Scenario #3.
[b] Estimated based on literature review results and judgment for situations with zero effects.
[c] With no hidden features.
[d] With hidden curve or ramp junction.

NOTE: Scenarios #1 through #3 are defined in Table 78; methods for estimating traffic safety effects are defined in Table 79.

fact, have no effect on crash frequency. A rank order for the remaining design criteria has been established based on literature review results and judgment.

6.2.4 Urban and Suburban Arterials

No formal sensitivity analyses have been conducted for urban and suburban arterials because there are very few design criteria for which quantitative traffic operational or safety relationships are available. In fact, design considerations that are outside the scope of this research, such as intersection design and access management, appear to have a much greater effect on traffic operations and safety for urban and suburban arterials than roadway design criteria.

6.3 Priorities for the Controlling Criteria

Based on the sensitivity analyses presented above, Table 82 presents the recommended priorities for the controlling criteria.

Table 82. Priorities for the 13 controlling criteria based on sensitivity analysis.

Priority rank	Roadway type		
	Rural two-lane highways	Rural multilane highways	Rural freeways
TRAFFIC OPERATIONS			
1 (highest priority)	Shoulder width	Lane width	Lane width
2	Lane width	Shoulder width	Shoulder width
3	Horizontal curve radius	Horizontal curve radius	Horizontal curve radius
4	Grade	Grade	Grade
5	Bridge width	Bridge width	Bridge width
6	Superelevation	Superelevation	Superelevation
7	Stopping sight distance[a]	Stopping sight distance[a]	Stopping sight distance[b]
8	Stopping sight distance[c]	Stopping sight distance[c]	Stopping sight distance[c]
9	Sag vertical curve length	Sag vertical curve length	Sag vertical curve length
10	Cross slope	Cross slope	Cross slope
11 (lowest priority)	Lateral offset	Lateral offset	Lateral offset
TRAFFIC SAFETY			
1 (highest priority)	Shoulder width	Shoulder width	Shoulder width
2	Lane width	Lane width	Lane width
3	Grade	Stopping sight distance[a]	Horizontal curve radius
4	Horizontal curve radius	Superelevation	Stopping sight distance[b]
5	Superelevation	Grade	Superelevation
6	Stopping sight distance[a]	Horizontal curve radius	Grade
7	Bridge width	Bridge width	Bridge width
8	Cross slope	Cross slope	Cross slope
9	Sag vertical curve length	Sag vertical curve length	Sag vertical curve length
10	Stopping sight distance[c]	Stopping sight distance[c]	Stopping sight distance[c]
11 (lowest priority)	Lateral offset	Lateral offset	Lateral offset

[a] With hidden curve, intersection, or driveway.
[b] With hidden curve or ramp junction.
[c] With no hidden features.

SECTION 7

Interpretation of Results

This section addresses the interpretation of the research results presented in Sections 2 through 6 and the formulation of the recommendations that will be presented in Section 8 of the report. The interpretation focuses mainly on application of controlling criteria in reconstruction projects, which includes most projects that are designed and constructed by highway agencies today. New construction projects have fewer constraints than reconstruction projects, and it is assumed that most new construction projects will be designed to full AASHTO design criteria, whether there are controlling criteria in place or not.

The 13 controlling criteria were implemented in 1985 as an administrative control on the design process, to ensure that certain design decisions with implications for traffic operations or for crash frequencies and severities were referred to management levels above the project design team within a highway agency or within FHWA. When the project team sees a need to design one or more particular geometric features of a project to criteria less than full AASHTO design criteria, a design exception document is prepared and submitted for approval, as appropriate, to higher management levels within the highway agency or to FHWA. There was a clear rationale for this process in 1985 because the traffic operational and safety implications of the 13 controlling criteria were largely unknown. Since knowledge was lacking, judgment was required, and the design exception process was created as an administrative control to elevate that judgment to a higher management level than the project design team.

The design process in 1985 and before was very much a standards-based process, in which compliance with the design criteria in the *Green Book* or highway agency design manuals was presumed to result in design of a safe and efficient roadway. This approach is referred to in the HSM (*12*) as achieving *nominal safety* for a project. Today, as this report demonstrates, much more is known about the traffic operational and safety effects of the 13 controlling criteria. There is, thus, the potential for more of a performance-based design process that has the potential to achieve what the HSM refers to as *substantive safety*, rather than mere nominal safety. Administrative controls like the 13 controlling criteria and the design exception process, using a standards-based process, should be less necessary in today's more knowledgeable environment, when designers can explicitly consider the traffic operational or safety implications of their decisions. At present, the state of knowledge may simply support changes to the controlling criteria. Given the current and likely future state of knowledge, it may be reasonable in the future to change to a performance-based design process in which highways are designed toward target levels of crash frequency and severity, and designers have flexibility about what combinations of geometric elements are used to achieve those levels on specific roadway sections. The appropriate administrative controls to be incorporated in a performance-based process will need to be determined at a later date.

The scope of this research is limited to the 13 specific design elements for roadway geometrics that have been selected as controlling criteria by FHWA. Neither FHWA's controlling criteria nor the scope of this research address other design features such as intersections, access control, or roadside features.

Ultimately, retaining, modifying, or dropping any of the 13 controlling criteria is a policy decision, and the portion of that decision that involves federal policy is beyond the scope of this research. However, documentation of knowledge about the traffic operational and safety effects of the 13 controlling criteria, establishment of priorities for the 13 controlling criteria, and recommendations concerning modification of the controlling criteria for application to non-federal projects is within the scope of the research. All recommendations given below or in Section 8 concerning modification of the 13 controlling criteria should be read as referring to projects to which the controlling criteria are applied based on state policy, rather than federal policy.

The interpretation discussion presented below focuses on traffic operational issues in terms of the effect of specific design features on average travel speed for roadway traffic and focuses on safety issues in terms of average fatal-and-injury crash frequency. Whenever the following discussion mentions crash frequency, it is referring specifically to fatal-and-injury crash frequency. The interpretations presented below tend to be based more on safety effects than on traffic operational effects, because safety concerns tend to be the focus of most discussions about design criteria.

The sensitivity analyses presented in Section 6 addressed rural two-lane highways, rural multilane highways, and rural freeways. While the sensitivity analyses addressed rural freeways, the results of these analyses appear applicable to urban freeways as well, so the interpretations below are applicable to freeways of all types.

Interpretation of the research results for each type of controlling criteria is presented below, followed by two brief summary sections.

7.1 Shoulder Width

The research results indicate that shoulder width should remain as a controlling criterion for rural two-lane highways, rural multilane highways, and rural and urban freeways. Shoulder width has the largest effect on crash frequency of any of the controlling criteria for rural highways. Shoulder width also has the largest effect on traffic speed of any of the controlling criteria for rural two-lane highways. Thus, it is reasonable that, if part of the current design exception process is to remain in place, highway agencies should require design exceptions for shoulder width on rural highways.

Shoulder width is less appropriate as a controlling criterion for urban and suburban arterials. There are no documented effects of shoulder width on traffic speed or crash frequency for urban and suburban arterials. Furthermore, it is acceptable to design urban and suburban arterials meant to be lower speed roads with curb-and-gutter sections, rather than with shoulders. Therefore, there does not appear to be a strong need to retain shoulder width as a controlling criterion for urban and suburban arterials.

7.2 Lane Width

Lane width appears to be the second most important design criterion with respect to crash frequency on rural highways, and generally the first or second most important design criterion with respect to traffic speed on rural highways. Thus, lane width appears very appropriate to retain as a controlling criterion for rural highways. It should be noted that the HSM shows very limited differences in crash frequency between 11- and 12-ft lanes on rural two-lane and multilane highways (nonfreeways). It appears reasonable that designers should be provided with great flexibility to choose between 11- and 12-ft lanes for rural two-lane and multilane highways (nonfreeways) and that the controlling criterion for lane width, and thus the need for design exceptions, should apply to lane widths less than 11 ft on rural two-lane and multilane highways (nonfreeways).

There are no documented relationships that indicate an effect of lane width on crash frequency for urban and suburban arterials, and research under NCHRP Project 17-53 found no effect of lane width on traffic speed for urban and suburban arterials. Recent research found no effect of lane width on safety for urban and suburban arterials, with only limited exceptions that may possibly represent random effects (*23*, *24*). The *Green Book* provides substantial flexibility in choosing among 10-, 11-, and 12-ft lanes for urban and suburban arterials (*4*, *5*). Using narrower lanes on urban and suburban arterials can provide space for incorporation of other features that are positive for operations and safety including medians, turn lanes, bicycle lanes, parking lanes, and shorter pedestrian crossings. It appears reasonable that designers should be provided with substantial flexibility to choose among 10-, 11-, and 12-ft lanes on urban and suburban arterials and that the controlling criterion for lane width, if retained, should apply only to lane widths less than 10 ft on urban and suburban arterials. This is not intended to imply that lane widths are not an important consideration in the design of urban and suburban arterials, or that any lane width can be used at any location, but rather that lane widths should be selected on a location-by-location basis to complement the other selected features of the roadway cross section within the available cross-section width. The *Green Book* should include clear design guidance that the needs of bicycles, trucks, and buses should be considered in any decision to use lane widths less than 12 ft across all lanes on an urban or suburban arterial. Where substantial volumes of bicycles, trucks, or buses are present, consideration should be given to maintaining wider curb lanes to accommodate them, even where other lanes are narrowed to less than 12 ft. Future research on lane width effects on urban and suburban arterials is planned under NCHRP Project 03-112.

7.3 Horizontal Curve Radius

Horizontal curve radius has a documented relationship to crash frequency, either the third or fourth largest effect of any design criterion, for rural highways of all types. The effect of horizontal curve radius on crash frequency is quite substantial on horizontal curves themselves, but is limited to third or fourth place overall because horizontal curves typically constitute only a portion of the length of any extended roadway section (e.g., 20 percent of the total roadway length in the sensitivity analyses reported in Section 6). It appears appropriate to retain horizontal curve radius as a controlling criterion for rural highways.

Horizontal curve radius does influence speeds on urban and suburban arterials. There is no definitive relationship of horizontal curve radius to crash frequency for urban and suburban arterials. Hauer at al. (*32*) found on-road crash frequencies for horizontal curves on urban four-lane undivided arterials to be lower than for tangent sections in the same corridors; the opposite was found to be the case for off-road crashes. Since on-road crashes are predominant on urban arterials, this suggests that horizontal curves do not have a role in increasing crash frequencies. Based on the available evidence, consideration might be given to dropping horizontal curve radius as a controlling criterion for urban and suburban arterials, at least for arterials with design speeds of 45 mph or less.

7.4 Superelevation

The sensitivity analysis in Section 6 found that the effect of superelevation on crash frequency for rural two-lane highways is similar in magnitude to, but slightly smaller than, the effect on crash frequency of horizontal curve radius. Among the documented effects of design criteria on crash frequency for rural two-lane highways, superelevation ranks fifth in magnitude. There has been no research on the effects of superelevation on safety for rural multilane highways or freeways, but there is no reason to presume that this effect is not similar to the effect for rural two-lane highways. It seems reasonable to retain superelevation as a controlling criterion for rural highways and freeways, as long as horizontal curve radius is also retained.

Because of generally lower speeds, superelevation of horizontal curves is likely to have a much less important influence on crash frequency on urban and suburban arterials than on rural highways or freeways. Curves on urban and suburban arterials that have limited radii typically have lower speeds such that normal cross slope can be retained throughout the curve rather than using a superelevated cross section. Based on the available evidence, consideration might be given to dropping superelevation as a controlling criterion for urban and suburban arterials, at least for arterials with design speeds of 45 mph or less.

7.5 Grade

Grade has a documented effect on crash frequency for rural two-lane highways that is slightly larger than the effect of horizontal curves discussed above. This result is based on sensitivity analyses in which the grades of interest extended throughout the length of the roadway section of interest, in rolling terrain consisting of alternating upgrades and downgrades. Grade may have an effect on other rural roadway types, but this is difficult to document because very few rural multilane highways and freeways have steep grades, except in mountainous terrain where only steeper grades are practical. It seems reasonable to retain grade as a controlling criterion for rural highways, as long as the design criteria recognized that steeper grades are needed in mountainous terrain.

Grade has no documented effect on traffic speed or crash frequency on urban and suburban arterials. Steep grades on urban and suburban arterials are rare except in locations where steep terrain gives designers little choice. There does not appear to be a strong need to retain grade as a controlling criterion for urban and suburban arterials.

7.6 Stopping Sight Distance

The results of research conducted in this project (see Section 4.7) provide an important perspective on the role of stopping sight distance in safety. These results indicate that stopping sight distance has no effect on safety at crest vertical curves except when the presence of a crest vertical curve hides a horizontal curve, intersection, or driveway from the view of approaching drivers. When no hidden curve, intersection, or driveway is present, the situations in which drivers might be called upon to make a stop on a rural highway are rare. Thus, the research results indicate that stopping sight distance is much more important in some locations than in others. Our current approach to design appears to treat stopping sight distance as if it were equally important at all locations on the highway system. New construction projects generally can and should be designed to provide the full stopping sight distances presented in the AASHTO *Green Book*. However, in improvement projects on existing roadways, where stopping sight distances less than (especially just less than) AASHTO criteria are present, consideration should be given to any history of sight-distance-related crashes at the site and to the presence of hidden features that might lead to future crashes as part of any decision to invest in sight distance improvements.

Table 73 shows that even where a hidden feature is present at each crest vertical curve, the effect on crash frequency for a 5-mi section of rural two-lane highway, as a whole, is extremely small (0.02 or 0.03 percent); it is likely, however, that some hidden features could influence crash frequency more substantially depending on the frequency of slowing, turning, or stopping vehicles. There is no reason to suppose that this research finding for vertical sight restrictions would not also apply to horizontal sight restrictions caused by sight obstructions on the inside of horizontal curves. Similarly, only hidden curves, intersections, or driveways on rural multilane highways or hidden curves or ramps on freeways appear likely to increase crash frequencies. It might be argued that queues of stalled traffic could be hidden by limited sight distance on freeways, but it should be kept in mind that vehicles are substantially taller than the 2-ft object height used in stopping sight distance design, and vehicles are typically 6-ft wide and, therefore, extend 3 ft both to the left and the right of the nominal sight line along the center of a lane.

Based on the available research findings, there does not appear to be a strong case for retaining stopping sight distance as a controlling criterion for rural highways and freeways. Many millions of dollars have been spent by highway agencies in improving crest vertical curves on existing rural highways and freeways to full AASHTO design criteria while providing little or no reduction in crash frequency. Funds available for safety improvement are too scarce to be spent in ways that provide little or no safety benefit. Better design guidance would be to improve stopping sight distance on existing rural highways or freeways only where specific crash patterns are present that indicate a need for such improvements or where an approaching curve, intersection, ramp, or driveway is hidden from the driver's view by the stopping sight distance limitation. This guidance is not meant to suggest that stopping sight distance is unimportant, but rather to suggest that its importance varies substantially from location to location and is best assessed on a location-by-location basis.

There is no research on the effect of stopping sight distance on crash frequency on urban and suburban arterials. However, there is no reason to suppose that the effect of stopping sight distance on urban and suburban arterials is much different than the effect of stopping sight distance on rural highways. Indeed, given the nature of urban and suburban arterials, it is likely that intersection sight distance, which is outside the scope of this research and outside the scope of the 13 controlling criteria, has a larger effect on crash frequency on urban and suburban arterials than stopping sight distance. Therefore, consideration should be given to dropping stopping sight distance as a controlling criterion for urban and suburban arterials, while emphasizing the importance of considering stopping sight distance where specific crash patterns are present that indicate a need for such improvements or where an approaching curve, intersection, ramp, or driveway is hidden by the stopping sight distance limitation.

7.7 Bridge Width

Research conducted in this project found no relationship between bridge width and crash frequency on rural two-lane highways (see Section 4.3). Current design guidance is to maintain the full roadway width of the approach to the bridge (lane width plus shoulder width) across the bridge. However, many existing bridges have roadway widths that are narrower than the approach roadway width. The analysis reported in Section 4.3 found no evidence that such bridges, on average, experience more crashes than bridges on which the full roadway width is carried across the bridge; in fact, narrower bridges in many cases appeared to have fewer crashes than bridges on which the full roadway width is carried across the bridge, although the differences were not statistically significant. The research did not address one-lane bridges.

The research results do not indicate any need to retain bridge width as a controlling criterion for rural two-lane highways. The logical interpretation of the research results is that if an existing bridge on a rural two-lane highway has a roadway narrower than the approach roadway, is in good structural condition (i.e., does not need replacement for structural reasons), and has no accompanying pattern of crashes (e.g., fixed-object, sideswipe, or head-on collisions) indicating a concern related to bridge width, the existing bridge may remain in place. Since funds for safety improvements are limited, it would likely be preferable to find a better investment of those funds than widening a bridge that is performing satisfactorily. There is no logical reason to believe that this same design approach is not applicable to bridges on other roadway types as well.

7.8 Cross Slope

There is no research that indicates a relationship between the normal cross slope of roadway pavements and crash frequency. In fact, such research has never been conducted because existing roadway inventory data sets do not generally include the cross slopes used on roadways. As a practical matter, the normal cross slope likely does not vary much from the recommended *Green Book* (*4*, *5*) design value of 1.5 to 2 percent, with appropriate adjustments for multilane pavements and areas that experience intense rainfall. Nevertheless, pavement cross slope is important to drainage, and improper drainage could contribute to potential vehicle loss of control under some circumstances. Given the lack of research (and, indeed, the lack of data for research) on this issue and the potential consequences of poor drainage, it makes logical sense to retain cross slope as a controlling criterion. While advances in knowledge have reduced the need for some other design elements to serve as controlling criteria, there have been no advances in knowledge about the traffic operational and safety effects of cross slope, so retaining it as a controlling criterion until better knowledge is available makes sense.

The *Green Book* establishes a maximum design value of 8 percent for the cross-slope break between the outside edge of a superelevated pavement on a horizontal curve and a shoulder that slopes away from the roadway. As discussed in Section 2.10, the NTSB (*37*) has requested that FHWA and AASHTO investigate this design criterion, and research to address this issue is being conducted under NCHRP Project 03-105. Pending completion of that research, no change in the inclusion of pavement cross-slope breaks in the controlling criterion for cross slope is recommended.

7.9 Sag Vertical Curve Length

Sag vertical curves by their nature appear to be less related to crash frequency than crest vertical curves. The entire length of a sag vertical curve is visible to drivers under daylight conditions except in the rare cases where an overpass structure is present. At night, vehicle headlights illuminate only a portion of a sag vertical curve. However, it is known that headlight illumination distance is less than stopping sight distance even on level tangent roadways, so drivers "outdrive their headlights" in many roadway situations, not just on sag vertical curves. The recent change in design criteria for crest vertical curves to use a 2-ft object height indicates that the small objects implied by the headlight sight distance model for sag vertical curve design may not represent an appropriate design approach. There does not appear to be justification for treating sag vertical curve length as a controlling criterion for design.

7.10 Horizontal Clearance/Lateral Offset

Sections 2.13 and 5.1.8 document the controlling criterion for horizontal clearance and the change in terminology to lateral offset that occurred in the 2011 *Green Book* (*5*) and the 2011 RDG (*40*). Lateral offset is essentially irrelevant as a controlling criterion for roadway types other than urban and suburban arterials, because the controlling criterion for shoulder width ensures that there will be a lateral offset to roadside objects of at least 18 in. On urban and suburban arterials, any effect on traffic speed due to roadside objects less than 18 in behind the curb would be minimal. The primary function of the lateral offset design criterion is to ensure that mirrors or other appurtenances of heavy vehicles do not strike roadside objects and that passengers in parked cars are able to open their doors. While these considerations are important, they do not appear to rise to the level of importance that attaches to other design criteria that may address the likelihood of fatal-and-injury crashes and, therefore, horizontal clearance/lateral offset does not appear to need administrative control as a controlling criterion for design.

7.11 Summary of Results for Rural Two-Lane Highways, Rural Multilane Highways, and Rural and Urban Freeways

It is recommended that the following design criteria should be retained as controlling criteria for rural two-lane highways, rural multilane highways, and rural and urban freeways: shoulder width, lane width (for lane width less than 11 ft), horizontal curve radius, superelevation, grade, stopping sight distance (for locations where a hidden curve, intersection, ramp, or driveway is present), and cross slope. There does not appear to be any need, based on their traffic operational and safety effects, for the following design criteria to be retained as controlling criteria: bridge width, sag vertical curve length, and horizontal clearance/lateral offset. This does not imply that bridge width, sag vertical curve length, and horizontal clearance/lateral offset should not continue to be important design considerations; clearly, they should continue to be addressed in the *Green Book*, in highway agency design manuals, and during the design process. Rather, it means that the traffic operational and safety effects of these design criteria do not appear to rise to the level that requires an administrative control like the controlling criteria. The priority rankings of the 13 controlling criteria in Table 82 and the quantitative sensitivity analysis results presented in Section 6 provide support to this recommendation.

7.12 Summary of Results for Urban and Suburban Arterials

The research results for urban and suburban arterials presented in Sections 2 through 6 do not indicate that the roadway design features represented by the 13 controlling criteria are critical factors in the design of urban and suburban arterials. More than other roadway types, the traffic operational and safety performance of urban and suburban arterials appears to depend on factors outside the scope of the 13 controlling criteria and outside the scope of this research, such as intersection design and access management. Well-reasoned and well-explained geometric design criteria, with flexibility to adapt roadway cross sections to the specific needs of each corridor, along with appropriate intersection design and access management criteria, would appear to be of greater importance to design of urban and suburban arterials than the administrative controls provided by the 13 controlling criteria and the design exception process. Therefore, it is recommended that consideration be given to dropping application of the 13 controlling criteria to urban and suburban arterials or restricting the controlling criteria to a minimal set, including lane width (for lane widths less than 10 ft), stopping sight distance (for locations with a hidden curve, intersection, or driveway), and cross slope. A possible exception to this recommendation is for urban and suburban arterials with design speeds over 45 mph; such arterials are designed more like rural highways and the same controlling criteria as for rural two-lane highways, rural multilane highways, and rural and urban freeways might be applied.

SECTION 8

Conclusions and Recommendations

This section presents the conclusions and recommendations of the research.

8.1 Conclusions

The conclusions of the research are presented below.

For rural two-lane highways:

- Quantitative relationships between traffic speed and roadway geometric design criteria have been established in the HCM (*13*) or previous research for lane width, shoulder width, horizontal curve radius, and grade. These relationships are documented in Section 2 of this report. The effects on traffic speed of other roadway geometric design criteria are understood in a qualitative sense.
- Quantitative relationships between crash frequency or severity and roadway geometric design criteria have been established in the HSM (*12*) or previous research for lane width, shoulder width, horizontal curve radius, superelevation, and grade. These relationships are documented in Section 2 of this report. The effects on crash frequency of other roadway geometric design criteria are understood in a qualitative sense.
- Analysis of traffic crash data for bridges on two-lane rural highways as part of this research found no evidence of increased crash frequencies or severities for bridges with roadway widths (lane width plus shoulder width) narrower than the roadway width on the approach roadway.
- Analysis of crash data as part of this research found no increase of crash frequencies by crash severity level on crest vertical curves as a function of stopping sight distance for a range of stopping sight distance levels above and below the AASHTO stopping sight distance criteria. Crash frequencies increased on a crest vertical curve only when a horizontal curve, intersection, or driveway hidden from the view of approaching drivers by the crest vertical curve was present.

For rural multilane highways:

- Quantitative relationships between traffic speed and roadway geometric design criteria have been established in the HCM (*13*) or previous research for lane width, shoulder width, and grade. These relationships are documented in Section 2 of this report. The effects on traffic speed of other roadway geometric design criteria are understood in a qualitative sense.
- Quantitative relationships between crash frequency or severity and roadway geometric design criteria have been established in the HSM (*12*) or previous research for lane width and shoulder width. These relationships are documented in Section 2 of this report. The effects on crash frequency of other roadway geometric design criteria are understood in a qualitative sense.
- Analysis of traffic speed data collected upstream of and within horizontal curves on rural multilane highways as part of this research developed a model to predict the reduction in traffic speed on horizontal curves, in comparison to the traffic speed upstream of the curve, as a function of curve radius.
- Analysis of crash data as part of this research developed models to predict the crash frequency by crash severity level on horizontal curves as a function of curve length and radius.

For freeways:

- Quantitative relationships between traffic speed and roadway geometric design criteria have been established in the HCM (*13*) or previous research for lane width, shoulder width, and grade. These relationships are documented in Section 2 of this report. The effects on traffic speed of other roadway geometric design criteria are understood in a qualitative sense.

- Quantitative relationships between crash frequency or severity and roadway geometric design criteria have been established in the HSM (*12*) or previous research for lane width, shoulder width, and horizontal curve radius. These relationships are documented in Section 2 of this report. The effects on crash frequency of other roadway geometric design criteria are understood in a qualitative sense.

For urban and suburban arterials:

- There are no quantitative relationships between traffic speed, crash frequency, or crash severity and roadway geometric design criteria that have been established for urban and suburban arterials in the HCM (*13*) or the HSM (*12*). Previous research by Potts et al. (*23*, *24*) found, with limited exceptions, no statistically significant effect of lane width on crash frequency for urban and suburban arterials in the range of lane widths from 10 to 12 ft. Some effects of roadway geometric design criteria for urban and suburban arterials are understood in a qualitative sense, but, in general, roadway geometric design features appear to be less important in the traffic operational and safety performance of urban and suburban arterials than intersection features and access management strategies.
- Analysis of traffic speed data collected upstream and downstream of lane-width transitions on urban and suburban arterials as part of this research found no statistically significant effect of lane width on traffic speed.
- Analysis of traffic speed data collected upstream of and within horizontal curves on urban and suburban arterials as part of this research developed a model to predict the reduction in traffic speed on horizontal curves, in comparison to the traffic speed upstream of the curve, as a function of curve radius.

Priorities for the 13 controlling criteria by roadway type, based on traffic operational and safety effects of roadway geometric design criteria, are presented in Table 82.

8.2 Recommendations

The recommendations developed in the research are presented below. Ultimately, retaining, modifying, or dropping any of the 13 controlling criteria is a policy decision, and the portion of that decision that involves federal policy is beyond the scope of this research. However, recommendations concerning modification of the controlling criteria for application to non-federal projects are within the scope of this research. All recommendations given below concerning modification of the 13 controlling criteria should be read as referring to projects to which the controlling criteria are applied based on state policy, rather than federal policy. Recommendations presented here for changes in the controlling criteria represent simply potential changes in an administrative process that determines when a particular form of design review is needed. Except where explicitly stated, no changes to the design criteria presented in the *Green Book* or highway agency design manuals are contemplated. The primary focus of these recommendations is on design practice for reconstruction of existing roads; new construction projects appear much less likely than reconstruction projects to require design exceptions under both current and potential future procedures.

The recommendations are the following:

1. If all of the current controlling criteria are retained, it is recommended that they be renamed to minimize any potential confusion over which design features are, or are not, included as part of the controlling criteria. The recommended names for the current controlling criteria are the following:
   - Design speed
   - Lane width
   - Shoulder width
   - Bridge width
   - Structural capacity
   - Horizontal curve radius
   - Superelevation
   - Grade
   - Stopping sight distance
   - Sag vertical curve length
   - Cross slope
   - Vertical clearance
   - Lateral offset

   If these recommended names are used, the accompanying documentation should make clear that the stopping sight distance criterion includes stopping sight distance as limited by any roadway or roadside feature including crest vertical curves, sight obstructions on the inside of horizontal curves, and overpass structures. Thus, the controlling criterion for stopping sight distance directly influences the minimum crest vertical curve length for any given algebraic difference in grade and the offset to roadside sight obstructions for any curve radius on horizontal curves.
2. No need to add any new controlling criteria to the current 13 controlling criteria has been identified.
3. For rural two-lane highways, rural multilane highways, and rural and urban freeways, it is recommended that the following design criteria should be retained as controlling criteria and that design exceptions should be required: shoulder width, lane width (for lane widths less than 11 ft), horizontal curve radius, superelevation, grade, stopping sight distance (for locations where a hidden curve,

intersection, ramp, or driveway is present), and cross slope. The rationale for retention of these controlling criteria is presented in Section 7 of this report. There does not appear to be any need, based on their traffic operational and safety effects, for the following design criteria to be retained as controlling criteria: bridge width, sag vertical curve length, and horizontal clearance/lateral offset. This does not imply that bridge width, sag vertical curve length, and horizontal clearance/lateral offset are not important or that they do not need to be addressed in the *Green Book*, in highway agency design manuals, and during the design process. Rather, it means that the traffic operational and safety effects of these design criteria do not appear to rise to the level that requires an administrative control involving management review like the design exception process.

4. For rural two-lane highways, the *Green Book* and highway agency design policies for reconstruction projects should permit existing locations with limited stopping sight distance to remain in place unless there is a specific crash pattern present that indicates a need for such an improvement, or there is an approaching curve, intersection, or driveway that is hidden from the driver's view by the stopping sight distance limitation. This same guidance is likely applicable to rural multilane highways and to rural and urban freeways, but stopping sight distance limitations on these roadway types were not specifically investigated in the research.
5. For rural two-lane highways, the *Green Book* and highway agency design policies for reconstruction projects should permit existing bridges with roadway widths (lane width plus shoulder width) less than the approach width to remain in place if the bridge is in good structural condition (i.e., does not require replacement for structural reasons), and has no accompanying pattern of crashes (e.g., fixed-object, sideswipe, or head-on collisions) indicating a concern related to bridge width. This guidance is not applicable to one-lane bridges. This guidance is likely applicable to rural multilane highways and to rural and urban freeways, but narrow bridges on these roadway types were not specifically investigated in the research.
6. The implications for sag vertical curve design of the change in the target object height for crest vertical curve design to 2 ft (representing the taillight height of a vehicle), first implemented in the 2001 *Green Book* (*3*), need to be assessed in future research. If the target object for sag vertical curve design is another vehicle, the need for the current headlight sight distance criterion in sag vertical curve design appears to be moot because a vehicle's headlights are not needed to see a same-direction vehicle with illuminated taillights or an oncoming vehicle with illuminated headlights. It appears that sag vertical curve design could be based solely on considerations of drainage and driver comfort (except where an overpass structure is present). Until such research is completed, it may be premature to recommend a specific change in the *Green Book*, but there appears to be little rationale for retaining sag vertical curve length in the controlling criteria.
7. Horizontal clearance, renamed lateral offset in the 2011 *Green Book* (*5*) and the 2011 RDG (*40*), is not needed as a controlling criterion for rural two-lane highways, rural multilane highways, and rural and urban freeways because the controlling criterion for shoulder width ensures that there will be sufficient horizontal clearance/lateral offset. On urban and suburban arterials, any effect on traffic speed due to roadside objects less than 18 in behind the curb would be minimal. The primary function of the lateral offset design criterion is to ensure that mirrors or other appurtenances of heavy vehicles do not strike roadside objects and that passengers in parked cars are able to open their doors. While these considerations are important, they do not appear to rise to the level of importance that attaches to other design criteria that may address the likelihood of fatal-and-injury crashes and, therefore, horizontal clearance/lateral offset does not appear to need administrative control as a controlling criterion for design.
8. If Recommendation 7 is not acted upon and horizontal clearance is retained as a controlling criterion, it should be renamed lateral offset, with an accompanying clarification that this controlling criterion applies only to the 1.5-ft operational offset and not to wider lateral offsets now presented in the RDG (*40*) that are intended to reduce fixed-object collision for vehicles that run off the road. Alternatively, the RDG could be changed to use different terms for the 1.5-ft offset intended as an operational offset and the wider offsets intended to reduce fixed-object collision for vehicles that run off the road.
9. It is recommended that the concept of controlling criteria for roadway geometrics not be applied to urban and suburban arterials or that only a minimum set of controlling criteria be applied, including lane width (for lane widths less than 10 ft), stopping sight distance (for locations where a hidden curve, intersection, or driveway is present), and cross slope. The *Green Book* and existing highway agency design policies provide excellent guidance for the geometric design of urban and suburban arterials. More than other roadway types, the traffic operational and safety performance of urban and suburban arterials appears to depend on factors such as intersection design and access management, which are outside the scope of the 13 controlling criteria and outside the scope of this research. Well-reasoned and well-explained geometric design

criteria, with flexibility to adapt roadway cross sections to the specific needs of each corridor, along with appropriate intersection design and access management criteria, would appear to be of greater importance to design of urban and suburban arterials than the administrative controls provided by the 13 controlling criteria and the design exception process. A possible exception to this recommendation is for urban and suburban arterials with design speeds over 45 mph; such arterials are designed more like rural highways, and the same controlling criteria as for rural two-lane highways, rural multilane highways, and rural and urban freeways might be applied.

10. The established concept of the controlling criteria and the design exception process has served the profession well since 1985, given the lack of quantitative knowledge about the traffic operational and safety effects of geometric design criteria. As more knowledge has become available, it now appears appropriate to make some changes to the controlling criteria. Ultimately, the current design process itself might be replaced with a performance-based design process in which highway designers assess the traffic operational and safety effects of each design decision to develop an overall project design whose traffic operational and safety performance can be accurately estimated. The appropriate administrative controls to be incorporated into a performance-based process will need to be determined at a later date. Research is being conducted under NCHRP Project 15-47 to consider possible updates to the geometric design process, including performance-based approaches.
11. Future research on traffic operational effects of geometric design elements would be desirable for the following:
    - Shoulder width on urban and suburban arterials
    - Bridge width on rural two-lane highways, rural multilane highways, urban and suburban arterials, and freeways
    - Limited stopping sight distance on rural two-lane highways, rural multilane highways, urban and suburban arterials, and freeways
    - Lateral offset to roadside objects on urban and suburban arterials
12. Future research on safety effects of geometric design elements would be desirable for:
    - Shoulder width on urban and suburban arterials
    - Bridge width on rural multilane highways, urban and suburban arterials, and freeways
    - Horizontal curve radius on urban and suburban arterials
    - Horizontal curve superelevation on rural multilane highways, urban and suburban arterials, and freeways
    - Limited stopping sight distance on rural multilane highways, urban and suburban arterials, and freeways
    - Lateral offset to roadside objects on urban and suburban arterials

References

1. *Code of Federal Regulations*, 23 CFR 625, April 1, 2013.
2. Federal Highway Administration, *Application of Design Standards, Uniform Federal Accessibility Standards, and Bridges*, updated April 2009, http://www.fhwa.dot.gov/design/0625sup.cfm (formerly called the Non-Regulatory Supplement of the *Federal-Aid Policy Guide*, 23 *CFR* 625, March 1, 2005).
3. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, 2001.
4. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, 2004.
5. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, 2011.
6. *Special Report 214: Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation*, Transportation Research Board, National Research Council, Washington, D.C., 1987.
7. Stein, W. J., and T. R. Neuman, *Mitigation Strategies for Design Exceptions*, Report No. FHWA-SA-07-011, Federal Highway Administration, July 2007.
8. American Association of State Highway and Transportation Officials, *A Guide for Achieving Flexibility in Highway Design*, 2004.
9. Harwood, D. W., D. J. Torbic, K. R. Richard, and M. M. Meyer, *SafetyAnalyst: Software Tools for Safety Management of Specific Highway Sites*, Report No. FHWA-HRT-10-063, Federal Highway Administration, 2010.
10. FHWA Interactive Highway Safety Design Model website, www.ihsdm.org.
11. American Association of State Highway and Transportation Officials, *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)*, 2001.
12. American Association of State Highway and Transportation Officials, *Highway Safety Manual*, 2010.
13. *Highway Capacity Manual 2010*,Transportation Research Board of the National Academies, Washington, D.C., 2010.
14. American Association of State Highway and Transportation Officials, *A Policy on Design Standards—Interstate System*, January 2005.
15. FHWA Crash Modification Factors Clearinghouse website, www.cmfclearinghouse.org.
16. Federal Highway Administration, *Manual on Uniform Traffic Control Devices for Streets and Highways*, 2009.
17. Hauer, E., "Speed and Safety," *Transportation Research Record: Journal of the Transportation Research Board, No. 2103*, Transportation Research Board of the National Academies, 2009, pp. 10–17.
18. Zegeer, C. V., R. C. Deen, and J. G. Mayes, "Effect of Lane and Shoulder Width on Accident Reduction on Rural, Two-Lane Roads," *Transportation Research Record 806*, Transportation Research Board, National Research Council, Washington, D.C., 1981, pp. 33–43.
19. Griffin, L. I., and K. K. Mak, *The Benefits to be Achieved From Widening Rural, Two-Lane Farm-to-Market Roads in Texas*, Report No. IAC (86-87)-1039, Texas Transportation Institute, College Station, Tx., April 1987.
20. Harkey, D. L., R. Srinivasan, J. Baek, F. M. Council, K. Eccles, N. Lefler, F. Gross, B. Persaud, C. Lyon, E. Hauer, and J. Bonneson, *NCHRP Report 617: Crash Reduction Factors for Traffic Engineering and ITS Improvements*, Transportation Research Board of the National Academies, Washington, D.C., 2008.
21. American Association of State Highway and Transportation Officials, *Guide for the Development of Bicycle Facilities*, 1999.
22. *Highway Capacity Manual*, Transportation Research Board, National Research Council, Washington, D.C., 2000.
23. Potts, I. B., D. W. Harwood, and K. R. Richard, "Relationship of Lane Width to Safety on Urban and Suburban Arterials," *Transportation Research Record: Journal of the Transportation Research Board, No. 2023*, Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 63–82.
24. Potts, I. B., D. W. Harwood, D. J. Torbic, K. M. Bauer, K. R. Richard, D. K. Gilmore, D. K. Lay, J. F. Ringert, J. D. Zegeer, D. L. Harkey, and J. M. Barlow, "Lane Widths, Channelized Right Turns, and Right-Turn Deceleration Lanes on Urban and Suburban Arterials," Final Report of NCHRP Project 03-72, Midwest Research Institute, August 2006.
25. Bonneson, J., S. Geedipally, M. Pratt, and D. Lord, Preliminary Draft of HSM Chapter Entitled "Predictive Method for Freeways," Final Report of NCHRP Project 17-45, Texas Transportation Institute, College Station, Tx., August 2012.
26. Lord, D., S. R. Geedipally, B. N. Persaud, S. P. Washington, I. van Schalkwyk, J. N. Ivan, C. Lyon, and T. Jonsson, *NCHRP Web-Only Document 126: Methodology to Predict the Safety Performance of Rural Multilane Highways*, Transportation Research Board of the National Academies, Washington, D.C., 2008.
27. Turner, D. S., "Prediction of Bridge Accident Rates," *Journal of Transportation Engineering*, Vol. 110, No. 1, American Society of Civil Engineers, January 1984, pp. 45–54.
28. Bigelow, J., Z. Hans, and B. Phares, "Bridge Rail and Approach Railing for Low-Volume Roads in Iowa," Iowa Highway Research Board Project TR-592, Institute for Transportation, Iowa State University, 2010.
29. Fitzpatrick, K., L. Elefteriadou, D. W. Harwood, J. M. Collins, J. McFadden, I B. Anderson, R. A. Krammes, N. Irazarry, K. D. Parma,

K. M. Bauer, and K. Passetti, *Speed Prediction for Two-Lane Rural Highways*, Report No. FHWA-R0-00-171, Federal Highway Administration, August 2000.

30. Zegeer, C. V., J. R. Stewart, F. M. Council, D. W. Reinfurt, and E. Hamilton, "Safety Effects of Geometric Improvements on Horizontal Curves," *Transportation Research Record 1356*, Transportation Research Board, National Research Council, Washington, D.C.,1992, pp. 11–19.
31. Bauer, K. M., and D. W. Harwood, *Safety Effects of Horizontal Curve and Grade Combinations on Rural Two-Lane Highways*, Report No. FHWA-HRT-13-077, Federal Highway Administration, January 2014.
32. Hauer, E., F. M. Council, and Y. Mohammedshah, "Safety Models for Urban Four-Lane Undivided Road Segments," *Transportation Research Record: Journal of the Transportation Research Board, No. 1897*, Transportation Research Board of the National Academies, Washington. D.C., 2004, pp. 96–105.
33. Harwood, D. W., D. J. Torbic, K. R. Richard, W. D. Glauz, and L. Elefteriadou, *NCHRP Report 505: Review of Truck Characteristics as Factors in Roadway Design*, Transportation Research Board of the National Academies, Washington, D.C., 2003.
34. Maiou, S-P., *Vertical Grade Analysis Summary*, unpublished, May 1998.
35. Harwood, D. W., F. M. Council, E. Hauer, W. E. Hughes, and A. Vogt, *Prediction of the Expected Safety Performance of Rural Two-Lane Highways*, Report No. FHWA-RD-99-207, Federal Highway Administration, December 2000.
36. Fambro, D. B., K. Fitzpatrick, and R. J. Koppa, *NCHRP Report 400: Determination of Stopping Sight Distances*, Transportation Research Board, National Research Council, Washington, D.C., 1997.
37. *Highway Accident Report: Tanker Truck Overturn and Fire, Indianapolis, Indiana, October 22, 2009*, National Transportation Safety Board, 2011, http://www.ntsb.gov/news/events/2011/indianapolis_in/synopsis.html.
38. Glennon, J. C., T. R. Neuman, R. R. McHenry, and B. G. McHenry, *HVOSM Studies of Cross Slope Breaks on Highway Curves, Phase II Task C*, Report No. FHWA-RD-82-54, Federal Highway Administration, May 1982.
39. American Association of State Highway and Transportation Officials, *Roadside Design Guide*, 2006.
40. American Association of State Highway and Transportation Officials, *Roadside Design Guide*, 2011.
41. Mason, J. M., and K. M. Mahoney, *NCHRP Synthesis 316: Design Exception Practices*, Transportation Research Board of the National Academies, Washington, D.C., 2003.
42. McGee, H. W., W. E. Hughes, and K. Daily, *NCHRP Report 374: Effect of Highway Standards on Safety*, Transportation Research Board, National Research Council, Washington, D.C., 1995.
43. Stamatiadis, N., K. R. Agent, and J. G. Pigman, "Design Exceptions and Safety," Compendium of Papers, Third International Symposium on Highway Geometric Design, Transportation Research Board, Chicago, Il., 2005.
44. Gross, F., P. P. Jovanis, and K. A. Eccles, *Safety Evaluation of Lane and Shoulder Width Combinations on Rural, Two-Lane, Undivided Roads*, Report No. FHWA-HRT-09-031, Federal Highway Administration, June 2009.
45. American Association of State Highway and Transportation Officials, *Manual for Assessing Safety Hardware*, 2009.
46. Mak, K. K., and D. L. Sicking, *NCHRP Report 492: Roadside Safety Analysis Program (RSAP)—Engineer's Manual*, Transportation Research Board of the National Academies, Washington, D.C., 2003.
47. RoadSafe LLC, Appendix A: RSAPv3 User's Manual. In "Roadside Safety Analysis Program (RSAP) Update," Final Report of NCHRP Project 22-27, http://rsap.roadsafellc.com, October 2012.
48. RoadSafe LLC, Appendix B: RSAPv3 Engineer's Manual. In "Roadside Safety Analysis Program (RSAP) Update," Final Report of NCHRP Project 22-27, http://rsap.roadsafellc.com, October 2012.
49. Dixon, K. K., M. Liebler, H. Zhu, M. P. Hunter, and B. Mattox, *NCHRP Report 612: Safe and Aesthetic Design of Urban Roadside Treatments*, Transportation Research Board of the National Academies, Washington, D.C., 2008.
50. McGee, H. W., *NCHRP Synthesis 417: Geometric Design Practices for Resurfacing, Restoration, and Rehabilitation*, Transportation Research Board of the National Academies, Washington, D.C., 2011.

Abbreviations and acronyms used without definitions in TRB publications:

A4A	Airlines for America
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
MAP-21	Moving Ahead for Progress in the 21st Century Act (2012)
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