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This report describes the development of recommended revisions to the stopping sight distance (SSD) design policy that appears in portions of Chapters II and III of the 1994 AASHTO publication, *A Policy on Geometric Design of Highways and Streets* (referred to as the *Green Book*). It also proposes modifications to other sections of the *Green Book* that currently reference stopping sight distance. The contents of this report are, therefore, of immediate interest to highway designers; highway-operations, capacity, and traffic-control personnel; and others concerned with highway safety. The report's conclusions are derived from field observations of driver performance, driver visual capacity, driver eye heights, and vehicle heights, as well as safety and operational studies.

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Report 400

Determination of Stopping Sight Distances

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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

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

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FOREWORD

By Staff
Transportation Research Board

This report describes the development of recommended revisions to the stopping sight distance (SSD) design policy that appears in portions of Chapters II and III of the 1994 AASHTO publication, *A Policy on Geometric Design of Highways and Streets* (referred to as the *Green Book*). It also proposes modifications to other sections of the *Green Book* that currently reference stopping sight distance. The contents of this report are, therefore, of immediate interest to highway designers; highway-operations, capacity, and traffic-control personnel; and others concerned with highway safety. The report's conclusions are derived from field observations of driver performance, driver visual capacity, driver eye heights, and vehicle heights, as well as safety and operational studies.

The current AASHTO stopping sight distance model has two components: (1) perception-reaction time, which is equated to the distance a vehicle travels at a fixed speed while these actions occur, and (2) braking distance, the distance the vehicle travels during the braking maneuver. This model has been altered only slightly since its inception in the 1940s, and it continues to result in well-designed roads. However, the hypothesis that the worst-case scenario—with its conservative assumptions of reaction time and pavement friction values and unproven driver visual capabilities—combined with an assumed below average driver, results in a model that provides a considerable margin of safety but is difficult to justify or defend as representative of either a real-life environment or a safe driving behavior.

The Texas Transportation Institute (TTI) at Texas A&M University was awarded NCHRP Project 3-42, *Determination of Stopping Sight Distances*, to evaluate, on the basis of the impact on vertical and horizontal curve design, the current AASHTO methodology and alternative approaches to establishing stopping sight distance. TTI produced five working papers describing their research, under controlled testing environments, into the different aspects that make up the components of the SSD: *(1) Driver Braking Performance*, which studied drivers' perception and reaction times in unexpected situations, deceleration characteristics of unexpected braking, and braking distances associated with those events; *(2) Driver Visual Capacity*, which measured driver capability in detecting and recognizing objects of various sizes and contrasts under different lighting conditions; *(3) Driver Eye and Vehicle Heights*, which collected real-world data to construct a cumulative distribution of driver eye, headlight, taillight, and vehicle heights as determined by a more current (1994) vehicle fleet; *(4) Safety Effects,* which collected and analyzed accident data for identified roadway segments containing limited sight distance crest vertical curves; and *(5) Operational Effects*, which evaluated the relationship between design and operating speeds at crest vertical curves with limited stopping sight distance. The information developed in these working papers is the basis for this report.

The recommended SSD model remains conceptually the same as the existing AASHTO model, that is, SSD equals reaction distance plus braking distance, but with initial speed equal to design speed and design deceleration substituted for friction coefficient. As with the current model, the minimum SSD, driver eye height, and object height values are used to calculate required minimum length of vertical curve (VC) and required minimum rate of curvature or lateral clearance on horizontal curves. The recommended changes result in proposed SSDs that fall between current AASHTO minimum and desirable design values, crest K values (K is the VC length divided by the algebraic difference of adjoining grades) that are slightly below AASHTO current minimum design values, sag K design values that fall between current AASHTO minimum and desirable design values, and horizontal curve offsets that are also between AASHTO current minimum and desirable design values. This research was undertaken, not because of safety concerns with the current AASHTO SSD model, but to propose scientifically-based, reproducible, rational SSD design values reflecting driver capabilities and performance.

CONTENTS

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This work was sponsored by the American Association of State Highway and Transportation Officials in cooperation with the Federal Highway Administration, and was conducted through the National Cooperative Highway Research Program, which is administrated by the Transportation Research Board of the National Research Council.

The research reported herein was performed under NCHRP Project 3-42 by the Texas Transportation Institute (TTI). The majority of the work was performed by personnel in TTI's Design and Operations Program. Primary authors of this report were Dr. Daniel B. Fambro, Dr. Kay Fitzpatrick, and Dr. Rodger J. Koppa. Supporting authors, in alphabetical order, were Dr. Joseph D. Blaschke, Dr. Lindsey I. Griffin III, Ms. Karen B. Kahl, Mr. Torsten Lienau, Mr. Valmon Pezoldt, Mr. Dale L. Picha, Mr. Charles W. Russell, and Ms. Angela M. Stoddard.

Dr. Daniel B. Fambro served as the principal investigator for NCHRP Project 3-42 and as such, was responsible for overall project coordination. Dr. Fitzpatrick coordinated the studies on safety effects, operational effects, and driver eye and vehicle heights described in Appendixes E, F, and G, respectively, and Dr. Koppa coordinated the driver performance studies and vehicle and roadway performance studies described in Appendixes C and D, respectively. Mr. Richard A. Zimmer and Mr. John Curik developed the instrumentation for the driver performance studies. Other project staff members at TTI who assisted with the data collection and reduction included Mr. Keith Behrens, Ms. Andre Berman, Mr. Scott Cooner, Mr. Aldolfo Garcia, Mr. Michael Lloyd, Ms. Sherrill Packenbush, Mr. Brian Shamburger, Ms. Lee Anne Shull, Mr. Lance Simmons, and Mr. Dan Walker.

The staffs of the Illinois Department of Transportation, Texas Department of Transportation, and Washington Department of Transportation were of great assistance during the data collection and analysis of this project. The research team is especially grateful for the contributions of Mr. Ken Lazar and Mr. John Blair of the Illinois Department of Transportation, Mr. Jim Templeton of the Texas Department of Transportation, and Mr. Larry Messmer of the Washington Department of Transportation.

The diligence, thoroughness, and thought-provoking suggestions of the NCHRP Project 3-42 panel was greatly appreciated. Panel members included Dr. Jerome Hall (Chair), University of New Mexico; Mr. Philip H. DeCabooter, Wisconsin Department of Transportation; Mr. Kenneth Hintzman, California Department of Transportation; Mr. C. Larry King, Federal Highway Administration (retired); Mr. Mark Marek, Texas Department of Transportation; Mr. Richard Radlinski, National Highway Transportation Administration (retired); Mr. Robert H. Raymond, Pennsylvania Department of Transportation; Mr. John Sanford, Illinois Department of Transportation; Ms. Alison Smiley, Human Factors North, Inc.; Dr. Daniel S. Turner, University of Alabama; Mr. Robert Walters, Arkansas Department of Transportation; Mr. Justin True, Federal Highway Administration. Lastly the patience, support, and guidance from Mr. Lloyd R. Crowther of the National Highway Cooperative Research Program are gratefully acknowledged.

DETERMINATION OF STOPPING SIGHT DISTANCES

SUMMARY

According to the American Association of State Highways and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets* (herein referred to as the *Green Book*), sight distance is the length of roadway ahead that is visible to the driver. The Green Book also states that the minimum sight distance at any point on the roadway should be long enough to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below average driver or vehicle to stop in this distance.

Problem Statement. The current procedure for determining required stopping sight distances (SSDs) is intended to allow a normally alert passenger-car driver, traveling at or near the design speed on wet pavement, to react and stop the vehicle before striking a stationary object in the road. AASHTO's basic model to describe this situation was developed in 1940. Although parameter values within the model have changed to reflect changes in the driver/vehicle fleet, the basic model has remained unchanged. Recently, several researchers have questioned the model's validity and applicability to a stopping situation. Additionally, the safety benefits of longer or shorter stopping sight distances have never been documented.

Research Objective. This research evaluated the AASHTO and other stopping sight distance models in the literature and developed recommended design procedures for AASHTO's consideration. Specific tasks were as follows:

- Review Pertinent Literature
- Evaluate Existing Data Bases
- Critical Analysis of SSD Models
- Prepare Interim Report
- Driver Performance Studies
- Driver Visual Capability Studies
- Driver Eye and Vehicle Height Studies
- Accident Studies
- Operating Speed Studies
- Tort Liability Survey
- Prepare Final Report

Background. The AASHTO stopping sight distance model consists of two components (perception-reaction and braking) and is based on the simple laws of physics; that is, the vehicle travels a certain distance during perception-reaction time and a certain distance while braking to a stop. Parameter values within the model are based on a below average driver, vehicle, and roadway, and the driver's capability to detect and stop for a small object in the roadway; however, the probability of all parameters being critical at the same time is extremely small. Thus, the resultant model includes a considerable margin of safety.

Why Change? Despite the criticisms in the literature, most people agree that the AASHTO stopping sight distance model results in well-designed roads; i.e., roads that are safe, efficient, and economical. If so, why initiate a research project to develop a revised model? The major criticism of the current model is that its parameters are not representative of the driving environment or safe driving behavior. Thus, even though its use results in a good design, it is difficult to justify, validate, and defend as a good model.

Driver Performance SSD Model. This research proposed a revised stopping sight distance model based on driver capabilities and performance in response to an unexpected object in the road. The recommended model is as follows:

$$SSD = 0.278Vt + 0.039\,V^2/a$$

where: SSD = stopping sight distance (m);
V = initial speed (km/h);
t = driver perception-brake reaction time (sec);
a = driver deceleration (m/s²).

An implicit assumption of a driver performance SSD model is that the tire-pavement friction must meet or exceed the driver's demands for stopping. The following sections describe the calibration and validation studies that support the proposed SSD model.

Comparison with Other Countries. When comparing the current AASHTO stopping sight distance model with those used by other countries, it was noted that many countries use measured or estimated 85th percentile operating speeds as the design speed. In addition, many countries use shorter perception-brake reaction times and friction values than those assumed by AASHTO. As a result, AASHTO stopping sight distance values are near the top of the range of values.

AASHTO eye heights are in the middle of the range of values and their object heights are near the bottom of the range of values. Thus, AASHTO vertical curve lengths (*K* values) are near the top of the range of values. In summary, AASHTO's stopping sight distances and vertical curve lengths are longer than those of most other countries; that is, AASHTO's values are more conservative.

Vehicle Braking. Several authors have debated locked-wheel versus controlled braking as the assumed behavior for a stopping sight distance model. Controlled braking requires longer braking distances, but offers greater steering control. Antilock braking distances can approach locked-wheel braking distances without loss of steering control. Braking simulation and field studies documented in the literature support these general statements.

Large trucks require longer braking distances than passenger cars; however, most large trucks are capable of stopping within AASHTO design braking distances on dry pavements. With antilock braking systems, they are also capable of stopping within AASHTO design braking distances on wet pavements.

Pavement Friction. Pavement friction data were obtained from California, Texas, and a North American data base. These data showed that the friction capabilities for the large majority of roadways exceeded the friction values assumed by the AASHTO stopping sight distance model. Thus, the roadway provides an additional factor of safety even in wet weather conditions.

Driver Performance. This task involved assessing perception-brake reaction times and driver deceleration in response to an unexpected object in the roadway. In addition to a review of the literature, 45 drivers and 3,000 braking maneuvers were recorded and analyzed under a variety of geometric, weather, and surprise conditions. Data were collected under closed-course and open-roadway conditions.

The perception-reaction time results showed 2.5 sec as a 90th to a 95th percentile value; that is, most drivers were capable of perception-brake reaction to a stopping sight situation within 2.5 sec. These findings were consistent with those in the literature. The braking studies and the literature also showed no differences in the perception-brake reaction times of younger and older drivers.

The driver deceleration results showed 3.4 m/sec^2 as the 10th percentile value; that is, when asked to stop as quickly as possible on wet pavements, most drivers selected decelerations of 3.4 m/sec^2 or greater. This value can be attained without a loss of steering control and is near values defined as "comfortable" by traffic engineering textbooks.

Driver Visual Capabilities. This task involved assessing driver visual capabilities in detecting objects in the roadway. In addition to a literature review, the ability of 65 drivers to detect 13 different objects (450 driver-object combinations) during both day and night were recorded and analyzed. Data were collected under closed-course conditions.

The driver visual capability results showed that during daytime conditions most drivers were able to detect (but not recognize) small, high contrast objects at the minimum stopping sight distance for most rural highways (130 m). Under nighttime conditions, however, drivers could not detect or recognize objects of any size at 130 m unless the object was illuminated or retro reflective.

Driver Eye and Vehicle Heights. This task involved assessing driver eye and vehicle heights important to stopping sight distance models. In addition to a literature review, more than 1,500 driver eye, headlight, taillight, and vehicle heights were collected and analyzed. Passenger-car, multipurpose vehicle, and large truck data were collected in four different geographic regions.

The passenger-car results showed 10th percentile driver eye, headlight, and taillight heights of 1,080 mm, 600 mm, and 640 mm, respectively. The 90th percentile vehicle height was 1,315 mm. The data and the literature also showed the vehicle fleet as approximately 2/3 passenger cars and 1/3 multipurpose vehicles. These values are higher than the current AASHTO parameter values.

Safety Studies. This task involved assessing the safety impacts of providing less than the minimum stopping sight distances required by the AASHTO *Green Book.* Forty-three limited stopping sight distance sites (439 accidents) in three states were identified and studied. Detailed geometric and accident data were collected and analyzed to determine the frequency of limited stopping sight distance as a causal factor in accidents on these roadways.

The safety study results showed that neither limited stopping sight distance nor moderate reductions in available sight distance appeared to create a safety problem on the roadways in the study's data base. Additionally, moderate reductions in stopping sight distance do not appear to create a safety problem for large trucks or older drivers.

Operational Studies. This task involved assessing the operational effects of providing less than the minimum stopping sight distances required by the AASHTO *Green Book.* Thirty-six limited stopping sight distance sites were identified in three states.

Detailed geometric data and paired speeds were collected and analyzed for more than 3,500 vehicles to determine the effect of limited stopping sight distance on desired operating speeds of drivers.

The operational study results showed that the 85th percentile free flow speeds were well above the inferred design speed of the crest vertical curves. The results also suggested that reductions in available sight distance resulted in reductions in operating speed; however, the reduction is less than that assumed by AASHTO.

Recommendations. For consistency, it is recommended that the parameters within the stopping sight distance model represent common percentile values from the underlying probability distributions. Specifically, 90th (or 10th) percentile values are recommended for design. The resultant values for design are as follows:

- One design speed and stopping sight distance;
- Perception-brake reaction time—2.5 sec;
- Driver deceleration—3.4 m/sec^2;
- Driver eye height—1,080 mm; and
- Object height—600 mm.

Impacts of Recommended Changes. The impacts of the recommended changes are design stopping sight distances that are between current minimum and desirable values. Also, K values for crests will be slightly below current minimum values and K values for sags will be between current minimum and desirable values. Finally, offsets for horizontal curves will be between current minimum and desirable values.

The recommended changes should have no impact on safety as there is no evidence that accident rates increase when stopping sight distance and vertical curve lengths are decreased by small amounts unless there is a nearby intersection or horizontal curve. Because there is no evidence of an SSD-related accident problem for large trucks or older drivers, the recommended changes should also have no impact on large truck or older driver safety.

Necessary Changes to the *Green Book*. To implement the recommended changes, the following changes to AASHTO's 1994 *A Policy on Geometric Design for Streets and Highways* will be required:

- Revise design, operating, and running speed definitions;
- Delete locked-wheel braking and friction discussion and replace it with driver deceleration discussion;
- Revise driver eye and object height discussions; and
- Revise related tables and graphs.

Advantages of the Recommended Model. The recommended model is based on driver capabilities and performance that can be validated and defended as representative of the driving environment and safe driving behavior. These findings and recommendations have been presented to the AASHTO Task Force on Geometric Design for its consideration in future revisions of the *Green Book.*

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

According to the American Association of State Highway and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets* (*1*, *2*, *3*) (referred to herein as the *Green Book*), sight distance is the length of roadway ahead that is visible to the driver. The *Green Book* also states that the minimum sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average driver or vehicle to stop in this distance.

The current procedures for determining stopping sight distance (SSD) are intended to allow a normally alert passenger-car driver, traveling at or near the highway design speed on a wet pavement, to react and bring the vehicle to a stop before striking a stationary object in the road. The basic model for this situation was formalized by the then American Association of State Highway Officials (AASHO) in 1940 (*4*). Over the past 50 years, several of the model's parameters have been modified to account for changes in the vehicle-driver-roadway system (*5*, *6*, *7*).

PROBLEM STATEMENT

NCHRP Report 270, Parameters Affecting Stopping Sight Distance, (1984) raised concerns about the model's validity as well as the appropriateness of certain parameter values used to calculate stopping sight distance (*8*). Subsequent research has revealed additional concerns about the validity of the model. Examples of such research are cited in the *Transportation Research Record 1208,* Highway Sight Distance Design Issues (1989) (*9*).

Stopping sight distance influences the geometric design of streets and highways, most notably horizontal and vertical alignment. These design features add to the cost of new highway construction and can dramatically increase the cost of major roadway construction. Use of decreased pavement-tire values, and lower eye and object heights (as recommended in *NCHRP Report 270*) in the current AASHTO SSD model would lengthen the desirable stopping sight distance and increase vertical curve length. State highway agencies report that increasing SSD would create serious problems and substantially increase costs without a demonstrated safety benefit.

Some older research studies, attempting to relate stopping sight distance to safety, appear to be inconclusive and inconsistent. Research results also have suggested that the current SSD model does not properly reflect the actual driving environment. Some recent studies, however, have shown that safety is apparently not compromised when actual stopping sight distances are marginally less than current standards. Considering the high construction costs and uncertain safety benefits associated with longer stopping sight distances, state highway officials concluded that a substantial research effort was needed to evaluate available information, add to it, and recommend improvements to current practice.

RESEARCH OBJECTIVES

The objective of this research was to evaluate, on the basis of the impact on vertical and horizontal curve design, the current AASHTO methodology and alternative approaches to establishing stopping sight distance. Based on a review of current and alternative practices, updated vehicle-performance characteristics, and updated driver-behavior data, the research team was to develop recommended design procedures for specific applications. Issues such as the variability of the roadway facility, cost-safety-effectiveness of the design and ease of applying the SSD model were to be taken into account.

To accomplish this objective, this research was performed in two phases and ten tasks. The phases and tasks are described briefly as follows.

Phase I

Task 1—Critical Review of Pertinent Literature. This task involved reviewing the literature to identify and analyze the state of the art as it pertains to stopping sight distance for driver-vehicle-roadway relationships; accident data, tort liability exposure, and cost-safety effectiveness methodologies; and alternative SSD models including those considered or adopted by other countries.

Task 2—Assessment of Existing Data Bases. This task involved assessing the adequacy and accessibility of existing local, state, and national data bases to identify pertinent information on national vehicle fleet, roadway pavement characteristics, measures of effectiveness of changes in stopping sight distances, and accident experience to assess the impact of various stopping sight distance models.

Task 3—Critical Analysis of SSD Models. This task involved critical analysis of the AASHTO and other promising SSD models including the parameters in each model. Advantages and disadvantages of each model were documented, including model practicality, complexity, and representativeness of the driver-vehicle-roadway system. Deficiencies in the data needed to accurately assess the model's impact, validity, or practicality were documented. Finally, recommendations were made concerning retention or refinement of the current AASHTO SSD model or an alternative to the AASHTO SSD model.

Task 4—Preparation of Interim Report. This task involved preparation of an interim report in two parts. The first part of the report documented the results of Tasks 1, 2, and 3. The second part of the report was a detailed revision of the Phase II portion of the original work plan describing the rationale, methods, required data, and data collection plan and schedule.

Phase II

Task 5—Driver Braking Performance Studies. This task involved assessing perception-brake reaction times and driver deceleration in response to an unexpected object in the roadway. In addition to a literature review, more than 3,000 braking maneuvers were recorded and analyzed under a variety of geometric, weather, and surprise conditions.

Task 6—Driver Visual Capability Studies. This task involved assessing driver visual capabilities in detecting objects in the roadway. In addition to a literature review, the ability of approximately 100 drivers to detect a variety of common objects (450 driver-object combinations) during both day and night were recorded and analyzed.

Task 7—Driver Eye and Vehicle Height Studies. This task involved assessing driver eye and vehicle heights important to stopping sight distance models. In addition to a literature review, more than 1,500 driver eye, headlight, taillight, and vehicle heights were collected and analyzed. The data were collected in four geographic regions.

Task 8—Safety Studies. This task involved assessing the safety impacts of providing less than the minimum stopping sight distances required by the AASHTO *Green Book.* Forty-three limited stopping sight distance sites in three states were identified. Detailed geometric and accident data were collected and analyzed to determine the frequency of limited stopping sight distance as a causal factor in accidents.

Task 9—Operational Studies. This task involved assessing the operational effects of providing less than the minimum stopping sight distances required by the AASHTO *Green Book.* Thirty-six limited stopping sight distance sites were identified in three states. Detailed geometric data and paired speeds were collected and analyzed for more than 3,500 vehicles to determine the effect of limited stopping sight distance on desired operating speeds of drivers.

Task 10—Preparation of Final Report. This task involved preparation of a final report covering the entire project. The final report contains the research findings, the recommended model and parameter values, recommendations for implementation, and the associated data to support these recommendations.

REPORT ORGANIZATION

This report is divided into four chapters and nine appendixes. Of the appendixes, only Appendix I is published herein. Chapter 1 describes the research problem, objective, and approach. Chapter 2 presents the research findings including the recommended SSD model. Chapter 3 describes the implications and practical applications of the recommended model. Chapter 4 presents the conclusions and recommendations from this research.

Appendix A discusses the history of the AASHTO stopping sight distance model and provides comparisons with international models. Appendix B describes vehicle braking and pavement friction characteristics of importance to stopping sight distance models. Appendix C describes driver braking performance measures (perception-reaction time and deceleration) in response to an unexpected object in the roadway. Appendix D discusses driver visual capabilities in detecting unexpected objects in the roadway.

Appendix E discusses driver eye, taillight, and vehicle heights that could be used in stopping sight distance models. Appendix F discusses the safety studies of limited stopping sight distance. Appendix G discusses the operating studies of limited stopping sight distance. Appendix H discusses tort liability issues. Appendix I describes the recommended revisions to the design policies in the AASHTO publication, *A Policy on Geometric Design of Highways and Streets*, known as the *Green Book.*

CHAPTER 2

FINDINGS

This research examined the current AASHTO methodology, alternative approaches, and related driver, vehicle, and roadway parameters for establishing stopping sight distances for vertical and horizontal curve design. This chapter presents a summary of the major findings—from previous research documented in the literature and the field studies—that were a part of this research.Topics include stopping sight distance models and issues, vehicle and roadway performance, driver braking performance, driver visual capabilities, driver eye and vehicle heights, safety effects, and operational effects. Each of these topics is discussed in the sections that follow.

BACKGROUND AND ISSUES

Sight distance is the length of roadway ahead that is visible to the driver. From a geometric design standpoint, the minimum sight distance available on a roadway should be long enough to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at each and every point along the highway should be at least the same as that required for a below-average driver or vehicle to stop.

The current procedures for determining stopping sight distance are intended to allow a normally alert passenger-car driver, traveling at or near the highway design speed on wet pavement, to react and bring the vehicle to a stop before striking a stationary object in the road. The basic model for this situation was formalized by AASHO in 1940 (*4*). Since that time, several modifications of the model's parameters have been made to account for changes in the vehicle-driver-roadway system; however, differences of opinion exist concerning the appropriateness of the model and the parameter values used to determine minimum required stopping sight distances.

Despite the alleged shortcomings of the current model (*1*), no evidence has been found to show that use of the AASHTO stopping sight distance model (SSD model) results in unsafe or badly designed roadways. Roadways with limited stopping sight distance may have safety problems, but these problems do not appear to be related to the available stopping sight distance. In addition, driver visual capabilities and vehicle headlights during nighttime conditions limit the visible length of roadway to distances less than the minimum stopping sight distance. Thus, from both a safety and a practical point of view, there is no apparent basis for recommending longer stopping sight distances for design.

The lack of a safety problem is the result of the AASHTO SSD model's use of extreme (upper percentile) values for each individual parameter in the model. In reality, the probability that all of the parameters will be critical at the same time is extremely low. For example, assuming independent events, the probability of occurrence of a driver with an 85th percentile speed and perception-brake reaction time and a 15th percentile deceleration and eye height is 0.0005; whereas, the probability of occurrence of a driver with a 90th percentile speed and perception-brake reaction time and a 15th percentile deceleration and eye height is 0.0001. The probability of occurrence is extremely small even if the events are dependent. Add the probability of there being an unexpected object in the roadway located over the crest of a hill, and the probability of occurrence is even smaller.

Given that the current AASHTO model appears to be conservative, and its use does not result in unsafe or badly designed roadways, why pursue a study to develop a new model? The need for such a study has been described elsewhere (*10*) as follows:

- The current SSD model was based on common sense, engineering judgment, and the laws of physics; however, the parameters within the model are not representative of the driving environment. Thus, the parameters are difficult to justify, validate, and defend.
- It has never been established on the basis of data that the provision of longer sight distance on curves results in fewer accidents. Conversely, it has never been established on the basis of data that, at least for marginal reductions, provision of shorter sight distance on curves results in more accidents.

Several problems with the individual SSD model parameters have been identified. For example, individual model parameters have been criticized as representative of drivers that do not exist; objects that do not exist and cannot be seen; and an assumed braking condition that does not exist, is unsafe, and not representative of real-world driver behavior. The question then becomes, what to do with a model that produces good answers, but does so for the wrong reasons?

Given the concerns with some of the parameters in the existing model, should these parameters be retained and what form should any new model take? Should newly developed models be simple or complex? Do more complex models produce more accurate or better estimates of stopping sight distances, and does the improved accuracy outweigh the extra effort required to use the model? Generally, simple models with parameters representative of the situation being modeled are better than more complex models as long as the simple model produces reasonable results that can be defended as representative of safe driving behavior.

Building on that idea, it appears that any stopping sight distance model, whether simple or complex, will be made up of two components: a pre-braking component and a braking component. The next concern is the basic form of the model and its components, and the complexity of the two components. The current model is composed of two simple components that are based on a simple linear perception-brake response time model and the basic laws of physics. As noted in the Interim Report for this project *(11)*, a number of more complex models exist which describe the two components of the SSD model; however, they do not appear to provide better estimates of stopping sight distance than the simpler models. Thus, with changes to several of the existing model's parameters and the substitution of deceleration for tire/pavement friction values, the following relatively simple model is recommended as a replacement for the current AASHTO model.

$$SSD = 0.278Vt + 0.039V^2/a \qquad (1)$$

where: SSD = stopping sight distance, m;
V = design or initial speed, km/h;
t = driver perception-response time, sec; and
a = driver deceleration, m/sec^2.

Note that the proposed model is similar to the existing AASHTO model (pre-braking and braking components); however, the recommended changes (braking based on driver performance and capabilities rather than vehicle-roadway capabilities) result in a model that has **realistic** parameters, can be **validated** with field data, and can be **defended** as representative of safe driving behavior. This report documents the development of the recommended model and its parameters. Figure 1 is a flow diagram of the studies undertaken to validate the recommended stopping sight distance model and its parameter values. The findings from each of these studies are summarized later in this chapter.

AASHTO STOPPING SIGHT DISTANCE MODEL

One of the most important requirements in highway design is to provide adequate stopping sight distance at every point along the roadway. Horizontal and vertical curves can limit available sight distance; however, when designed in accordance with AASHTO criteria, adequate stopping sight distance should be available at each and every point along the curve. The design of horizontal and vertical curves, therefore, is dependent on the minimum required stopping sight distance. Providing AASHTO's minimum stopping sight distances allows below average drivers to detect unexpected, stationary objects in the road and to stop their vehicles before striking the objects *(1)*.

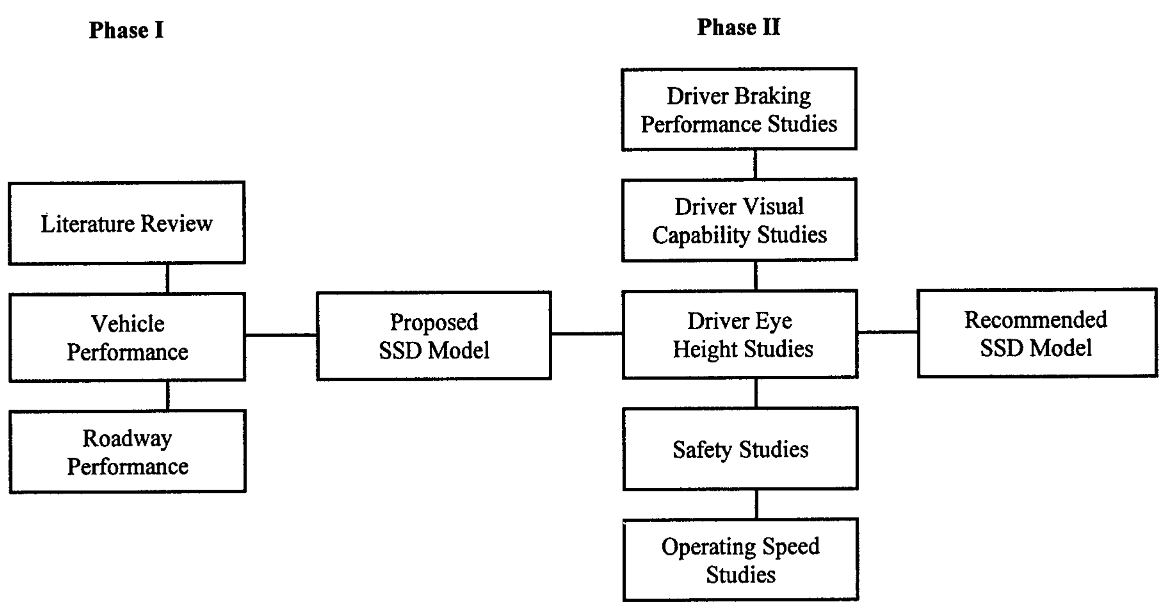


Figure 1. Flow diagram of work plan.

Stopping sight distance is calculated using basic principles of physics and relationships among the various design parameters. AASHTO defines stopping sight distance as the sum of two components, brake reaction distance (distance traveled from the instant of object detection to the instant the brakes are applied) and braking distance (distance traveled from the instant the brakes are applied to when the vehicle is decelerated to a stop) (*1*). Conceptually, required stopping sight distances can be expressed by the following equation:

$$SSD = \text{Brake Reaction Distance} + \text{Braking Distance} \qquad (2)$$

More specifically, these two components can be mathematically expressed as follows:

$$SSD = 0.278Vt + \frac{V^2}{254(f \pm G)} \qquad (3)$$

where: SSD = stopping sight distance, m;
V = design or initial speed, km/h;
t = driver perception-reaction time, sec;
f = friction between the tires and the pavement surface; and
G = percent grade/100, + for upgrades and − for downgrades.

The minimum stopping sight distance values are used to calculate the required length of horizontal and vertical curves. The minimum length of vertical curves is controlled by the minimum required stopping sight distances, the driver eye height (h_e), and the object height (h_o). This required length of curve is such that, at a minimum, the stopping sight distance calculated by Equation 3 is available at all points along the curve. Figure 2 illustrates the stopping sight distance model parameters as they relate to crest curve geometry.

Changes Over Time

Although the basic stopping sight distance model has remained the same, changes in design parameter values within the model have been addressed in several AASHO and AASHTO publications during the past 50 years. The fundamental principles of highway design were discussed in engineering textbooks as early as 1921 (*12*); however, it was not until 1940 that seven documents were published by AASHO, formally recognizing policies on certain aspects of geometric design. In that same year, these seven policies were reprinted and bound as a single volume entitled *Policies on Geometric Design* (*4*).

The AASHO policies were revised and amended in a 1954 document, *A Policy on Design of Rural Highways* (*5*). In 1965 and again in 1971, this document was revised and republished under the same title, and because of the color of its cover, was referred to as the *Blue Book* (*6, 7*). The current comprehensive 1994 document (*1*) and its 1990 (*2*) and 1984 (*3*) predecessors were entitled *A Policy on Geometric Design of Highways and Streets.* These are commonly referred to as the *Green Book*. The 1994 document is the first AASHTO design policy in metric units. The changes in the values of the parameters in the stopping sight distance model and minimum curve length equations that have occurred from 1940 to the present are summarized in Table 1 and described in subsequent sections. Appendix A, which is not published here, contains additional in formation regarding the history of the AASHTO SSD model.

Design Speed. The use of design speed in calculating stopping sight distance was first adopted by AASHO in the 1940 *A Policy on Sight Distance for Highways* (*4*). Design speed was defined as the maximum uniform speed which will be adopted by the faster group of drivers, but not necessarily by the small percentage of reckless drivers. In 1954,

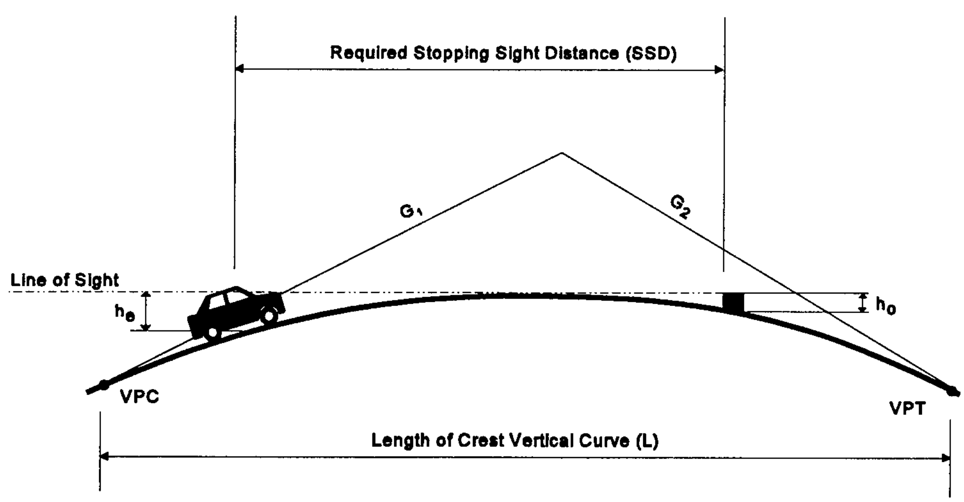


Figure 2. AASHTO'S SSD model and crest curve geometry.

TABLE 1 History of AASHTO stopping sight distance parameters

Parameters	1940 A Policy on Sight Distance for Highways	1954 A Policy on Geometric Design - Rural Highways	1965 A Policy on Geometric Design - Rural Highways	1971 A Policy on Geometric Design of Highways and Streets	1984 and 1990 A Policy on Geometric Design Highways and Streets
Design Speed	Design Speed	85 to 95 percent of design speed.	80 to 93 percent of design speed.	Min. - 80 to 93 percent of design speed. Des. - design speed.	Min. - 80 to 93 percent of design speed. Des. - design speed.
Perception - Reaction Time	Variable: 3.0 sec at 30 mph 2.0 sec at 70 mph	2.5 sec	2.5 sec	2.5 sec	2.5 sec
Design Pavement/ Stop	Dry Pavement Locked-wheel Stop	Wet Pavement Locked-wheel Stop	Wet Pavement Locked-wheel Stop	Wet Pavement Locked-wheel Stop	Wet Pavement Locked-wheel Stop
Friction Factors	Ranges from 0.50 at 30 mph to 0.40 at 70 mph	Ranges from 0.36 to 30 mph to 0.29 to 70 mph	Ranges from 0.36 to 30 mph to 0.27 at 70 mph	Ranges from 0.35 at 0.30 mph to 0.27 at 70 mph	Slightly higher at higher speeds than 1970 values
Eye Height	4.5 ft	4.5 ft	3.75 ft	3.75 ft	3.5 ft
Object Height	4.0 in	4.0 in	6.0 in	6.0 in	6.0 in

AASHO approximated the assumed speed on wet pavements to be a percentage varying from 85 to 95 percent of the design speed on the assumption that most drivers will not travel at full design speed when pavements are wet (*5*). In 1965, AASHO changed the approximated speed on wet pavements to a percentage varying from 80 to 93 percent of the design speed (*6*).

In 1971, AASHO published *A Policy on Design Standards for Stopping Sight Distance* (*7*). The policy introduced a range of design speeds, defined by a minimum and a desirable value, that were used for computing stopping sight distance. The minimum value was based on a percentage varying from 80 to 93 percent of the design speed (1965 assumed speeds on wet pavements), while desirable values were based on the design speed. AASHTO retained the concept of minimum and desirable values in its 1984, 1990, and 1994 policies, but noted that recent observations show that many operators drive just as fast on wet pavements as they do on dry (*1, 2, 3*).

Perception-Reaction Time. Perception-reaction time is the summation of brake reaction time and perception time. Brake reaction time was assumed as 1 sec in 1940 (*4*); since then, there have been no changes in the recommended value for brake reaction time. Total perception-reaction time, however, ranged from 2 to 3 sec, depending on design speed. In 1954, the *Blue Book* (*5*) adopted a policy for a total perception-reaction time of 2.5 sec for all design speeds. The 1954 *Blue Book* stated available references do not justify distinction over the range in design speed. The available references were not cited; therefore, the reason for this change is not known.

Design Pavement/Stop Conditions. The assumption for calculating braking distances since the 1940s has been that of a passenger car with locked-wheel tires throughout the braking maneuver. When compared to dry pavements, wet pavements result in lower coefficient of friction values and longer braking distances. Thus, design is governed by wet conditions. Friction values should be characteristic of variations in vehicle performance, pavement surface condition, and tire condition. As noted in Table 1, the friction factors were determined in accordance with the prevailing knowledge of the time. The 1940 AASHO Policy (*4*) used a safety factor of 1.25 to allow for the variations because of a lack of extensive field data. As more studies were completed, empirical friction factors were utilized in design. In all cases, friction factors decreased with increases in speed; this phenomenon is referred to as a speed gradient.

Driver Eye Height. Driver eye height values are a combination of the height of driver stature and driver seat height. The design value for driver eye height is selected so that the majority of driver eye heights in current vehicles will be greater than design values. As shown in Table 1, this design parameter has decreased from 54 to 42 in. during the past 50 years. The change in eye height can be attributed to increased numbers of small vehicles, vehicle design changes, and different seat angle designs. At the time of each AASHTO publication, the eye height was based on the prevailing distribution of drivers and vehicles. The most significant decrease in driver eye height took place between 1954 and 1965, when the eye height changed from 54 to 45 in.

Object Height. The changes in object height used in calculating stopping sight distance from 1940 to the present are shown in Table 1. The object height was equal to the driver eye height, 5.5 ft, in a 1921 highway engineering textbook (*12*). In 1940, a 4-in. object height was adopted by AASHO as an average control value (*4*). It was noted that the stationary object may be a vehicle or some high object, but it may be a very low object such as merchandise dropped from a truck or small rocks from side cuts (*2*). The surface of the roadway would have provided the safest design, but an object height of 4 in. was chosen because large holes in modern pavements were uncommon and other very small objects could be easily avoided.

In 1954, the 4-in. object height was justified as *the approximate point of diminishing returns* (*5*). The use of a zero object height was not used because of the excessive construction costs; however, too high an object height would exclude lower hazards and produce dangerously short lengths of vertical curves. AASHO noted a significant relationship between object height and vertical curve length: the vertical curve length decreased rapidly as the object height was increased from zero to 4 in. and decreased less rapidly for greater increases in object height.

When the driver eye height was decreased to 3.75 ft in 1965 (*6*) an object height of 6 in. was adopted. Figure 3 shows the percent reduction in vertical curve length for different object heights in the 1940, 1954, and 1965 SSD models. The same wording was used in 1954 to justify a 4-in. object height was also used to justify the 6-in. object height in 1965 (*5,6*). The 1984 and subsequent *Green Books* (*1,2,3*) considered a 6-in. object height to be representative of the lowest object that can create a hazardous condition and be perceived as a hazard by a driver in time to stop before reaching it. They also noted that it is an arbitrary rationalization of possible hazardous objects and a driver's ability to perceive and react to a hazardous situation.

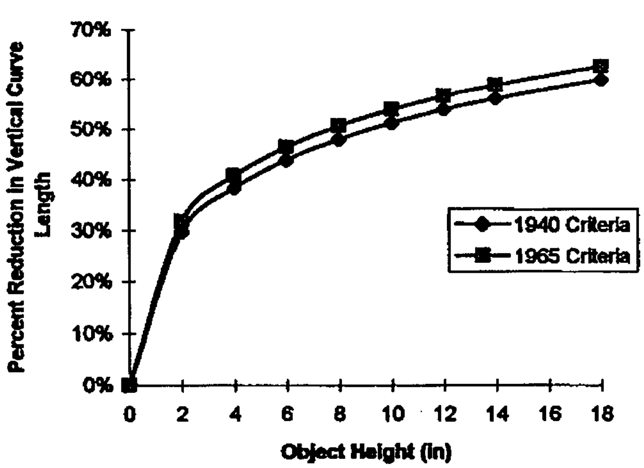


Figure 3. Sensitivity of vertical curve length to object height (S > L).

Functional Analysis

Vertical curves restrict available stopping sight distance whenever the approach grades are steep, the vertical curve is short, or both. Current design standards (*1*) for lengths of vertical curves are based on combinations of design speed (v) and algebraic difference in the approach grades (A). The minimum and desirable lengths (L) of vertical curves defined by AASHTO produce minimum and desirable stopping sight distances at the assumed design speed.

To avoid separate tabulations for A and L, design controls for vertical curves are expressed as K factors, that is, the length of vertical curve for a 1 percent change in A. These K factors are calculated so that they provide either minimum or desirable stopping sight distance at the assumed design speed. Thus, a single K value encompasses all combinations of L and A for any design speed, and plan sheets can be easily checked by comparing K values for all vertical curves with the design K value.

The most important characteristics of vertical curves are the K value and the available stopping sight distance throughout the vertical curve. A common misconception by non-design engineers is that minimum stopping sight distance is manifest over the entire length of the curve (*13*). A plot of available sight distance along the vertical curve, however, reveals available sight distance decreasing to a minimum value and then rapidly increasing as the vehicle reaches the crest of the curve (see Figure 4). Such plots are referred to as sight-distance profiles (*13*).

Sight-distance profiles are useful because they reveal the relationship between curve length, approach grade, and available stopping sight distance. The sight-distance profiles shown in Figure 4 represent crest vertical curves for different combinations of K factors and an algebraic grade difference of 6 percent. The different K values represent minimum stopping sight distances for design speeds of 45 and 55 mph (*2*). Horizontal lines represent minimum stopping sight distance for a design speed of 55 mph (*2*). Thus, if the available stopping sight distance curve falls below the horizontal line, stopping sight distance is less than the minimum AASHTO criteria for a 55-mph design speed.

Inspection of the sight-distance profiles shown in Figure 4 reveal three characteristics of stopping sight distance at vertical curves (*13*):

1. Vertical curves that restrict available stopping sight distances do so over relatively short lengths of highway. Similarly, less severe stopping sight distance restrictions (higher K values) affect longer sections of highway;
2. The length of highway over which stopping sight distance is at a minimum is relatively short compared with the length of a vertical curve; and
3. For a constant K factor, the length of highway over which stopping sight distance is limited increases as the

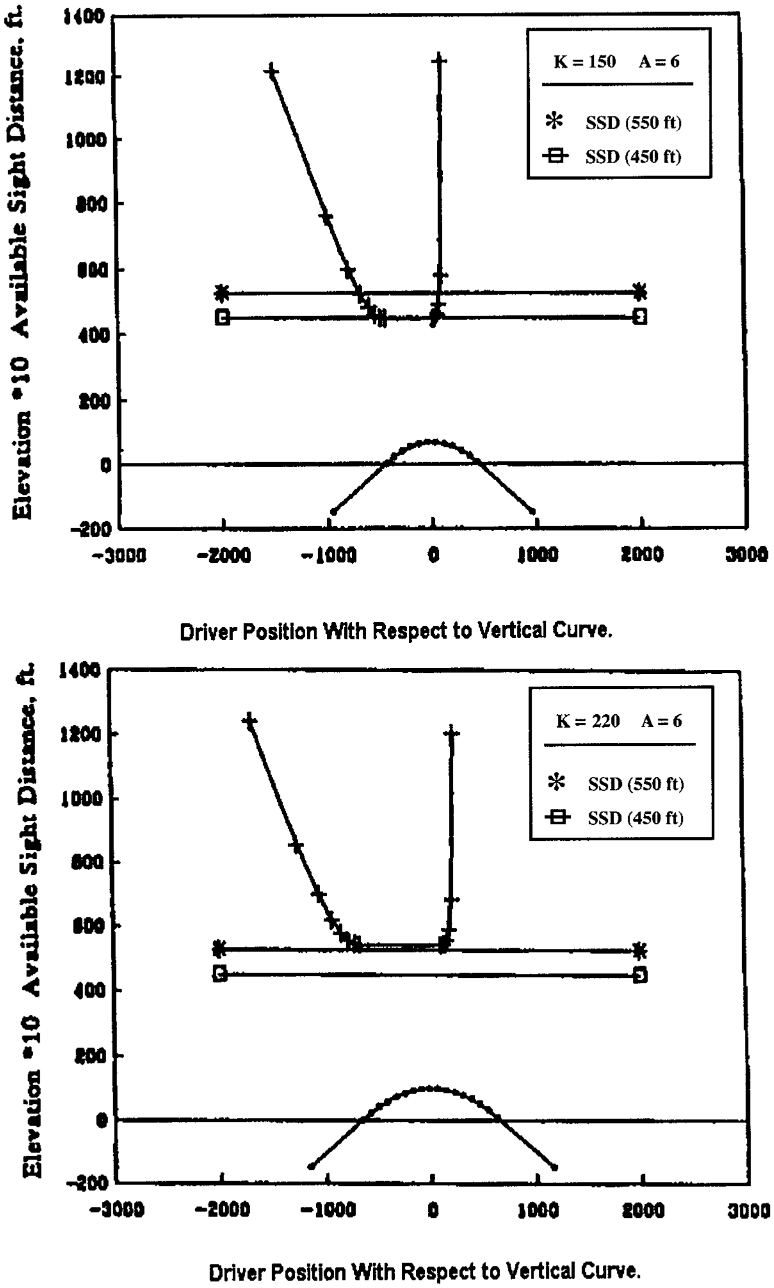


Figure 4. Available sight distance as a function of curve geometry.

difference in grade increases. The minimum available sight distance, however, remains the same.

Comparison with Other Countries

There are many similarities between the AASHTO stopping sight distance model and those used by other countries. Most countries' policies on stopping sight distance are based on a two component model that differs from AASHTO's only in specific assumptions regarding parameter values. There are, however, differences in philosophy and approach. **For example, most European countries emphasize anticipated operating speeds and consistency between design elements.** The differences in stopping sight distance design policies are described below with emphasis on the parameter values assumed in the stopping sight distance models.

The minimum required stopping sight distances used by other countries are shown in Table 2 (*14*). Note that the AASHTO stopping sight distance values are near the upper

TABLE 2 Comparison of minimum required stopping distances

Country	t_{pr} sec	Design or Operating Speed (km/h)												
		20	30	40	50	60	70	80	90	100	110	120	130	140
		Stopping Sight Distance (m)												
AASHTO	2.5	20	30	44	63	85	111	139	169	205	246	286		
Australia														
Normal Design	2.5	--	--	--	--	--	--	115	140	170	210	250	300	--
Normal Design	2.0	--	--	--	45	65	85	105	130	--	--	--	--	--
Restricted Design	1.5	--	--	--	40	55	70	--	--	--	--	--	--	--
Austria	2.0	--	--	35	50	70	90	120	--	185	--	275	--	380
Canada	2.5	--	--	45	65	85	110	140	170	200	220	240	--	--
France	2.0	15	25	35	50	65	85	105	130	160	--	--	--	--
Germany	2.0	--	--	--	--	65	85	110	140	170	210	255	--	--
Great Britain	2.0	--	--	--	70	90	120	--	--	215	--	295	--	--
Greece	2.0	--	--	--	--	65	85	110	140	170	205	245	--	--
South Africa	2.5	--	--	50	65	80	95	115	135	155	180	210	--	--
Sweden	2.0	--	35	--	70	--	165	--	--	--	195	--	--	--
Switzerland	2.0	--	--	35	50	70	95	120	150	195	230	280	--	--

end of the range for the countries surveyed, and Canadian stopping sight distances are near the lower end of the range. The principal assumptions in determining required stopping sight distance are the perception-brake reaction time and the braking coefficients used for various design speeds. All countries reviewed use a perception-reaction time of 2.0 sec for rural roads, except Australia (for higher speeds only), Canada, South Africa, and AASHTO, which all use 2.5 sec.

The braking coefficients of frictions assumed in determining design stopping sight distances are shown in Table 3 (*14*). In interpreting this data, keep in mind that most of the values represent assumed constant values of braking coefficient over the entire speed range, while the Austrian, German, and Greek values vary with speed over the braking maneuver. AASHTO generally has the lowest friction values and the smallest difference in friction values between 50 and 120 km/h.

Driver eye height and object height for determining vertical curve lengths are summarized in Table 4 (*14*). The assumed driver eye heights for a passenger-car driver range from 1.0 to 1.1.5 m. Object height assumptions are more var-

TABLE 3 Comparison of longitudinal friction coefficients

Country	Design or Operating Speed (km/h)									
	30	40	50	60	70	80	90	100	110	120
	Stopping Sight Distance (m)									
AASHTO	0.40	0.38	0.35	0.33	0.31	0.30	0.30	0.29	0.28	0.28
Australia	--	--	0.52	0.48	0.45	0.43	0.41	0.39	0.37	0.35
Austria	0.44	0.39	0.35	0.31	0.27	0.24	0.21	0.19	0.17	0.16
France	--	0.37	-	0.37	--	0.33	--	0.30	--	0.27
Germany	0.51	0.46	0.41	0.36	0.32	0.29	0.25	0.23	0.21	0.19
Greece	0.46	0.42	0.39	0.35	0.32	0.30	0.28	0.26	0.24	0.23
South Africa										
(passenger cars)	0.42	0.38	0.35	0.32	--	0.30	--	0.29	--	0.28
(heavy vehicles)	0.28	0.25	0.23	0.23	--	--	--	--	--	--
Sweden	0.46	0.45	0.42	0.40	0.37	0.35	0.33	0.32	0.30	--
Switzerland	--	0.43	0.37	0.33	0.29	0.27	0.25	0.24	0.23	0.22

TABLE 4 Comparison of criteria for driver eye height and object height used in vertical curve design

Country	Driver Eye Height (m)		Object Height (m)
	Passenger Car	Truck	
AASHTO	1.07	--	0.15
Australia	1.15	1.80	0.20
Austria	1.00	--	0.00-0.19
Canada	1.05	--	0.38
France	1.00	--	0.35
Germany	1.00	--	0.00-0.45
Great Britain	1.05	2.0	0.26
Greece	1.00	--	0.00-0.45
Sweden	1.10	--	0.20
Switzerland	1.00	2.50	0.15

ied. Australia, Great Britain, Sweden, Switzerland, and the U.S. each assume a small object with a height between 0.15 and 0.26 m. Canada and France use an object height based on vehicle taillight height in the range from 0.35 to 0.38 m. Germany uses an object height value that varies with design speed from 0.0 m at low speeds to 0.45 m at high speeds. A unique feature of the Swedish guidelines is that they specify a minimum portion of the object (1 min of arc) that must be visible above the driver's line of sight.

The minimum *K* values are based on the required stopping sight distance, as well as driver eye and object heights. Many countries specify circular vertical curves, but for convenience, lay them out in the field as parabolic curves (*15*). For a circular curve, the *K* value represents the radius of the vertical curve; however, it should be noted that for any given *K* value, the alignment of parabolic and circular vertical curves differs by only a few centimeters. As with stopping sight distances, AASHTO curve lengths are near the upper end of the range. Several countries use headlight sight distance criteria similar to AASHTO's for determining sag vertical curve lengths. Other countries view sag vertical curves as less critical with respect to safety and base their guidelines on comfort and appearance.

VEHICLE AND ROADWAY PERFORMANCE

Vehicle and roadway performance parameters related to stopping sight distance situations are important in that they must be greater than or equal to the driver braking requirements. The following sections summarize the literature related to these two parameters. Appendix B, which is not published here, contains additional information regarding vehicle and roadway performance.

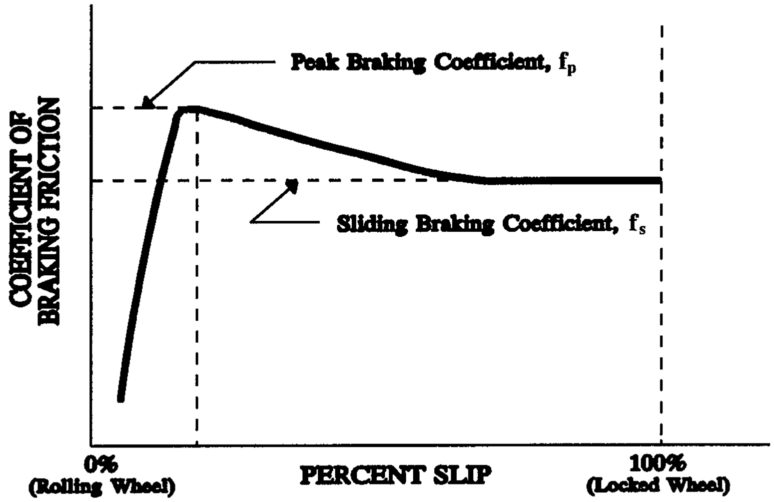
Vehicle Braking Performance

Figure 5 illustrates that both braking and cornering friction vary as a function of percent slip. Braking friction is the ratio of the braking force generated at the tire-pavement interface to the vertical load carried by the tire. Cornering friction is the ratio of the cornering force generated at the tire-pavement interface to the vertical load carried by the tire. Percent slip is the percent decrease in the angular velocity of a wheel relative to the pavement surface as a vehicle undergoes braking. A freely rolling wheel is operating at zero percent slip. A locked wheel is operating at 100 percent slip with the tires sliding across the pavement.

Figure 5 also shows that the coefficient of braking friction increases rapidly with percent slip to a peak value that typically occurs between 10 and 15 percent slip. The coefficient of braking friction then decreases as percent slip increases, reaching a level known as the coefficient of sliding friction at 100 percent slip. The coefficient of cornering friction has its maximum value at zero percent slip and decreases to a minimum at 100 percent slip. Thus, when a braking vehicle locks its wheels, it loses its steering capability because of a lack of cornering friction.

Locked Wheel versus Controlled Braking. Braking maneuvers can be performed in two general modes: locked-wheel braking and controlled braking. Another term associated with locked-wheel braking is *panic stops*. In panic stops, the driver *stomps on* the brake pedal and holds it depressed until the vehicle stops. Braking in this mode causes the vehicle to slide or skid over the pavement surface on its nonrotating or *locked* tires. A significant consequence of a panic stop is the loss of control of the vehicle. Locked-wheel braking also uses sliding friction, f_s, rather than rolling or peak friction. The sliding coefficient of friction takes advantage of most of the friction available from the pavement surface, but is generally less than the peak available friction.

Controlled braking is the application of the brakes in such a way that the wheels continue to roll without locking up while the vehicle is decelerating. Controlled braking distances are governed by the rolling coefficient of friction, which occurs at a value of percent slip to the left of the peak available friction (see Figure 5). Drivers generally achieve controlled braking by *modulating* the brake pedal to vary the braking force and avoid locking the wheels. Harwood (*16*) noted that because of the steep slope of the braking friction curve to the left of the peak and because of the braking techniques used by drivers to avoid wheel lock up, the average rolling friction used when braking is generally less than the maximum available sliding friction. Thus, driver-controlled braking distances are usually longer than locked-wheel braking distances, although theoretically they would be less if the driver could use peak braking friction. Antilock braking, another type of controlled braking, is when a microprocessor evaluates the vehicle's wheel and makes adjustments when wheel lock occurs or is anticipated.



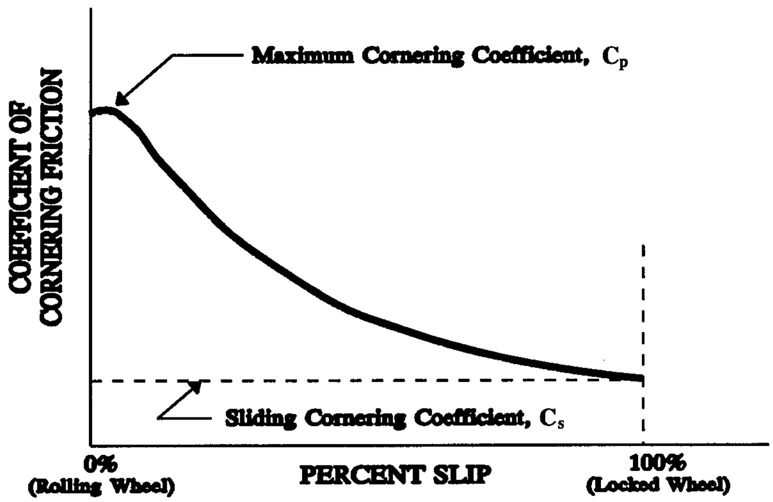


Figure 5. Variation of braking and cornering friction coefficients with percent slip (16).

Braking Simulation Studies. A major criticism of the AASHTO stopping sight distance model has been that drivers are highly unlikely, when traveling at high speed on a wet pavement, to brake sufficiently to lock the wheels on their vehicles and hold them at lockup (*8*). Rather, they will probably modulate their brakes to retain the ability to steer the vehicle. This argument forms the basis of Olson's recommended braking distances, which are significantly longer than the existing AASHTO values. Olson et al. (*8*) developed sets of equations based on an examination of the influences of pavement, tire, vehicle, and driver properties on vehicle braking distance to predict braking distance capabilities of cars and trucks operating on poor, wet roads. These equations were used in connection with a numerical integration algorithm to determine braking distances. Integration is needed because the frictional characteristics at the tire-road interface change as the vehicle's velocity changes during braking. Aerodynamic drag also changes as velocity changes.

Figures 6 and 7 illustrate the braking performance of passenger cars and trucks on poor, wet roads based on Olson's equations (*8*). Figure 6 shows values for type of braking (locked-wheel or controlled), type of tire condition (new tire or worn—2⁄32 in., which is the legal limit), driver control efficiency, and the braking distances given in the 1984 AASHTO *Green Book*. Figure 7 shows truck braking distances for type of braking and type of tire condition.

Roadway Characteristics. The available friction from a pavement is characterized by American Society for Testing and Materials (ASTM) skid numbers. Olson et al. (*8*)

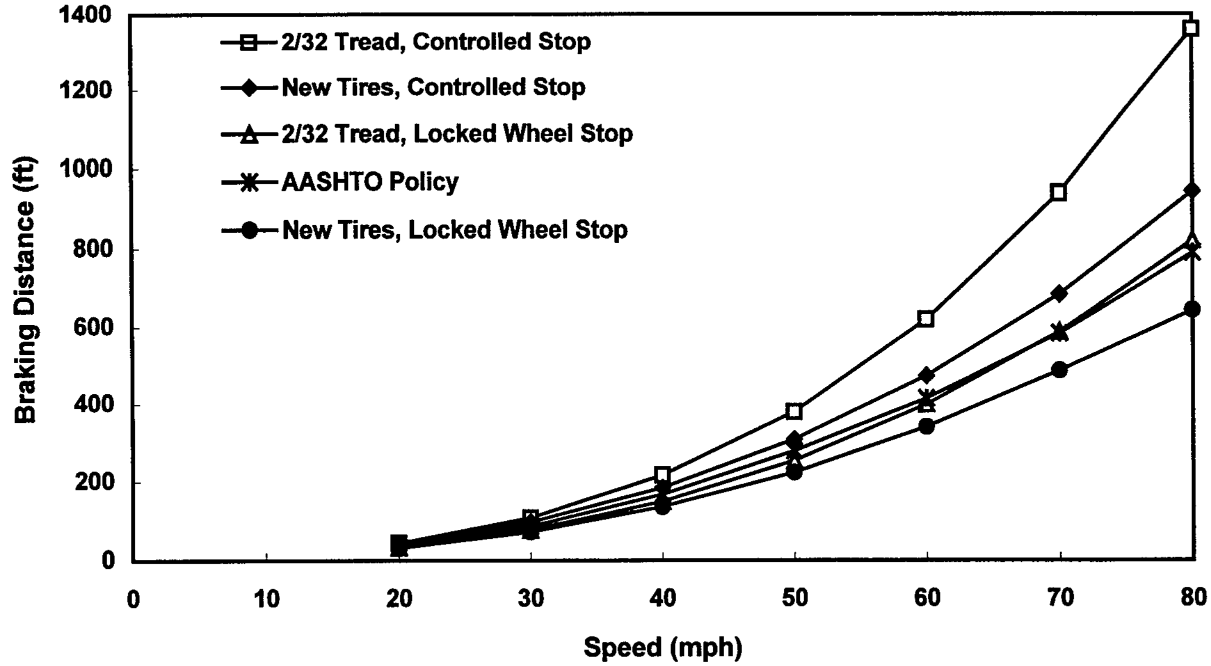


Figure 6. Passenger-car braking distances on a poor, wet road (8).

included equations that express the relationship between skid numbers for various speeds and qualities of pavement. These equations were from other research studies (*17, 18*). The average pavement texture depth and skid number represent the quality of the pavement. A texture depth of 0.015 in. and a skid number of 28 were selected to represent a 15th percentile, wet road in the calculations.

Tire Properties. Olson stated that on the basis of a number of studies, new car tires are approximately 1.2 times more effective than the ASTM tire (used to determine skid numbers) and that truck tires are approximately 0.7 times as effective as car tires. These values were used to determine the relationship between maximum rolling friction (f_p) and sliding friction (f_s). The differences between the frictional capability of worn tires and new tires is accounted for in a separate equation. Truck tires are assumed to have the same wear characteristics as those attributed to car tires when tire groove depth wears below 12/32 inch.

Vehicle Properties. Braking efficiency represents the influences of the proportioning of the braking effort amongst the various wheel locations and the overall distribution of mass throughout the vehicle. The braking systems used in new passenger cars are designed to be very efficient in using the peak friction available at the tire-road interface. On the basis of data from Radlinski and Flick (*19*), Olson estimated that the average efficiency of a sample of 1982 passenger cars was approximately 91 percent. He also estimated that the braking efficiencies of empty heavy trucks will range from 55 to 59 percent as peak friction varies from 0.43 to 0.21.

Driver Characteristics. Driver control efficiency (CE) predicts the ability of the driver to use the deceleration capability afforded by friction and braking efficiency. A CE prediction equation was developed based on experiments conducted by Mortimer et al. (*21*). Automobile drivers were to stop as quickly as possible while following a slightly curv-

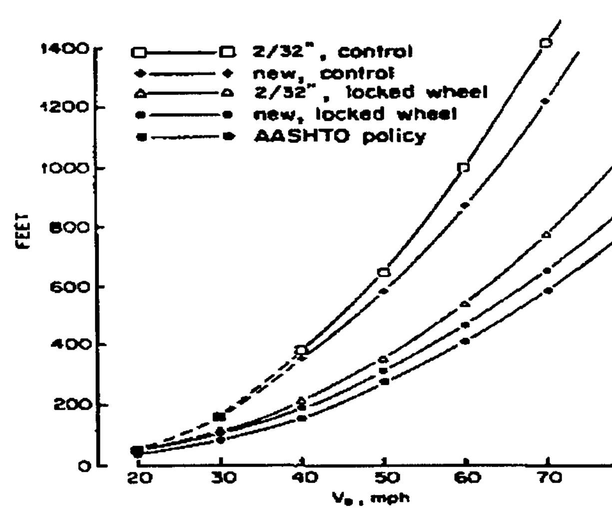


Figure 7. Truck braking distances on a poor, wet road (20).

ing 10-ft lane. The results showed that drivers would not use the ultimate braking capabilities of their vehicles because they could not modulate their brake pedals well enough to use the peak-tire-road friction available while simultaneously avoiding loss of directional control because of wheel lockup.

Because of the lack of information on braking control efficiencies of truck drivers, Olson et al. (*8*) performed a limited set of experiments. The results of these studies indicated that professional truck drivers could usually achieve 62 percent or more of the braking capabilities of empty heavy trucks during a braking-in-a-turn maneuver. In approximately one-sixth of their attempts to stop quickly from 40 mph, this group of drivers failed to stay within a 12-ft lane.

Heavy Truck Braking. Truck stopping distance requirements must consider that truck drivers cannot make a locked-wheel stop without the risk of losing control of the vehicle. The process of bringing a truck to a stop requires a complex interaction between the driver, the brake system, the truck tires, the dimensions and loading characteristics of the truck, and the pavement surface characteristics. Harwood et al. (*16*) presented a detailed discussion on this complex interaction. Following is a summary of their discussion.

The shape of the braking friction curve in Figure 5 is a function of both pavement and tire properties. Highway agencies generally measure pavement friction by means of locked-wheel skid tests with a standard tire. Olson et al. (*8*) estimated the peak coefficient of friction for truck tires from the sliding coefficient of friction found in locked-wheel skid tests.

Truck tires tend to have lower wet friction coefficients than passenger-car tires because they are designed primarily for wear resistance. Olson et al. (*8*) estimated that truck tires have coefficients of friction that are about 70 percent of those of passenger-car tires; however, passenger-car tires generally have coefficients of friction that are about 120 percent of the friction coefficients of the standard tires used in skid testing. The coefficient of friction for truck tires decreases as the tires wear and their tread depth decreases. Dijks (*22*) reported that the tread wear of truck tires has very little effect on their frictional properties until the tread depth falls below $^{12}/_{32}$ inch. Tire tread depth also has little effect on the frictional properties of pavements with high macro texture, but the coefficient of friction does decrease substantially with tread depth for smooth, poorly textured pavements.

Current truck braking systems are limited in their ability to take advantage of the available friction at the tire-pavement interface. Fancher (*23*) estimated that the braking efficiency for single-unit trucks is between 55 and 59 percent of the peak available friction. Braking efficiency is influenced by disconnected front brakes (20 to 25 percent longer braking distance) (*24*), automatic limiting valves that limit braking achievable on the front axle (8 to 29 percent longer braking distances) (*19*), and antilock braking systems (shorter braking distances).

Most truck drivers have little or no practice in emergency braking situations. This lack of expertise in modulating the brakes results in braking distances that are longer than the vehicle's capability. Harwood et al. (*16*) evaluated three different scenarios for determining truck braking distances: an empty tractor-trailer truck with a conventional brake system and an inexperienced driver (*worst-performance driver*); the same truck and braking system operated by an experienced driver (*best-performance driver*); and finally, the same truck with an antilock braking system. The antilock stopping distances were based on braking tests conducted by National Highway Traffic Safety Administration (NHTSA) for Harwood's study. The results for the three scenarios, listed in Table 5, indicate that braking distances for trucks with antilock brakes on wet pavements are similar to the AASHTO criteria for passenger cars.

Truck Braking Field Tests. Tests were conducted by NHTSA to evaluate the braking performance of a two-axle straight truck with and without an antilock braking system (*25*). Straight-line stops, stops in a turn, and stops in a lane change were used. Table 6 lists the results for the different types of brakes, surfaces, loadings, and stopping maneuvers.

TABLE 5 Braking distance for trucks on wet pavement (*16*)

Design Speed (mph)	AASHTO Criteria for Passenger Cars (ft)	Braking Distances for Trucks (ft)		
		Worst-Performance Driver	Best-Performance Driver	Antilock Brake System
20	33	77	48	37
30	86	186	115	88
40	167	344	213	172
50	278	538	333	267
60	414	744	462	375
70	583	1013	628	510

TABLE 6 NHTSA wet pavement braking test results (*25*)

Surface	Skid Number (SN)	Maneuver	Speed (mph)	Loaded (ft)			Empty (ft)		
				AL*	LW	CB	AL	LW	CB
Dry Asphalt	80	St. Line	60	297	318	302	195	179	205
Dry Asphalt	80	St. Line	35	90	94	92	64	64	72
Wet Asphalt	60	St. Line	60	278	315	338	226	248	222
Wet Asphalt	60	St. Line	35	71	66	78	69	71	85
Wet Pol Con.	30	St. Line	60	333	---	405	325	373	356
Wet Pol Con.	30	St. Line	35	87	97	117	93	98	100
Wet Jennite	20	St. Line	35	134	216	232	148	232	155
Split Mu	20/60	St. Line	35	110	107	129	110	106	119
Ice	5/10	St. Line	20	198	180	261	116	144	201
Wet Jennite	20	500 ft Curve	35	154	222	221	162	219	191
Ice	5/10	500 ft Curve	20	225	227	284	155	172	215
Wet Jennite	20	Lane Change	35	150	232	221	148	228	178

* *AL = antilock braking system; LW = locked-wheel braking, best of two stops; CB = controlled braking, best of six stops*

In general, the authors concluded that the antilock system provided improved stopping capability. Most of the stops with antilock brakes were performed in shorter distances (up to 42 percent shorter) than without the antilock brakes. The vehicle also was under the control of the driver. For a straight-line stop from 60 mph on a wet, polished concrete pavement (SN_{40} is approximately 30), a 15 percent reduction in braking distance was found between controlled braking and braking with antilock brakes.

Flick reported on 1990 tests to determine the straight-line stopping capability of two single-unit trucks and six truck tractors (*24*). The single-unit trucks were tested empty and fully loaded while the truck tractors were tested bobtail, with an empty trailer, and with a fully loaded trailer. Tests were conducted to determine the effect of antilock braking systems (ABS) automatic front axle automatic limiting valves (ALV), and bobtail proportioning systems (BPS). All braking tests were straight-line stops from 60 mph on a dry concrete surface (nominal dry skid number was 81). Table 7 presents the results from these tests.

The results of the single-unit truck tests showed that the loaded stopping distances were significantly shorter than the empty stopping distances. The stopping distances with the antilock brakes operational were shorter than the base vehicle stopping distances in both load configurations; however, the percentage decrease in stopping distance for the empty vehicle was larger than that for the loaded vehicle. Performance of the bobtail with the BPS was significantly better than the performance of the base vehicle, and an even larger stopping distance improvement was seen with the antilock system. The AVLs, however, generally increased the stopping distance because they reduced the pressure applied to the front brakes in a situation where front brake force was already too low.

With empty trailers, combination vehicle stopping distance for the base vehicles was shorter than the bobtail case. Those tractors with antilock brakes, once again, showed significant improvements. The ALV had a significant negative effect on the performance of three of the tractors/empty trailer combinations, but essentially no effect on the other three. The combination vehicle tests with loaded trailers stopped shorter than the same configurations with empty trailers. The two tractors that were tested with ABS did not show a significant improvement in their loaded trailer stopping distances primarily because brake balance was near optimum at this load condition.

Light Vehicle ABS Performance Evaluation. NHTSA reported on 1991 tests conducted on ten light vehicles (seven passenger cars, two light trucks, and one van) that evaluated the improvements in braking performance and vehicle control resulting from adding an antilock braking system (*26*). The vehicles were tested empty and loaded on different surfaces and at speeds from 35 to 60 mph. Eight of the vehicles had all-wheel antilock brake systems and two vehicles had rear-wheel only systems. The test series was intended to evaluate the benefits of ABS to individual vehicles. Table 8 lists the results from the locked-wheel braking tests on wet, polished concrete for an empty vehicle braking in a straight line.

Each vehicle's stability improved during braking. Without the ABS, vehicles were more likely to spin because the back wheels locked up. The all-wheel antilock system improved directional control, but the rear-wheel antilock system did not because the front wheels locked up. Stopping distances in panic situations were shortened for the all-wheel system on most hard surfaces. On wet or dry surfaces with high coefficients of friction, the difference was relatively small or neg-

TABLE 7 Dry pavement braking distances for 1988 heavy vehicles (*26*)

Vehicle	Braking Distances (ft)			
	Base	With ABS	With ALV	With BPS
Single-Unit Truck, Empty				
Ford 4X2	375	na	412	na
Freightliner 6X4	438	233	456	na
Single-Unit Truck, Loaded				
Ford 4X2	272	na	na	na
Freightliner 6X4	307	282	na	na
Bobtail Tractor				
Ford 4X2	375	na	412	286
Freightliner 6X4	438	233	456	359
International 6X4	356	na	348	275
Peterbilt 4X2	350	na	414	356
Volvo White 6X4	333	248	353	291
Volvo White 4X2	463	na	531	345
Empty Tractor/Trailer				
Ford 4X2	263	na	300	na
Freightliner 6X4	319	225	322	na
International 6X4	260	na	285	na
Peterbilt 4X2	282	na	287	na
Volvo White 6X4	282	226	279	na
Volvo White 4X2	301	na	316	na
Loaded Tractor/Trailer				
Ford 4X2	230	na	na	na
Freightliner 6X4	266	256	na	na
International 6X4	261	na	na	na
Peterbilt 4X2	273	na	na	na
Volvo White 6X4	253	262	na	na
Volvo White 4X2	261	na	na	na

Notes: ABS = antilock brake system
AVL = automatic limiting valve for front-axle brakes
BPS = bobtail proportioning systems

ligible. On wet, highly polished concrete, improvements of 25 percent were observed. On wet asphalt, improvements of more than 50 percent were observed with the all-wheel systems. In rear-wheel systems, the stopping distance was not shorter; and in panic situations, it actually increased. All of these results were presumed to be the minimum that an ABS could provide because drivers were trained professionals. A non-trained driver could not be expected to perform as well without antilock brakes and would, therefore, have greater improvements.

Summary. Several parameters can affect the distance a vehicle requires to stop. Some of these issues, especially for heavy trucks, result in significantly longer braking distances. Olson argued that rather than locked-wheel braking, drivers will attempt to maintain the steering ability of their vehicles especially when braking on a horizontal curve or during a lane-changing maneuver. Using controlled-braking distances rather than locked-wheel braking distances results in significantly longer stopping distances. Widespread use of ABS could resolve the issue of controlled versus locked-wheel braking. Antilock brakes would provide the driver with steering control while using near peak braking friction. Thus, controlled braking distances for large trucks with ABS can approach locked-wheel braking distances.

Roadway Performance

Friction coefficient is the variable that reflects numerous vehicle and roadway conditions that impact braking capabilities. It is a reflection of the tire condition, the pavement condition, and the interaction between the tire and the pavement. It varies depending on the pavement type, whether the pave-

TABLE 8 Wet pavement braking distances for vehicles with and without ABS (*24*)

Vehicle	Speed (mph)	Braking Distance (ft) With ABS	Braking Distance (ft) W/O ABS	Percent Improvement
Toyota Supra	35	63	79	20.3
	50	131	198	33.8
Acura Legend	35	75	84	10.7
	50	152	196	22.4
GMC Safari GT Mini Van	35	83	90	7.8
	50	175	197	11.2
Cadillac Brougham	35	72	98	26.5
	50	144	244	41.0
Chrysler Imperial	35	78	98	20.4
	50	164	221	25.8
Pontiac Grand Prix SE	35	78	87	10.3
	50	151	202	25.2
Ford F-150 Pickup Truck	35	100	99	-1.0
	50	228	236	3.4
Mazda B2200 Pickup Truck	35	105	94	-11.7
	50	239	241	0.8
Oldsmobile Cutlass Calais I	35	76	90	15.6
	50	160	225	28.9
Buick Electra	35	78	96	18.8
	50	220	323	31.9

Braking characteristics: straight line, locked-wheel braking on wet highly polished concrete (SN=28), test weight was empty (driver and instrumentation only), average of three stops

ment is dry or wet, whether the tires are new or worn, and many other conditions. The friction values selected for use in the 1994 *Green Book* were based on studies conducted by Moyer and Shuppe in 1951 (*27*) and were selected to reflect **locked- wheel braking on a poor, wet pavement with worn tires**. The *Green Book* states that "the friction values used for design should be nearly all-inclusive, rather than average."

To select friction values to use in design or to use skid data to verify or compare existing assumptions with actual data requires that such data be available. For this study, skid data from two states, Texas and California, and from the Strategic Highway Research Program (SHRP) data base were obtained. The skid numbers and other relevant data were extracted from the data bases and then sorted into functional roadway classes. Cumulative frequencies of the individual skid numbers were developed for each functional class. The cumulative frequencies can be used to determine the 15th percentile (or any other desirable percentile) skid number that can then be used in design. To present a representative picture of the available friction levels on existing pavements, the most current data were used.

Texas Data. The high, low, and average skid numbers for different highway segments were provided by the Texas Department of Transportation. The skid data represented skidding efforts between 1986 and 1991. The data base provided by the state contained several pieces of information pertinent to the skid number; however, data pertaining to the functional class of the segment were not present. Extensive manipulations between the skid number data base and the Texas Roadway Inventory Log provided information needed to sort the skid numbers by functional class.

The average skid number, rather than the high or low value, was selected for use in this analysis. If more than one skid number was available for a particular road (because the road was skidded twice or more within the 5-year period), then only the most current value was used in the evaluation. Table 9 lists the 15th, 50th, and 85th percentile skid number value for each functional class, as well as the number of records (or skid numbers) present for the functional class. Each functional class has a 15th percentile skid number equal to or greater than the 32 skid number for the 40 mph design speed in the *Green Book* data.

California Data. The California DOT provided magnetic tapes of their Skid Resistance Inventory (SRI) file. This file includes skid number by section of roadway. Because California generally skids its pavements every two years, the

TABLE 9 Skid numbers from Texas data base

Functional Class	Percentile			Number of Records
	15th*	50th	85th	
Rural Freeway	33	41	51	191
Rural Multilane Divided	34	47	58	127
Rural Multilane Undivided	34	45	57	219
Rural Two-Lane High	32	43	56	121
Rural Two-Lane Low	33	48	59	133
Urban Freeway	33	41	50	66
Urban Multilane Divided	36	52	62	437
Urban Multilane Undivided	38	48	59	287
Urban Two-Lane High	35	48	62	669
Urban Two-Lane Low	38	51	61	1,097

* 15 percent of the pavements have this skid number or worse (or stated another way, 85 percent of the pavements have this skid number or better).

two most recent years of data (July 1990 to June 1992) were used for analysis in the study. The data were sorted into eight functional classes, and cumulative frequency curves were generated for each functional class. The 15th, 50th, and 85th percentile values, along with the number of records, are listed in Table 10.

Most of the rural functional classes have a 15th percentile skid number that is at or just above the 32 skid number for the 40 mph design speed in the *Green Book* (the multilane divided is just below). Most of the urban functional classes, however, are below the 32 value. Only the urban freeway class has a higher skid number. The California SRI file also was used in a 1986 study that related skid numbers to wet pavement accident frequency (*28*).

SHRP Data. The SHRP data were from the Long-Term Pavement Performance (LTPP) Information Management System (IMS). Data stored in the IMS are collected from SHRP test sections located throughout the United States and Canada. A variety of information and data are collected for each section including climatic, material properties, traffic loads, friction, and numerous other types of data. The data of primary interest for this project is the skid numbers.

Skid number or friction measurements are taken at least every 2 years by state agencies and recorded in the IMS. The skid number, time of day, surface type, vehicle speed, and test method are some of the principal elements stored. Each section in the data base is 500 ft in length and the skid tests

TABLE 10 Skid numbers from California data base

Functional Class	Percentile			Number of Records
	15th[1]	50th	85th	
Rural Freeway Rural Multilane Divided Rural Multilane Undivided Rural Two-Lane[2]	33 30 32 32	42 41 43 43	50 49 52 51	18,990 904 1,192 22,457
Urban Freeway Urban Multilane Divided Urban Multilane Undivided Urban Two-Lane[2]	33 28 27 27	40 36 36 36	47 44 43 45	22,274 5,297 1,173 2,211
All Roadways	32	41	49	74,498

[1] 15 percent of the pavements have this skid number or worse (or stated another way, 85 percent of the pavements have this skid number or better).
[2] Because information on shoulders was lacking, the two-lane roadways could not be sorted into high and low type categories.

were all conducted at 40 mph. Two skid numbers were reported for each section, one at the beginning of the section and the other at the end of the section. The values reported in this project reflect an average of these two numbers. Skid data were available for 687 sections. The data were from the following regions: Western, Southern, North Central, and North Atlantic.

The skid data were divided into functional classes that would be similar to the classes used to evaluate the Texas and California data. The multilane class includes both divided and undivided sections because information needed to separate the records was not available in the data base. Approximately 90 of the 687 records (13 percent) did not have sufficient data to be placed into a functional class. Table 11 lists the 15th, 50th, and 85th percentile skid numbers for each functional class along with the number of records available for the class. Except for rural freeways, each functional class had fewer than 100 records.

A 1986 FHWA report included a distribution of skid numbers that were used in a study of highway geometrics and wet pavement accident frequency (*28*). Data from three California Department of Transportation (CALTRANS) districts for a 1-year period were used. The skid numbers for the three districts ranged from 17 to 54 with a mean of 37 and a standard deviation of 6.6. Table 12 lists the average skid number for different pavement types in the California data base.

Summary. Using a 15th-percentile value, the data indicated that all functional classes of Texas roads had a skid number greater than or equal to the 32 value assumed in the *Green Book*. The skid numbers from the SHRP data base also were greater than the AASHTO assumed value except for the *Rural Two-Lane Low* class, which had a 15th percentile skid number of 31. The California pavements in the rural area were near the 32 value (*the Rural Multilane Divided* class had a value of 30); however, the urban class (except the *Freeway* class) had values below 32. Thus, most roadways have skid numbers greater than or equal to the assumed friction coefficients in the AASHTO stopping sight distance model.

DRIVER BRAKING PERFORMANCE

Driver braking performance parameters related to stopping sight distance situations are perception-reaction time and driver deceleration. The following sections summarize the literature related to these two parameters and the results of several closed and open roadway studies that quantified these parameters (*29*). Appendix C contains additional information regarding driver performance related to stopping sight distance situations.

Perception-Brake Reaction Time

In recent years, models used to predict human perception-response time have become probabilistic, starting with Fitts and his *random walk* model (*30*), to more recent stochastic network models, such as those proposed by Wickens (*31*). Wickens discusses the assumption of linearity in the Hick-Hyman Law (*32*) and suggests that reaction time tends to follow an exponential function if the decision is in the context of danger or severe penalty. In other words, the larger the amount of information that is processed (i.e., the nature of the oncoming situation and the number of alternatives the driver has to choose from in that situation), the longer the driver takes to react.

Perception-Reaction Time Models. Perception-brake reaction time represents the total time it takes a driver to

TABLE 11 Skid numbers from LTPP IMS data base

Functional Class	Percentile			Number of Records
	15%[1]	50%	85%	
Rural Freeway	40	48	56	426
Rural Multilane[2]	34	52	60	20
Rural Two-Lane High	35	45	55	43
Rural Two-Lane Low	31	40	49	6
Urban Freeway	34	43	51	72
Urban Multilane[2]	40	51	59	23
Urban Two-Lane High	40	49	54	6
Urban Two-Lane Low	46	46	46	1
All functional class records	38	47	54	597
All records (includes those records without classification information)	38	47	56	687

[1] 15 percent of the pavements have this skid number or worse (or stated another way, 85 percent of the pavements have this skid number or better).
[2] Includes both divided and undivided roadways —information was not present to segregate data.

TABLE 12 Average skid numbers by pavement type for three California districts (*28*)

Surface	Average Skid Number	Number of Samples
Dense Graded Asphalt Concrete	38	3,932
Open Graded Asphalt Concrete	37	704
Portland Cement Concrete	36	3,103
Portland Cement Concrete—Grooved	37	1,433
Chip Seal	44	243
Slurry Seal	38	48
Epoxy	34	16
Patch	34	7
Other	38	4

detect an object, recognize it as a hazard, decide on an action, and initiate that action. The more complex the decision, the longer the response time. Fortunately, the decision in a daytime stopping sight distance situation is relatively simple compared to decisions at intersections and interchanges. In the AASHTO model, it is presumed that the driver's response consists of moving the foot from the accelerator to the brake to initiate braking. In determining stopping sight distances, it is further assumed that the driver always applies the brake pedal with enough force to immediately lock the wheels.

At this point, the AASHTO model takes no further driver response into account. The driver is removed as controller of the vehicle, and the laws of physics as they relate to speed, tire-pavement friction, and roadway grade control stopping distance. Values used to represent these latter variables differ as a function of design speed, pavement type and condition, and roadway alignment. The perception-brake reaction time of the driver is constant for all combinations of conditions.

The current AASHTO model uses a 2.5 sec perception-reaction time for all stopping sight distance calculations. A model sensitive to actual human behavior would likely require this parameter to vary as a function of both vehicle speed and highway type; however, even though such variation seems logical and several researchers have offered recommendations in this regard, no studies or data were found in the literature to support this distinction. In addition, any differences would probably be so small that the effect on stopping sight distances would be insignificant.

Hooper and McGee (*33*) recommended different decision sight distance perception-response times for different design speeds. The range of recommended values was from 1.5 to 3.0 sec, which encompasses AASHTO's 2.5 sec value. One interpretation of this recommendation is that 2.5 sec is inclusive of almost all drivers under nearly all stopping sight distance situations, and it is only in more complex decision situations, such as intersections or interchanges, that longer perception-response times are needed.

Perception-Brake Reaction Time Studies. One of the least documented but most highly referenced studies on driver perception-brake reaction time was conducted by two Swedish researchers in 1971 (*34*). The primary importance of this study is the fact that AASHTO (*1, 2, 3*) identified this study as the fundamental basis for the 2.5 sec perception-brake reaction time used in the stopping sight distance equation. AASHTO states that "for approximately 90 percent of the drivers (in the Johansson and Rumar study), a reaction time of 2.5 sec was found to be adequate" (*34*).

Johansson and Rumar collected both *unexpected surprise* ($\bar{x}$ = 0.73 sec) and *anticipated surprise* ($\bar{x}$ = 0.54 sec) perception-brake reaction times (PRBT) for drivers on rural Swedish highways. The measured brake reaction times were in response to an auditory signal. They used these data to calculate an empirical correction factor or the relationship between a surprise brake reaction time to an anticipated brake reaction time as follows:

$$\text{Correction Factor} = \frac{\text{Surprise PRBT}}{\text{Anticipated PRBT}} = \frac{0.73\text{sec}}{0.54\text{sec}} = 1.35 \qquad (4)$$

The Johansson and Rumar data suggest that perception-response times collected under the usual conditions in which the driver anticipates the need to respond can be corrected for estimating a surprise perception-response time by simply multiplying the anticipated perception-response time by 1.35. This factor could be used to adjust *anticipated* perception-response times from other studies to produce a larger data base of *surprise* perception-response times.

Table 13 summarizes 10 perception-response time studies that have been documented in the literature. The top part of the table represents surprise conditions and the bottom part of the table represents anticipated conditions. The *Stimulus* column provides very brief descriptions of the stimulus to which the subject reacted. Note that three studies under the

TABLE 13 Summary of surprise and alerted perception-brake reaction time studies

Study Condition: Surprise (Unsuspecting Driver), Perception to Start of Brake Actuation (PBRT)					
	N	Ages	Mean	Std.Dev.	Stimulus
Covert:					
Sivak et al. (*35*)	1644	Mix	1.21	0.63	Unexpected Signal
Wortman (*36*)	839	Mix	1.30	0.60	Unexpected Signal
Chang et al. (*37*)	579	Mix	1.30	0.74	Unexpected Signal
MEAN ESTIMATES			1.27	0.66	
Surprise:					
Olson, Sivak (*40*)	49	Young	1.10	0.15	Unexpected Object
Olson, Sivak (*40*)	15	Old	1.06	0.10	Unexpected Object
Lerner (*38*)	56	Mix	1.50	0.40	Unexpected Object
MEAN ESTIMATES			1.28	0.20	
Study Condition: Anticipated (Alerted Driver), Onset to Start of Brake Actuation					
Driving Simulator:					
Bracket et al. (*41*)	114	Mix	0.31	0.11	Onset Red Light
Retchin et al. (*42*)	61	Old	0.66	0.66	Bumpa-Tel Test
Retchin et al. (*42*)	38	Old	0.84	0.10	Bumpa-Tel Test
Cation et al. (*43*)	104	Mix	0.43	0.10	Onset Red Light
MEAN ESTIMATES			0.56	0.10	
Behind the Wheel:					
Olson, Sivak (*40*)	49	Young	0.72	0.11	Anticipated Object
Olson, Sivak (*40*)	15	Old	0.73	0.10	Anticipated Object
Johansson, Rumar (*34*)	321	Mix	0.75	0.28	Anticipated Horn
MEAN ESTIMATES			0.73	0.16	

Covert category involve drivers stopping at traffic signals who had no idea that they were test subjects. The consistency and large sample sizes associated with the covert studies of Sivak (*35*), Wortman (*36*), and Chang (*37*) suggest that these findings should be considered a good estimate of the true perception-brake reaction time for this situation (unsuspecting driver, unexpected signal).

The study by Lerner (*38*) in the *Surprise* category is significant because he compared perception-brake reaction times for older drivers to those of younger drivers in a stopping sight distance situation. Fifty-nine of 116 subjects reacted to an unexpected object by braking; the remainder reacted by steering and/or braking. For those subjects that braked, the mean perception-brake reaction time was 1.5 sec with a standard deviation of 0.4 sec. The 85th percentile perception-brake reaction time was 1.9 sec, and the longest observed perception-brake reaction time was 2.54 sec. **There was no significant difference in perception-brake response time because of age.** Although this result is important for stopping sight distance situations, it does not suggest that there is no age-related slowing for more complex driving situations (*38*).

The Johansson and Rumar data (*34*) and the Olson data (*8*) in the alerted driver behind-the-wheel category are also quite consistent with one another. The Olson data were in response to a visual signal, and the Johansson and Rumar data were in response to an auditory signal. Because of the consistency, they should be considered good estimates of the true perception-brake reaction time for an anticipated object. As expected, perception-brake reaction times for unexpected signals and objects were longer than perception-brake reaction times for anticipated signals and objects. The ratio between the unexpected and anticipated perception-brake reaction times during daytime conditions is 1.75, slightly higher than the Johansson and Rumar correction factor.

Driver Braking Behavior

The assumed driver braking behavior in an emergency situation is not consistent in the literature. Some confusion exists from the pre-antilock brake era (1960s) to the more recent studies (1980s). A 1955 study by Starks and Lister states that in an emergency situation "it is suspected that drivers apply their brakes as hard as possible" (*39*). This idea differs from *NCHRP Report 270*, in which the authors state that drivers will "modulate" their braking to maintain directional control (*8*).

Deceleration Studies. Limited research has been conducted on driver-performance and deceleration characteris-

TABLE 14 Deceleration and braking distances for vehicles with one axle locked (*44*)

Load	Pavement Quality	Maximum Deceleration (g) Range	Median	Braking Distances (ft) Range	Median
Passenger Cars (10-post 1980 cars)					
Unloaded	Poor, wet	0.388-0.521	0.414	195-262	247
Unloaded	Good, dry	0.804-0.964	0.881	105-126	116
Loaded	Poor, wet	0.327-0.440	0.400	231-310	256
Loaded	Good, dry	0.671-0.946	0.822	107-151	125
Pickups (3 pickups, 1 representative of pickups produced in the 1970s, 2 representative of more recent models that have similar load capacity)					
Unloaded	Poor, wet	0.493-0.411	0.452	206-247	226
Unloaded	Good, dry	0.943-0.872	0.908	108-116	112
Loaded	Poor, wet	0.445-0.407	0.426	228-249	239
Loaded	Good, dry	0.924	0.924	110	110
Van (1 van, representative of vans produced in the 1970s)					
Unloaded	Poor, wet	---	0.508	---	200
Unloaded	Good, dry	---	0.953	---	107
Loaded	Poor, wet	---	0.462	---	220
Loaded	Good, dry	---	0.890	---	114

* Calculated using the given deceleration and V = 55 mph in the following formula: $BD = 1.47V^2/2sg$.

tics in emergency situations; however, a number of studies have been conducted on braking performance of trucks and other motor vehicles, including a NHTSA study to evaluate antilock braking performance of two-axle trucks (*25*) and a light truck and passenger car braking performance study in the mid-1980s (*26*). These research activities, however, focused primarily on vehicle performance characteristics only, such as in extreme vehicle-maneuver conditions. Test drivers were aware of the study, and the maneuvers were anticipated at certain locations on the test track.

In 1983, Shadle, Emery, and Brewer reported test results on 10 representative passenger cars, all 1980 or later vintage, 3 pickup trucks, and 1 van (*44*). The vehicles were equipped to provide deceleration histories, pedal forces, and other parameters. They were not actually braked to a stop, but instead, measurements of brake pedal force were taken after wheel lock or steady state was obtained. Stopping times were extrapolated using standard equations of kinematics and deceleration values determined from the brake pedal force. Readings were taken just before either front or rear axle locked and then with either or both axles locked. Table 14 illustrates the stopping distance ranges and median values calculated from the deceleration values presented in the paper for vehicles with one axle locked. The generated data cannot be considered completely empirical because partial stops were extrapolated.

Design Decelerations. The Institute of Transportation Engineer's (ITE) Handbook states that *decelerations up to 10 ft/sec² are reasonably comfortable for passenger car occupants* (*45*). An earlier version of the ITE Handbook suggested 15 ft/sec² as the comfort threshold value. Table 15 lists decelerations derived from information provided by AASHTO that shows the distance traveled by passenger cars during deceleration to a stop (*2, 3*). Decelerations for minimum braking distance on a dry pavement are near 20 ft/sec². Note that AASHTO-defined comfortable decelerations are less than the ITE rate of 10 ft/sec² (0.32 g).

TABLE 15 Deceleration based on Figure II-17 in the *Green Book* (*1,2,3*)

Type of Deceleration	Speed (mph)	Stopping Distance (ft)	Friction	Deceleration (ft/sec²)	Deceleration (m/sec²)
Comfortable deceleration	60 30	475 180	0.25 0.17	8.13 5.37	2.49 1.64
Minimum braking Dry pavement	60 30	210 50	0.57 0.60	18.4 19.3	5.61 5.88
Minimum braking Wet pavement	60 30	295 70	0.41 0.43	13.1 13.8	3.99 4.21

Driver Performance Field Studies

To provide additional information on driver braking performance to an unexpected object in the roadway, four different but similar field studies were undertaken. The study design used an instrumentation package to measure driver perception-brake response times, braking distances, and decelerations to unexpected and anticipated stops. The study design took into account vehicle handling differences and driver capabilities associated with ABS, wet and dry pavement conditions, and the effects of roadway geometry.

Vehicle instrumentation consisted of a Compaq 386 laptop computer, a data acquisition software program and translator, an accelerometer, and a fifth wheel/distance measuring device. This system recorded time, longitudinal acceleration, lateral acceleration, brake pedal status, and gas pedal status at 1-ft intervals along the test course. Vehicle speeds, perception-brake reaction times, braking distances, and deceleration profiles were determined for each braking maneuver in the driver performance studies.

The driver performance studies consisted of the four studies described in Table 16. Study 1—conducted on a closed course—was the largest, requiring more than 2,000 braking maneuvers. Studies 2 and 3 also were conducted on closed courses and built on the results from Study 1. The difference in the second two studies was that Study 2 used a single test vehicle, and Study 3 used multiple personal vehicles. Part A of Studies 2 and 3 evaluated driver performance to an unexpected object scenario, and Part B evaluated driver performance to an expected object scenario. Study 2 also had a Part C, testing a driver's baseline perception-response time. Study 4, an open-road study, measured driver performance data for an unexpected object scenario.

Study 1—Closed-Course Braking Study. The purpose of Study 1 was two-fold. First, it served as a pilot study to determine the amount of testing and under what conditions the remaining braking performance studies would be conducted. Nine TTI employees, three of whom were high performance drivers, participated in Study 1. Each of the subjects experienced several combination of test conditions, as well as several repetitions of these conditions. It was hoped that by having a sizable data base, test conditions that were not significantly different in terms of driver braking performance could be eliminated prior to Studies 2 and 3. Second, Study 1 provided high speed braking performance data. Previous studies have been limited because of the dangers involved with braking at speeds in excess of 60 mph; however, this study was able to use a closed-course test track and high performance drivers in order to obtain high speed braking performance data.

To evaluate a large number of variables that affect driver/vehicle braking performance, several test conditions were established for Study 1. All nine subjects performed braking maneuvers at speeds of 40 and 55 mph, and the three *expert* subjects were further tested at 70 mph. The nine subjects were also tested under several other conditions: with antilock brakes enabled or disabled, with pavement conditions wet or dry, and on a tangent section and a left and right horizontal curve. Additional test conditions included either braking at the onset of an anticipated signal (anticipated stop) or braking at the onset of a randomly activated signal (surprise stop). The different test conditions for the nine subjects in Study 1 are summarized in Table 17.

Subjects were instructed to drive their vehicles through the test course at the required speed for the condition being evaluated. The first series of tests for each test subject were the *anticipated signal* maneuvers. The test administrator counted down by saying "Ready, set . . ." and then illuminated the windshield-mounted signal. When the test subjects saw the signal, they were instructed to bring the vehicle to a stop as quickly as possible, but to stay within the 12-ft lane. For each test condition combination (speed, pavement, geometry, and ABS) three trial runs were performed. The second series of tests for each test subject were the *anticipated surprise* maneuvers. Each test subject was instructed that somewhere along the test course the windshield-mounted signal would illuminate. At the onset of this surprise signal, the driver was

TABLE 16 Summary of driver braking performance studies

Study	Part	Test Condition	Test Vehicle	Test Subjects	Encounter
Study 1		Closed	TTI Vehicle	TTI	Planned/Surprise
Study 2	Part A	Closed	TTI Vehicle	Pool	Unexpected
	Part B	Closed	TTI Vehicle	Pool	Expected
	Part C	Baseline	TTI Vehicle	Pool	Expected
		Baseline	TTI Vehicle	Church	Expected
Study 3	Part A	Closed	Personal	Pool	Unexpected
	Part B	Closed	Personal	Pool	Expected
Study 4		Open-Road	Personal	Pool	Unexpected

TABLE 17 Summary of test conditions for Study 1

	Number of Test Conditions Per TTI Driver					
Condition	Speed *(40/55)*	ABS *(On/Off)*	Pavement *(Wet/Dry)*	Geometry *(Tangent/Curve)*	# Trials	Total
Planned	2	2	2	3	3	72
Surprise	2	2	2	3	5	120
No Signal	2	2	2		5	40
Total						232
	Number of Test Variables Per Expert Subject					
Condition	Speed *(40/55/70)*	ABS *(On/Off)*	Pavement *(Wet/Dry)*	Geometry *(Tangent/Curve)*	# Trials	Total
Planned	3	2	2	3	3	108
Surprise	3	2	2	3	5	180
No Signal	3	2	2		5	60
Total						348
Total Maneuvers =	6 "TTI" x 232 Runs + 3 "Expert" x 348 Runs =					2436

to bring the vehicle to a stop as quickly as possible. Approximately 20 percent of the time no signal was given in an effort to minimize driver expectancy. Five trials were performed for each test condition.

Study 1 Results. Overall perception-brake reaction time across all test conditions was 0.34 sec, with a standard deviation of 0.173 sec. Overall foot movement time from accelerator to the brake pedal was 0.18 sec, with a standard deviation of 0.094 sec. Neither the main effects of *stopping condition, speed,* nor the interaction of *condition and speed* was statistically significant for reaction time. Foot-movement time was slower for maneuvers under the *anticipated signal* conditions (0.21 sec) than under the *anticipated surprise* conditions (0.17 sec), that is, foot movement time was slightly faster when drivers did not know when to expect the onset of windshield-mounted signal.

Analysis of variance (ANOVA) techniques were used to test *braking distances* for differences between the 40 and 55 mph data sets. Five independent variables, including *condition, geometry, ABS, pavement,* and *speed* (plus *subjects*), were considered. The variable *speed* had two levels for this analysis: 40 and 55 mph. Neither the *condition* nor the *geometry* levels of those variables were significantly different from one another. Observed braking distances for the *anticipated surprise* signal condition were about the same as they were for the *anticipated signal* condition, and observed braking distances on tangents were about the same as they were for left and right horizontal curves.

As expected, both *pavement* and *speed* conditions resulted in significantly different braking distances. The ABS condition was also significant, meaning that braking distance is affected by whether ABS is enabled. Figure 8 shows the braking distance comparisons between wet and dry conditions, with and without ABS at the three nominal speeds. Note the differences between wet and dry pavement and the relatively small benefits of ABS at 40 mph. The braking distance differences between ABS and no ABS on wet pavements are larger at the higher speeds, that is, 50 ft at 55 mph and 90 ft at 70 mph.

ANOVA techniques were also used to test the maximum longitudinal deceleration, *Max* G_x, achieved during the braking maneuver. Note that *Max* G_x is not equivalent to a sustained deceleration for the entire maneuver. A typical deceleration profile (Figure 9) yields a maximum value at some time during the maneuver (in this case, at 4.3 sec) and then falls off or fluctuates at a lesser deceleration until the vehicle is at a standstill. As shown in Figure 9, deceleration profiles are not linear.

Consistent with the braking distance analysis, the *condition* and *geometry* conditions resulted in no differences in maximum decelerations, and the *ABS* and *pavement* conditions resulted in significantly different maximum decelerations; however, the *ABS* condition differences were not large enough to be of practical significance. Interestingly, maximum deceleration was insensitive to speed prior to braking. In other words, even though different drivers reached different maximum decelerations, the average of the maximum decelerations was the same for both the 40 and 55 mph data sets. The average maximum deceleration across all conditions and speeds was 21.3 ft/s^2 with ABS enabled and 24.4 ft/s^2 with ABS disabled. The average maximum deceleration was 21.3 ft/s^2 for the wet pavement conditions and 28.3 ft/s^2 for dry pavement conditions.

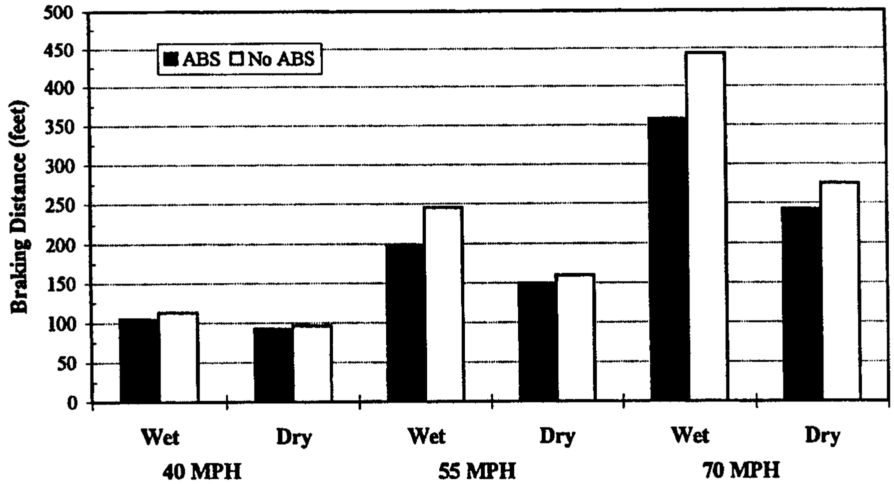


Figure 8. Braking distances for wet and dry pavement braking maneuvers at test speeds of 40, 55, and 70 mph.

The data set for the nine test subjects in Study 1 also afforded an opportunity to test for differences in equivalent constant decelerations because of subjects and/or test conditions. Initial speed and braking distance were used to calculate the equivalent constant deceleration for each of the braking maneuvers in the data base. *Condition* and *geometry* were dropped from further testing because they did not result in significant differences in the previous analysis. *ABS, pavement*, and *subjects* remained as independent variables for the ANOVA. ABS did not result in different equivalent constant decelerations for the 40 mph initial speeds. *Pavement* conditions, however, were statistically significant for both the 40 and 55 mph initial speeds. As expected, there were also statistically significant differences among drivers. Equivalent constant decelerations for individual drivers ranged from 6.8 ft/s^2 to 10.0 ft/s^2 at 40 mph and from 9.0 ft/s^2 to 12.6 ft/s^2 at 55 mph.

Table 18 is a summary of the equivalent constant deceleration percentile values. Nominal speed, ABS condition, pavement condition, and number of observations are presented in columns 1 through 4, respectively. The mean and standard deviation for the equivalent constant deceleration values are in column 5 and 6, respectively. The last 3 columns provide an estimate of the percentiles of equivalent constant decelerations in the population represented by this sample. For example, under wet conditions with no ABS and at 55 mph, only 5 percent of the braking maneuvers generated an equivalent constant deceleration of 0.30 g (9.3 ft/s^2) or less. Conversely, 95 percent of the braking maneuvers generated an equivalent constant deceleration of 0.30 g or more.

In the final part of Study 1, a series of runs at initial speeds of 40 and 55 mph were made with the test vehicle using one of the expert drivers. The procedure was very simple: the dri-

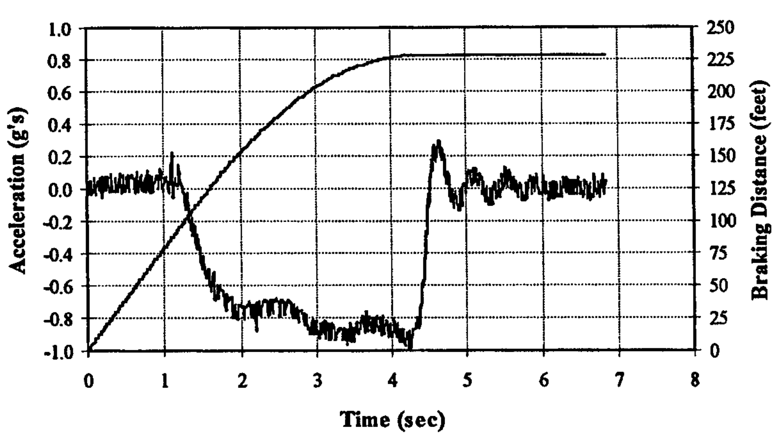


Figure 9. Typical deceleration profile during braking maneuver.

TABLE 18 Summary of equivalent constant deceleration percentile values*

Speed (mph)	Pavement	ABS	N	Constant G_x	STD	85%	90%	95%
40	Dry	No	191	0.60	0.122	0.47	0.44	0.40
40	Dry	Yes	176	0.62	0.134	0.48	0.45	0.40
40	Wet	No	203	0.49	0.067	0.42	0.40	0.38
40	Wet	Yes	186	0.54	0.071	0.47	0.45	0.42
55	Dry	No	216	0.65	0.135	0.51	0.48	0.43
55	Dry	Yes	203	0.71	0.163	0.54	0.50	0.44
55	Wet	No	146	0.42	0.074	0.34	0.33	0.30
55	Wet	Yes	171	0.53	0.206	0.32	0.26	0.19

* All drivers

ver was instructed to perform a series of straight-line stopping maneuvers on a wet pavement (same area as the 40 and 55 mph tests reported above) using the maximum pedal effort possible (locked wheel if no ABS). Table 19 summarizes the results from these test runs, which should be compared to the performance of the Expert and TTI drivers under similar test conditions.

For example, under locked wheel conditions at 40 mph with no ABS on a wet pavement, the mean locked-wheel braking distance was 98 ft, and the average controlled braking distance under the same test conditions was 113 ft. At 55 mph, the difference between these two conditions is larger, 178 ft versus 246 ft. This analysis summarized the differences between what the vehicle can achieve versus what drivers actually do, in terms of percentage of potential braking performance that was exhibited by the Study 1 drivers. At highway speeds with or without ABS, driver braking performance is approximately 75 percent of full braking capability.

Summary of Study 1. Under closed-course conditions with drivers anticipating braking at some time during the test runs, all drivers were capable of a shorter perception-response time than AASHTO's 2.5 sec. Thus, remaining studies focused on conditions that were not so predictable. Braking performance and maximum deceleration were similar for the two stopping condition scenarios, *anticipated* and *surprise*. Because *surprise* is more closely associated with real-world stopping situations, that procedure was used in the remaining field studies. Braking performance and maximum deceleration also were similar for the different geometric conditions—tangents and left and right horizontal curves. Thus, remaining studies included braking on horizontal curves and tangent sections, but not systematic variation of tangents and left and right curves as was done in Study 1.

The differences in the nine subjects' maximum and equivalent constant deceleration at 55 and 40 mph suggested that additional braking performances studies with volunteer subjects be done at 55 mph. As expected, *ABS* conditions made a difference in braking distance and maximum deceleration, indicating a need for additional study of that condition. The *pavement* condition, wet or dry, also made significant differences in braking distance and maximum deceleration. Thus, both wet and dry pavement conditions were included in the remaining studies.

Between individual drivers there were differences in the level of performance in both braking distance and maximum deceleration. Maximum decelerations ranged from 0.7 g to 0.9 g (22.5 ft/s^2 to 2.90 ft/s^2). The average value for

TABLE 19 Summary of locked-wheel vehicle performance on a wet pavement*

Speed (mph)	ABS	Statistic	Braking Distance (ft)	Max G_x	Constant G_x
40	On	Mean Std. Deviation	85.8 2.3	0.82 0.04	0.60 0.02
40	Off	Mean Std. Deviation	97.8 4.3	0.77 0.02	0.56 0.03
55	On	Mean Std. Deviation	153.6 3.7	0.82 0.02	0.59 0.08
55	Off	Mean Std. Deviation	178.2 8.5	0.68 0.03	0.53 0.02

* Expert driver only

all drivers was 0.7 g (25.1 ft/s^2) at 40 and 55 mph and 0.82 g (26.4 ft/s^2) at 70 mph. Equivalent constant deceleration also varied between individual drivers, ranging from 0.46 g to 0.70 g (14.8 to 22.5 ft/s^2) depending on initial velocity before braking. A mean value for all drivers was 0.56 g (18.0 ft/s^2) at 40 mph, and 0.60 g (19.3 ft/s^2) at 55 mph. Based on the 55 mph data, 85 percent of all drivers will produce derived equivalent constant decelerations of at least 0.34 g (10.9 ft/s^2) on wet pavements without ABS and at least 0.54 g (17.4 ft/s^2) on dry pavements with ABS; and 95 percent of all drivers will produce derived equivalent constant decelerations of at least 0.30 g (9.3 ft/s^2) on wet pavements without ABS and at least 0.44 g (13.2 ft/s^2) on dry pavements with ABS.

Study 2—Closed-Course Braking Study. The purpose of Study 2 was to gain additional insight into driver braking performance under closed-course conditions. Twenty-six subjects representative of the general driving population participated in the study. All 26 participants performed braking maneuvers at a speed of 55 mph using the following test conditions: with antilock brakes enabled or disabled, with pavement conditions wet or dry, and with two different geometric conditions, a tangent section and a horizontal curve section. The antilock brake variable remained on or off throughout the testing for each test subject, as opposed to the test conditions in Study 1 where each test subject performed with and without ABS. To avoid potential bias, the test subjects in Study 2 were not informed whether or not ABS was enabled.

Study 2 included three different parts: an unexpected object segment, an expected object segment, and a simple brake-reaction time test. The unexpected object segment, Part A, was different than the *surprise* condition in Study 1 in that this study required test subjects to react to a truly unexpected object in the roadway. Since an unsuspecting driver can only be truly surprised once in a testing environment, each of the 26 test subjects provided a single data point. The expected object segment, Part B, followed the unexpected object segment, and the test subjects performed a series of *anticipated* braking maneuvers to a surprise condition, similar to the *surprise* condition in Study 1, that is, they stopped their vehicles at the onset of the windshield mounted signal. The simple brake reaction time segment, Part C, required each of the 26 test subjects to perform a simple brake-reaction time test while sitting in the driver's seat of the test vehicle. A separate control group of subjects that did not participate in Parts A and B also performed the simple reaction time test. All the test conditions for Study 2 are summarized in Table 15.

The unexpected object scenario, or Part A of Study 2, was located at the end of a long tangent and consisted of a 3-ft-high fabric barricade, spanning both lanes of the roadway, that suddenly appeared in the path of the driver. Before deployment, it was stored in a small, 2-in.-wide trench in the concrete pavement that was covered by black rubber strips to prevent it from being seen by the test subject. The fabric barricade was a lightweight, black landscaping material with four 36-in. by 36-in. stop signs attached to its face. The barricade scenario was designed to represent an unexpected object that might suddenly appear in the roadway and compel the driver to stop the vehicle before hitting it. For safety purposes, it was designed to break-away on impact. Each test subject's braking performance was recorded in response to this scenario.

The unexpected object appeared 210 ft (2.5 sec at 55 mph) in front of the test subject. This distance was selected based on 1.0 sec perception-brake reaction time and a dry pavement friction value of 0.80. If the subject was provided more than 1.0 sec of perception-brake reaction time, it was thought that he or she might have time to initiate an evasive maneuver. The intent of this study was not to allow time for an evasive maneuver, but to have the test subject to *stomp* the brakes to stop the vehicle before striking the barrier. Stopping before striking the barrier was secondary to having the subjects take evasive actions.

Study 3—Closed-Course Braking Study. The purpose of Study 3 was to determine driver/vehicle braking performance characteristics of drivers operating their own vehicles, rather than a vehicle owned and maintained by someone else. This study procedure was identical to Study 2 with the exception of the type of vehicle being tested. Study 3 was a three-part study that included the unexpected object segment, the expected object segment, and the simple brake-reaction time tests that were used in Study 2. Twelve different subjects representative of the general driving population participated in Study 3, five younger drivers and seven older drivers. All braking maneuvers were conducted at a speed of 55 mph including the unexpected object part of the study. The only difference in test conditions between Studies 2 and 3 was that none of the personal vehicles had ABS. The test conditions for Study 3 are summarized in Table 20.

Study 4—Open-Roadway Braking Study. The purpose of Study 4 was to determine a driver's perception-response times when presented with an unexpected object on the open road. Unlike the methods used in Studies 2 and 3, this study's experimental approach required participants to drive along a rural, low-volume, two-lane roadway, in their own vehicle. Each subject was led to believe that they were involved in a roadway evaluation test, not a driver performance test. As they approached a particular location on the roadway, an unexpected object suddenly appeared in their field of vision and moved toward them from the right side of the roadway. The intent of this study was not to have the subject brake to a complete stop to this unexpected object, but instead to react to the object by braking, making a steering maneuver, or both, and then drive on after realizing that the object could be avoided.

TABLE 20 Summary of test conditions for Study 2

	Number of Variables Per Test Subject					
Segment - Condition	Speed (mph)	ABS	Pavement	Geometry	# Trials	Total
Study 2						
Part A - Unexpected	55	On/Off	Dry	T	1	1
Part B - Expected	55	On/Off	Wet/Dry	T/C	5	20
Part C - RT Test					10	10
Total						31
Total Maneuvers = 26 Test Subjects x 31 Trials						806
Study 3						
Part A - Unexpected	55	Off	Dry	T	1	1
Part B - Expected	55	Off	Wet/Dry	T/C	5	20
Part C - RT Test					10	10
Total						31
Total Maneuvers = 12 Test Subjects x 31 Trials						372

Study 4, similar to Studies 2 and 3, required a group of volunteer subjects representative of the driving population. Twelve additional subjects participated in Study 4, including six younger drivers and six older drivers. No preset test conditions, such as a particular speed or the required use of antilock brakes, applied to this study. Only one test run was conducted for each subject with the primary variable of interest being the subject's reaction to the unexpected object. Other than dry pavement, no other test conditions applied to this study. All test runs were conducted at a speed of approximately 45 mph, but varied slightly according to the test subject's speed preference.

Study 4 was conducted along a portion of a state-maintained, rural, two-lane roadway with an average daily traffic of approximately 300 vehicles per day. The test site was in the middle of a long, flat tangent section. The unexpected object was a 30-in. diameter, empty cardboard barrel that rolled down a ramp on the back of a pick-up parked alongside the roadway. A remote-controlled mechanism in the back of the truck released the barrel when a radio frequency signal was activated by the test administrator in the vehicle. A photograph of this scenario is shown in Figure 10.

Similar to the unexpected object scenario in Studies 2 and 3, it was assumed that if the subject had more than 1.0 sec of perception-brake reaction time, he or she would have time to decide on an appropriate evasive maneuver. Thus, the barrel appeared to roll toward the travel lane approximately 75 ft (1.1 sec at 45 mph) in front of the test subject. The situation was designed to appear as if the barrel would strike the front of the vehicle; however, for safety purposes, small cords attached to the barrel prevented it from rolling any further than the edge of the roadway.

Results of Studies 2, 3, and 4. The results of Studies 2, 3, and 4 are divided into three parts: perception-brake reaction times, braking distances, and driver decelerations, each of which are discussed in the following sections.

Perception-Brake Reaction Times. Table 21 summarizes the observed perception-brake reaction times to the one-time *unexpected* object in Studies 2, 3, and 4. The ANOVA results indicated that there were significant differences because of the type of study conducted. The perception-brake reaction times in Study 2, where test subjects drove the TTI vehicle, were significantly shorter than the perception-brake reaction times in Studies 3 and 4 where test subjects drove their own vehicles. These results seem to indicate that drivers may be more alert in an unfamiliar vehicle. Upper-percentile values were calculated and a perception-brake reaction time of approximately 1.98 sec

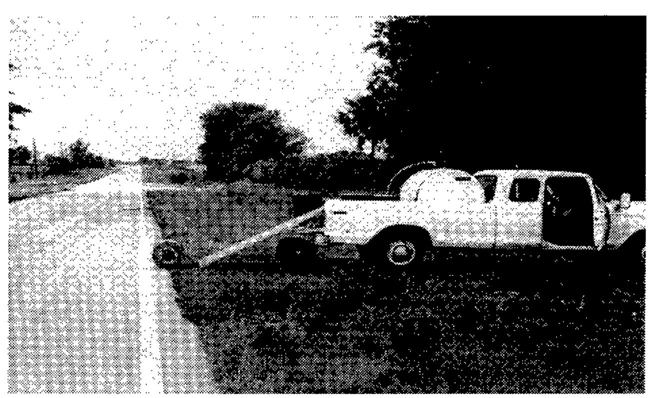


Figure 10. Unexpected object used in Study 2.

TABLE 21 Summary of perception-response time to an unexpected object

Study	Age	No. of Test Subjects	Mean PRT (sec)	Standard Deviation (sec)
Study 2	Older Younger	12 10	0.82 0.82	0.159 0.203
Study 3	Older Younger	7 3	1.14 0.93	0.353 0.191
Study 4	Older Younger	5 6	1.06 1.14	0.222 0.204

includes almost all (95 percent) of the drivers in these three studies.

The analysis of drivers' perception-response times to the *expected* object scenarios, (the onset of the windshield-mounted signal) was limited to Part B of Studies 2 and 3. Each participant performed a series of 20 braking maneuvers on a combination of dry and wet pavements and tangent and horizontal curve sections. The braking maneuver was initiated when the test subject reacted to the signal mounted on the windshield. This signal was the *expected* object in that the test subjects knew that the signal would be initiated, but not when it would be illuminated. One driver chose not to participate in Parts B and C of the study after panicking, veering off the course, and sustaining minor vehicle damage to his vehicle in the *unexpected* object scenario. A summary of the observed perception-brake reaction times in Part B of Studies 2 and 3 is presented in Table 22.

In addition to the perception-brake reaction times from the *unexpected* and *expected* object scenarios, participants in Studies 2 and 3 also provided data for a baseline perception-brake reaction time comparison. Each subject sat behind the wheel of the stationary test vehicle, either the TTI vehicle or the subject's personal vehicle. With a foot on the accelerator pedal, the subject watched for the illumination of the windshield-mounted signal and initiated a brake response as fast as possible after that point in time. A total of 10 repetitions was conducted for each test subject. These data were compared with baseline data from a group of 18 subjects from a local church to ensure the test subjects were representative of the driver population. A summary of the observed baseline perception-brake reaction times for Studies 2, 3 and 4 is presented in Table 23.

The results of the *unexpected* object perception-brake reaction time observations from Studies 2, 3, and 4 indicated that a perception-brake reaction time of approximately 2.0 sec seems to be inclusive of nearly all the subjects' responses to the *unexpected* object that suddenly appeared in the road. Most of the test subjects responded to this *unexpected* condition by braking the vehicle to a stop; however, some drivers in Study 4 steered to avoid the object because the opposite lane was open for this type of maneuver. The test subjects responded quicker to the *unexpected* object when driving in the unfamiliar vehicle than when driving their own vehicles. Observed perception-brake reaction times for the subjects in the TTI vehicle in Study 2 were approximately 75 percent of the perception-brake reaction times for the subjects in their own vehicles in Studies 3 and 4. For subjects in their own vehicles, no significant differences were noted between the *unexpected* object perception-brake reaction times for a closed-course versus open-road environment.

The *expected* object perception-brake reaction time observations from Studies 2, 3, and 4 indicated that the mean value for this type of scenario was approximately 0.55 sec. Because of the large sample size, statistically significant differences in mean reaction times were noted between the age and gender groups. The mean perception-brake reaction time for the

TABLE 22 Summary of perception-response time to an expected object

Study	Age	Gender	No. of Test Subjects	Total No. Repetitions	Mean PRT (sec)	Standard Deviation (sec)
Study 2	Older	Female Male	7 7	134 129	0.66 0.65	0.216 0.228
	Younger	Female Male	6 6	117 113	0.57 0.48	0.167 0.088
Study 3	Older	Female Male	5 3	90 52	0.67 0.65	0.252 0.345
	Younger	Female Male	2 1	40 20	0.49 0.55	0.168 0.078

TABLE 23 Results of baseline perception-response time observations

Study	Age	Gender	No. of Test Subjects	Total No. Repetitions	Mean PRT (sec)	Standard Deviation (sec)
Study 2	Older	Female	7	70	0.50	0.09
		Male	7	71	0.47	0.06
	Younger	Female	6	60	0.47	0.13
		Male	6	60	0.42	0.08
Study 3	Older	Female	5	50	0.47	0.08
		Male	3	30	0.44	0.10
	Younger	Female	2	20	0.39	0.06
		Male	1	10	0.48	0.04
Church	Older	Female	10	94	0.52	0.19
		Male	8	81	0.43	0.14

younger and older driver groups was 0.52 and 0.66 sec, respectively. The mean perception-brake reaction time for the male and female driver groups was 0.59 and 0.63 sec, respectively. Significant differences were also found between pavement and geometric conditions. The test subjects responded quicker for the wet pavement braking maneuvers, with the likely explanation being that they knew the maneuver would be performed on the wet pavement section and were anticipating the signal activation. Because of lower workloads, the test subjects also responded faster on the tangent sections that they did on the horizontal curve sections.

The perception-brake reaction times of the test subjects in the control group did not differ from the perception-brake reaction times of the older subjects that participated in Studies 2 and 3. Statistically significant differences were noted between age and gender groups, but no other significant differences existed among the three groups of subjects. From this analysis, it appears that the older subjects tested in Studies 2 and 3 are representative of the older drivers in the community.

Braking Distances. The braking distance values were analyzed from Part A of Studies 2 and 3. This part of each study provided braking distances for a completely unexpected object, much like the assumed scenario in AASHTO's stopping sight distance model. These studies provided a unique opportunity to test not only vehicle braking performance, but also driver braking performance to an unexpected object. As previously mentioned, all of the *unexpected* object braking maneuvers were performed on a dry tangent section of the course. The test vehicle in Study 2 had the option of having the ABS processor enabled or disabled, and none of the test vehicles in Study 3 was equipped with ABS. There were no noticeable improvements in braking distances for vehicles with ABS. Many of the subjects did not *stomp* the brakes during the entire braking maneuver, unless it was evident that the obstacle might be struck.

The analysis of subjects' braking performance to the *expected* object scenarios was limited to Part B of Studies 2 and 3. As mentioned, each test subject performed a series of 20 braking maneuvers on a combination of dry and wet pavements and tangent and horizontal curve sections. The braking maneuver began when the test subject reacted to the windshield-mounted signal. Because the driver was instructed to stay within the 12-ft lane at all times, braking maneuvers that left the travel lane were excluded from further analysis. These loss-of-control braking maneuvers occurred more frequently on the wet pavement and on horizontal curves, and generally only the first or second maneuver for a particular test subject. Once the subject realized the vehicle's capabilities, vehicle controllability improved for subsequent test runs. Test subjects performing with ABS enabled in Study 2 did not experience this problem.

The braking distances in the braking performance studies were compared with the AASHTO braking distances for a design speed of 55 mph, that is, 336 ft. For these studies, test subjects in their own vehicle without ABS exhibited a 90th percentile braking distances of 330 ft on a wet tangent and 342 ft on a wet curve. Test subjects in the TTI vehicle without ABS exhibited a 90th percentile braking distances of 244 ft on a wet tangent and 278 ft on a wet curve without ABS. Thus, it appears that the AASHTO braking distance is inclusive of nearly all wet pavement braking distances for these studies.

The test subjects did not perform as well (i.e., longer braking distances) to the *expected* object condition as they did to the *unexpected* object condition. These differences were statistically significant. Drivers exhibited approximately 25 percent shorter braking distances to the *unexpected* object, probably because this scenario appeared hazardous and they were willing to accept higher decelerations in order to stop. There were no significant differences in the braking distances because of geometric conditions, that is, horizontal curves and tangent sections. One possible explanation of this result is that the test subjects were using about 75 percent of the frictional capabilities of the pavement/braking ability of the test vehicle. The use of ABS resulted in approximately 10 to 15 percent shorter braking distances. These improvements were most noticeable for braking maneuvers on wet pave-

TABLE 24 Summary of maximum deceleration to an unexpected object

			Maximum Deceleration		Constant Deceleration	
Study	ABS	No. of Test Subjects	Mean Max G_x* (g)	Standard Deviation (g)	Equivalent Constant G_x* (g)	Standard Deviations (g)
Study 2	No	6	0.91	0.08	0.62	0.07
	Yes	7	0.91	0.14	0.63	0.08
Study 3	No	7	0.74	0.09	0.55	0.07

* Dry Pavement Conditions

ments and horizontal curves. At 55 mph, the test subjects operating the TTI vehicle with ABS enabled could stop the vehicle on a wet horizontal curve in approximately 200 ft. For these same conditions, test subjects operating the TTI vehicle with the ABS disabled or in their own vehicles without ABS stopped in 225 ft and 237 ft, respectively.

Driver Decelerations. Analysis of the observed driver decelerations from the braking maneuvers in Studies 2 and 3 are presented in this section. The deceleration value of interest was the maximum deceleration achieved during a particular braking maneuver, referred to as *Max* G_x. This value is not sustained throughout the braking maneuver, but reached at some time during the braking maneuver. The instrumentation also recorded lateral accelerations, but these data were not analyzed for this study. The analysis of the maximum deceleration data involved determining the average and upper-percentile values from the data set, and using ANOVA techniques to tests for significant differences because of vehicle type, study condition, ABS condition, pavement condition, and geometry condition. The driver braking distance data were used to build the deceleration data set.

A summary of the *Max* G_x deceleration to the *unexpected* object is presented in Table 24. As mentioned, all of the maneuvers were performed on a dry tangent section of the course, the test vehicle in Study 2 either had the ABS processor enabled or disabled, and none of the test vehicles in Study 3 was equipped with ABS. Note that the average *Max* G_x values are near the pavement's coefficient of friction; however, the maximum longitudinal deceleration was usually not exhibited until the last portion of the braking maneuver. As the vehicle was constantly decreasing in speed and as the driver came closer to the *unexpected object*, the pedal pressure on the brake increased to near locked-wheel braking. The data also indicate that the test subjects chose higher decelerations in an unfamiliar vehicle than they did in their own vehicles. Test subjects in Study 3 without ABS chose a mean *Max* G_x of 0.74 g (23.8 ft/s^2) and test subjects in Study 2 without ABS chose a mean *Max* G_x value of 0.91 g (29.3 ft/s^2). Given that the deceleration profiles were similar, the difference in *Max* G_x values between the two studies corresponds to the differences in braking distances between the two studies.

The analysis of the deceleration data was taken one step further to examine one of the assumptions of the AASHTO model: uniform or constant deceleration throughout the braking maneuver. Initial speed and braking distance were used to calculate the equivalent constant deceleration for each of the braking maneuvers in the data set. Upper-percentile estimates from the equivalent constant deceleration data set are shown in Table 25. Note that the values presented in the table are a result of dry pavement braking maneuvers in response to the *unexpected* object scenario.

The same data set used for analysis of the braking distances to an *expected* object was used for this portion of the analysis. The analysis indicated that the maximum decelera-

TABLE 25 Percentile estimates of equivalent constant deceleration to an unexpected object

	Equivalent Constant Deceleration (g)		
	Study 2 ABS	Study 2 No ABS	Study 3 No ABS
Mean	0.63	0.62	0.55
75th	0.50	0.49	0.43
90th	0.42	0.42	0.37
95th	0.38	0.38	0.32
99th	0.28	0.29	0.24
AASHTO*	0.30	0.30	0.30

*Assumed Wet Pavement Friction Coefficient at 55 mph

TABLE 26 Summary of maximium deceleration to an expected object

					Maximum Deceleration		Equivalent Constant Deceleration	
Study	ABS	Pavement	Geometry	Total No. Repetitions	Mean Max G_x (g)	Standard Deviation (g)	Mean Constant G_x (g)	Standard Deviation (g)
Study 2	No	Dry	Curve	62	0.68	0.11	0.54	0.20
			Tangent	54	0.70	0.13	0.53	0.08
		Wet	Curve	56	0.61	0.06	0.45	0.04
			Tangent	50	0.63	0.06	0.49	0.04
	Yes	Dry	Curve	48	0.73	0.18	0.54	0.11
			Tangent	40	0.76	0.18	0.57	0.12
		Wet	Curve	51	0.68	0.11	0.51	0.09
			Tangent	49	0.71	0.09	0.55	0.08
Study 3	No	Dry	Curve	38	0.66	0.14	0.53	0.11
			Tangent	38	0.68	0.14	0.54	0.11
		Wet	Curve	38	0.58	0.13	0.42	0.06
			Tangent	43	0.63	0.09	0.45	0.06

tion to an *expected* object was lower than maximum deceleration to an *unexpected* object. In Study 2 for example, the mean *Max* G_x for drivers in the TTI vehicle without ABS was 0.63 g (20.3 ft/s^2) in response to the *expected* object scenarios and 0.91 g (29.3 ft/s^2) for the *unexpected* object scenario. In Study 3, the mean *Max* G_x for drivers on dry pavements was 0.68 g (21.9 ft/s^2) in response to the *expected* object scenarios and 0.74 g (23.8 ft/s^2) for the *unexpected* object scenarios.

Antilock brakes also had a significant effect on the maximum peak longitudinal deceleration achieved during the *expected* object braking maneuvers, i.e., shorter braking distances translate to higher decelerations. In Study 2, the mean *Max* G_x for drivers in the TTI vehicle on wet pavements was 0.71 g (22.9 ft/s^2) with ABS and 0.63 g (20.3 ft/s^2) without ABS. In Study 3, the *Max* G_x for drivers in their own vehicles (without ABS) on wet pavement also was 0.63 g (20.3 ft/s^2). A summary of the observed *Max* G_x to the *expected* object scenario is presented in Table 26.

As mentioned, the equivalent constant deceleration in response to the *expected* object scenario was calculated for each braking maneuver in the data set. These values are shown in Table 26. A comparison of the constant equivalent deceleration percentile values with the current AASHTO model is shown in Table 27. Note that the 99th percentile controlled deceleration on a wet pavement with and without ABS is close to the deceleration in an AASHTO braking maneuver, a maneuver that assumes locked-wheel braking on a poor, wet pavement.

The equivalent constant decelerations of 0.49 g (15.8 ft/s^2) and 0.45 g (14.5 ft/s^2) for Studies 2 and 3 indicate that the test pavement had higher wet frictional capabilities than anticipated from the skid trailer results. The average *Max* G_x, 0.63 g (20.3 ft/s^2) in Studies 2 and 3, also was higher than anticipated for wet pavement braking maneuvers. The maximum deceleration was usually achieved near the end of the braking maneuver when the friction capabilities of the pavement increased as the vehicle slowed to a stop. The vehicle's tires also were a factor in what level of deceleration could be

TABLE 27 Percentile estimates of equivalent constant deceleration to an expected object

	Equivalent Constant Deceleration (g)		
	Study 2 ABS	Study 2 No ABS	Study 3 No ABS
Mean	0.55	0.49	0.45
75th	0.46	0.44	0.36
90th	0.40	0.41	0.31
95th	0.37	0.39	0.27
99th	0.30	0.35	0.21
AASHTO*	0.30	0.30	0.30

*Assumed Wet Pavement Friction Coefficient at 55 mph achieved. Good tires can provide frictional capabilities higher than the measured skid number of the pavement, especially toward the end of the braking maneuver.

The ANOVA indicated significant differences in *Max* G_x between the *unexpected* and *expected* object scenarios. For the *expected* object scenarios, test subjects driving the TTI vehicle chose higher levels of deceleration than when they were driving their own vehicles. The ANOVA also indicated significant differences in *Max* G_x and *Equivalent Constant* G_x between wet and dry pavement conditions. As expected, the average *Max* G_x was higher on dry pavements, primarily because dry pavements provide higher frictional qualities than wet pavements. Finally, the ANOVA indicated significant differences in *Max* G_x and *Equivalent Constant* G_x with and without ABS. Higher decelerations were exhibited with ABS enabled. Thus, not only does ABS provide improved vehicle control, it also results in improved use of the friction capabilities of the tire-pavement surface.

Summaries of Studies 2, 3, and 4. An mean perception brake-reaction time to an *unexpected* object scenario under controlled and open-road conditions is about 1.10 sec. The 95th percentile perception-brake reaction times for these same conditions was 2.0 sec. The findings from the braking performance studies are consistent with those in the literature which state that almost all drivers are capable of responding to an unexpected hazard in 2.0 sec or less. With regard to observed braking distances, ABS resulted in shorter braking distances by a much as 100 ft at 55 mph. These differences were most noticeable on wet pavements where ABS resulted in better control and shorter braking distances. Braking distances on horizontal curves were slightly longer than they were on tangent sections, however, they were not large enough to be of practical significance.

Deceleration profiles for a sudden stop are not linear. Rather, they resemble a step input with higher-order components. The maximum deceleration during braking is independent of initial velocity, at least in the range of speeds tested (40 to 70 mph). Differences were noted in individual driver performance levels in terms of resultant maximum deceleration. Although the maximum deceleration was equal to the pavement's coefficient of friction for some drivers, the mean *Max* G_x was about 75 percent of that level. Overall, drivers generated maximum decelerations from 0.7 g (22.5 ft/s^2) to 0.9 g (30.0 ft/s^2). The equivalent constant deceleration also varied between drivers. Based on the 55 mph data, 95 percent of all drivers without ABS chose equivalent constant decelerations of at least 0.29 g (9.3 ft/s^2) under wet conditions, and 95 percent of all drivers with ABS chose equivalent constant decelerations of at least 0.41 g (13.2 ft/s^2) on dry pavements.

DRIVER VISUAL CAPABILITIES

Driver visual capabilities related to stopping sight distance are important in that they are generally the limiting condition in object detection and recognition at night and at very long stopping distances. The following sections summarize the literature related to driver visual capabilities and the results of two closed-roadway studies that quantified these capabilities (*46*). Appendix D contains additional information regarding the driver visibility field studies.

Driver Visibility Limits

Throughout the history of the AASHTO sight distance model, it appears no one has ever determined if a driver can actually see an object at the specified distance. Hall and Turner (*9*) expressed concern for the driver's ability to see a 6-in. object at the required stopping sight distance on rural high-speed highways. They determined that at a distance of 600 ft, a driver with 20/40 static visual acuity is required to see an object that is much smaller than the driver's vision allows, that is, at 600 ft, a 6-in. object would have to be 3.5 times larger (21 in.) to be visible to the driver. Thus, perception or recognition of small objects may be beyond the driver's visual capability at long sight distances.

Because perception and recognition are two different aspects of vision, Hall and Turner (*9*) also questioned the assumption that the driver only needs to see the top of the object to recognize, react, and stop for a hazard. They calculated that in the distance and time that it takes for the entire 6-in. object to become visible at 60 mph, the available stopping sight distance has decreased by 165 ft. Hall and Turner even questioned whether height is enough to describe the critical obstacle that should be visible to the driver.

Object Visibility Limits. American and British design guides imply that the moment the top of the object comes into view it is visible to the driver; however, other visibility factors must be considered before the object can be assumed to be visible: luminance contrast, color contrast, ambient luminance level, and glare. Hills (*47*) offered two conclusions concerning the visibility of the object. First, it is the portion of the object above the specified object height that the driver responds to, called the *object cut off height*, and second, objects of the same height but different sizes and contrasts are not equally visible. For example, a vehicle rooftop would be more visible than a child because of its greater surface area and contrast. For these reasons, Hills stated that the line of sight should not be equated with visibility.

The Swedish Design Standards (*48*) address the problem of object recognition by specifying an obstruction height and a visibility angle. The obstruction height of 200 mm (8 in.) is the perpendicular distance from the top of an obstruction to the roadway surface. "The obstruction must be visible to a normal eye" (*48*). Under bright light conditions, 1 min of arc is the minimum angle that part of the obstruction must cover to allow a driver with 20/20 static visual acuity to perceive it as an object if he or she is looking for it.

The visible portion of the obstruction at 1 min of arc is subtracted from the obstruction height to obtain the effective object height. The portion of the obstruction that must be visible on a crest vertical curve is dependent on the distance to the object and the speed of the vehicle. Table 28 shows the speed, sight distance, visible portion, and effective height of the object. The effective height of the object is close to AASHTO's 6-in. object; however, Sweden first presumed that the driver would detect a portion of the object before recognizing and reacting to it.

McLean (*49*) identified drivers' visual limitations within the stopping sight distance model in an evaluation of the 1980 National Association of Australian State Road Authorities (NAASRA) Geometric Design Guide. McLean stated that an observer could resolve detail under ideal lighting and contrast conditions when an object spanned 1 min of arc. Considering the atmospheric and environmental conditions that a driver encounters on the roadway, 5 min of arc would be necessary to perceive an object on the roadway surface.

Using these conclusions, 100 mm of an object must be above the line of sight to detect it at a distance of 65 m. At a distance of 130 m, the object must have 200 mm above the line of sight to be detected. "For distances greater than 130 m, it is likely that the design object would not be seen even with completely clear sight distance (*49*)." McLean hypothesized that required stopping sight distances for speeds above 90 km/h were beyond the visual capability of the driver to detect the hazard.

Object Visibility Studies. Ketvirtis studied the major factors that contributed to detecting a hazardous object at a safe stopping sight distance (*50*). Four separate topics were considered in his study: the visual capability of the drivers, the characteristics of the object, the quality of light acting as an intermediary, and the qualities of the pavement. Ketvirtis first attempted to define the critical object that would be visible at the safe stopping sight distance. The psychological impact and physical reality of the objects were also studied. A critical object height of 7 in. was chosen because that is the average undercarriage clearance of a vehicle. The visible surface area of the object was considered to be significant, but the object weight and type of material were not significant because they were indeterminable at the required stopping sight distance. The width also was not considered critical because objects located within 2 ft of either edge of the lane could be avoided if they were narrower than 0.5 m.

The drivers' reactions and object detection distances were tested during both day and night. This study included the use of 12 objects ranging in size from 60 by 90 by 270 mm to 610 by 200 by 150 mm. A video was placed along the roadway at the site where objects were encountered to observe drivers' responses. Test subjects were asked at what distance they would make a decision to react to an object in the roadway and how they would rate the object as a potential hazard. When given the option to stop, go around, or pass over, very few subjects chose the full-stop option. Most subjects chose to go around the 150 mm object and almost all chose to pass over the 100 mm object; however, the subjects were instructed that the traffic volume was very low, which allowed the option to go around the object safely (*50*).

When the objects were rated from the film, 100-mm objects were rated as a minor hazard and 200-mm objects were rated as a moderate hazard. An automobile muffler with a height of 150 mm was rated as a hazard probably because of its 610 mm length. The two animals used as objects had lower hazard ratings than inanimate objects of the same height. Ketvirtis presented three conclusions from his first two studies:

1. The degree of hazard is often overrated, especially at night;
2. An object of 100 to 120 mm may precipitate a response of moderate degree (e.g., initiate the use of brakes or a change of lane); and
3. The actual smallest size to constitute a perceived physical hazard is approximately 200 mm (approximately 8 in.) in height.

The second part of Ketvirtis' study attempted to establish the visibility distance of objects under different luminance levels. The distances were based upon detection only, and three objects from the previous study were used for the follow-up study. The results showed that a luminance level of 1.0 cd/m^2 was needed to see a high contrast object at approximately 160 m (525 ft) and a speed of 100 km/h. Luminance above 1.2 cd/m^2 added little to the visibility dis-

TABLE 28 Effective height of object with 200 mm visible height—Sweden

Height	Speed, km/h (mph)			
	50 (31)	70 (44)	90 (56)	110 (68)
Sight Distance, m (ft)	70 (230)	120 (394)	165 (541)	195 (640)
Visible Portion, mm (in.)	20 (0.08)	35 (1.37)	50 (1.89)	55 (2.23)
Effective Height, mm (in.)	180 (7.07)	165 (6.50)	150 (5.98)	145 (5.64)

tance. Driver reactions, however, suggested that at 130 m the drivers could react and make required adjustments in their driving task to avoid a hazard, with a minimum luminance of 0.8 cd/m^2.

Headlight Visibility Limits. Basic performance requirements that all headlamps are designed to meet are contained in *Federal Motor Vehicle Safety Standard 108* (*51*) and Society of Automotive Engineers (SAE) J579 (*52*). These requirements apply to all vehicles registered in the United States, regardless of the design of the headlamp filament or light source. Two-headlamp systems use Type 2 and Type 2A (rectangular) headlamp units, both of which must meet the same requirements. A four-headlamp system uses two Type 1 or 1A and two Type 2 or 2A headlamp units. Two- and four-headlamp systems have the following performance ranges:

Type 2 or 2A Sealed Beam
  Upper Beam (Each lamp): 20,000 to 75,000 candela
  Lower Beam (Each lamp): 15,000 to 20,000 candela.
Type 1 or 1A Sealed Beam
  Upper Beam (Each lamp): 18,000 to 60,000 candela.

These illumination levels are the *hot spot* levels for each type of lamp, and they decrease rapidly from these maximum values as the beam pattern diverges from the nominal *hot spot*. Because these headlamp arrays provide an extended illuminated field with either two or four *hot spots*, the level to which a given object will be illuminated can be estimated to a first order approximation by considering a single headlamp. Research by Bhise (*53*) at Ford Motor Company suggests that headlight illumination levels encountered in highway situations vary by as much as a factor of two. Low voltages and the use of many accessories decrease illumination levels. High charging rates and overvoltages increase illumination levels, but to the detriment of lamp life.

Nominally, high beams are aimed to provide the *hot spot* in a direction parallel to a level highway surface with the vehicle unloaded. Low beams are aimed to place the *hot spot* several hundred feet ahead of the vehicle and at a location somewhat to the right of directly ahead, although the pattern is more complex than this description. The principal hot spot for low beam headlamps is specifically aimed 0.5 degrees below the straight ahead (horizontal/vertical reference point) and 1.5 degrees to the right. At a nominal 600 mm height above the pavement (the minimum headlamp height), the low beam hits the pavement at a distance of 70 m, with a deviation of 1.8 m to the right of straight ahead. Small variations in mounting height can result in considerable differences in the distance at which the *hot spot* hits the pavement. For example, a 125 mm difference in mounting height results in 15 m change in where the *hot spot* hits the pavement.

Driver Visual Capability Field Studies

To quantify driver visual capabilities in object detection and recognition, this research incorporated two different, but similar, field studies. The study design measured drivers' capabilities in detecting and recognizing different-sized objects under different lighting conditions. Study 1 measured driver visual capabilities during daylight conditions. Six objects of various sizes were evaluated in this study. Study 2 measured driver visual capabilities under nighttime conditions. Seven different objects of various sizes were evaluated in this study.

Study 1— Daylight Visual Capability Study. Study 1 was conducted on a closed-course test track and involved a range of driver ages and a variety of 45 different objects. Subjects were asked to drive the course as if it were a rural two-lane highway and indicate when an object was first detected by saying "now" or in some way indicating that there was something in the road that should not be there. After detecting the object, the subject was to identify the object when it was recognizable. The study was conducted during the daytime between the hours of 10:00 a.m. and 5:00 p.m. to take advantage of maximum lighting conditions. In addition, objects were placed on the tangent section of the course in an effort to represent a best-case scenario. Any conditions other than these would probably result in shorter object detection and recognition distances.

Each subject drove the course several times and encountered a different object on each run. Six different-sized objects, from 100 to 450 mm in height and with varying contrasts, were used for Study 1. Four different presentation orders were used to minimize possible learning effects. Two 150 mm objects of different contrasts were tested because of their similarity to AASHTO's 150-mm object height used for determining required stopping sight distances. The objects and their heights used in Study 1 were a 100 mm piece of wood, a 150 mm stuffed black dog, a 150 mm stuffed white dog, a 200 mm tire tread, a 300 mm tree limb, and a 450 mm hay bale. The pavement was portland cement concrete so dark objects provided a high contrast and light objects provided a low contrast.

Distance, time, and velocity were recorded at points of object detection and recognition with the aid of a distance-measuring instrument (DMI) that had been installed in the test vehicle. The distance to the object was then calculated to determine detection and recognition distances for each object in the study. The subject's braking and steering behaviors were also recorded. Differences in driver behavior allowed a comparison between animate and inanimate objects, different-sized objects, and the object's perceived hazard.

Study 2—Nighttime Visual Capability Study. Study 2 was also conducted on a closed-course test track and involved

20 subjects and seven objects of different size and contrast. Ten of the subjects were *younger* drivers (25 years old or younger) and 10 of the subjects were *older* drivers (55 years old or older). Subjects were asked to indicate when an object was first detected by saying "now" in some way indicating that there was something in the road that should not be there. After detecting the object, the subject was to identify the object when it was recognizable to them. The study was conducted during the nighttime between the hours of 9:00 p.m. and 12:00 midnight to ensure total darkness. Each object was placed only on the tangent section of the course to represent a best-case scenario under these lighting conditions.

For each test run, zero, one or two objects were placed at predetermined positions on the tangent sections of the course. Target detection and recognition distances were easily calculated by subtracting the distance from the starting point to the subject's detection and recognition response from the known distance from the starting point to the target object. Each object was placed so that it extended 1.0 m into the travel lane as measured from the roadway centerline. The seven objects used in this study provided a variety of contrasts under nighttime lighting and the pavement against which they were viewed, encompassed a range of physical sizes, and represented objects frequently encountered on rural roadways. The objects used in Study 2 were a side view of a passenger car, a rear view of a passenger car, a rear view of a motorcycle, a 0.5 m traffic cone, a 1.0 m stuffed deer, a 0.2 m tire tread, and a 1.8 m mannequin dressed in dark clothing.

Results. Object detection and recognition distances for the 45 subjects in the daylight visibility studies are summarized in Table 29. The 15th and 50th percentile values for detection and recognition of the six objects are presented in the table. As expected, the large objects and high contrast objects were detected at longer distances than the small objects and the low contrast objects. Note that, except for the 100 mm piece of wood and the 150 mm white dog, the 15th percentile values for object detection were at or above 131 m (minimum stopping sight distance for 90 km/h); however, the 15th percentile recognition distances were all below 131 m. There were no statistically significant differences between the younger and older driver groups.

Object detection and recognition distances for the 20 subjects in the nighttime visibility studies are summarized in Figures 11 and 12. The 15th and 50th percentile values for detection and recognition of the seven objects under low- and high-beam headlight conditions are illustrated in the figures. Under low-beam illumination, very few drivers could detect or recognize roadway hazards at the minimum stopping sight distance required on rural high-speed highways. The only objects for which the average detection distance exceeded the 131 m minimum stopping sight distance were the side and rear views of the passenger car. The only object that was correctly recognized from more than 131 m was the rear view of the passenger car. The objects with the longest 15th percentile detection distance (116 m) and recognition distances (78 m) were the side view of the passenger car and the rear view of the passenger car, respectively.

As expected, objects were detected and recognized from substantially greater distances under high-beam illumination. Note that the mean detection distance exceeded 131 m for all objects except the tire tread and the traffic cone; however, only the deer and the side and rear views of the passenger car resulted in mean recognition distances greater than 131 m. The passenger car, from both perspectives, was the only object for which the 15th percentile detection and recognition distances exceeded 131 m.

Summary. The findings from the daytime visual capability studies indicate that drivers on level roadways can *detect* a high contrast 150 mm object at or beyond the AASHTO minimum stopping sight distances for a driver traveling at 90 km/h (131 m). The same can be said for objects that are greater than 300 mm in height, regardless of contrast. The findings also indicate that drivers do not have the visual capabilities to *recognize* objects that are less than 300 mm in height at or beyond the minimum stopping sight distances. Recognition is not totally necessary for stopping sight distance, but the driver must be able to recognize the object as a hazard.

TABLE 29 Daylight object detection and recognition distances (m)

	15th Percentile		50th Percentile	
Object	Detect	Recognize	Detect	Recognize
2-25 by 100 cm boards	0	0	113	44
Black Dog	180	5	277	39
White Dog	70	1	213	30
Tire Tread	272	40	333	155
Tree Limb	153	22	218	81
Bale of Hay	254	41	371	169

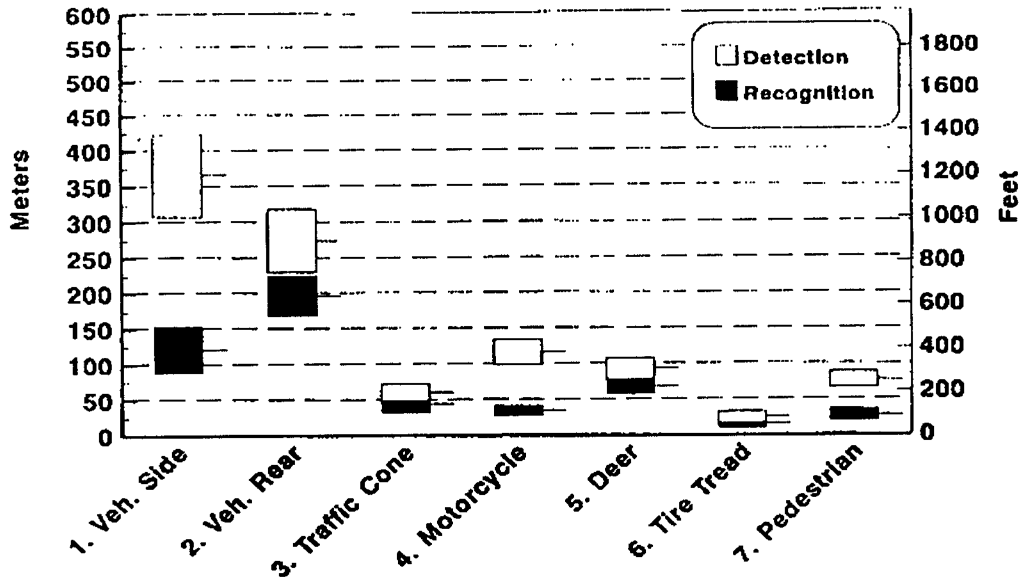


Figure 11. Low-beam detection and recognition distances for all subjects.

The findings from the nighttime visual capability studies suggest that a substantial proportion of the driving population are *not* able to detect or recognize hazardous objects in the roadway at the AASHTO minimum stopping sight distance for a driver traveling at 90 km/h (131 m). The only exception to this statement is when the object is externally illuminated or retro reflective, that is, has vehicle taillights or side reflectors. Detection, and more especially recognition of potentially hazardous objects at 131 m distances, is even more unlikely when low-beam headlights are in use.

DRIVER EYE AND VEHICLE HEIGHTS

Driver eye heights and vehicle dimensions are important parameters in the determination of vertical curve lengths. The following sections summarize the literature related to these two parameters and the results of several field studies that quantified these parameters (*46*). Appendix E contains additional information regarding driver eye and vehicle heights for use in highway geometric design.

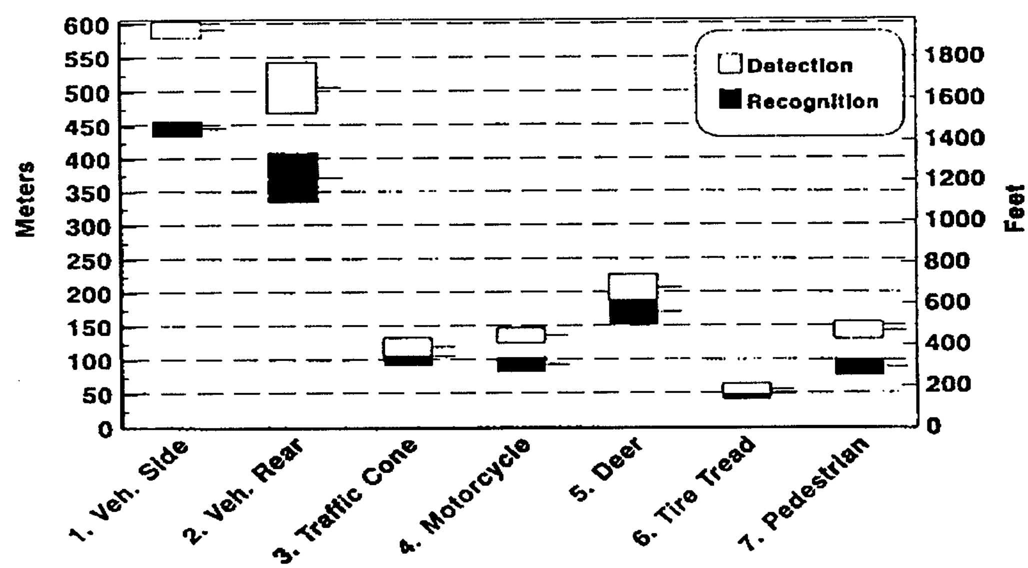


Figure 12. High-beam detection and recognition distances for all subjects.

Background

In the 1920s, the assumed driver eye height for design was 5.5 ft (1,676 mm), and with changes in the vehicle fleet over the past 70 years, the assumed driver eye height for design has decreased to 1,070 mm (*1*), with some studies recommending an even lower value. The decrease in driver eye height over the years can be attributed to changes in vehicle design, most recently in the 1970s and early 1980s when fuel economy became an important issue and vehicle manufacturers responded to the challenge with smaller and more compact vehicles.

In addition to driver eye height, object and headlight heights are important elements in the procedure for determining horizontal and vertical curve lengths that provide required stopping sight distances. Object height, like driver eye height, has varied significantly since its inception in the 1920s, when its value was suggested as 1,676 mm (5.5 ft), or the same as driver eye height. The object height used in AASHTO's current design standards is 150 mm (*1*). Some studies have suggested lower values so that drivers can pass over the object in the road without damaging their vehicles, and other studies have suggested higher values because drivers cannot see small objects at the required stopping sight distances for high speeds. The height of a vehicle's taillight has been suggested as an alternative object height.

Analytical Eye Height Studies. One of the earliest and most comprehensive efforts at quantifying driver eye height was done by General Motors. Driver eye heights had been calculated for each model year vehicle from 1936 to 1957 at the General Motors Proving Ground, and a summary of driver eye height for each vehicle and model year was reported by Stonex (*54*) in 1957. Driver eye heights were determined based on the average vehicle seat cushion height and the average seated eye height of a group of males. This latter value was found to be approximately 725 mm. In addition, it was found that the average 1936 vehicle seat cushion depressed 50 mm.

Based on these values, it was determined that the mean driver eye height for each model year fell from 4.75 ft (1,445 mm) in 1936 to 4.25 ft (1,295 mm) in 1957. Additional investigation by Stone revealed that the average 1957 seat cushion depressed approximately 100 mm, further reducing the mean driver eye height. Stone concluded his study with a prediction that the mean driver eye height would not fall much below 1,092 mm (43 in.) based on future vehicle height estimates and an assumed minimum vertical clearance between the driver's eye and the top of the vehicle.

NCHRP Report 270 (*8*) recommended a design value of 3.33 ft (1,016 mm) for driver eye height based on an analytical approach, rather than experimental data. The analytical approach, in simplistic terms, involved determining the distance from the ground to a *seating reference point* (a point defined by the driver's seated position) and from the seating reference point to the driver's eye. This latter distance was obtained by using a method recommended by Hammond (*55*).

In Hammond's method, a vertical and horizontal component of eye height is needed because a driver's eye height is dependent on both the height of the driver and the position in which that person sits. Plotting a multitude of these points creates a cluster of points in the general shape of an ellipse, hence the name *eyellipse*. The lowest percentile eyellipse value would be represented by a tangent at the lowest point of the eyellipse, whereas the 50th percentile value would be represented by the midpoint of the eyellipse. Other percentiles are determined in the same manner and are illustrated in Figure 13.

A cumulative distribution for the distance from the ground to the seating reference point was obtained from all foreign and domestic vehicle models that comprised a measurable volume of the 1981 U.S. sales market. The two distributions for distance from the seating reference point to the eyellipse and ground were combined, *assuming independence and normality*, to obtain a distribution of driver eye heights. The study concluded that almost 25 percent of the driver eye heights were below the current 3.5-ft (1,067-mm) standard

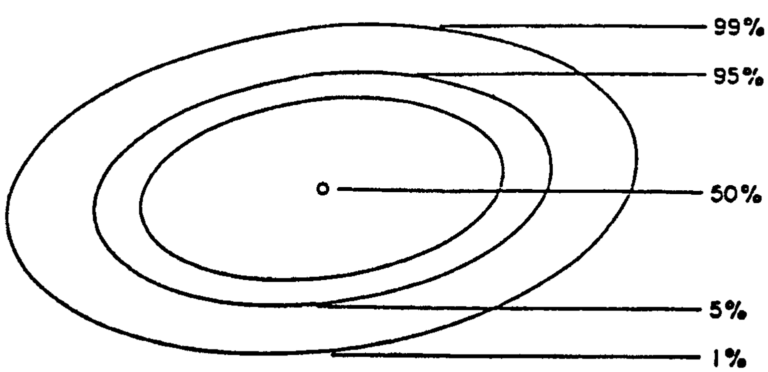


Figure 13. Percentile tangent lines of an eyellipse. (55)

and recommended that driver eye height be reduced to 40 in. (1,016 mm) to encompass 95 percent of the driving population, that is, 95 percent of the driver eye height eyellipses were above 1,016 mm (40 in.).

Empirical Eye Height Studies. Empirical driver eye height studies generally involve photography of moving vehicles such that both the driver's profile and the vehicle are captured on film. Reference markers are placed on the pavement in the camera's field of view to convert scaled values from the photo to their corresponding actual values. A summary of the empirical driver eye height studies since 1957 is illustrated in Table 30. Note that the empirical studies concluded that 85 percent of the driver eye heights exceed 1,067 mm. In contrast, the analytical methodology using the eyellipse concluded that only 75 percent of the driver eye heights exceed 1,067 mm; however, the analytical procedure assumes independence between the seat height and seated eye height distributions, an assumption that may or may not be true. Shorter drivers may favor larger vehicles over smaller vehicles.

Headlight and Taillight Studies. *Standard 108* of the *Federal Motor Vehicle Standards* (*51*) provides requirements for lighting equipment and its placement on motor vehicles. Taillights and headlights for all motor vehicles must be located on either side of the vertical centerline of the vehicle and as far apart as *practicable*. Headlights may be no lower than 22 in. (559 mm) and no higher than 54 in. (1,370 mm). Both the upper and lower beam lamps should be at the same height. It should be noted that the minimum headlight height requirements were reduced from 24 in. (610 mm) to 22 in. (559 mm) in the 1980s. Taillights are required for trailers as well as motor vehicles. Taillights must be mounted no lower than 15 in. (381 mm) and no higher than 72 in. (1,829 mm). The point of reference is to be taken as the height above the road surface measured from the center of the item on the vehicle.

Driver Eye and Vehicle Height Field Studies

To quantify the driver eye height and vehicle dimensions associated with the current vehicle fleet, several field studies were undertaken. The methodology described in the following sections documents the data collection, data reduction, and statistical analysis procedures that were followed in these studies. Driver eye, headlight, taillight, and vehicle height data were collected in the field during times of normal travel. Data were separated into new vehicles and older vehicles in an effort to determine whether the newer vehicle fleet is continuing a downward trend for driver eye and vehicle heights.

Data Collection and Reduction Procedures. Three different data collection schemes were developed and used for collecting dynamic vehicle data, heavy vehicle data, and new vehicle fleet data. Initially, video records of the dynamic vehicle data were to provide the driver eye height data for all vehicles in the current vehicle fleet; however, it was not possible to obtain accurate driver eye and vehicle height data for both passenger cars and heavy vehicles from the same video tape because of their large differences in height. The camera had to be level and mounted at about the same height as the object being measured for accurate measurements. Thus, passenger car or heavy vehicle data could be obtained from a single video, but not both.

To obtain driver eye and vehicle heights that were representative of national conditions, data were collected from several different geographic regions. This approach allowed comparisons of different regions to determine whether driver eye heights around the country are similar. For these reasons, data were collected at seven sites in four states: Washington, Illinois, Texas, and Virginia. These states represent the northwest, midwest, southwest, and east regions of the country.

The procedure for collecting the dynamic vehicle data involved videotaping traffic traveling along a roadway so that the vehicle's driver, headlights, taillights, and rooftop could be seen. This process required a two-camera set up, with driver eye height, headlight height, and vehicle height being obtained from one camera and taillight height being obtained from the second camera. Color film was used to maximize contrast. Temporary pavement markers were placed on the roadway surface for use as a scale in data reduction. For calibration purposes during data reduction, reference vehicles with known dimensions were driven through the roadway segment being filmed.

The procedure for collecting the heavy vehicle data involved manually measuring the driver eye height, headlight height, taillight height, and vehicle height of stationary vehicles at a truck weigh station. The procedure involved two data recorders, a surveying rod, and a step ladder. The surveying rod was used to measure the driver's eye and the truck height. The rod was more than 13 ft. in length when extended and was marked in one-tenth of an inch increments. A step ladder was used to place the data collector on the same level as the driver's eye and vehicle heights. Headlight and taillight heights were measured using a meter stick.

The procedure for collecting the 1993 vehicle fleet data involved manually measuring the headlight heights and taillight heights of the vehicle fleet at local dealerships. This procedure involved a data collector and a meter stick. Sales volumes for each domestic and import passenger car or light truck in the 1993 vehicle fleet were obtained from *Automotive News* (*61*). These sales volumes could then be applied as weighting factors to the heights of the headlights and taillights of each vehicle to obtain the cumulative distribution for all vehicles. Local automotive dealerships were visited, and the heights of the top, center, bottom, and bulbs of the headlights and taillights for each vehicle type were measured using the meter stick.

TABLE 30 Summary of driver eye height studies

	Study					
Author	Stonex (*54*)	Lee (*56*)	Boyd, et al. (*57*)	Cunagin & Abrahamson (*58*)	Haslegrave (*59*)	Barker (*60*)
Mean/Median Driver Eye Height Value	1295 mm (4.25 ft)	1289 mm (4.23 ft)	1125 mm (3.69 ft)	n/a	1145 mm (3.76 ft)	1130 mm (3.71 ft)
15th Percentile Driver Eye Height Value	1242 mm (4.07 ft)	1204 mm (3.95 ft)	1064 mm (3.49 ft)	1067 mm (3.50 ft)	1092 mm (3.58 ft)	1070 mm (3.51 ft)
Vehicle Types*	PC	PC	PC	PC, PU	PC, PU, V	PC, PU
Data Points	1957 Domestics	761	195	1478	825	1124
Data Collection Method	@ GM Proving Grounds	Photography	Photography	Photography	Photography	Video
Year of Study	1957	1960	1978	1979	1979	1987
Country	U.S.	U.S.	U.S.	U.S.	U.K.	Australia

* *PC = Passenger Cars* *PU = Pickup Trucks/Sport Utilities* *V = Vans*

Results. Table 31 presents summary statistics for the passenger cars in the dynamic vehicle data set. The driver eye, headlight, and vehicle height results are based on data from four states and the taillight results are based on data from three states. Headlight and taillight heights represent the center of the lights as specified by *Federal Motor Vehicle Standard 108* (*51*). Of the 875 passenger-car driver eye heights in the data base, the 5th percentile value of 1,060 mm was very close to the current AASHTO driver eye height of 1,070 mm. Passenger car driver eye height values ranged from a low of 955 mm to a high of 1,422 mm with 15th and 50th percentile values of 1,094 mm and 1,149 mm, respectively. Percentile values refer to the percentage of the total observation that were below these values.

Of the 1,318 passenger-car headlights in this data base, fewer than 1.0 percent were below the 559 mm minimum height requirement of *Federal Motor Vehicle Standard 108* (*51*); however these few vehicles were all within 20 mm of the standard. None of the headlights measures was above the 1,372 mm requirement. Headlight height values ranged from a low of 531 mm to a high of 947 mm with 5th, 15th, and 50th percentile values of 590 mm, 608 mm, and 649 mm, respectively. Of the 858 passenger-car taillights in the data base, none was lower than the 381 mm requirement or higher than the 1,829 mm requirement in *Federal Motor Vehicle Standard 108* (*51*). Taillight height values ranged from a low of 385 mm to a high of 999 mm with 5th, 15th, and 5th percentile values of 616 mm, 660 mm, and 726 mm, respectively.

The 1,378 passenger-car heights in the data base ranged from a low of 1,156 mm to a high of 1,690 mm with 5th, 15th, and 50th percentile values of 1,282 mm, 1,331 mm, and 1,384 mm, respectively. AASHTO uses a height of 1,300 mm to establish design criteria for passing and intersection sight distances. This value encompasses more than 90 percent of the 1,378 vehicles in this study's data base, that is, more than 90 percent of the vehicles had heights greater than 1,300 mm.

Table 32 presents the summary statistics for the multipurpose vehicles in the dynamic vehicle data set. The multipurpose vehicle category contained pick-up trucks, sport utility vehicles, minivans, and vans, and represented almost 37 percent of the observations in the study's data base. The 5th percentile driver eye height for multipurpose vehicles was 1,264 mm, which was 20 mm greater than the 5th percentile driver eye height for passenger cars. Also, the lowest driver eye height for multipurpose vehicles was approximately equal to the 5th percentile driver eye height for passenger cars.

The lowest values measured for multipurpose vehicle headlight and vehicle heights were also approximately equal to the 5th percentile headlight and vehicle heights for passenger cars. The 5th percentile headlight and vehicle heights for multipurpose vehicles were 691 mm and 1,523 mm, respectively. The taillight heights for the vehicles in the multipurpose vehicle category were also higher than the passenger-car values. The 5th percentile taillight height was 780 mm, which was 160 mm above the 5th percentile passenger-car value.

Table 33 presents summary statistics for the trucks in the heavy vehicle data base. Note that this vehicle category includes only tractor-trailer combination vehicles. The 5th, 15th, and 50th percentile eye heights for the 163 trucks in the data base were 2,304 mm, 2,341 mm, and 2,447 mm, respectively. The lowest tractor-trailer eye height was higher than the highest value for a passenger car. The headlight and taillight height values for tractor-trailers were all within the *Federal Motor Vehicle Standard 108* (*51*) requirements. The highest point on the tractor or trailer was used in the vehicle height measurement. The 5th percentile height was 2,652 mm, and the 85th percentile height was 4,054 mm, which was slightly lower than the maximum legal height for combination trucks (4,115 mm or 13.5 ft).

TABLE 31 Descriptive statistics for passenger cars

Descriptive Statistic*	Driver Eye Height	Headlight Height	Taillight Height	Vehicle Height
Sample Size	875	1,318	858	1,378
Mean	1,149 (3.77)	649 (2.13)	726 (2.38)	1,384 (4.54)
Standard Deviation	55 (0.18)	41 (0.13)	70 (0.23)	59 (0.19)
High Value	1,422 (4.67)	947 (3.11)	999 (3.28)	1,690 (5.54)
Low Value	955 (3.13)	541 (1.77)	385 (1.26)	1,156 (3.79)
Range	467 (1.53)	406 (1.33)	614 (2.01)	534 (1.75)
5th Percentile	1,060 (3.48)	590 (1.94)	616 (2.02)	1,282 (4.21)
10th Percentile	1,082 (3.55)	602 (1.98)	642 (2.11)	1,315 (4.31)
15th Percentile	1,094 (3.59)	608 (1.99)	660 (2.17)	1,331 (4.37)

* Descriptive Statistics presented in millimeters (and ft) where applicable.

TABLE 32 Descriptive statistics for multipurpose vehicles

Descriptive Statistic*	Driver Eye Height	Headlight Height	Taillight Height	Vehicle Height
Sample Size	629	992	534	987
Mean	1,482 (4.86)	842 (2.76)	963 (3.16)	1,759 (5.77)
Standard Deviation	130 (0.43)	95 (0.31)	132 (0.43)	155 (0.51)
High Value	2,034 (6.67)	1,174 (3.85)	1,436 (4.71)	2,501 (8.21)
Low Value	1,053 (3.45)	569 (1.87)	420 (1.38)	1,279 (4.20)
Range	981 (3.22)	605 (1.98)	1,016 (3.33)	1,222 (4.01)
5th Percentile	1,264 (4.15)	691 (2.27)	780 (2.56)	1,523 (5.00)
10th Percentile	1,306 (4.28)	713 (2.34)	818 (2.68)	1,564 (5.13)
15th Percentile	1,331 (4.37)	728 (2.39)	839 (2.75)	1,613 (5.29)

* Descriptive Statistics presented in millimeters (and ft) where applicable.

Table 34 presents summary statistics for the headlight and taillight heights for passenger cars and multipurpose vehicles from the dynamic vehicle and 1993 vehicle data bases. As shown, there is very little difference (approximately 10 mm) between the 5th percentile headlight height for passenger cars and that of the combination of passenger cars and multipurpose vehicles for either the dynamic or new vehicle data. For comparison purposes, 5th percentile taillight height for passenger cars and the combination of passenger cars and multipurpose vehicles differed by less than 20 mm for both the dynamic vehicle data and the 1993 vehicle data. Note that the 5th percentile taillight height for the 1993 vehicle fleet is approximately 10 percent higher than the dynamic vehicle data.

Summary. The field studies quantified driver eye heights, headlight heights, taillight heights, and vehicle heights used in establishing geometric design criteria. The major findings from this effort were that approximately 92 percent of the measured passenger-car driver eye heights in four states exceeded the AASHTO design value of 1,070 mm. Of the 875 passenger-car driver eye heights in the data base, the 5th and 15th percentile driver eye heights were 1,060 mm and 1,094 mm, respectively. The 5th and 15th percentile driver eye heights for the 163 heavy trucks in the data base were 2,304 mm and 2,341 mm, respectively.

Of the 1,318 passenger-car headlight heights in the data base, 10 were below the 559 mm requirements for *Federal Motor Vehicle Standard 108* (*51*). No multipurpose vehicles or heavy trucks had headlight heights below this standard. The 5th and 15th percentile passenger-car headlight heights were 590 mm and 608 mm, respectively. Of the 1,652 taillight heights in the data base, none was below the *Standard 108* (*51*) requirements of 381 mm. Of the 858 passenger-car

TABLE 33 Descriptive Statistics for Heavy Trucks

Descriptive Statistic*	Driver Eye Height	Headlight Height	Taillight Height	Vehicle Height
Sample Size	163	337	260	158
Mean	2447 (8.03)	1121 (3.68)	1058 (3.47)	3590 (11.78)
Standard Deviation	107 (0.35)	88 (0.29)	159 (0.52)	581 (1.91)
High Value	2816 (9.24)	1351 (4.43)	1690 (5.54)	4639 (15.22)
Low Value	2103 (6.90)	915 (3.00)	415 (1.36)	2396 (7.86)
Range	713 (2.34)	436 (1.43)	1275 (4.18)	2243 (7.36)
5th Percentile	2304 (7.56)	972 (3.19)	719 (2.36)	2652 (8.71)
10th Percentile	2329 (7.64)	1008 (3.31)	908 (2.98)	2719 (8.92)
15th Percentile	2341 (7.68)	1022 (3.35)	953 (3.13)	2774 (9.10)
85th Percentile	2560 (8.40)	1220 (4.00)	1185 (3.89)	4054 (13.30)
90th Percentile	2579 (8.46)	1236 (4.06)	1208 (3.96)	4084 (13.40)
95th Percentile	2597 (8.52)	1258 (4.13)	1265 (4.15)	4084 (13.40)

* Descriptive Statistics presented in millimeters (and ft) where applicable.

TABLE 34 **Headlight and Taillight Height Descriptive Statistics**

		Dynamic Vehicle Fleet			1993 Vehicle Fleet		
		PC**	MV	PC & MV	PC	MV	PC & MV
Headlight Statistics*	Mean	649 (2.13)	842 (2.76)	732 (2.40)	634 (2.08)	830 (2.72)	709 (2.33)
	50th Percentile	646 (2.12)	847 (2.78)	688 (2.26)	630 (2.07)	835 (2.74)	660 (2.17)
	15th Percentile	608 (1.99)	728 (2.39)	623 (2.04)	595 (1.95)	760 (2.49)	600 (1.97)
	5th Percentile	590 (1.94)	691 (2.27)	599 (1.97)	580 (1.90)	710 (2.33)	590 (1.94)
Taillight Statistics*	Mean	726 (2.38)	963 (3.16)	817 (2.68)	771 (2.53)	957 (3.14)	842 (2.76)
	50th Percentile	728 (2.39)	949 (3.11)	774 (2.54)	780 (2.56)	955 (3.13)	810 (2.66)
	15th Percentile	660 (2.17)	839 (2.75)	682 (2.24)	715 (2.35)	835 (2.74)	735 (2.41)
	5th Percentile	616 (2.02)	780 (2.56)	632 (2.07)	665 (2.18)	798 (2.62)	678 (2.22)

** Descriptive Statistics presented in millimeters (and ft)*
*** PC = Passenger Cars, MV = Multipurpose Vehicles*

taillight heights in the data base, the 5th and 15th percentiles were 616 mm and 660 mm, respectively.

Of the 1,378 passenger car vehicle heights in the data base, the 5th and 15th percentile values were 1,282 mm and 1,331 mm, respectively. Of the 158 heavy trucks in the data base, 157 had vehicle heights less than or equal to the 4,115 mm (13.5 ft) maximum height value for these vehicles. The 95th percentile vehicle height value for heavy trucks in this study was 4,084 mm.

SAFETY EFFECTS

The relationship between stopping sight distance and safety is of utmost importance in establishing geometric design criteria. The following sections summarize the literature on the relationship between stopping sight distance and safety, and the results of an accident causation study that investigated this relationship (*62*). Appendix F contains additional information regarding accident causation on roadways with limited stopping sight distance.

Limited Sight Distance Accident Studies

Three studies in the literature have examined the relationship between accident frequency and stopping sight distance. The first of these studies is documented in *NCHRP Report 270* (*8*) and is based on a matched pair comparison of accident rates on crest vertical curves in Michigan. The second study is documented in a Texas Transportation Institute (TTI) research report (*63*) that used multiple regression analysis to analyze accident rates and available sight distance at crest vertical curves in Texas. The third study, described in a 1991 issue of *Public Roads* (*64*), used the Highway Safety Information System (HSIS) data base to analyze accident rates at crest vertical curves in Utah.

In the Michigan study (*8*), ten crest vertical curves with limited stopping sight distance (35 to 90 m) were paired with ten nearby curves with adequate stopping sight distance (greater than 200 m). The 20 curves in the study ranged from 0.24 km to 0.80 km in length, but paired curves were of equal length. Paired curves were also matched in terms of traffic volumes, abutting land use, vegetation, road geometry, lane widths, and shoulders. Five-year accident histories were obtained for each of the 20 curves in the study.

When compared with its adjacent control site, the curves with limited sight distance had more accidents at seven of the sites, the same number of accidents at two of the sites, and fewer accidents at one of the sites. Collectively, the ten limited sight distance curves had more than 50 percent more accidents than the corresponding control sites. Thus, it was concluded that limited stopping sight distance resulted in increased accident rates (*8*). It should be noted, however, that nine of the limited sight distance sites had less than 90 m of stopping sight distance. A more accurate conclusion would have been that vertical curves with less than 90 m of stopping sight distance resulted in increased accident rates.

In the Texas study (*63*), accident and roadway data from 222 segments of highway were collected and analyzed. Collectively 1,500 accidents occurred at these locations during a 4-year period. For each highway segment in the Texas study, several descriptive variables were available: traffic volumes, number of intersecting roads influenced by limited sight distance, and the percent of the segment with stopping sight distance below a certain length. This last measure was of primary interest to this study. The Texas study's basic hypothesis was that accident rates were a function of the amount of limited stopping sight distance on the segment; however, when accident frequency and accident rates were regressed on the amount of limited sight distance, the percent of the segment with limited sight distance was not a significant contributor to the regression model. Thus, it was concluded that **in the sight distance ranges studies, limited stopping sight distance had no discernable effect on accident frequency or rate**.

The Utah study used accident data from 2,396 crest vertical curves in the HSIS data base (*64*) to analyze the relationship between vertical curve geometry and safety. Three-year accident histories were merged with each of the crest vertical curve locations in terms of distance from the crest to the reported accident locations. For each curve location, accident totals were determined for each 30 m interval in both directions from the curve's crest. To remove the possibility of adjacent curves influencing the accident experience at the vertical curve of interest, curves with less than 240 m of separation between approach grades were omitted from the data set. After this check was made, the remaining data set contained 1,424 crest vertical curves.

The number of accidents in each distance interval was divided by the number of curve locations to produce the number of accidents per crest curve location. Accident rates as a function of distance from the crest were plotted for curves with grade differences of 1 to 3 percent, 3 to 6 percent, and greater than 6 percent. The results indicated that accident rates are highest near the crest of the vertical curve, then level off to a relatively constant rate between 30 m and 120 m from the crest. Higher accident rates were seen for the curves with grade differentials greater than 6 percent, but the difference is most noticeable in the first 30 m interval from the crest, that is, more accidents occurred within the 60 m interval centered at the middle of the crest vertical, and greater changes in grades are associated with more accidents near the crest of the vertical curve.

Large Truck Accidents. The American Automobile Association (AAA) Foundation for Highway Safety has a data base containing information on 231 large truck accidents that occurred between 1983 and 1984 on interstate highways in six western states (*65*). Limited stopping sight distance was not cited as a contributing factor in any of the 231 accidents. There also was no indication from the factors contributing to the accidents that the roadway's geometry limited the driver's view of the roadway. This result was not unexpected because interstates have relatively few locations with limited stopping sight distance.

Six of the 231 accidents involved objects or animals in the roadway. None of the six object-related accidents resulted in a fatality. One object was not identified, the second object was a parked car, and four objects were animals: two cows, one deer, and one porcupine. Thus, only one of the five objects was a small unexpected object in the roadway. The four accidents involving large objects occurred at night, and the one accident involving a small object occurred during the day; however, in this particular accident, it was raining and the pavement was wet. The truck's driver swerved to miss the porcupine and hit a bridge abutment. The roadway was straight and level in all five object-related accidents, so limited stopping sight distance was not a contributing factor to any of the accidents.

In addition to the AAA data base, all 1990 single-vehicle accidents involving large trucks in Texas were reviewed to ascertain the extent to which limited stopping sight distance might be a contributing factor to large truck accidents. Of the 2,230 single-vehicle truck accidents identified, 14 (0.6 percent) occurred on hill crests (that may or may not have had adequate stopping sight distance), with 8 (0.4 percent) of the 14 accidents occurring during the day when roadway geometry rather than vehicle headlights limit available sight distance. Thus, even if limited sight distance was a contributing factor in all eight daytime accidents, it contributed to a very small number of large truck accidents.

Object-Related Accidents. Kahl (*66*) examined object-related accident reports and narratives from two states to determine the type and size of objects struck in accidents that might be related to stopping sight distance. Three types of accidents were evaluated: other object accidents, animal accidents, and evasive action accidents. The results of this study were as follows:

- Two percent of all reported accidents involved objects or animals on the roadway, however, only 0.07 percent of all reported accidents involved objects or animals less than 150 mm high. Thus, accidents involving small objects are extremely rare events.
- More than 90 percent of the object- and animal-related accidents occurred on straight, level roads where the driver's visibility was not limited by the geometry of the roadway. Thus, the overwhelming majority of other object- and animal-related accidents occur where the driver's visibility is not limited by the roadway's geometry.
- Most of the object- and animal-related accidents occurred at night when the driver's visibility is limited by the vehicle's headlights. Longer crest vertical curves will not necessarily decrease object- and animal-related accidents because headlight sight distances are generally less than minimum required stopping sight distances.
- More than 95 percent of the object- and animal-related accidents resulted in low severity accidents. Small objects do not represent a hazard to most drivers.

Summary. The previous discussion pointed out the difficulty in quantifying the relationship between safety and stopping sight distance. The conclusion from the Michigan study was that limited stopping sight distance creates safety problems, and the conclusion from the Texas study was that limited stopping sight distance did not create safety problems. At first glance, these results appear contradictory and confusing; however, collectively, they point out that it is the degree of deficiency that produces safety problems. In other words, if the available sight distance is marginally less than the AASHTO recommended value, it had no effect on accident rates; however, once the available sight distance was less than some *threshold value*, it did have an effect on accident rates.

In the Michigan study, Olsen et al. (*8*) concluded that vertical curves with stopping sight distances less than 90 m had a higher number of accidents than vertical curves with very long stopping sight distances. The Michigan data also suggested that the largest increase in accidents occurred at the study sites which had the shortest stopping sight distances. The Texas study (*63*) concluded that stopping sight distances in the range from 100 to 135 m had no discernable effect on accident rates unless there was an intersection within the limited sight distance segment. This finding suggests there are no safety benefits from providing additional stopping sight distance beyond 90 m unless there is a hazard within the limited sight distance section. It also is consistent with Glennon's conclusion that alignment changes are cost-effective only on high-traffic-volume highways with major hazards (such as intersections and sharp curves) within the limited sight distance section (*67*).

Conceptually, the relationship between stopping sight distance and safety at crest vertical curves can be illustrated as shown in Figure 14. Accident rates are high for short sight distances and relatively insensitive to sight distance beyond some *threshold value(s)*. That is, there is a wide range of sight distances that satisfies driver needs for safety and has similar accident rates. Additional hazards within the limited sight distance section may affect this threshold value by moving it farther to the left (lower values). The challenge is defining the point or conditions at which accident rates begin to increase.

Accident Causation Field Studies

Past studies that examined the relationship between stopping sight distance and safety have been inconclusive and inconsistent; however, the fact that a relationship has not been identified does not mean that stopping sight distance is not a contributing factor in some accidents. Instead, it means that any relationship that might exist has not been quantified with existing data bases. The most promising approach to quantify such a relationship appears to be a detailed examination or case study of accidents from a relatively large sample of limited sight distance roadways. If limited sight distance is a contributing factor to accidents, it should show up in such a study.

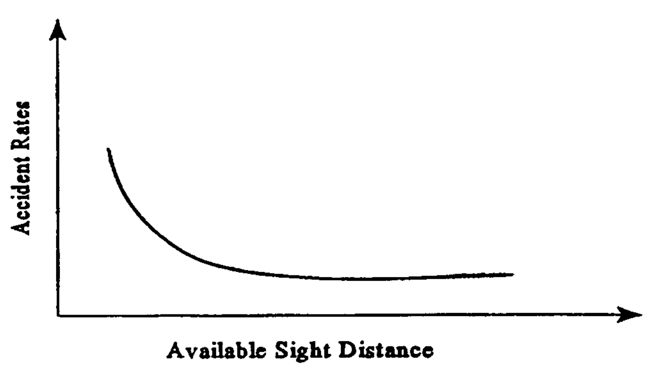


Figure 14. Conceptual relationship between available sight distance and safety at crest vertical curves.

Case Study Approach. The approach selected for this study was an in-depth diagnostic case study of the accidents that occurred on a sample of roadways containing limited sight distance crest vertical curves. The case study approach involves analyzing accident narratives for selected sites to identify patterns or common characteristics that might be related to limited sight distance. The narrative descriptions and the geometric characteristics of the sites were used to identify accidents in which limited stopping sight distance was a possible contributor to the accident.

Potential study sites were identified in three regions of the country (Washington, Texas, and Illinois) to represent a wide range of geographic conditions. These states were selected because they contain rolling terrain where limited sight distance crest vertical curves are generally found, and they have sufficient roadway geometry data bases to identify limited sight distance curves. For example, Texas and Washington have computerized roadway geometry and accident data bases, and the Illinois roadway geometry and accident data are contained in the HSIS data base.

After a list of potential sites was developed, visits were made to each site to verify that the field conditions were the same as the conditions in the roadway geometry data bases. The site visits provided an opportunity to collect information that was not available from other sources. For example, potential accident sites were videotaped and photographed to provide a permanent record of the roadway geometry and site-specific characteristics. The widths of the travel lanes, shoulders, and clear zones were documented. The number and location of driveways, intersections, median openings, and bridges were also recorded.

The 37 study sites—by location and roadway type—are shown in Table 35. Traffic and geometry data for the 33 nonfreeway sites are shown in Table 36. It should be noted that for this study *with shoulder* sites were those with paved shoulders 1.8 m in width or greater, while *without shoulder* sites were those with unpaved or narrow (less than 1.8 m in width) paved shoulders.

Results. The results of the accident studies are subdivided into three major areas: characteristics of limited sight distance sites, comparison between different groups of limited sight distance sites, and examination of individual accidents. *Accident Characteristics* provides comparisons of

TABLE 35 Number of sites selected for accident studies

Type of Roadway	State			Total
	Texas	Washington	Illinois	
Freeway	0	4	0	4
Multilane	2	0	0	2
Two-Lane with Shoulders	3	3	1	7
Two-Lane without Shoulders	9	10	5	24
Total	**14**	**17**	**6**	**37**

these sites with all rural sites. A summary of the characteristics of limited sight distance sites is also given showing the percentage of younger and older driver accidents, the percentage of tractor-trailer accidents, and the percentage of accidents occurring during daylight conditions. *Accident Comparisons* describe several different groups such as design speed and roadway type used to investigate differences in accident rates between the limited stopping sight distance sites in this study. *Accident Examination* describes the results of the case study of accidents that might have been influenced by the presence of a limited sight distance curve.

Accident Characteristics. Because of the large number of freeway accidents (826), an in-depth analysis of contributing factors was not done. To evaluate the characteristics of the freeway accidents, one-line accident summaries were requested from Washington state. These summaries contained computer codes for several of the accident characteristics, including pavement condition and type of accident. Most of the freeway accidents (545 of 826) occurred during the day. The most frequent factor contributing to accidents was snow, ice, or water on the pavement. More than 70 percent of the accidents at five of the seven study sites involved snow, ice, or wet pavement. Approximately one-third of the accidents involved multiple vehicles while another one-third of the accidents involved single vehicles striking a roadside appurtenance. Two of the six accidents on the WA-A04 site involved snow or ice on the pavement.

The multilane highway sites had lower accident rates than the interstate sites but had higher accident rates than the two-lane sites. As shown in Table 37, the two multilane sites had an average of 0.86 accidents per million vehicle miles and 3.60 accidents per year per mile. The accidents per million vehicle miles for the two sites were 0.57 and 1.16, with the primary difference being the number of wet pavement-related accidents at one of the sites. The percentage of intersection or driveway-related accidents, large truck-related accidents, and wet pavement-related accidents were 16, 8, and 20 percent, respectively. As noted, most of the wet pavement-related accidents occurred at one site. Approximately 36 percent of the accidents at the two sites occurred at night.

The two-lane highway *with shoulder* sites had slightly higher accident rates than the multilane highway sites, but had lower accident rates than the two-lane highway *without shoulder* sites. As shown in Table 38, the seven two-lane *with shoulder* sites had an average of 0.97 accidents per million vehicle miles and 1.37 accidents per year per mile. The accidents per million vehicle miles ranged from 0.48 to 1.73, with the higher rates associated with the sites having the highest number of intersection and wet pavement-related accidents. The percentage of intersection- or driveway-related accidents, large truck-related accidents, and wet pavement-related accidents were 25, 2.5, and 10 percent, respectively. Most of the intersection- or driveway-related accidents and all of the wet pavement-related accidents occurred at two of the seven sites. Approximately 54 percent of the accidents on the two-lane highway *with shoulder* roadways occurred at night (44 of the 81 accidents).

As expected, the two-lane *without shoulder* sites had higher accident rates than the multilane and two-lane *with shoulder* sites. As shown in Table 39, the 24 two-lane *without shoulder* sites had an average of 1.68 accidents per million vehicle miles and 1.71 accidents per year per mile. With one exception, the accidents per million vehicle miles ranged from 0.39 to 2.50. The number of accidents per year per mile ranged between 0.12 and 4.04. The percentage of intersection- or driveway-related accidents, large truck-related accidents, and wet pavement-related accidents were 22, 8, and 14, respectively. Eight of the sites had no intersection- or driveway-related accidents, four of the sites had more than 50 percent of the intersection-related accidents, and 10 of the 26 truck accidents occurred at the site that had the highest number of intersection-related accidents. Approximately 47 percent of the accidents on the two-lane highway *without shoulder* roadways occurred at night.

Accident Rates. In another study, accident rates were computed for sections of urban and rural freeways and two-lane highways in two of the states in the HSIS data base (*68*). Highway sections were divided into eight roadway types and defined as not including intersections or interchanges. The HSIS study's objective was to combine accident data from

TABLE 36 Accident study sites

Route	Length (mi)	Appr/ mile	AADT	LSD Curves	Lane Width (ft)	Shoulder Width (ft)	Subject Rating
Multilane Roadways							
TX-A01	0.9	12.22	14000	1	12	0[b]	1
TX-A02	3.0	16.33	4375[a]	8	12		3
Two-Lane Roadways with Shoulders							
IL-A01	4.4	10.0	3450	10	12	8-10	3
TX-A03	0.9	7.8	3300	1	12	10	1
TX-A04	3.0	9.0	4800	6	12	8	2
TX-A05	2.3	10.4	7600	3	12	8	2
WA-A05	2.8	12.5	3275	6[c]	12	6	2
WA-A06	0.7	5.7	2742	2	12	6	1
WA-A07	0.8	5.0	3625	1	12	6	1
Two-Lane Roadways without Shoulders							
IL-A02	1.7	5.88	5400	4	12	3-5	3
IL-A03	5.2	4.62	5200	17	12	3-5	3
IL-A04	1.7	15.29	4500	5	11	1-3	3
IL-A05	1.5	4.67	2400	3	12	4	2
IL-A06	1.5	8.39	1700	5	12	1	3
TX-A06	3.8	22.89	2800	10	11	0	3
TX-A07	2.2	20.91	3600	4	12	0	2
TX-A08	2.7	31.48	5150	5	12	0	2
TX-A09	4.0	8.75	1000	4[c]	10.5	0	2
TX-A10	2.4	21.25	1500	3	11	0	2
TX-A11	1.2	14.17	3700	2	11	0	2
TX-A12	2.2	18.64	2000	4	10.5	0	2
TX-A13	2.8	13.93	940	10	10.5	0	2
TX-A14	2.3	11.30	940	2	11	4	1
WA-A08	1.5	4.00	4150	3	11	4	2
WA-A09	0.9	23.33	2725	3	11	4	2
WA-A10	1.6	1.88	2750	3	11-14	3	2
WA-A11	1.9	3.16	2075	6[c]	12	3	2
WA-A12	1.5	4.67	6025	2	12.5	5	1
WA-A13	3.6	3.89	1545	11	10.6	4	3
WA-A14	2.0	2.00	1535	2	11.5	1-4	1
WA-A15	1.5	12.00	5200	4	12	4	1
WA-A16	1.7	1.76	800	5	10	4	2
WA-A17	1.5	23.33	4500	6[c]	12	3	3

[a] 1/2 of AADT, because of NB only, [b] 1-ft paved, 3-ft gravel, [c] Excludes curves 100 ft or shorter

Notes: Appr/mile = Number of approaches (intersections and driveways) per mile
LSD Curves = Number of limited sight distance curves
Subj Rate = Subjective rating of the vertical curve geometry—1 = isolated limited sight distance curve, 3=closely spaced limited sight distance curves, and 2 = not 1 or 3

TABLE 37 Multilane accident summary

Route	Number of Accidents*					Accident Rates		% Appr Acc.
	per yr	Total	Appr	Truck	Snow/ Ice/Wet	Acc/ MVM	Acc/Yr/ Mi	
TX-A01	5.67	17	3	1	5	1.16	6.30	18
TX-A02	2.67	8	1	1	0	0.57	0.89	13
Average/Total	**4.17**	**25**	**4**	**2**	**5**	**0.86**	**3.60**	**16**

Total column represents 3 years of data.
*Appr = Approach-related (intersection or driveway) accidents
Truck = Large truck-related accidents

TABLE 38 Two-lane with shoulders accident summary

Route	Number of Accidents*					Accident Rates		% Appr Acc.
	per yr	Total	Appr	Snow/ Ice/Wet	Truck	Acc/ MVM	Acc/Yr/Mi	
IL-A01	9.33	28	5	5	2	1.73	2.12	18
TX-A03	0.67	2	0	0	0	0.61	0.74	0
TX-A04	7.67	23	8	3	0	1.54	2.56	35
TX-A05	3.00	9	2	0	0	0.48	1.30	22
WA-A05	2.10	11	3	0	0	0.67	0.79	27
WA-A06	0.60	3	0	0	0	0.80	0.86	0
WA-A07	1.00	5	2	0	0	0.98	1.25	40
Average/Total	**3.48**	**81**	**20**	**8**	**2**	**0.97**	**1.37**	**20**

Total column represents 3 years of data for Illinois and Texas sites and 5 years of data for Washington sites.
*Appr = Approach-related (intersection or driveway) accidents
Truck = Large truck-related accidents

the two states to provide a single estimate for accident rates. They suggested that if the accident rates for similar highway types were not significantly different from a statistical standpoint, the data bases could be combined, but if the accident rates were significantly different, the data bases should not be combined. Although their results showed that the accident rates for the rural two-lane highways in the two states were significantly different and could not be combined, the reported accident rates provide an estimate of the range in accident rates for rural two-lane highways.

TABLE 39 Two-lane without shoulders accident summary

Route	Number of Accidents*					Accident Rates		% Appr Acc
	per year	Total	Appr	Truck	Snow/ Ice/Wet	Acc/ MVM	Acc/Yr/ Mi	
IL-A02	6.00	18	4	1	2	1.82	3.53	22
IL-A03	21.00	63	14	10	9	2.31	4.04	22
IL-A04	6.00	18	3	2	2	2.50	3.53	17
IL-A05	2.00	6	0	0	0	1.64	1.33	0
IL-A06	1.67	5	0	0	1	0.98	0.54	0
TX-A06	6.00	18	9	1	2	1.58	1.58	50
TX-A07	1.00	3	1	0	0	0.39	0.45	33
TX-A08	6.33	19	11	0	0	1.29	2.35	58
TX-A09	3.00	9	2	0	2	2.00	0.75	22
TX-A10	5.33	16	6	2	1	4.35	2.22	38
TX-A11	3.33	10	2	0	2	2.20	2.78	20
TX-A12	2.67	8	3	0	0	1.71	1.21	38
TX-A13	2.00	6	3	0	2	2.22	0.71	50
TX-A14	0.67	2	0	0	0	0.89	0.29	0
WA-A08	2.30	14	1	0	2	1.31	1.87	7
WA-A09	1.20	6	0	0	2	1.40	1.33	0
WA-A10	1.80	9	0	1	2	1.10	1.12	0
WA-A11	3.60	18	0	2	4	2.47	1.89	0
WA-A12	3.00	15	1	0	2	0.92	2.00	7
WA-A13	3.60	18	2	4	2	1.56	1.00	11
WA-A14	2.40	12	0	1	5	2.22	1.20	0
WA-A15	3.40	17	4	0	2	1.32	2.27	24
WA-A16	0.20	1	0	0	0	0.41	0.12	0
WA-A17	4.40	22	8	2	4	1.79	2.93	36
Average/Total	**3.87**	**333**	**74**	**26**	**48**	**1.68**	**1.71**	**19**

Total column represents 3 years of data for Illinois and Texas sites and 5 years of data for Washington sites.
*Appr = Approach-related (intersection or driveway) accidents
Truck = Large truck-related accidents

The two states had total accident rates of 1.07 and 1.86 accidents per million vehicle miles of travel per year, respectively (*68*). For the limited sight distance sites, the average accident rate for the two-lane *without shoulder* sites in Washington, Illinois and Texas were 1.45, 1.85, and 1.85 accidents per million vehicle miles, respectively, all of which are between the accident rates for the two HSIS states. Average accident rates for the two-lane *with shoulder* sites were 0.97 accidents per million vehicle miles, which is slightly below the accident rate in one of the HSIS states. These results suggest that the accident rates for the limited stopping sight distance sites are similar to the accident rates for rural two-lane highways in the two HSIS states.

Older and Younger Driver Accidents. One question of interest was whether roadways with limited sight distance create significant safety problems for older drivers and/or inexperienced drivers. A comparison of the percentage of accidents involving older and younger drivers in the data base with the percentage of all accidents involving older and younger drivers should indicate whether limited sight distance presents a significant problem for either type of driver. Of the 609 drivers involved in accidents at the selected study sites, 91 of them (14.9 percent) were 55 years of age or older. The National Safety Council reports that 14.8 percent of the drivers involved in all accidents are 55 years of age or older (*69*). Examination of the number of younger drivers involved in accidents at the limited sight distance sites produced similar findings. Approximately, 16.1 percent (98 drivers) of the drivers at the study sites were 20 years of age or younger while the National Safety Council reports that 17.2 percent of the drivers involved in accidents are 20 years of age or younger (*69*). These comparisons suggest that younger and older drivers are not over represented in accidents on roadways with limited sight distance vertical curves.

Large Truck Accidents. A second question of interest was whether limited stopping sight distance vertical curves create safety problems for large trucks because of their generally poorer braking performance. If the current model creates safety problems for large trucks, the percentage of large truck accidents on roadways with limited stopping sight distance vertical curves should be greater than the percentage of large truck accidents on all roadways. In 1992, 3.4 percent of vehicles involved in all accidents were medium or heavy trucks (*70*). Medium or heavy trucks are tractors with or without the semi-trailer. The limited stopping sight distance sites in this study experienced two large trucks accidents on multilane roadways, two large trucks accidents on two-lane *with shoulder* roadways, and 26 large trucks accidents on two-lane highways *without shoulder* roadways (see Table 39).

These 30 large truck accidents represent 4.9 percent (30 of the 609 vehicles) of the accidents at the 33 study sites; however, 10 of the 30 accidents involving large trucks occurred at a single site. This site also experienced the largest number of intersection-related accidents. If this site is removed from the sample, the percentage of truck-related accidents decreases to 3.3 percent. These percentages are similar to the percentage of large trucks involved in accidents on all roadway types (3.4 percent) and suggest that large trucks are not over represented in accidents on roadways with limited sight distance vertical curves.

Accident Comparisons. Several methods of classifying the study sites were developed to investigate the variation in accident rates between the sites. The sites were first classified according to roadway type (multilane, two-lane with shoulder, two-lane without shoulder) to test for differences by roadway type. The average accident rate per million vehicle miles was 0.86 for the multilane sites, 0.97 for the two-lane *with shoulder* roadways, and 1.68 for the two-lane *without shoulder* roadways. Note, however, that the accident rate at one of the two-lane *without shoulder* roadways is much higher than at the other 23 sites, which adds greatly to the within roadway variability. Analysis of variance tests at the 95 percent confidence level showed no differences in mean accident rate for the three types of roadways.

The sites were then classified by state to determine if sites with limited sight distance vertical curves in one state had higher or lower accident rates than similar sites in the other two states. The two-lane *without shoulder* roadways' average accident rate per million vehicle miles was 1.85 for Illinois sites; 1.45 for Washington sites; and, 1.85 for Texas sites. The ANOVA tests at the 95 percent confidence level showed no difference in mean accident rate on two-lane *without shoulder* roadways for the three states in this study.

The sites were then classified by subjective rating and the inferred design speed of the limited sight distance crest curves within the segment to test for differences in accident rates because of the type of curve environment and design speed. Sites with a subjective rating of 1 represented an isolated curve and sites with a subjective rating of 3 represented several closely spaced curves with a consistent design speed. Sites with a subjective rating of 2 were those sites that could not be classified as a 1 or a 3, that is, multiple curves and variable curve geometry within the section.

The two-lane *without shoulder* sites had multiple design speeds in each of the subjective rating categories. Except for two sites, the sites with the highest accident rates for *subjective ratings 1 and 3*, had design speeds of 35 to 40 mph. This comparison suggests those sites with *higher* accident rates correspond to sites with *lower* design speeds, that is, higher accident rates were associated with significantly shorter stopping sight distances.

Accident Examination. Each individual accident report was reviewed to determine contributing factors to the accident. While the accident narrative provided the greatest amount of information regarding the cause of the accident, information on the specific elements or sequence of events that contributed to individual accidents was supplemented from several sources. For example, the roadway geometry available from plan and profile sheets provided information on the horizontal and vertical curvature near the accident site, and data on the accident report, in addition to the narrative box, provided information such as light and pavement conditions. Approximately one-half the accidents had more than one contributing factor.

Reviewing the entire list of contributing factors to accidents on roadways with limited sight distance gives an overview of characteristics that appear over represented in the data base. Tables 40 and 41 list the most frequent contributing factors for the multilane sites, two-lane *with shoulder* sites, and two-lane *without shoulder* sites. Of the 499 contributing factors to accidents on the two-lane *without shoulder* sites, 81 were in the object-related category, that is, 81 of the 333 accidents (24 percent) involved objects in the roadway. The most frequent contributing factor to accidents on the two-lane *with shoulder* sites also was an object in the roadway, that is, almost 40 percent of the accidents on the two-lane *with shoulder* sites involved objects in the roadway. Surprisingly, only 8 percent of the accidents on multilane highways involved objects in the roadway.

A subset of the object-related accidents included accidents where the object was struck at night. This contributing factor category was called "headlight sight distance" and represented those accidents where stopping sight distance probably was limited more by the vehicle's headlights than by the roadway's geometry. Accidents where an object was struck at night were slightly more than 10 percent at the two-lane *without shoulder* sites, 12 percent at the two-lane *with shoulder* sites, and 4 percent at the multilane sites.

The second most frequent contributing factor to accidents on all three types of roadways was "speed too fast for conditions." This factor contributed to approximately 17 percent of the two-lane roadway accidents and 44 percent of the multilane roadway accidents. Other frequent contributing factors to accidents at the study sites were wet or icy pavements—10 percent of the two-lane roadway accidents and 20 percent of the multilane roadway accidents; driver impairment—8 percent of the two-lane *without shoulder* roadway accidents and 20 percent of the multilane roadway accidents; and driver negligence—8 percent of the two-lane roadway accidents.

Limited Stopping Sight Distance. Judgment was used when determining if the accident occurred on or near a crest curve because of the difficulty in determining an accident's exact location. Thus, all accidents where limited stopping sight distance could have been a contributing factor were identified as a "limited stopping sight distance" (LSSD)-related accident. LSSD was identified as a contributing factor if the accident occurred on or near a limited sight distance curve, the accident occurred during daylight conditions, and there was an object in the roadway.

None of the accident narratives specifically stated that the accident was caused by limited sight distance; however,

TABLE 40 Multilane roadways and two-lane with shoulder roadways contributing factors

State Route	Contributing Factors*					
	Object	**HLSD**	**Speed**	**Wet**	**Imp**	**Other**
Multilane Roadways						
TX-A01	2	1	8	5	4	4
TX-A02	0	0	3	0	1	10
TOTAL	**2**	**1**	**11**	**5**	**5**	**14**
Two-Lane with Shoulder Roadways						
IL-A01	15	0	5	4	1	11
TX-A03	2	2	0	0	0	0
TX-A04	7	4	6	2	0	19
TX-A05	2	1	0	0	0	7
WA-A05	3	1	2	0	2	6
WA-A06	1	1	0	0	1	1
WA-A07	2	1	0	0	3	3
TOTAL	**32**	**10**	**13**	**6**	**7**	**47**

* Object = Object on Road
HLSD = Headlight Sight Distance
Speed = Speed too Fast for Conditions
Wet = Water on Pavement
Imp = Driver was Impaired or Negligent
Other = All Remaining Contributing Factors

TABLE 41 Two-lane without shoulders contributing factors

State Route	Contributing Factors*						
	Object	HLSD	Speed	Icy	Imp	Neg	Other
IL-A02	8	6	4	1	1	1	15
IL-A03	17	9	4	5	5	3	43
IL-A04	11	5	0	0	0	0	2
IL-A05	6	1	0	1	0	0	0
IL-A06	4	4	2	2	0	0	7
TX-A06	2	0	4	2	2	2	14
TX-A07	0	0	0	0	0	0	3
TX-A08	1	0	3	0	5	0	14
TX-A09	1	0	6	0	0	0	13
TX-A10	1	0	3	0	1	1	14
TX-A11	4	2	5	0	0	1	7
TX-A12	1	0	2	0	1	0	8
TX-A13	1	0	2	1	1	0	5
TX-A14	0	0	1	0	1	0	0
WA-A08	4	4	2	2	0	4	9
WA-A09	0	0	3	2	1	1	5
WA-A10	3	1	3	2	0	1	6
WA-A11	2	1	2	4	3	1	16
WA-A12	3	1	1	2	3	4	4
WA-A13	2	1	2	1	1	3	16
WA-A14	3	0	3	5	0	1	12
WA-A15	5	3	3	2	1	2	9
WA-A16	0	0	0	0	0	0	1
WA-A17	2	0	2	1	2	4	14
TOTAL	**81**	**38**	**57**	**33**	**28**	**29**	**233**

*Object = Object on Road
HLSD = Headlight Sight Distance
Speed = Speed too Fast for Conditions
Icy = Snow or Ice on Pavement
Imp = Driver was Impaired
Neg = Driver was Negligent
Other = All Remaining Contributing Factors

several accident narratives with stopping sight distance as a possible contributing factor stated that the hill crest blocked the driver's view. For example, the narrative for one of the accidents on WA-A14 stated: "Hit deer. Did not see it in time to slow down or avoid it." LSSD was identified as a possible contributing factor to this accident because it occurred near a limited sight distance vertical curve.

Other accident narratives provided more clues into the possible involvement or contribution of the crest vertical curve. For example, the narrative for an accident where a vehicle stuck another vehicle involved in a previous accident stated that "Unit 1 . . . drove up a small hill . . ." Because this accident occurred at night (4:49 a.m.) and at a crest vertical curve, its contributing factors were identified as headlight sight distance and object in the road. Another example of sight distance as a contributing factor to an accident was the following narrative: "Unit 1 . . . was stopped for a school bus unloading when unit 2 . . . came over a hill and was unable to stop veered to the left of roadway striking unit 1."

Although the sites were selected to maximize the possibility of stopping sight distance accidents, LSSD was involved in only a small portion of the accidents at the study sites. Only 14 of the 439 accidents investigated (3 percent) had LSSD as a possible contributing factor to the accident. By roadway type, 3.0 percent (10 of 333 accidents) of the accidents on the two-lane *without shoulder* sites, 4.9 percent (4 of 81 accidents) of the accidents on two-lane *with shoulder* sites, and none of the accidents on the multilane sites had LSSD as a possible contributing factor to the accident.

Table 42 summarizes the 14 accidents with LSSD as a possible contributing factor to the accident. Note that most of the accidents (9 of 14) involved striking another vehicle. The non-vehicle objects struck were deer (two), deep water, a dog, and an unknown object. Only one of the accidents with sight distance as a possible contributing factor involved a large truck. A farm truck pulling a small trailer was traveling on a parallel, private access road to the two-lane roadway. The trailer came loose and entered the two-lane highway. A gravel dump truck struck the trailer head-on, left the roadway, and overturned.

Four of the 24 drivers (17 percent) involved in accidents with LSSD as a possible contributing factor were 55 years of age or older. Another four drivers (17 percent) were 20 years in age or younger. These percentages are near those reported by the National Safety Council (*69*) which reports that 14.8 percent of the drivers involved in all accidents are 55 years or older and 17.2 percent are 20 years or younger. These data suggest that older and younger drivers are not over represented in LSSD-related accidents.

Headlight Sight Distance. Accidents with headlight sight distance as a contributing factor were defined as those accidents in which a driver struck an object in the roadway at night (dark conditions). These accidents could occur on vertical curves, horizontal curves, or tangent sections of roadway. Table 43 lists the 19 accidents that occurred at night, near a vertical curve, and involved striking an object in the roadway. The table is divided into two sections—those accidents where LSSD could have been a contributing factor and those accidents where limited sight distance was not believed to be a contributing factor. An example of the latter situation is a deer running into a vehicle rather than a vehicle striking a deer. Somewhat surprisingly, 15 of the 19 headlight sight distance accidents that occurred near crest vertical curves involved a deer in the roadway.

Twenty-seven additional accidents beyond those listed in Table 43 had headlight sight distance as a contributing factor to the accident. The characteristics of these accidents were that they occurred at night (i.e., headlights were in use), they involved striking an object, and they occurred on a tangent section of roadway (i.e., stopping sight distance was not limited by a vertical curve). Of these 27 headlight-related accidents, a deer was struck in 18 accidents, a bull or cow was struck in 6 accidents, another vehicle was struck in 2 accidents, and a tree was struck in 1 accident.

Object in Roadway. The contributing factor, "object-related," was used for all accidents in which a driver struck an object in the roadway. Although, the same contributing factor designation was used for both day and night conditions, the percentage of *object-related* accidents was not consistent among the three states. Texas and Washington had similar percentages (16 percent in Texas and 20 percent in Washington), but the object-related accident percentage was much higher in Illinois. More than 44 percent of the accidents in Illinois involved striking an object in the roadway. This disparity is caused by a much larger number of deer accidents in Illinois (more than 6,000 in 1990).

Deer were the most common object struck (83 of 115 objects). In fact, deer-related accidents were a major contribution to accidents on the rural two-lane highways in this study. Deer are a significant safety problem in many other states, and represent a problem that is likely to get worse before getting better. According to recent estimates, the number of white tail deer in the lower 48 states (approximately 25 million) has almost doubled in the past decade and is expected to continue increasing in the future (*71*). The increase in num-

TABLE 42 Accidents with sight distance as a possible contributing factor

Route	Class[a]	Object Struck	Age of Driver[b]	K-value	Subject Rating
IL-A02	w/o	Vehicle	B	50	3
IL-A02	w/o	Vehicle	B	125	3
IL-A03	w/o	Vehicle	B	30	3
IL-A03	w/o	Vehicle	B	65	3
IL-A01	with	Deer	Older	122	3
TX-A04	with	Vehicle	Younger	92	2
TX-A04	with	Vehicle	B	92	2
TX-A09	w/o	Vehicle	B	41	2
TX-A12	w/o	Vehicle	Younger	43	2
WA-A07	w/o	Dog	B	105	1
WA-A13	w/o	Deep Water	Younger	75	3
WA-A13	w/o	Vehicle	B	75	3
WA-A14	w/o	Unknown	Older	122	1
WA-A14	w/o	Deer	Younger	122	1

[a] All accidents listed in the above table occurred on two-lane highways. When 6 ft or more of paved shoulder is present, the roadway is classified as having shoulders (with), when less than 6 ft of paved shoulders is present, the two-lane roadway is without shoulders (w/o).

[b] The age of one of the involved drivers was 20 years old or younger (younger), 55 years old or older (older), or the accident did not involve a younger or older driver (B).

TABLE 43 Accidents with headlight sight distance as a contributing factor

State Route	Class[a]	Object Struck	Age of Driver[b]	K-Value of Curve	Subjective Rating
Accidents where sight distance could have been a contributing factor					
IL-A03	w/o	Deer	B	40	3
TX-A05	with	Vehicle	B	93	2
WA-A08	w/o	Deer	Older	71	2
WA-A08	w/o	Deer	B	71	2
Accidents that occurred on a vertical curve where sight distance is not believed to be a contributing factor					
IL-A02	w/o	Deer	B	50	3
IL-A02	w/o	Deer	B	50	3
IL-A02	w/o	Deer	B	50	3
IL-A03	w/o	Deer	B	136	3
IL-A03	w/o	Deer	B	95	3
IL-A03	w/o	Deer	B	95	3
IL-A03	w/o	Deer	Older	95	3
IL-A04	w/o	Deer	B	58	3
IL-A04	w/o	Deer	B	143	3
IL-A05	w/o	Deer	B	90	2
TX-A04	with	Vehicle	B	104	2
WA-A08	w/o	Deer	B	71	2
WA-A07	with	Deer	B	105	1
WA-A11	w/o	Pony	Older	84	2
WA-A13	w/o	Bike	B	109	3

[a] All accidents listed in the above table occurred on two-lane highways. When 6 ft or more of paved shoulder is present, the roadway is classified as having shoulders (with), when less than 6 ft of paved shoulders is present, the two-lane roadway is without shoulders (w/o).

[b] The age of one of the involved drivers was 20 years old or younger (younger), 55 years old or older (older), or the accident did not involve a younger or older driver (B).

bers and their behavior around highways may explain why deer are involved in so many accidents on rural highways. Deer are attracted to highways, partly because of salt leeching into the surrounding soil, and partly because of forage planted in the median and along the roadside. Additionally, deer cross roadways to move from open feeding areas to protected bedding areas in regular cycles, sometimes several times a day.

The problem for motorists is that deer react to cars in seemingly illogical ways, especially at night. For example, it appears that deer often wait until a car is quite close and then run out in front of the vehicle just in time to get hit; however, this seemingly illogical behavior to drivers is really quite logical for a deer. Deer freeze as a standard response to an approaching threat, especially under bright lights. Essentially, the bright lights of the approaching vehicle negate the deer's extremely effective night vision.

Summary. The objective of the accident causation study was to determine if stopping sight distance was a contributing factor in accidents on roadway segments containing limited sight distance crest vertical curves. This objective was accomplished by reviewing the 439 narratives from accidents that occurred on selected multilane and two-lane roadways with limited sight distance crest vertical curves. The findings were that the accident rates on rural two-lane highways with LSSD are similar to the accident rates on all two-lane rural highways. **Thus, LSSD does not appear to cause a safety problem.**

Approximately 4 percent of the accident narratives reviewed had LSSD as a possible contributing factor to the accident. Thus, even on limited sight distance roadways, LSSD is not a major safety problem. All the accidents with limited sight distance as a possible contributing factor occurred on crest vertical curves with *K* values of 125 or less (stopping sight distance of 400 ft or less). Additionally, most of the accidents with LSSD as a possible contributing factor occurred on vertical curves with *K* values of 100 or less (stopping sight distances of 360 ft or less). Current AASHTO policy requires a minimum stopping sight distance of 450 ft (*K* value of 150) for a 55-mph design speed. Thus, **moderate reductions in minimum stopping sight distance do not appear to cause a safety problem.**

Most of the objects struck on roadways with limited sight distance crest vertical curves were large objects such as deer, cattle, horses, and other vehicles. Most of the accidents with LSSD as a contributing factor were caused by another vehicle stopped in the roadway to make a turn into a driveway or intersection. Thus, the placement of driveways and intersections near crest vertical curves should be carefully considered in the design of new roadways or reconstruction of existing roadways. The percentage of accidents in this study involving large trucks was comparable to the percentage of all accidents involving large trucks reported by the National Safety Council. Thus, **LSSD does not appear to cause a safety problem for large trucks.** The percentage of accidents involving younger and older drivers in this study was comparable to the percentage of all accidents involving younger and older drivers reported by the National Safety Council. Thus, **LSSD does not appear to cause a safety problem for either inexperienced or older drivers.**

OPERATIONAL EFFECTS

The relationship between driver behavior and available stopping sight distance is important in the selection of the appropriate design speed. The following sections summarize the literature related to design and operating speed and the results of several field studies that quantified this relationship (*72*). Appendix G contains additional information regarding the relationship between operating speed and available stopping sight distance on rural highways.

Design Speed Concept

Horizontal and vertical elements of a highway are designed based on an assumed design speed. The design speed concept was developed in the 1930s as a mechanism for designing rural alignments to permit most drivers to operate uniformly at their desired speed. In 1938, AASHO recognized that drivers will select a speed influenced by the roadway environment rather than an assumed design speed (*4*). It states "A low design speed should not be assumed for a secondary road, however, if the topography is such that vehicle operators probably will travel at high speeds . . . Drivers do not adjust their speed to the importance of the road but to the physical limitations of curvature, grade, sight distance, smoothness of pavement . . ."

The problem of how to decide what the design speed should be for a particular set of conditions was posed by the design speed concept, that is, what was the "maximum approximately uniform speed adopted by the faster group of drivers?" To find a solution to that question for roads not yet built, the Bureau of Public Roads engineers used data from 260,000 vehicles measured at 40 different locations in 1934, 1935, and 1937. Ratios of the speeds of the fastest drivers to the average speed of all drivers for various percentiles of total traffic were developed. Based on the resulting curves, they recommended that the design speed of a future highway be the speed that only 5 or possibly 2 percent of the drivers will exceed after the road is built.

The design speed concept was developed because of safety problems resulting from a discrepancy between speeds for which horizontal curves were designed and the speeds at which drivers negotiated those curves. The basis for the use of a design speed concept was a result of work done by Barnett (*73*). Barnett recommended that, "the assumed design speed of a highway should be the maximum reasonably uniform speed that would be adopted by the faster driving group of vehicle operators, once clear of urban areas." He urged that all features of geometric design be made consistent with the chosen design speed. The design speed or "balanced design" concept became a permanent feature of geometric design policy in the United States when it was adopted by AASHO in 1938. AASHO defined design speed as "the maximum approximately uniform speed which probably will be adopted by the faster group of drivers but not, necessarily, by the small percentage of reckless ones."

Recent research (*74*), however, argues that "Design speed is no longer the speed adopted by the faster driving group of vehicle operators, but has become a value used for the correlation of design elements which is also a maximum safe speed." Good (*75*) states ". . . there seems to have been a change in emphasis from design speed as a speed which might be expected from driver behavior, to a speed which is *safe* from the designer's point of view."

Current Use of Design Speed. AASHTO (*1*) currently defines design speed as "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern." A fundamental limitation of the design speed concept, however, is that design speed applies only to horizontal and vertical curves. Design speed has no practical meaning on flat tangents, and therefore, provides no basis for establishing maximum tangent lengths to promote consistency by controlling the maximum operating speeds that can be reached. In fact, AASHTO's encouragement of the use of above minimum values may negatively affect the consistency among alignment elements. Use of above minimum values encourages operating speeds that exceed the design speed of the controlling element.

The 1990 *Green Book* (*1*) provides general guidance on both the selection and application of design speed. Examples of guidance provided in the Green Book include

- The assumed design speed should be a logical one with respect to the topography, the adjacent land use, and the functional classification of the highway.
- Except for local streets where speed controls are frequently included intentionally, every effort should be made to use as high a design speed as practicable to

attain a desired degree of safety, mobility, and efficiency while under the constraints of environmental quality, economics, aesthetics, and social or political impacts.

- The design speed chosen should be consistent with the speed a driver is likely to expect. Where a difficult condition is obvious, drivers are more apt to accept lower speed operation than where there is no apparent reason for it.
- Above minimum design values should be used where feasible, but in view of the numerous constraints often encountered, practical values would be recognized and used.
- Where it is necessary to reduce design speed, many drivers may not perceive the lower speed condition ahead, and it is important that they be warned well in advance. The changing condition should be indicated by such controls as speed-zone signs and curve-speed signs.

Influences on Operating Speeds. In a 1962 study on operating speeds within the urban environment, Rowan et al. concluded that substantial speed reductions occurred when sight distance was below 300 to 360 m and that the introduction of a curbed urban cross-section and the adjacent land use (residential or commercial development) had a speed-reduction influence. Lateral restrictions were found to be a greater speed-reduction influence than development density (*76*).

In 1966, Oppenlander reviewed the literature to identify variables influencing spot speed. The variables were organized into driver, vehicle type, roadway, traffic, and environment categories. The roadway characteristics determined to be most significant included functional classification, curvature, gradient, length of grade, number of lanes, and surface type. Sight distance, lateral clearance, and frequency of intersections were also determined to have an influence (*77*).

Garber and Gadiraju examined speed variances of 36 roadway locations including interstates, arterials, and rural collectors in 1989. The ANOVA tests were used to determine which traffic characteristics significantly affected average speed and speed variance. Design speed and highway type were significant, and the year in which data were obtained and the traffic volume were not significant (*78*).

Lefeve (*79*) found that as drivers approach vertical curves with short sight distances, they reduce to their speeds to some extent. When the minimum sight distance was 45 m, the average decrease in speed as drivers approached the point of minimum sight distance was 10 km/h. When the minimum sight distance was 120 m, the average decrease in speed was only 3 km/h. This reduction in speed, however, is much less than the speed reduction assumed by the AASHTO SSD model. Lefeve hypothesized that drivers feel that their reduction in speed is much greater than it actually is, or that individual drivers so seldom encounter critical situations on vertical curves that they are not aware of the hazard involved and their perception of risk is low.

Design and Operating Speed. Recent studies have documented a noticeable disparity between design and operating speeds. A 1992 FHWA study on design consistency collected speed data at 138 horizontal curves on 29 rural two-lane highways in five states (New York, Oregon, Pennsylvania, Texas, and Washington) (*80*). The data in Figure 15 showed that the 85th percentile speed exceeded the inferred design speed on all but two curves whose design speed was 55 mph or less. Whereas the 85th percentile speed was less than the inferred design speed for all curves whose design speed was 65 mph or more. Of the curves with 60 mph design speeds, an almost equal number had 85th percentile speeds greater than and less than the design speed. The disparity between the 85th percentile speeds and inferred design speeds is greatest for the lowest design speeds. For curves with design speeds between 25 and 40 mph, 85th percentile speeds average 11 to 12 mph faster than the design speed (*80*).

McLean (*81, 82*) also found similar design speed/operating speed disparities on rural two-lane highways in Australia. McLean found that horizontal curves with design speeds less than 90 km/h had 85th percentile speeds that were consistently faster than the design speed, whereas curves with design speeds greater than 90 km/h had 85th percentile speeds that were consistently slower than the design speed. McLean's findings prompted a revision of the Australian design procedures for lower-design speed roadways.

Operating Speed Field Studies

To determine the relationship between operating speeds and available stopping sight distance, this research used a procedure for collecting speeds on both tangent and limited sight distance sections, and then analyzing the data to determine whether motorists drove slower on limited sight distance sections. The following sections describe the site plan information data bases, site selection criteria, data collection, and data analysis.

Data Collection and Analysis. The process of determining potential study sites involved finding rural roadways with limited sight distance vertical curves. Efforts were focused on older roadways in rolling terrain because they generally had more limited sight distance curves. To identify potential study sites, information for several types of roadways was requested from three state DOTs and FHWA. This information included the following: average daily traffic (ADT), length and radius of horizontal curves, length and approach grades of vertical curves, number and width of lanes, shoulder width and type (if any), and milepost location of geometric features.

The roadways used in this study were found in three geographic regions: Washington, Texas, and Illinois. These locations were selected because of the differing geographic areas they represent, and the quality of the traffic and road-

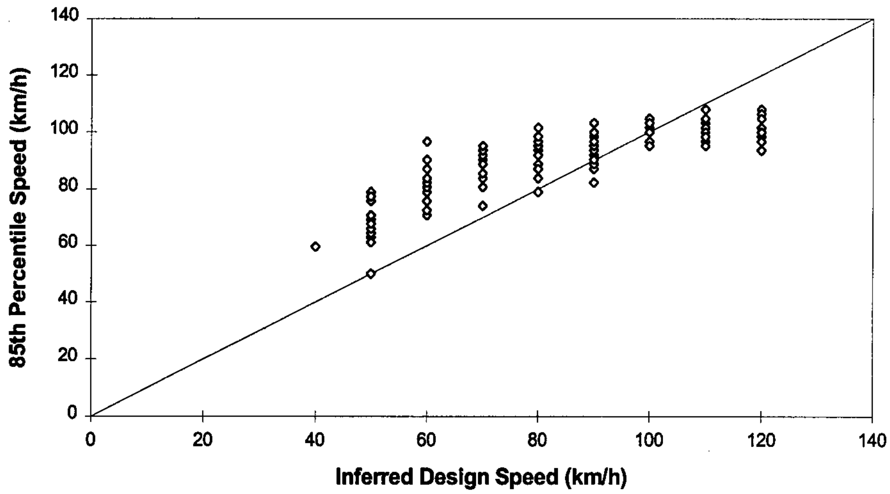


Figure 15. 85th percentile speed versus inferred design speed for 138 curves in five states.

way information data bases available from each of the respective departments of transportation. Traffic and roadway geometry information were obtained from each of these states and used in selecting potential study sites. Additionally, a functional classification scheme was developed to aid in selection of potential study sites. Roadway types included multilane roadways, two-lane *with shoulder* roadways, and two-lane *without shoulder* roadways (roadways were categorized as having shoulders if the shoulder was 1.8 m or wider).

Criteria for identification as a potential operating speed study site were that the roadway have crest vertical curves with less than the AASHTO required minimum stopping sight distance for a design speed of 55 mph, that the roadway section was in a rural area, that the roadway's cross section and adjacent land use were consistent throughout the section, and that there were no intersections controlled by traffic signals or multi-way stop signs within the section.

Speeds were collected for individual vehicles at both a control section and a limited sight distance crest curve section. The speeds on the control section were collected at a location where the vehicles were expected to be operating at their desired speeds, while the speeds for the vertical curve were collected just before the intersection of the approach and departure grades. Distances between the control and crest section varied from 100 m to approximately 2 km; however, most of the distances were in the 300 to 1000 m range. The minimum speed in the limited sight distance section was needed so that the maximum speed differential between the control section and the limited sight distance section could be determined.

Most speed data were collected with radar guns that were undetectable by most commonly used radar detectors; however, some data were collected with normally tuned radar guns. At these locations, care was used in ensuring that drivers did not detect the presence of the radar. All data that showed vehicles slowing down dramatically were discarded. Data were collected for a minimum of 4 hours or 100 vehicles. The time limit of 4 hours eliminated undue delays while collecting speed data, and the quantity of 100 vehicles provided for a reliable data base on which to perform a statistical analysis.

As mentioned, three types of roadways (multilane, two-lane *with shoulder* and two-lane *without shoulder*) were defined for this study. Three volume levels were also defined. Within these nine roadway-type, volume-level categories, 42 operating speed studies were conducted at 39 sites in three states (data were collected twice at three sites—once during the day and once during the night). Table 44 provides a breakdown of the 42 studies in each of the nine defined study categories. Except for one of the high-volume categories, there were at least two sites in each category studied.

In addition to roadway type and volume level, each site was further subdivided by the design speed of the vertical curve where data were collected. Design speed levels are analogous to available sight distance and were set at Level 1—50 to 55 mph (400 to 450 ft of available sight distance), Level 2—40 to 49 mph (275 to 400 ft of available sight distance), and Level 3—40 mph and below (275 ft or less of available sight distance). Thus, there were 27 categories (3 roadway types × 3 volume levels × 3 design speed categories) in the final study design.

Results. The results of the operating speed studies are presented in the following section. A statistical comparison was made between speeds at the control and crest sections for various roadway types and available sight distance (design speed levels). A comparison was also made between states to determine any differences in 85th percentile speeds for dif-

TABLE 44 Study sites by roadway type and traffic volume

Type of Roadway	Traffic Volume Levels/Number of Sites			Total
	Low	Medium	High	
Multilane	-	5	3	8
Two-Lane with Shoulder	7	6	1	14
Two-Lane without Shoulder	6	10	4	20
Total	13	21	8	42

ferent roadway types. Finally, a regression analysis was performed to determine if design speed is a good predictor of 85th percentile speed.

As the first step in the analysis process, the mean speed reductions between the control and crest sections were calculated for each of the 42 studies where data were collected. These data are summarized by site in Table 45. When examining the matrix of speed study sites, it can be seen that 10 cells contain no sites and 10 additional cells contain one site. These cells do not contain more sites because certain combinations of conditions, such as multilane and two-lane *with shoulder* roadways with low design speeds and very short sight distances, simply do not exist in large numbers.

The mean speed reductions for the design speed categories were then plotted for each of the three roadway categories to gain initial insight in the relationship between design and operating speed. Knowing that lower design speed crest curves provide less sight distance than higher design speed curves, it was expected that roads with lower design speed curves would have larger mean speed reductions between the control and crest sections. In other words, assuming drivers associate risk levels with how much sight distance is provided and reduce their speed as their perceived risk level increases, it was expected that drivers would decrease their speed with decreasing amounts of sight distance.

Multilane Roadways. The mean speed reductions between the control and crest section for each design speed and volume level were plotted for the *multilane roadway category*, and this plot showed that as the design speed (available sight distance) decreased, the mean speed reductions between the control and crest sections increased. Also, it showed that as traffic volume increased, the mean speed reductions increased for one design speed category and decreased for the other design speed category; however, it should be noted that data were available for only four of the nine cells in this roadway category. Only two hypothesis could be tested because of the missing cells in this roadway category: differences because of traffic volume and differences because of design speed levels.

In the first set of tests, it was concluded that one design speed category, increases in traffic volume caused a decrease in mean speed reductions between control and crest locations, and in the other design speed category, increases in traffic volume caused a significant increase in mean speed reductions; however, in both cases, the magnitude of these differences was not large enough to be meaningful. In the second set of tests, it was concluded that decreases in design speed and available sight distance caused significant increases in mean speed reductions between control and crest sections for high-volume sites, but not for medium-volume sites. In this study, the magnitude of the difference in mean speed reductions for the high-volume study sites was approximately 2.7 mph.

Two-lane Roadways with Shoulder. The mean speed reductions between the control and crest sections for each design speed and volume level were plotted for the *two-lane roadways with shoulder category.* As expected, the mean speed reductions between the control and crest sections increased as design speed and available sight distance decreased. No noticeable effects because of changes in traffic volumes were noted.

The first two tests involved single design speed categories and multiple volume levels, and concluded that increases in traffic volume on two-lane roadways with shoulder do not appear to affect mean speed reductions between control and crest sections. The third test involved a single traffic volume category and multiple design speed categories, and concluded that decreases in design speed and available sight distance on low-volume, two-lane roadways with shoulder caused an increase in mean speed reductions between the control and crest sections.

Two-lane Roadways without Shoulder. The mean speed reductions between the control and crest section for each design speed and volume level for the *two-lane roadways without shoulder category* were also plotted. As expected, the mean speed reductions between the control and crest sections increased as design speed and available sight distance decreased. It also appears that mean speed reductions decrease with increasing volumes.

The first two tests involved single design speed categories and multiple volume levels and concluded that increases in traffic volume appear to have a statistically significant effect on mean speed reductions for two-lane roadways without shoulder; however, the magnitude of this effect is not large enough to be meaningful. The next three tests involved single

TABLE 45 Operating speed field study sites and mean speed reductions by volume level and design speed level

Roadway Type	Design Level	Volume Class		
		Low	Medium	High
Multilane	50-55 mph	-	TX SH 6 day - 2.3 TX SH 6 night - 2.2	IL IL 29 - 1.3
	40-49 mph	-	TX US 69 - 4.9 IL US 51 - 1.3 IL US 28 - 2.3	TX SH 31 - 3.6 IL US 12 - 4.5
	< 40 mph	-	-	-
Two-Lane With Shoulder		Low	Medium	High
	50-55 mph	IL SH 23 - 0.4 WA SR 97 - 2.1 WA SR 97 - 0.9 WA SR 101 - 1.2		WA SR 101 - 1.4
	40-49 mph	WA SR 14 - 3.1 WA SR 410 - 2.8	IL SH 127 - 1.2 TX US 80 - 3.3 TX US 80 - 0.8 TX SH 64 - 3.4 TX SH 64 - 3.7 WA SR 101 - 3.9	-
	< 40 mph	IL SH 116 - 3.1	-	-
Two-Lane Without Shoulder	50-55 mph	IL SH 72 - 3.7 TX SH 19 - 0.5 WA SR 97 - 2.3	TX FM 14 - 2.3	WA SR 101 day - 1.2 WA SR 101 night - 1.5
	40-49 mph	TX FM 3058 - 3.8 IL IL 29 - 1.3	TX FM 14 - 6.6 TX FM 1179 - 1.7 TX FM 1179 - 3.2 WA SR 2 - 3.3 WA SR 203 - 0.4	IL SH 173 - 3.4 WA SR 507 - 0.1
	< 40 mph	*WA SR 7 - 1.8	IL US 20 day - 5.3 IL US 20 night - 4.4 TX FM 315 - 2.1 WA SR 14 - 3.5	-

volume levels and multiple design speed categories and concluded that decreases in design speed and available sight distance caused an increase in the mean speed reductions between control and crest sections; however, the magnitude of this increase tends to diminish with increases in traffic volumes.

Day and Night Comparisons. As shown in Table 45, both day and night data were collected for one multilane roadway site and two two-lanes *without shoulder* sites. Mean speed reductions between day and night conditions at two of the sites were similar, but the third site had noticeably larger mean speed reductions and the largest difference between day and night conditions. Interestingly, the two similar sites were the multilane site and one of the two-lane sites, rather than the two two-lane sites. The probable explanation for this result is that the two similar sites were in the 50 to 55 mph design speed category, whereas the third site was in less than 40 mph design speed category. This much larger speed reduction at the US 20 site is consistent with other comparisons that show that the largest reduction in speeds between control and crest sections are associated with those crest curves that have the lowest design speeds and shortest sight distances.

Operating and Design Speed Comparison. To compare daytime operating speeds with design speeds, 85th percentile speeds for each of the 39 crest curve sections were computed and plotted versus their corresponding design speed as shown in Figures 16 and 17. Data points are located at the intersection of the 85th percentile speed and the design speed of the

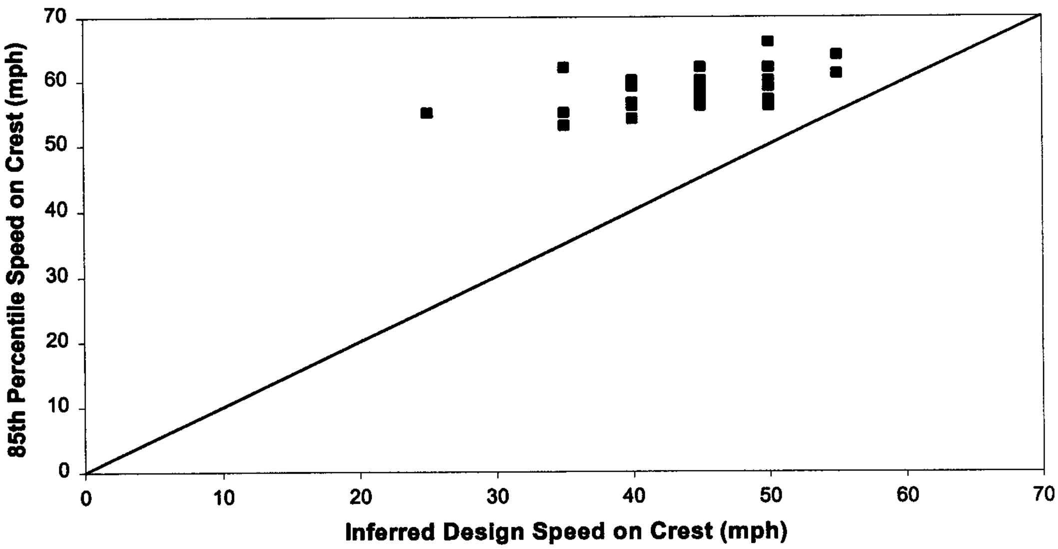


Figure 16. 85th percentile speeds for two-lane roadways with paved shoulders.

crest vertical curve. The diagonal line represents those points where the 85th percentile speed at the crest equals the design speed of the crest curve. The difference between the 85th percentile speed and design speed of the crest curve was greater for the lower design speeds and smaller for the higher design speeds; however, the difference at the higher design speeds is still quite large. Note that it appears that 85th percentile speeds would be less than the design speed at design speeds greater than about 60 to 65 mph.

Figure 16 suggests a relationship between 85th percentile speed and inferred design speed. In the multilane roadway and two-lane *with shoulder* roadway categories, the level of significance of the regression coefficient is less than 95 percent. This result indicates that the inferred design speed (available sight distance) of the crest curve is not a good predictor of 85th percentile speeds for these types of roadways. In other words, the 85th percentile speeds are not changing as the available sight distance of the crest curve changes on multilane and two-lane *with shoulder* roadways.

In the two-lane *without shoulder* roadway category, the regression coefficient was significant at the 99 percent confidence level with a coefficient of determination of 0.48. This result indicates that the inferred design speed (available sight distance) of a crest curve is a moderately good predictor of 85th percentile crest speeds for these types of roadways. In other words, the operating speed selected by drivers varies

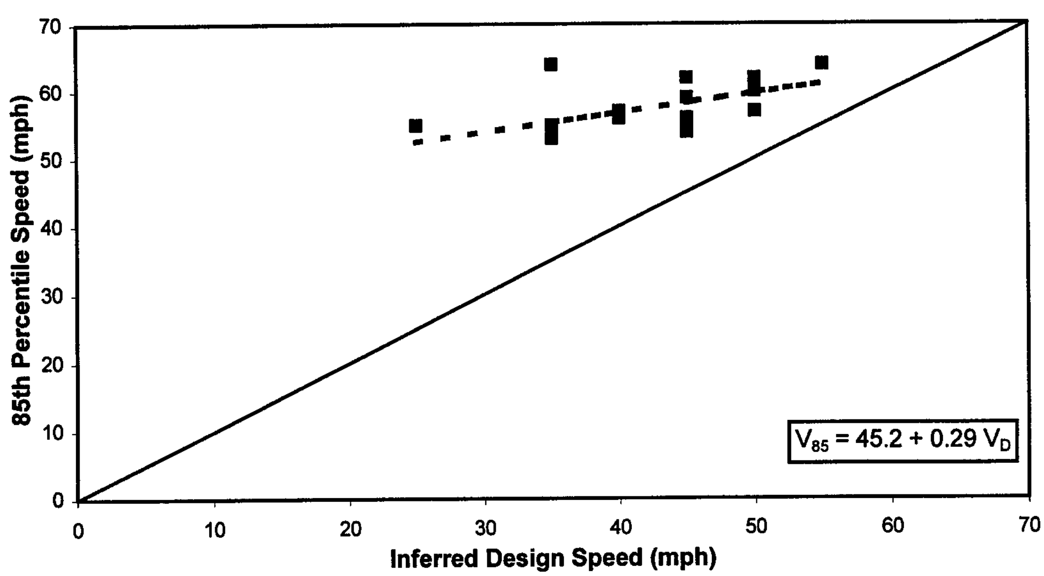


Figure 17. Mean difference in 85th percentile inferred design speed.

with the available sight distance at crest curves on two-lane *without shoulder* roadways; however, it should be noted that 85th percentile speeds are greater than crest curve design speeds for all design speeds less than about 55 mph. Figure 17 illustrates the regression equation for the two-lane *with* and *without shoulder* roadway categories.

These results are consistent with the findings of other research studies (*81, 82, 83*). Messer (*83*) found that 85th percentile operating speeds were higher than the design speed of 50 and 60 mph design speed roadways and less than the design speed of 70 mph design speed roadways. Krammes (*80*) found that horizontal curves with design speeds less than 50 mph had 85th percentile speeds that were consistently higher than the design speed of the curve, while horizontal curves with design speeds of 65 mph and above had 85th percentile speeds that were less than the design speed of the curve. McLean (*81*) found that horizontal curves with design speeds less than 90 km/h had 85th percentile speeds that were consistently higher than the design speed of horizontal curves.

Summary. The operating speed studies evaluated the relationship between design and operating speeds for crest vertical curves with limited sight distance. The questions studied included the mean reductions in operating speeds between control and crest sections, and the relationship between operating and design speeds on crest vertical curves. The mean speed reductions were compared statistically to determine any differences in operating speeds for various roadway types, traffic volumes, and design speed levels.

The study results indicated that both the 85th percentile and the mean operating speeds were well above the design speeds of the crest vertical curves in the range of conditions studied. The data from all of the roadways studied suggest that the lower the design speed the larger the difference between the 85th percentile speed and the design speed. Available sight distance appears to influence the mean speed reductions between the control and crest sections. As available sight distance is decreased, the mean speed reductions between the control and crest sections tend to increase; however, the reduction in speed is less than that suggested by the current AASHTO criteria.

For two-lane roadways without shoulder, a relationship exists that can be used to predict the 85th percentile speed using available sight distance at crest vertical curves. For the range of conditions studied, traffic volume and roadway type appeared to have little influence on the mean speed reductions between the control and crest sections. For the range of conditions studied, no significant were noted between 85th percentile speeds for the three states in this study.

CHAPTER 3
INTERPRETATION, APPRAISAL, AND APPLICATION

Sight distance is the length of roadway ahead that is visible to the driver. The minimum sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below average driver or vehicle to stop in this distance.

Currently, stopping sight distance is calculated using basic principles of physics and relationships between the various design parameters. AASHTO defines stopping sight distance as the sum of two components—brake-reaction distance (distance traveled from the instant of object detection to the instant the brakes are applied) and the braking distance (distance traveled from the instant the brakes are applied to when the vehicle is decelerated to a stop) (*1*). Conceptually, required stopping sight distances can be expressed by the following equation:

$$SSD = \text{Brake Reaction Distance} + \text{Braking Distance} \quad (5)$$

The findings presented in Chapter 2 illustrate the driver, vehicle, and roadway characteristics related to stopping sight distance. These findings provided the basis for recommending a stopping sight distance model that represents safe driving behavior and has been validated with field data. This chapter summarizes the key factors affecting stopping sight distance for geometric design for highways, the potential impacts on vertical curve design, and the safety implications of adopting the new model.

KEY FACTORS

One of the principal objectives in roadway design is to ensure that the driver can recognize hazardous obstacles in the roadway in time to take proper action and avoid an accident. The sight distance concept is used to provide a quantifiable parameter that can be related to the geometry of the roadway. This concept is based on a number of assumptions regarding particular hazards and corresponding driver behavior. The hazard is assumed to be an unexpected object of sufficient size to require the driver to take evasive action. The driver's action is assumed to be either braking to a stop or going around the obstacle.

Specific values are assumed for the driver's initial speed, perception-reaction time, deceleration, eye height, and visual capabilities (though in practice, a distribution of values would be present) to establish the geometry of the clear line of sight. Values selected for each parameter should represent the majority of drivers, vehicles, and roadways; however, selection of extreme values for every parameter is not appropriate, as the probability of their all occurring together is extremely low. For example, assuming independent events, the probability of occurrence of a driver with an 85th percentile speed and perception-reaction time, and a 15th percentile deceleration, and eye height is 0.0005; whereas, the probability of occurrence of a driver with a 90th percentile speed and perception-reaction time, and a 10th percentile deceleration, and eye height is 0.0001. It follows that the probability of occurrence is very small even if the events are dependent. Add the probability of there being an unexpected object in the roadway and it being located over the crest of a hill, and the probability of occurrence is even smaller.

As an objective, the parameter values assumed for design should lead to sight distances that produce a safe, comfortable, and aesthetically pleasing design. Additionally, the selected values should represent the majority of the driver, vehicle, and roadway population, but not the extremes. Finally and for consistency, the selected values should represent about the same percentiles from the underlying distribution of parameter values (e.g. the 85th or 90th percentile for parameters where larger values are critical and the 10th or 15th percentile where smaller values are critical). Percentile values from the literature and this research for the key factors affecting stopping sight distance and the design of horizontal and vertical curves are shown in Table 46.

Initial Speed

The initial speed for determining stopping sight distance requirements should be a speed that encompasses the desired speed of all but the small percentage of reckless drivers. This statement is consistent with AASHO's original definition of design speed in 1938 (4): "the maximum approximately uniform speed that will probably be adopted by the faster group of drivers but not, necessarily, by the small percentage of reckless ones." This concept was introduced so that an appropriate speed based on the reasonable desires of the majority

TABLE 46 Percentile values for factors effecting stopping sight distance and vertical curve design

Parameters	50th Percentile	15th Percentile	10th Percentile	5th Percentile
Initial Speed*		✓		
Perception-Brake Reaction Time*			2.0-2.5	2.5
Friction Coefficient	0.43	0.32		
Deceleration	0.42 g	0.36 g	0.34 g	0.32 g
Eye Height (Passenger Cars)	1150 mm	1090 mm	1080 mm	1060 mm
Taillight Height (Passenger Cars)	720 mm	660 mm	640 mm	610 mm
Headlight Height (Passenger Cars)	650 mm	610 mm	600 mm	590 mm

** Initial speed and perception-brake reaction times are 85th, 90th, and 95th percentile values.*

of drivers could be selected, and subsequent highway geometry features designed to accommodate that speed.

Design criteria accommodate near worst-case conditions; therefore, they incorporate considerable margins of safety. As a result, exceeding the design speed is not necessarily unsafe; however, it does reduce the margin of safety. During the past 50 years, vehicle and roadway design have improved and driver behavior has changed. As a result, the majority of today's drivers exceed the inferred design speed of horizontal and vertical curves; however, accident rates have not increased indicating that driver's speed selection is still reasonable and prudent. These findings suggest that the current AASHTO design criteria include a considerable margin of safety.

The term operating speed refers to the speed at which drivers are observed operating their vehicles. The 85th percentile of a sample of free flow speeds is the most frequently used descriptive statistic for defining the operating speed associated with a particular location or geometric feature. The 85th percentile speed is the speed at or below which 85 percent of drivers are operating their vehicles. It is thought to represent the reasonable desires of the majority of drivers, but not those of the small percentage of reckless drivers. It also is well understood and widely used by most state departments of transportation. Thus, it seems reasonable that the initial speed for determining stopping sight distance design criteria should be the roadway's anticipated 85th percentile operating speed.

Perception-Reaction Time

Perception-reaction time for stopping sight distance is defined as the interval of time between the moment the driver recognizes the existence of an object or hazard on the roadway ahead and the moment the driver applies the brakes or makes an evasive maneuver. This interval includes the time required to decide that a stop or path correction is necessary. Under most conditions, drivers must associate the object ahead with fixed objects adjacent to the roadway to decide whether the object is stationary or moving at a slow speed. The time required to make such decisions varies considerably, depending on the distance to the object, the visual capability of the driver, the speed with which the driver reacts, atmospheric visibility, the type and condition of the roadway, and the type, color, and condition of the hazard. Vehicle speed and the roadway environment also influence perception-reaction time.

Perception-Brake Reaction Time. The critical decision in a stopping sight distance situation is that a stop is necessary. As shown in Table 46, this research and other studies concur that under semi-alerted conditions, the 90th and 95th percentile perception-brake reaction times to an unexpected object in the roadway are approximately 2.0 and 2.5 sec, respectively. These values encompass the capabilities of most drivers (including those of older drivers). The distances traveled during perception-brake reaction are shown in Table 47.

TABLE 47 Perception-brake reaction distances (m)

Initial Speed (km/h)	Perception-Brake Reaction Time (sec)	
	2.0	2.5
50	28	35
60	33	42
70	39	49
80	44	56
90	50	63
100	56	69
110	61	76
120	67	83

TABLE 48 Maneuver sight distance (m)

Initial Speed (km/h)	Maneuver Time (sec)	Maneuver Sight Distance (m)
50	3.0	41.7
60	3.4	56.7
70	3.8	73.9
80	4.2	93.3
90	4.6	115.0
100	5.0	138.9

Note: Use where normal stopping sight distance is difficult or costly to achieve on consistent alignment sections below 100 km/h design speed when drivers are assumed to be alert or on isolated features up to 100 km/h when roadway width includes sufficient maneuver width.

Although it requires longer stopping sight distances, the 2.5 sec value is well established and should continue to be used for establishing desirable stopping sight distances. It should be noted, however, that at locations or geometric features where something other than stopping sight distance may be the appropriate design control, different perception-reaction times may be appropriate. For example, shorter perception-brake reaction times may be appropriate for traffic signal design where drivers are generally more alert, and longer perception-reaction times may be appropriate for intersection or interchange design where driver speed and path corrections may be required.

Equivalent Maneuver Time. An alternative decision to stopping is to drive around the obstacle. In fact, the literature indicates that a driver is more likely to avoid a hazard through lateral maneuvering than to bring the vehicle to a stop; therefore, an alternative to the provision of stopping sight distance is to ensure that

- The roadway is wide enough to provide a reasonable space for evasive action (which normally means ensuring that a crest vertical curve is not combined with minimum traffic lane and shoulder widths).
- The driver can perceive a hazard in time to take evasive action. Times from about 3.0 sec at 50 km/h and 5.0 sec at 100 km/h (*84*) appear to provide reasonable values (the speeds being the design speeds of corresponding horizontal geometry).

Estimated maneuver times and sight distances are shown in Table 48. The corresponding maneuver distances are longer than perception-brake reaction distances, but less than stopping distances because the response is an evasive maneuver rather than braking. For maneuver sight distance, the critical object is the pavement surface (zero object height), and the roadway's cross section should provide reasonable space for evasive action. Note that larger objects are visible sooner and provide longer maneuver times and sight distances. Note also that maneuver sight distances can only be extended up to 100 km/h because of the driver's visual capabilities and headlight sight distance limits.

Design Deceleration and Pavement Friction

Recent braking tests documented in the literature have found that modern passenger cars can achieve deceleration rates in excess of 1.0 g on good, dry pavements; however, the values used for design purposes should allow for the degradation of pavement skid resistance when wet, and for a reasonable amount of surface polishing. Design criteria should be based on deceleration values that encompass the decelerations selected by the majority of drivers stopping for an unexpected hazard in the roadway. Note that the pavement's skid resistance should be greater than the design decelerations.

The 15th percentile equivalent constant deceleration for drivers stopping on wet and dry pavements were 3.5 m/s^2 and 4.1 m/s^2, respectively. The 10th percentile values were 3.4 m/s^2 and 3.8 m/s^2, respectively. These values represent controlled stopping and, from a human behavioral standpoint, are near what is considered comfortable decelerations by many drivers. They are also relatively close to the skid resistance of a 15th percentile pavement. It should be noted that most drivers choose a higher deceleration and that most pavements have a higher skid resistance than these values.

Stopping Sight Distance

For level roadways, the two stopping sight distance components can be expressed mathematically as follows:

$$SSD = (0.278)(V)(t) + 0.039\ V^2/a \qquad (6)$$

where: SSD = stopping sight distance, m;
V = initial speed, km/h;
t = perception-brake reaction time, sec; and
a = deceleration, m/sec^2.

Stopping sight distances for different combinations of perception-brake reaction time and decelerations are shown in Table 49. For comparison, AASHTO stopping sight distances are shown in the second column. Note that most of the stopping sight distances in Table 49 are between the minimum and desirable values the 1994 AASHTO policy (*1*). For example, Figure 18 illustrates the resultant stopping sight distances for a 2.5 perception-brake reaction time and a 3.4 m/s^2 deceleration, and the minimum and desirable AASHTO values.

Headlight Sight Distance

The most common object on a rural highway is another vehicle that may or may not be stopped. Stopping at night is the critical condition when visibility is restricted because of darkness and the limits of the vehicle's headlights. Even if the other vehicle's lights are not illuminated, it will have retroreflective material at locations higher than the object height used in stopping sight distance calculations. As for small unilluminated objects, this research and others in the literature have shown that

- Only large, high contrast objects can be perceived at distances greater than 130 m on illuminated roadways.
- Significant improvement in visibility distances is unlikely because a five-fold light increase is necessary for a 15km/h increase in speed, and a ten-fold light increase is necessary for a 50 percent reduction in object size.
- The conflicting requirements of providing for driver visibility of the roadway ahead and minimizing glare to oncoming traffic sets limits on beam intensity.

A general limit of 130 m of sight distance is all that can be safely assumed for visibility of a small or low contrast object on a unilluminated roadway. This value corresponds to a satisfactory stopping distance for 90 km/h and a maneuver time of 5.0 sec at 100 km/h. Beyond this distance, only large, high contrast objects can be perceived in time to take evasive action on unlit roadways. The relatively small number of accidents involving objects in the roadway at night is probably due to the low probability of objects being in the roadway, and the factor of safety implicit in the various assumptions in the stopping sight distance model.

It is apparent that vehicle headlights limit the available sight distance on unlit roadways to about 130 m, assuming unlit obstacles. The only method for achieving full compatibility between theoretical day and night sight distances is by roadway lighting; however, two factors act to mitigate this imbalance. First, other vehicles constitute the large majority of hazards encountered on roadways. These vehicles are either illuminated or visible because of the required presence of retroreflective fittings. Second, because retroreflective materials respond to much lower light levels than nonreflective objects, they are perceived well outside the direct headlight beam. Thus, the provision of retroreflective buttons partially offsets the limitations of vehicle headlights.

Driver Eye Height

The height of the driver's eye above the road when seated in a vehicle varies with the type and condition of the vehicle, and the build of the driver and the driving position. The 10th and 15th percentile driver eye height values for passenger-car drivers are 1,080 mm and 1,090 mm, respectively. Both values are above the current AASHTO driver eye height of 1,070 mm. It should be noted that these eye heights are con-

TABLE 49 Stopping sight distances for different combinations of perception-brake reaction time and decelerations (m)

Initial Speed (km/h)	AASHTO SSD (m)	Perception-Brake Reaction Time (2.0 sec)			Perception-Brake Reaction Time (2.5 sec)		
		Deceleration (m/s^2)			Deceleration (m/s^2)		
		3.2	3.3	3.4	3.2	3.3	3.4
30	30-30	27.5	27.2	26.9	31.7	31.4	31.0
40	44-44	41.5	40.9	40.4	47.1	46.5	45.9
50	57-63	57.9	57.0	56.1	64.9	63.9	63.4
60	74-85	76.7	75.4	74.2	85.1	83.8	83.0
70	94-111	98.0	96.2	94.5	107.7	105.9	104.8
80	113-139	121.6	119.3	117.1	132.7	130.4	129.0
90	131-169	147.7	144.7	141.9	160.2	157.2	155.4
100	157-205	176.1	172.5	169.0	190.0	186.4	184.2
110	180-246	207.0	202.6	198.4	222.3	217.8	215.2
120	203-286	240.3	235.0	230.1	256.9	251.7	248.5

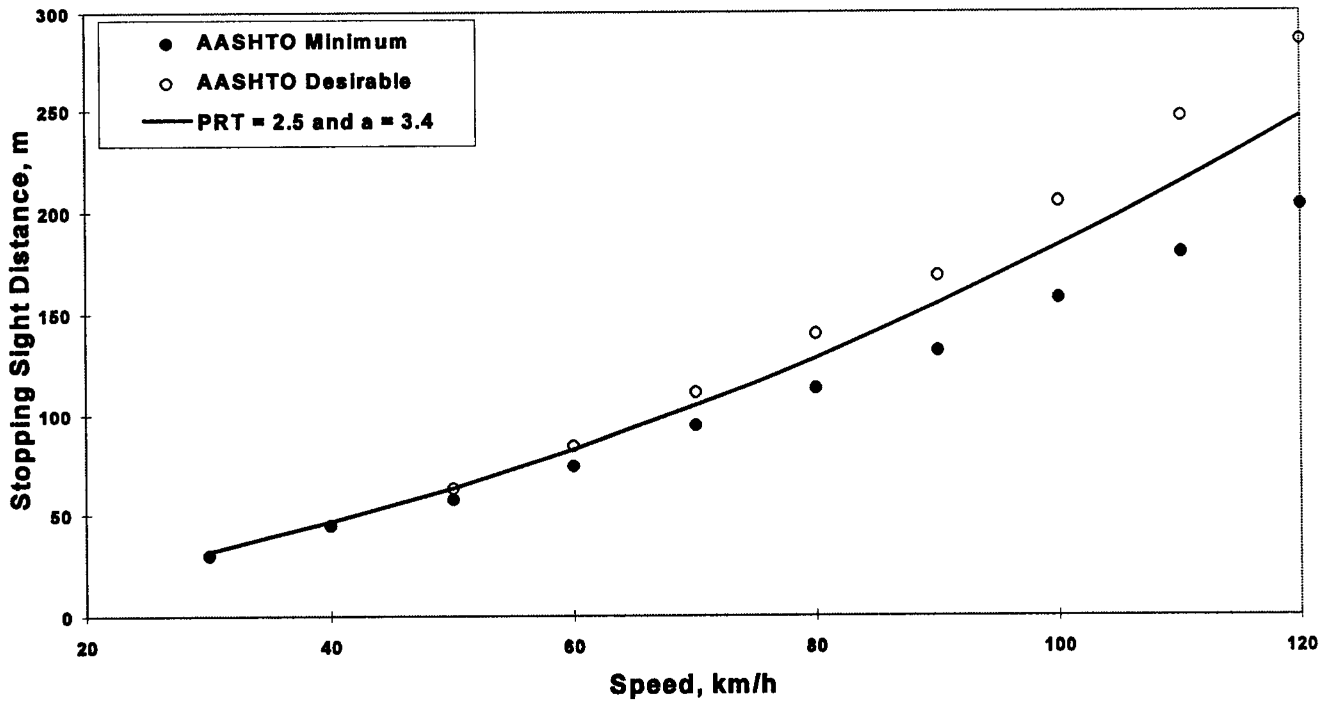


Figure 18. Stopping sight distances for perception-brake reaction time of 2.5 sec and deceleration of 3.4m/s².

servative given that the current vehicle fleet is approximately two-thirds passenger cars and one-third vans, pickups, and sport utility vehicles. The driver's eye height for a heavy vehicle is greater than for passenger-car drivers, and 2,300 mm (the 5th percentile value) or 2,600 mm (the 95th percentile value) are appropriate for purposes where heavy vehicles are the design vehicle.

Object Height

The object height to be used in calculation of stopping sight distance is a compromise between the length of sight distance and the cost of construction. Stopping is generally in response to another vehicle or large hazard in the roadway. To recognize a vehicle as hazard at night, a line of sight to its headlights or taillights would be necessary. Larger objects would be visible sooner and provide longer stopping distances. To perceive a very small hazard, for example, a surface obstruction, a zero object height would be necessary; however, at the required stopping sight distances for high speeds, small pavement variations and small objects (especially at night) may not be visible to most drivers. Thus, most drivers traveling at high speeds would have difficulty in stopping before reaching such a small obstruction.

The length of vertical curve required at crests increases significantly as the object height approaches zero and the general figure adopted which produces satisfactory design is between 150 (the current AASHTO object height) and 600 mm (headlight and taillight object height). Shorter object heights can be used in intersections or roadway sections susceptible to flooding or standing water. Shorter object heights can also be used at locations where there is a high probability of rocks or other debris being in the road.

A driver will most likely attempt to take evasive action rather than to stop for small objects on the roadway. Although not recommended as a design parameter, the time available to maneuver is a useful measure when examining variations of geometry in restricted situations or reconstruction projects. In this case, the appropriate object is the pavement surface.

VERTICAL CURVE DESIGN

The longitudinal profile of a road consists of a series of straight grades and vertical curves. Vertical curves smooth the passage of the vehicle from one grade to another and increase the available sight distance over crests at the junction of the grades. Convex curves are known as crest curves and concave curves are known as sag curves.

At crest vertical curves, the minimum length is determined by sight distance requirements, appearance requirements, comfort requirements, or drainage requirements. Use of above minimum lengths may increase the available passing sight distance on the approaches. At sag vertical curves, the length may be fixed by sight distance requirements, appearance requirements, comfort requirements, drainage requirements, headlight performance, or overhead restrictions to the line of sight.

Various curve forms exist that are suitable for use as vertical curves; however, the parabola has been used because of the ease of calculation. It is convenient to specify parabolic vertical curves by the length of curve required for a change in grade of 1.0 percent, this value being a constant for the parabola:

$$K = L/A \tag{7}$$

where: K = length required for a 1 percent change of grade;
L = length of vertical curve (m); and
A = algebraic change in grade (%).

The length of vertical curve for a given sight distance is given by the following expression:

$$L = AS^2/C \tag{8}$$

where: L = length of curve (m);
S = sight distance (m);
A = algebraic difference in grade (%); and
C = a constant dependent on the parameter values used to define the sight line.

The vertical curve parameter K may be substituted for L and A in the Equation 8 to give the following:

$$K = S^2/C \tag{9}$$

This value is a constant for a given sight distance and method of defining the sight line. The calculated length of vertical curve ($L = KA$) is usually rounded, and may be modified to comply with the appearance and comfort criteria. For crest curves, the sight line constant C to be used with this expression is given by

$$C = 200\left(\sqrt{h_e} + \sqrt{h_o}\right)^2 \tag{10}$$

where: h_e = height of the eye above the road (m); and
h_o = object cut-off height above the road (m).

Values of C for selected values of h_e and h_o are given in Table 50. Values of K for stopping sight distances from the recommended model and selected values of C are given in Tables 51 and 52. Note that the K values for 400 and 600 mm objects are near or slightly less than AASHTO minimum values.

Length of Vertical Curves—Appearance Criterion. For a particular design speed, the required length of crest curve is usually governed by the sight distance requirements; however, appearance considerations may suggest longer lengths for small changes in grade. For very small changes in grade, a vertical curve has little influence on the roadway's appearance and may be omitted. Short vertical curves detract from the roadway's appearance with any significant change of grade. This distraction is particularly evident on high-speed roads and on sag curves.

Most states use a minimum length of vertical curve, expressed as either a single value, a range for different design speeds, or a function of A. Values currently being used range from 30 to 100 m. To recognize the distinction in design speed and to approximate current practice, minimum lengths of crest vertical curves are expressed as 0.6 times the design speed (*1*). Table 53 shows minimum crest vertical curve lengths for satisfactory appearance. Longer curves may be preferred where they can be used without conflict with the other design requirements (e.g., passing) and provide a better fit to the topography. For satisfactory appearance of sag vertical curves, the minimum length is approximated as 30 times the algebraic difference in grade.

Length of Vertical Curves—Comfort Criterion. Discomfort is felt by a person subjected to rapid changes in vertical acceleration. To reduce discomfort when passing from one grade to another, it is common to limit the vertical acceleration generated on a sag curve to a value less than 0.03 g, where g is the acceleration because of gravity. On low-speed roads and at intersections, a limit of 0.10 g may be used. This latter value is also the limit for vertical acceleration on crest curves. The vertical component of the acceleration normal to the curve, when traversing the path of a parabolic curve at uniform speed, is given by

$$a = v^2A/100L = v^2/100K \tag{11}$$

TABLE 50 Sight line constants for crest vertical curves

Object Height, h_o (mm)	Driver Eye Height, h_e (mm)		
	1090 mm (15th percentile)	1080 mm (10th percentile)	1060 mm (5th percentile)
600 (90th percentile headlight)	662	658	651
400 (minimum legal taillight)	562	559	553
150 (1994 AASHTO object)	410	407	402
0 (roadway surface)	218	216	212

TABLE 51 *K* **values for crest vertical curves, perception-brake reaction time of 2.0 sec and deceleration of 3.3 m/s²**

Design or Initial Speed (km/h)	Driver Eye and Object Heights				
	h_e=1070 mm* h_o= 150 mm C=401	h_e=1080 mm h_o=600 mm C=658	h_e=1080 mm h_o=400 mm C=559	h_e=1080 mm h_o=150 mm C=407	h_e=1080 mm h_o= 0 mm C=216
30	3-3	1.1	1.3	1.8	3.4
40	5-5	2.5	3.0	4.8	7.0
50	9-10	4.9	5.8	8.0	15.0
60	14-18	8.6	10.2	14.0	26.3
70	22-31	14.1	16.5	22.7	42.8
80	32-49	21.6	25.5	35.0	65.9
90	43-71	31.8	37.5	51.4	96.9
100	62-105	45.2	53.2	73.1	137.7
110	80-151	62.4	73.4	100.8	190.0
120	102-202	83.9	98.8	137.8	255.7

* *Current AASHTO values*

TABLE 52 *K* **values for crest vertical curves, perception-brake reaction time of 2.5 sec, and deceleration of 3.4 m/s²**

Design or Initial Speed (km/h)	Driver Eye and Object Heights				
	h_e=1070 mm* h_o= 150 mm C=401	h_e=1080 mm h_o=600 mm C=658	h_e=1080 mm h_o=400 mm C=559	h_e=1080 mm h_o=150 mm C=407	h_e=1080 mm h_o= 0 mm C=216
30	3-3	1.5	1.7	2.4	4.5
40	5-5	3.2	3.8	5.2	9.8
50	9-10	6.0	7.1	9.8	18.4
60	14-18	10.3	12.2	16.7	31.5
70	22-31	16.5	19.4	26.7	50.3
80	32-49	25.0	29.4	40.4	76.1
90	43-71	36.2	42.7	58.6	110.4
100	62-105	50.8	59.9	82.2	154.9
110	80-151	69.4	81.7	112.2	211.4
120	102-202	92.5	108.9	149.6	281.8

* *Current AASHTO values*

TABLE 53 Length of vertical curves—appearance criterion

Design Speed (km/h)	Maximum Grade Change Without Vertical Curve (%)	Min. Length of Vertical Curve for Satisfactory Appearance (m)
40	1.0	24
60	0.8	36
80	0.6	48
100	0.4	60
120	0.2	72

Note: In practice, vertical curves are frequently provided for all changes in grade.

TABLE 54 ***K*** **values for vertical curves—comfort criteria**

Design Speed (km/h)	K = length of vertical curve in meters for a 1 percent change in grade	
	a = 0.03 g*	a = 0.10 g*
40	4	1.5
60	8	3
80	15	5
100	23	8
120	33	12

* *g = acceleration due to gravity, 9.81 m/s²*

where: a = vertical component of acceleration (m/s^2); and
v = speed of the vehicle (m/s);
$K = L/A$ as previously defined.
A = algebraic difference in grade (%); and
L = length of vertical curve (m).

The similarity of this formula to the expression for acceleration normal to the curve on a circular path $a = V^2/R$ leads to the use of *equivalent radius* of a vertical curve $R = 100K$. For large radii, the radius of a circular curve would differ very little from the parabolic curve. Some designers find the concept of an equivalent radius useful, and a satisfactory alternative to the use of K values. Rounded values for K for specific design speeds and vertical accelerations of 0.03 g and 0.10 g are shown in Table 54.

Sag Curves

Sight distance on sag curves is not restricted by the vertical geometry in daylight conditions or at night with full roadway lighting, unless overhead obstructions are present. Under night conditions on unlit roads, limitations of vehicle headlights restrict sight distance to approximately 130 m on crest curves. Therefore, sag curves should be designed to achieve the comfort criterion (0.03 g vertical acceleration or 0.10 g when economics dictate).

For headlight sight distance, the sight line constant C to be used is given by

$$C = 200\,(h + S \tan q) \tag{12}$$

where: h = mounting height of headlights (m);
S = sight distance (m); and
q = elevation of angle beam (+ upwards).

For a headlight mounting height of 600 mm and zero degree upward elevation, this equation yields $C = 120$. For any design speed, the length of vertical curve will be determined by either the comfort criteria or for headlight sight distance up to 120 m on more important roads. These values are shown in Table 55.

Overhead obstructions such as road or rail overpasses, sign bridges, or even overhanging trees may limit the sight

TABLE 55 Length of sag curves—comfort and headlight criteria

Design Speed (km/h)	K = Length of Vertical Curve in Meters for 1% Change in Grade				Headlight Maneuver Time (s)
	Comfort Considerations		Headlight Considerations (C = 120)		
	General Design a=0.03 g	Special Cases a=0.10 g	Sight Distance (m)	K	
50	7	2	50	21	3.0
60	9	3	65	35	3.4
70	13	4	85	60	3.8
80	17	5	105	92	4.2
90	21	6	120	120	4.6
100	26	8	188	188	5.0
110	32	10			
120	38	12			

TABLE 56 Sight line constant for overhead obstructions

Obstruction Height H (m)	Sight Line Constant (C)
4.0	1980
4.5	2390
5.0	2800
5.5	3200
6.0	3610

distance available on sag vertical curves. With the minimum overhead clearances normally required on public roadways, these obstructions should not interfere with minimum stopping sight distances; however, some consideration may be needed near the upper limit of stopping distance (including sight distance to intersections). The sight line constant C for this situation is given by

$$C = 200\left(\sqrt{H - h_e} + \sqrt{H - h_o}\right)^2 \tag{13}$$

where: H = height of overhead obstruction; and
h_e = eye height and object height
h_o = as before.

Using a driver eye height of 2,300 mm and an object height of 600 mm (commercial vehicle eye height to vehicle taillight), the sight line constants for a range of overhead obstruction heights are given in Table 56. Intermediate values may be interpolated.

SAFETY IMPLICATIONS

The current and recommended stopping sight distance criteria incorporate considerable margins of safety. Although excessive speeds, slow reactions, slippery pavements, or small cars may use up a part of the safety margin, it is extremely unlikely that all of these variables will be critical at the same time. The large majority of drivers select speeds that are reasonable and prudent on the basis of the roadway's appearance, and unless there are hidden hazards such as intersections or sharp horizontal curves, stopping sight distances less than the minimum design criteria do not seem to cause an increase in accident rates. Thus, even though the recommended stopping sight distance criteria will result in shorter vertical curve lengths, it should not result in increased accident rates.

In support of these observations, this research and the literature do not show an increase in accident rates for moderate reductions in stopping sight distance on rural high-speed highways. This research also showed accident rates on rural two-lane highways with limited stopping sight distance crest curves are similar to accident rates on all two-lane highways and that moderate reductions in stopping sight distance do not appear to cause a problem for large trucks or older drivers. Finally, this research showed that there are no tort problems associated with the current stopping sight distances.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations of this research address a revised model for determining required stopping sight distances for roadway design. The revised model is similar to AASHTO's current stopping sight distance model and suitable for inclusion in AASHTO's *A Policy on Geometric Design for Highways and Streets.*

This chapter provides a brief summary of each conclusion and recommendation from the research.

CONCLUSIONS

A revised stopping sight distance model based on driver behavior and vehicle and roadway characteristics was developed as a product of this research. Parameters for the revised model have been validated with field data and represent safe driving behavior. The parameters also reflect driver, vehicle, and roadway limitations related to the stopping sight distance situation.

The revised model is intended as the design control for locations or geometric features where stopping sight distance is the appropriate criterion, specifically vertical curves on tangents and horizontal curves near lateral obstructions. In general, the model is intended for use where speed and path changes are not required. At these locations, intersection or decision sight distance may be the appropriate control.

The following sections describe the revised model and the recommended parameter values for use in highway geometric design.

Revised Stopping Sight Distance Model

The revised stopping sight distance model developed as a product of this research is similar to the existing AASHTO model, but with initial speed equal to the design speed and design deceleration substituted for friction coefficient. Stopping sight distance is still the sum of two components—brake-reaction distance (distance traveled from the moment an unexpected object could be sighted to the moment the brakes are applied) and the braking distance (distance traveled from the moment the brakes are applied to the moment the vehicle is decelerated to a stop).

Conceptually, stopping sight distance can still be expressed by the following equation:

$$SSD = \text{Reaction Distance} + \text{Braking Distance} \qquad (14)$$

For level roadways, these two components can be mathematically expressed as follows:

$$SSD = 0.278Vt + 0.039V^2/a \qquad (15)$$

where: SSD = stopping sight distance, m;
V = initial speed, km/h;
t = driver perception-brake reaction time, sec; and
a = driver deceleration, m/sec^2.

As with the current AASHTO model, the minimum stopping sight distance, driver eye height, and object height values are used to calculate the minimum length of vertical curve required and the minimum rate of curvature or lateral clearance required on horizontal curves. This required length of curve is such that, at a minimum, the stopping sight distance calculated by Equation 15 is available at all points on the curve.

Initial Speed

This research and other studies documented in the literature show that many drivers exceed the inferred design speed (design speed calculated using current criteria and existing geometry) of horizontal and vertical curves. The consistency of these results does not support the use of initial speeds less than the roadway's design speed for determining stopping sight distance requirements.

Initial speeds for determining stopping sight distance requirements should be a speed that encompasses the desired speed of most drivers; e.g., the roadway's operating or 85th percentile free flow speed. When a roadway's operating speed is expected to change over time, the highest anticipated operating speed should be used to determine stopping sight distance requirements.

Perception-Brake Reaction Time

This research and other studies documented in the literature show that AASHTO's 2.5 sec perception-brake reaction time for stopping sight distance situations encompasses the capabilities of most drivers (including those of older drivers). In fact, the data shows that 2.0 sec exceeds the 85th percentile SSD perception-brake reaction time for all drivers, and 2.5 sec exceeds the 90th percentile SSD perception-brake reaction time for all drivers.

Thus, the 2.5 sec value should be used for determining required stopping sight distances; however, it should be noted that at locations where stopping sight distance is not the appropriate control, different perception-reaction times may be appropriate. For example, shorter perception-brake reaction times may be appropriate for traffic signal design where change intervals are expected, and longer perception-brake reaction times may be appropriate for intersection or interchange design where driver speed and path corrections are unexpected.

Design Deceleration

This research and other studies documented in the literature show that most drivers choose decelerations greater than 5.6 m/sec^2 when confronted with the need to stop for an unexpected object in the roadway. Approximately 90 percent of all drivers choose decelerations that are greater than 3.4 m/sec^2. These decelerations are within drivers' capability to stay within their lanes and maintain steering control during braking maneuvers on wet surfaces.

Thus, 3.4 m/sec^2 (a comfortable deceleration for most drivers) is recommended as the deceleration threshold for determining required stopping sight distance. Implicit in this deceleration threshold is the requirement that the vehicle braking system and pavement friction values are at least equivalent to 3.4 m/sec^2 (0.34 g). Skid data show that most wet pavement surfaces on state maintained roadways exceed this threshold. Braking data show that most vehicle braking systems can exceed the skidding friction values for the pavement.

Recommended Stopping Sight Distances

The recommended stopping sight distances for design are based on below average drivers detecting an unexpected object in the roadway and stopping a vehicle before striking the object. The recommended values are shown in Table 57. The values in the bottom five rows of the table represent those stopping sight distances beyond the driver's visual capabilities for detecting small objects (150 to 200 mm objects) during the day and large, low contrast objects at night.

For comparison purposes, AASHTO's 1994 design stopping sight distances are shown in Table 58 and Figure 19. Note that the recommended values are approximately midway between the 1994 minimum and desirable values for all initial speeds.

Eye Heights and Object Heights

This research and other studies documented in the literature show that more than **90 percent** of all passenger-car driver eye heights exceed 1,080 mm. This eye height encompasses an even larger proportion of the vehicle fleet when trucks and multipurpose vehicles are included in the population. Thus, 1,080 mm is recommended as the driver eye height for determining required stopping sight distances.

TABLE 57 Recommended stopping sight distances for design

Initial Speed (km/h)	Perception-Brake Reaction		Deceleration (m/s^2)	Braking Distance (m)	Stopping Sight Distance for Design (m)
	Time (s)	Distance (m)			
30	2.5	20.8	3.4	10.2	31.0
40	2.5	27.8	3.4	18.2	45.9
50	2.5	34.7	3.4	28.4	63.1
60	2.5	41.7	3.4	40.8	82.5
70	2.5	48.6	3.4	55.6	104.2
80	2.5	55.6	3.4	72.6	128.2
90	2.5	62.5	3.4	91.9	154.4
100	2.5	69.4	3.4	113.5	182.9
110	2.5	76.4	3.4	137.3	213.7
120	2.5	83.3	3.4	163.4	246.7

TABLE 58 Current stopping sight distances for design (AASHTO 1994)

Design Speed (km/h)	Assumed Speed for Condition (km/hr)	Brake Reaction Time (s)	Brake Reaction Distance (m)	Coefficient of Friction f	Braking Distance on Level (m)	Stopping Sight Distance for Design (m)
30	30-30	2.5	20.8-20.8	0.40	8.8-8.8	29.6-29.6
40	40-40	2.5	27.8-27.8	0.38	16.6-16.6	44.4-44.4
50	47-50	2.5	32.6-34.7	0.35	24.8-28.1	57.4-62.8
60	55-60	2.5	38.2-41.7	0.33	36.1-42.9	74.3-84.6
70	63-70	2.5	43.7-48.6	0.31	50.4-62.2	94.1-110.8
80	70-80	2.5	48.6-55.5	0.30	64.2-83.9	112.8-139.4
90	77-90	2.5	53.5-62.5	0.30	77.7-106.2	131.2-168.7
100	85-100	2.5	59.0-69.4	0.29	98.0-135.6	157.0-205.0
110	91-110	2.5	63.2-76.4	0.28	116.3-170.0	179.5-246.4
120	98-120	2.5	68.0-83.3	0.28	134.9-202.3	202.9-285.6

This research showed that accidents involving small objects are extremely rare events and almost never result in injuries to vehicle occupants. This research also showed that small objects are beyond most drivers' visual capabilities at the stopping sight distances required for most rural highways, especially at night. Specifically, small objects are beyond most drivers' visual capabilities at distances greater than 130 m, and except for reflective or illuminated objects. Large, low contrast objects are beyond most drivers' nighttime visual capabilities at distances greater than 100 m.

More realistic and frequent hazards to drivers are large animals (cattle, deer, etc.) and other vehicles. From a potential hazard standpoint, the critical object for stopping sight distance should be the smallest visible object during the day and at night that represents a hazard to the driver, that is, the taillight or headlight height of another vehicle.

Approximately 95 percent of the taillight heights and 90 percent of the headlight heights exceed 600 mm. Additionally, this research showed that accidents with smaller objects are extremely rare and of low severity in nature. Thus, 600 mm is

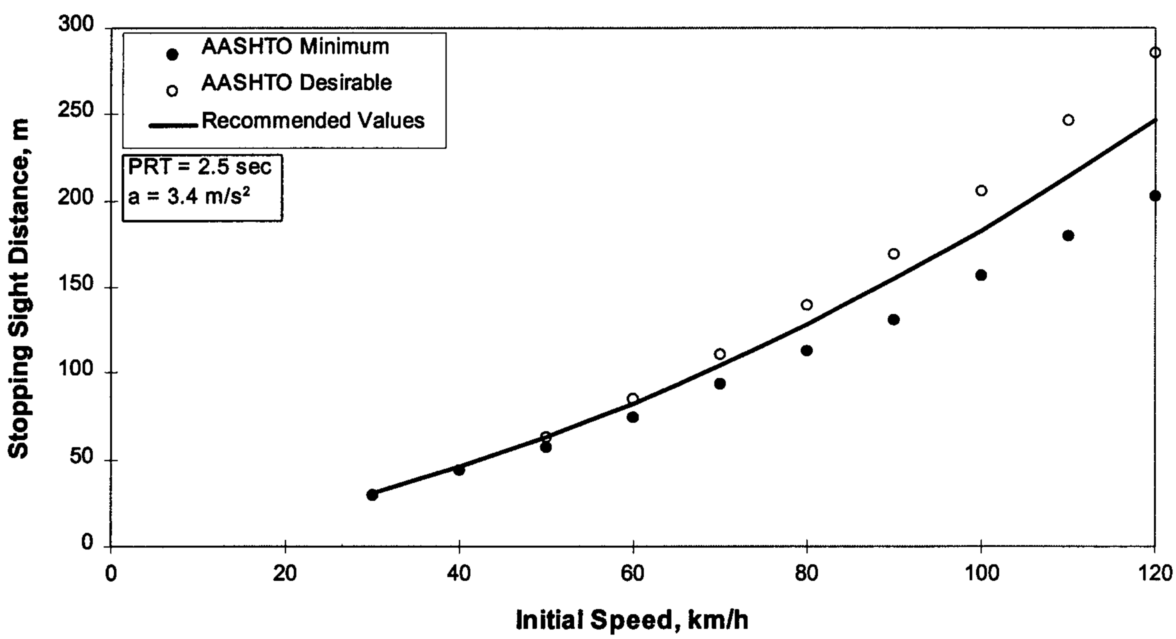


Figure 19. Comparison of 1994 AASHTO and recommended values for stopping sight distance.

recommended as the appropriate object height for determining required stopping sight distances except in those locations where the probability of rocks or other debris in the roadway is high. In those locations, a shorter object height is appropriate.

Design Controls for Vertical Curves

Minimum lengths of vertical curves are determined by the provision of ample sight distance for the initial speed before braking and the controlling situation. Required stopping sight distances should be the control where stopping sight distance is the appropriate control, and intersection, decision, or passing sight distance should be the control where speed reduction or path correction is the appropriate control. The largest control value determines the minimum length of vertical curve.

Crest Vertical Curves. When eye height and object height are 1,080 mm and 600 mm, respectively, as used for stopping sight distance, the required length of curve (L) in terms of algebraic difference in grade (A) and sight distance (S) can be computed as follows:

For S less than L,

$$L = AS^2/658 \tag{16}$$

For convenience in describing different combinations of approach and departure grades, the quantity L/A, termed "K" is the horizontal distance to effect a 1 percent change in gradient, that is, a measure of curvature. Table 59 shows the computed K values for lengths of crest vertical curves as required for the stopping sight distances shown in Table 57.

For comparison purposes, the 1994 AASHTO design controls for crest vertical curves are shown in Table 60 and Figure 20. Note that the recommended K values for crest vertical curves are slightly below the 1994 minimum values for all initial speeds.

Sag Vertical Curves. Headlight sight distance generally controls the minimum length of sag vertical curves. In this case, a headlight height of 600 mm and a 1 degree upward divergence of the light beam from the longitudinal axis of the vehicle is used to define the driver's line of sight. The following equation shows the relationship between S, L, and A, using S as the distance between the vehicle and the point where the 1 degree upward angle of light intersects the surface of the road:

For S less than L,

$$L = AS^2/(120 + 3.5S) \tag{17}$$

Table 59 shows the computed K values for lengths of sag vertical curves as required for the stopping sight distances shown in Table 57.

For comparison purposes, the 1994 AASHTO design controls for sag vertical curves are shown in Table 61 and Figure 21. Note that the recommended values are between the 1994 minimum and desirable values for all initial speeds.

Design Controls for Horizontal Curves

Minimum rate of curvature or lateral clearance for horizontal curves is determined by providing ample sight distance

TABLE 59 Recommended design controls for vertical curves

Initial Speed (km/h)	Stopping Sight Distance for Design (m)	Rate of Vertical Curvature, K [length (m) per % of A]	
		Crest Curves	Sag Curves
30	31.0	2	5
40	45.9	4	8
50	63.1	7	12
60	82.5	11	17
70	104.2	17	23
80	128.2	25	29
90	154.4	37	37
100	182.9	51	45
110	213.7	70	53
120	246.7	93	62

TABLE 60 Design controls for crest vertical curves (AASHTO 1994)

Design Speed (km/h)	Assumed Speed for Condition (km/h)	Coefficient of Friction f	Stopping Sight Distance for Design (m)	Rate of Vertical Curvature, K (length (m) per % of A)	
				Computed	Rounded for Design
30	30-30	0.40	29.6-29.6	2.17-2.17	3-3
40	40-40	0.38	44.4-44.4	4.88-4.88	5-5
50	47-50	0.35	57.4-62.8	8.16-9.76	9-10
60	55-60	0.33	74.3-84.6	13.66-17.72	14-18
70	63-70	0.31	94.1-110.8	21.92-30.39	22-31
80	70-80	0.30	112.8-139.4	31.49-48.10	32-49
90	77-90	0.30	131.2-168.7	42.61-70.44	43-71
100	85-100	0.29	157.0-205.0	61.01-104.02	62-105
110	91-110	0.28	179.5-246.4	79.75-150.28	80-151
120	98-120	0.28	202.9-285.6	101.90-201.90	102-202

for the initial speed before braking and the controlling situation. Required stopping sight distances should be the control where stopping sight distance is the appropriate control, and intersection, decision, or passing sight distance should be the control where speed reduction or path correction is the appropriate control. The largest control value determines the minimum rate of curvature or lateral clearance.

In design of horizontal curves, the sight line is a chord of the curve, and the applicable sight distance is measured along the centerline of the inside lane around the curve. The middle ordinates (M) for clear sight areas to satisfy required stopping sight distances (S) for curves of different radii (R) can be expressed as follows:

$$M = R\left[\left(1 - \cos\frac{28.65S}{R}\right)\right] \quad (18)$$

This formula applies only to circular curves longer than the sight distance for the initial speed. For any initial speed, the relationship between radius and middle ordinate is a straight line. The required middle ordinates to provide the stopping sight distance shown in Table 59 are shown in Table 62 and Figure 22.

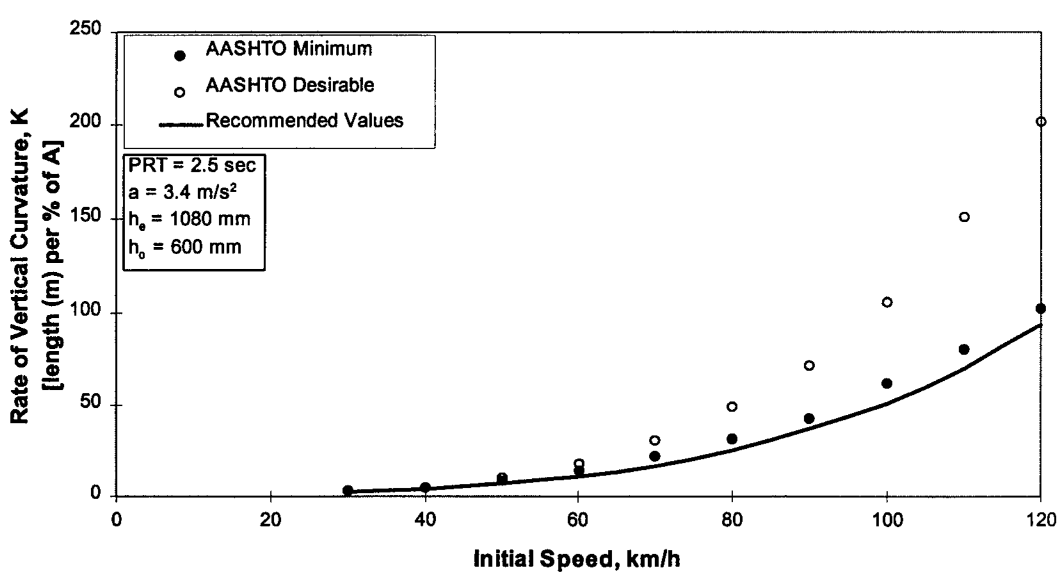


Figure 20. Comparison of 1994 AASHTO and recommended K values for crest vertical curves.

TABLE 61 Design controls for sag vertical curves (AASHTO 1994)

Design Speed (km/h)	Assumed Speed for Condition (km/h)	Coefficient of Friction f	Stopping Sight Distance for Design (m)	Rate of Vertical Curvature, K (length (m) per % of A)	
				Computed	Rounded for Design
30	30-30	0.40	29.6-29.6	3.88-3.88	4-4
40	40-40	0.38	44.4-44.4	7.11-7.11	8-8
50	47-50	0.35	57.4-62.8	10.20-11.54	11-12
60	55-60	0.33	74.3-84.6	14.45-17.12	15-18
70	63-70	0.31	94.1-110.8	19.62-24.08	20-25
80	70-80	0.30	112.8-139.4	24.62-31.86	25-32
90	77-90	0.30	131.2-168.7	29.62-39.95	30-40
100	85-100	0.29	157.0-205.0	36.71-50.06	37-51
110	91-110	0.28	179.5-246.4	42.95-61.68	43-62
120	98-120	0.28	202.9-285.6	49.47-72.72	50-73

For comparison purposes, the middle ordinates to provide the 1994 AASHTO minimum stopping sight distances are shown in Table 63. Note that the middle ordinates based on the recommended stopping sight distances are larger than those based on AASHTO minimum stopping sight distances; however, they are smaller than those based on AASHTO desirable stopping sight distances.

Safety Considerations

Safety is of the utmost importance when designing roadways; therefore, any recommended guidelines should result in designs that do not create hazards or unsafe conditions. This research and other studies show that for moderate reductions in available stopping sight distance, there are no noticeable safety problems associated with crest curves on rural high-speed highways. This research also showed that there are no tort problems associated with current stopping sight distances.

Specifically, this research shows that accident rates on rural two-lane highways with limited stopping sight distance crest vertical curves (curves with stopping sight distances slightly below current criteria) are similar to accident rates on all two-lane highways. Additionally, crest vertical curves with moderate reductions in stopping sight distance do not appear to cause a safety problem for large trucks or older

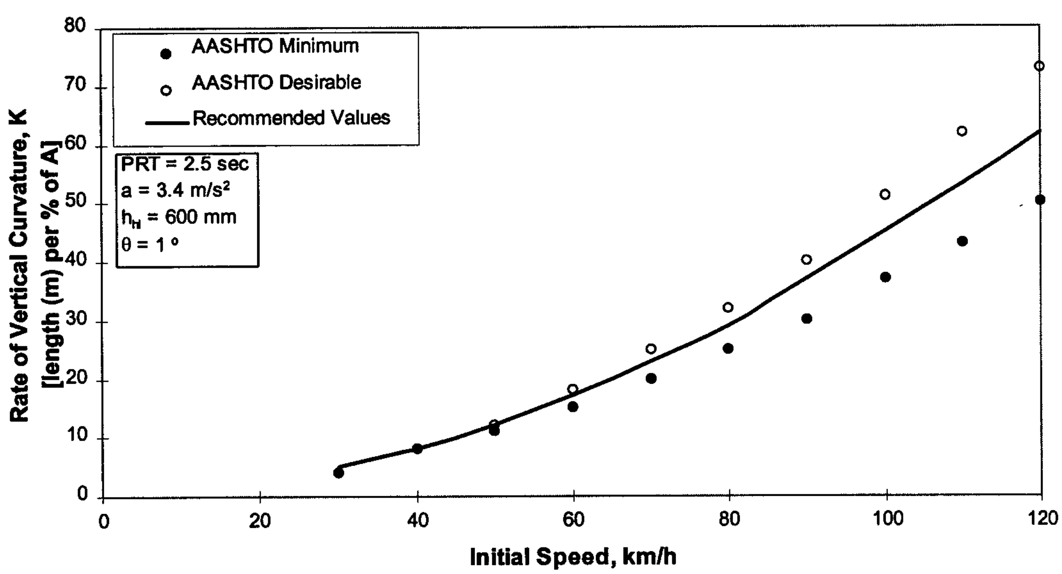


Figure 21. Comparison of 1994 AASHTO and recommended K values for sag vertical curves.

TABLE 62 Required middle ordinates for various initial speeds and horizontal curve radii

Initial Speed (km/h)	Stopping Sight Dist. (m)	Minimum* Radius (R)	Radius, R, Centerline of Inside Lane (m)						
			80	100	150	300	500	1000	1500
30	31.0	25	--	--	--	--	--	--	--
40	45.9	45	3.3	2.6	--	--	--	--	--
50	63.1	75	6.1	4.9	3.3	--	--	--	--
60	82.5	115	--	--	5.6	2.8	--	--	--
70	104.2	160	--	--	--	4.5	2.7	--	--
80	128.2	210	--	--	--	6.8	4.1	2.1	--
90	154.4	275	--	--	--	9.9	6.0	3.0	2.0
100	182.9	360	--	--	--	--	8.3	4.2	2.8
110	213.7	455	--	--	--	--	11.4	5.7	3.8
120	246.7	595	--	--	--	--	--	7.6	5.1

* Minimum radius when $e_{max} = 0.10$

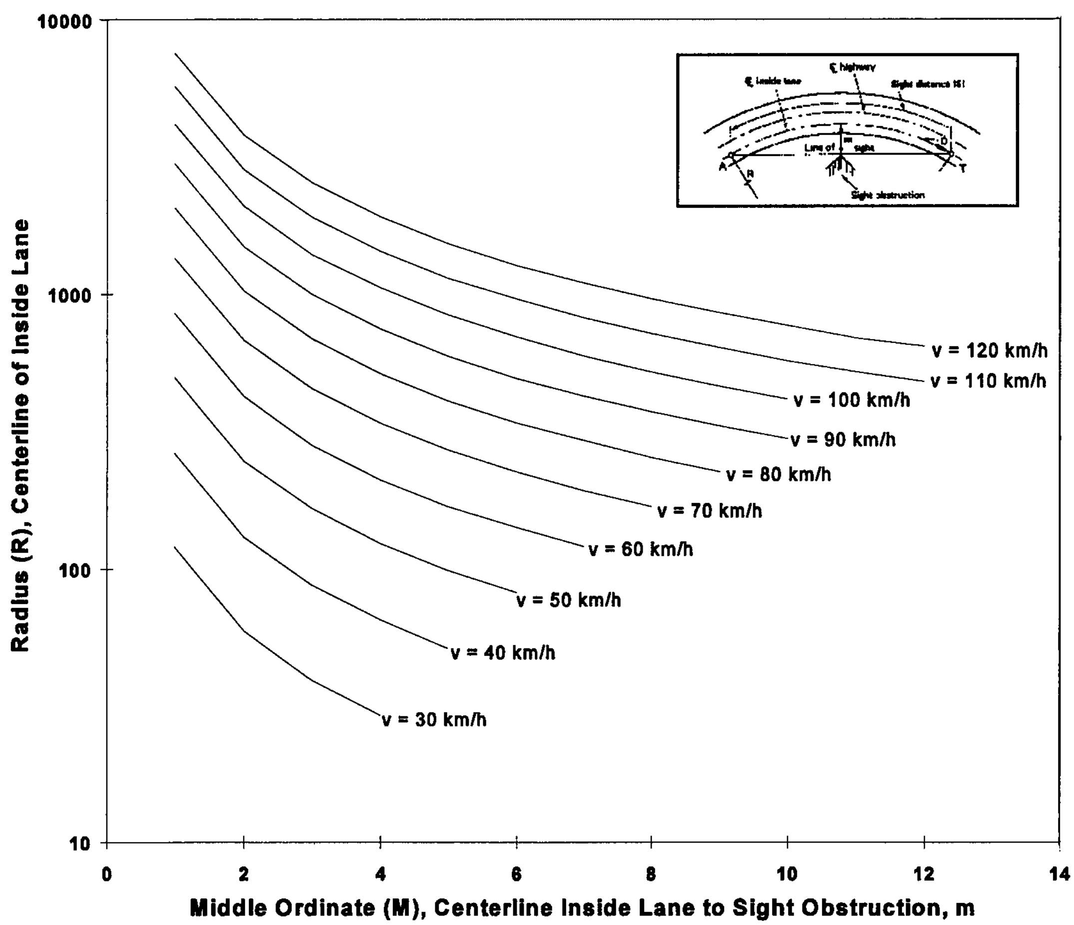


Figure 22. Relationship between radius and value of middle ordinate necessary to provide stopping sight distance on horizontal curves.

TABLE 63 Required middle ordinates for various design speeds and horizontal curve radii (AASHTO 1994, desirable values)

Design Speed (km/h)	Stopping Sight Dist. (m)	Minimum* Radius (R)	Radius, R, Centerline of Inside Lane (m)						
			80	100	150	300	500	1000	1500
30	29.6	25	--	--	--	--	--	--	--
40	44.4	45	3.1	2.5	--	--	--	--	--
50	62.8	75	6.1	4.9	3.3	--	--	--	--
60	84.6	115	--	--	5.9	3.0	--	--	--
70	110.8	160	--	--	--	5.1	3.1	--	--
80	139.4	210	--	--	--	8.1	4.9	2.4	--
90	168.7	275	--	--	--	--	7.1	3.6	2.4
100	205.0	360	--	--	--	--	--	5.2	3.5
110	246.4	455	--	--	--	--	--	7.6	5.1
120	285.6	595	--	--	--	--	--	--	6.8

* Minimum radius when $e_{max} = 0.10$

drivers. Finally, most accidents with limited stopping sight distance as a possible contributing factor occurred on vertical curves with stopping sight distances of 120 m or less and involved another vehicle entering or exiting an intersection or driveway.

It should be noted that the revised model is intended for use in designing those curves where stopping sight distance controls. If speed or path corrections are needed in addition to stopping sight distance, intersection or decision sight distance may control the design of curves in combination with other roadway features.

RECOMMENDATIONS

The revised stopping sight distance model and parameter values represent driver capabilities and performance that can be validated with field data and defended as representative of safe driving behavior. It is similar to the existing AASHTO model so department of transportation personnel will not need to learn a new methodology. The revised model does recommend stopping sight distances and crest curve lengths that are longer than the current minimum design values; however, it should be noted that these recommendations are based on driver capabilities and performance rather than on a need for additional safety.

Thus, it is recommended that the revised model, associated documentation, and suggested changes to the *Green Book* be presented to the AASHTO Task Force on Geometric Design for possible inclusion in the next update of the *Green Book*. It also is recommended that a research project be initiated to address the differences in AASHTO definitions of design and operating speed because of their importance in geometric design.

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APPENDIXES A-H

Appendixes A through H as submitted by the research agency are not published herein but are available for loan on request to the NCHRP.

APPENDIX A	Background
APPENDIX B	Vehicle and Roadway Performance
APPENDIX C	Driver Braking Performance
APPENDIX D	Driver Visual Capabilities
APPENDIX E	Driver Eye and Vehicle Heights
APPENDIX F	Safety Effects
APPENDIX G	Operational Effects
APPENDIX H	Tort Liability Issues

APPENDIX I

RECOMMENDED REVISIONS TO THE AASHTO GREEN BOOK

This appendix presents recommended revisions to the design policies in the AASHTO publication, *A Policy on Geometric Design of Highways and Streets*, known as the *Green Book (1)*. The *Green Book* is the primary geometric design guide used by many transportation departments and other geometric design practitioners.

The recommended revisions are derived from the findings of this research, and other than a few minor editorial suggestions, do not address topics other than stopping sight distances. Sections of the *Green Book* potentially affected by the research findings include portions of Chapters II and III. The remainder of this appendix presents the current text of the *Green Book* with recommended changes. Deletions of text are shown as ~~strikeouts~~ and additions to the text are shown in **bold.**

The *Green Book* text presented in this appendix is based on the 1994 edition of the *Green Book* that, for the first time, incorporates units in the SI or metric system. The revisions to the *Green Book* are shown in Table I-1 at the end of this appendix.

GREEN BOOK CHAPTER II (Design Controls and Criteria)

The following text shows the recommended revisions to the section on speed that appears on pages 61-71 in Chapter II (Design Controls and Criteria) of the 1994 *Green Book.* The recommended changes are intended primarily to incorporate the research findings.

Speed

Speed is one of the most important factors to the traveler in selecting alternate routes or transportation modes. The value of a transportation facility in carrying people and goods is judged by its convenience and economy, which are directly related to its speed. The attractiveness of a public transportation system or a new highway are each weighed by the traveler in terms of time, convenience, and money saved. Hence, the desirability of rapid transit may well rest with how rapid it actually is. The speed of vehicles on a road or highway depends, in addition to capabilities of the drivers and their vehicles, upon four general conditions: the physical characteristics of the highway and its roadsides, the weather, the presence of other vehicles, and the speed limitations (either legal or because of control devices). Although any one of these may govern, the effects of these conditions are usually combined.

The objective in design of any engineered facility to be used by the public is to satisfy the demands for service in the safest and most economical manner. The facility should therefore accommodate nearly all demands with reasonable adequacy and also not fail completely under the severe or extreme load. In applying this principle to the design of highways, with particular reference to speed demands, provision should be made for a speed that satisfies nearly all drivers. Only a small percentage of drivers travel at extremely high speed, and it is not economically feasible to design for them. They can use the highway, of course, but must travel at speeds somewhat less than they consider desirable. On the other hand, the speed chosen for design should not be that used by drivers under unfavorable conditions, such as inclement weather, because the highway then would be unsafe for drivers under favorable conditions, and would not satisfy reasonable demands.

Operating Speed

Operating speed is the ~~highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without any time exceeding the safe speed as determined by the design speed on a section-by-section basis.~~ **speed at which drivers are observed operating their vehicles during free flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used descriptive statistic for the operating speed associated with a particular location or geometric feature.**

Design Speed

Design speed is ~~the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern~~. **a selected speed used to determine the various geometric design features of the roadway.** The assumed design speed should be a logical one with respect to the topography, the adjacent land use, and the functional classification of highway. Except for local streets where speed controls are frequently included intentionally, every effort should be made to use as high a design speed as practicable to attain a desired degree of safety, mobility, and efficiency while under the constraints of environmental quality, economics, aesthetics, and social or political impacts. Once selected, all of the pertinent features of the highway should be related to the design speed to obtain a balanced design. Above-minimum design values should be used where feasible, but in view of the numerous constraints often encountered, practical values should be recognized and used. Some features, such as curvature, superelevation, and sight distance, are directly related to, and vary appreciably with, design speed. Other features, such as widths of lanes and shoulders and clearances to walls and rails, are not directly related to design speed, but they affect vehicle speed, and higher standards should be accorded these features for the higher design speeds. Thus, when a change is made in design speed, many design elements of the highway are subject to change.

The design speed chosen should be consistent with the speed a driver is likely to expect. Where a difficult condition is obvious, drivers are more apt to accept lower speed operation than where there is no apparent reason for it. A highway of higher functional classification may justify a higher design speed than a less important facility in similar topography, particularly where the savings in vehicle operation and other operating costs are sufficient to offset the increased costs of right-of-way and construction. A low design speed, however, should not be assumed where the topography is such that drivers are likely to travel at high speeds. Drivers do not adjust their speeds to the importance of the highway, but to their perception of the physical limitations and traffic thereon.

The speed selected for design should fit the travel desires and habits of nearly all drivers. Where traffic and roadway conditions are such that drivers can travel at their chosen speed, there is always a wide range in the speeds at which various individuals operate their vehicles. A cumulative distribution of **free flow** vehicle speeds has the typical S pattern when plotted as percent of vehicles versus observed speeds. The design speed chosen should be a high-percentile value in this speed distribution curve, i.e., nearly all inclusive of the typically desired speeds of drivers, wherever this is feasible.

The speed distribution curves in Figure II-21 illustrate the range in speed that should be considered in a determination of assumed design speed. A design speed of 110 km/h should be maintained on freeways, expressways, and other major highways. This speed will ensure an adequate design if the speed restriction is removed. Also, a larger percent of the vehicles traveling at the faster speeds will be safely accommodated.

These data lead to the conclusion that where physical features of the highway are the principal speed controls and where most drivers have been conditioned to operate near the speed limit, a top design speed of 120 km/h would fit a very high-percentile speed. On a highway designed for this speed, a small percent of drivers might still operate at higher speed when volume is low and all other conditions are favorable. However, for a design speed of 80 km/h, satisfactory performance could be expected only on certain highways. When the ~~minimum~~ **low** design speed**s** ~~is~~ **are** used as the criteria, it is important to have the speed limit enforced during off-peak hours.

On many freeways, particularly in suburban and rural areas, a design speed of 100 km/h or higher can be provided with little additional cost above that required for a design speed of 80 km/h. The corridor of the main line may be relatively straight and the character and location of interchanges permit high-speed design. Under these conditions a design speed of 110 km/h is desirable. Flat curvature and ample sight distance usually result in safer highways.

Generally, there is no design speed distinction between a ground-level, an elevated, or a depressed freeway. However, the operating characteristics on elevated freeways differ somewhat from those on depressed freeways. On an elevated highway, traffic exits the facility on downgrade ramps and enters on upgrade ramps. This condition is less desirable than the opposite one on a depressed highway because vehicles, particularly loaded trucks, entering the elevated freeway on an ascending grade require long distances to reach the running speed on the freeway (see section on "Running Speed"). Moreover, vehicles leaving the elevated freeway on a descending grade require additional braking distance to reach the running speed of the arterial street and consequently may tend to slow down on the through traffic lanes in advance of the ramp terminal. Parallel deceleration lanes or longer ramp lengths and lesser grades are frequently used to reduce the problem of vehicles slowing on the main lanes. Nevertheless, running speeds on elevated freeways are apt to be slightly lower than those on depressed freeways of the same standards, especially when access points are closely spaced. In northern climates, elevated structures are subject to rapid freezing of precipitation as a result of their exposure and may require the use of lesser superelevation rates, which affect both running and design speed. Although speeds on viaducts are less than those on comparable depressed sections, the difference probably is small. Therefore, the appropriate design speeds of 80 to 110 km/h apply to both elevated and depressed freeways.

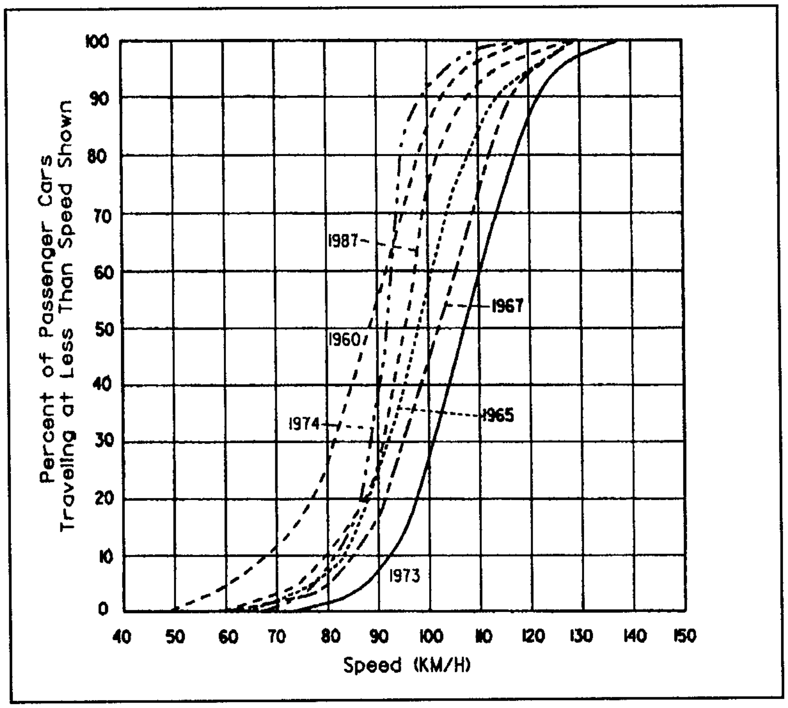


Figure II-21. Distribution of representative passenger car speeds on rural Interstate highways.

With an overall range in design speeds of 30 to 120 km/h, it has been found desirable to use increments of 10 km/h. Smaller increments show little distinction in design elements between one design speed and the next higher design speed, and larger increments of 25 to 30 km/h cause too large a difference in design dimensions of features between any two design speeds. In some instances, there may be an advantage in using intermediate increments to effect changes in the design speed. Increments of 10 km/h also may be pertinent to design of turning roadways, ramps, and low-speed roads. The use of 10 km/h increments in design speed below 120 km/h does not preclude the use of smaller increments for traffic control purposes, such as speed-zone signs.

Although the selected design speed establishes the minimum curve radius and minimum sight distance ~~necessary for safe operation~~ **requirements**, there should be no restriction on the use of flatter horizontal curves or greater sight distances where such improvements can be provided as a part of economic design. Even in rugged terrain an occasional tangent or flat curve may be desirable. These would not necessarily encourage drivers to speed up; but, if a succession of them is introduced, drivers will naturally resort to higher speeds, and that section of highway should be designed for a higher speed. A substantial length of tangent between sections of curved alinement also is apt to encourage high-speed operation. In such cases a higher speed should be assumed and all geometric features, particularly that of sight distance on crest vertical curves, should be related to it.

A pertinent consideration in selecting design speeds is the average trip length. The longer the trip, the greater the desire for expeditious movement. In design of a substantial length of highway it is desirable where feasible to assume a constant design speed. Changes in terrain and other physical controls may dictate a change in design speed on certain sections. If so, the introduction of a lower design speed should not be done abruptly but should be effected over sufficient distance to permit drivers to change speed gradually before reaching the section of highway with the lower design speed.

Where it is necessary to ~~reduce design speed~~, **shorten horizontal and vertical curvature,** many drivers may not perceive the lower speed condition ahead, and it is important that they be warned well in advance. The changing condition should be indicated by such controls as speed-zone signs and curve-speed signs.

On arterial streets, the design speed control applies to a lesser degree than on other high-type highways. On rural highways or on high-type urban facilities, a certain percentage of vehicles are able to travel at near the ~~safe~~ **operating** speed determined by geometric design elements, but on arterial streets the top speeds for several hours of the day are limited or regulated to that at which the recurring peak volumes can be handled. Speeds are governed by the presence of other vehicles traveling en masse both in and across the through lanes and by traffic control devices rather than by the physical characteristics of the street. During periods of low-to-moderate volume, speeds are governed by such factors as speed limits, midblock turns, and intersectional turns, traffic signal spacing and signal timing for progression. When arterial street improvements are being planned, the design speed should be considered with factors such as speed limits, physical and economic constraints, and the likely running speeds that can be attained during offpeak hours, which would influence the selection of the speed design.

Horizontal alinement generally is not the governing factor in restricting speeds on arterial streets. Proposed improvements generally are patterned to the existing street system, and minor horizontal alinement changes are commonly made at intersections. The effect of these alinement changes is usually minor because operation through the intersection is regulated by the type of traffic controls needed to handle the volume of cross and turning traffic. Superelevation may be provided at curves on arterial streets but is developed in a different manner than for open road rural conditions. The wide pavement areas, proximity of adjacent development, control of cross slope and profile for drainage, and the frequency of cross streets and entrances all contribute to the need for lower superelevation rates. Likewise, the width of lanes, offset to curbs, proximity of poles and trees to the traveled way, presence of pedestrians within the right-of-way, and nearness of business or residential buildings, singly and in combination, often nullify speed characteristics of a highway with good alinement and flat profiles. Yet, good alinement and flat profiles should always be strived for in the design of arterial streets, because safety and operating characteristics are improved, particularly during off-peak periods.

Topography can materially affect the choice of design speed on arterial streets. Many of our cities were developed along watercourses and include areas varying from gently rolling to mountainous terrain. The streets originally were constructed with minor grading to fit the topography. Because the arterial street usually is developed to fit an existing street, both through business and residential areas, it generally follows a varying vertical alinement. Once the design speed is determined, the proper sight distance should be assured at all crests. Profile conditions with long, continuous grades should also be designed with proper consideration of speeds of operation of mass transit and commercial vehicles.

Extra lanes on the upgrades may be needed so that this portion of the route can match other portions in capacity and enable vehicles that can proceed at reasonable speed to pass slower moving vehicles.

Arterial streets should be designed and control devices regulated, where feasible, to permit running speeds of 30 to 70 km/h. Lower speeds in this range are applicable for local and collector streets through residential areas and for the arterial streets through the more crowded business areas, while the higher speeds apply to the high-type arterials in the outlying suburban areas. For the arterial streets through the crowded business areas, coordinated signal control through successive intersections generally is necessary to permit even the lower speeds. Many cities have substantial lengths of streets controlled so as to operate at running speeds of 25 to 40 km/h. At the other extreme in suburban areas, it is common experience on preferred streets to adopt some form of speed zoning or speed control to prevent high operating speeds. In these areas, the infrequent pedestrian or occasional vehicles on a cross street may be unduly exposed to potential accidents from through drivers. Such through drivers gradually gain speed as the frequency of urban restrictions are left behind or such drivers retain their speed of the open road as they enter the city. Thus, although through traffic should be expedited to the extent feasible, it may be equally important to establish a certain speed to reduce potential hazards and to serve local traffic.

A posted speed limit as a matter of practicability, is not the highest speed that might be used by drivers. Instead, it usually approximates the 85-percentile speed value as determined by observing a sizable sample of vehicles. Such a value is within the "pace" or 15 km/h speed range used by most drivers. Speed zones cannot be made to operate properly if arbitrarily determined or selected. In addition, speed zones must be consistent with conditions along the street, the selected cross-section of the street from engineering studies, and must be subject to reasonable enforcement.

With running speeds of 30 to 70 km/h, it follows that pertinent design speeds for arterial streets and highways would range from 50 to 100 km/h. The selected design speed for an urban arterial highway would depend largely on the spacing of signalized at-grade intersections, the selected type of median cross-section, whether or not curb and gutter is used along the street, and the amount and type of access allowed to the street. As a desirable minimum, elements of a reconstructed urban arterial highway should be designed for an ~~safe~~ operating speed of at least 50 km/h.

The preceding paragraphs describe the basis for, and various considerations that need to be examined when selecting a design speed. From this discussion, it is evident that there are meaningful differences between the design criteria applicable to low- and high-speed designs. Because of these distinct differences, it is desirable to establish certain limits. For application in this publication, the upper limit for low or lower design speed usually is 60 km/h, and the minimum limit for high speed design is 80 km/h. The intermediate design speed of 70 km/h could be considered as either low speed or high speed depending upon the specific conditions along the street, and such conditions would govern in the selection of the appropriate design criteria.

Running Speed

In design it is necessary to know actual vehicle speeds for traffic en masse to be expected on highways of different design speeds and various volume conditions.

Speed of operation is one measure of the service that a highway renders, and it affords a means of evaluating road-user costs and benefits. The running speed is the speed of a vehicle over a specified section of highway, being the distance traveled divided by the running time (the time the vehicle is in motion).

One means of obtaining an equivalent average running speed on an existing facility where flow is reasonably continuous is to measure the spot speed. The average spot speed is the arithmetic mean of the speeds of all traffic at a specified point. For short sections of highway on which speed characteristics do not vary materially, the average spot speed may be considered as being representative of the average running speed. On longer stretches of rural highway, spot speeds measured at several points, where each represents the speed characteristics pertinent to a selected segment of highway, may be averaged (taking relative lengths into account) to represent the average running speed.

Average spot speeds, which generally are indicative of average running speeds, have been measured over a period of years in many States on highway sections of favorable alinement. The average speed slowly increased over the years, then leveled out, and subsequently dropped with the advent of the 55-mph speed limit. Since then it has decreased very slightly on highways where the 55-mph speed limit is still in effect. On interstate highways with 105 km/h speed limits, the increase has been greater.

Experience on horizontal curves shows that speeds are lower than those on tangent alinement and that the difference between average spot speed and ~~calculated~~ **inferred** design speed on such curves becomes less as the radius of curvature decreases. In this regard, it is generally accepted that a greater proportion of drivers operate near or at the design speed on highways with low design speed than on highways with high design speed. It is also known that some sections of low design speed highways are frequently overdriven, with an appreciable number of drivers exceeding the **inferred** design speed.

Observed speeds of free-moving vehicles on horizontal curves indicate that low design speed curves yield an average spot speed close to the design speed; on high design speed curves the average speed is substantially below the design speed and approaches the average spot speed found on long stretches of tangent alinement. Because horizontal curvature is the principal factor related to design speed of rural highways and since average spot speed approximates the average running speed for such conditions, a useful relation between the highway design speed and the average running speed (for low-volume conditions) may be established from these data. Comparing the observed average speeds with ~~calculated~~ **inferred** design speeds, it is found that on sections having a 50 km/h design speed, the average running speed is approximately 90 to 95 percent of the design speed.

The general relation between design speed and average running speed (the average for all traffic or component of traffic, being the summation of distances divided by the summation of running times; it is approximately equal to the average of the running speeds of all vehicles being considered) is illustrated in Figure II-22. The upper curve represents the conditions for low traffic volume as just described. As traffic volume increases on any highway, the average running speed decreases because of interference among vehicles. The curve labeled "Intermediate Volume" represents the relation between design speed and average running speed when the volume approximates the design service volume for rural highways. Should the volume exceed the intermediate level, the average running speed would be further lowered, and in the extreme case, where the volume is approaching the capacity of the highway, the speed of traffic is influenced more by congestion than by the design speed, especially where the design speed is above 80 km/h. The relation between design speed and average running speed for very high traffic volumes is illustrated by the lower cure in Figure II-22. This curve ~~is of academic interest only. It~~ establishes a limiting condition for average running speeds but it is of little value in design. Highways should usually be designed to accommodate their traffic volumes without being subjected to the high degree of congestion represented by this curve.

A design that satisfies the requirements for average running speed at low volume is adequate for traffic using the highway when the volumes are higher and the speeds are lower. At low volumes about 50 percent of all vehicles travel at speeds within 10 km/h of the average running speeds, as shown by the speed distribution curves in Figure II-21. For volumes in the intermediate range about 90 percent of all vehicles travel at or less than the average running speed representative of low volumes. For this reason, low volumes control certain highway elements, such as lane and shoulder widths, treatment of intersection curves, and speed-change lanes.

Average running speed on a given highway varies somewhat during the day, depending primarily on the volume of traffic. Therefore, when reference is made to running speed it should be clear whether this speed is for peak hours or offpeak hours or whether it is an average for the day. The first two are of concern in design and operation; the latter is of importance in economic analyses.

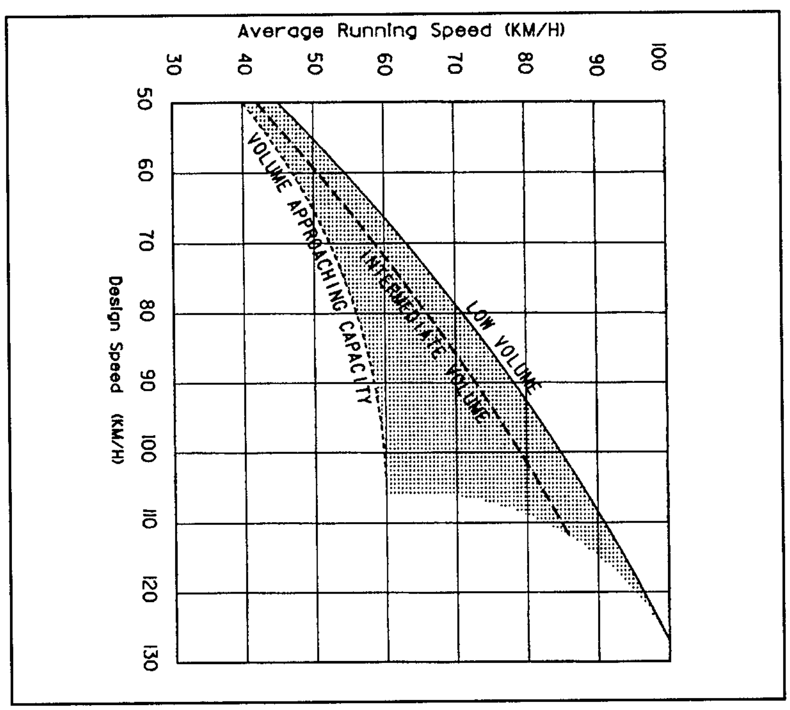


Figure II-22. Relation of average running speed and volume conditions.

Figure II-23 depicts the relationship between average speed of **an** ideal traffic stream and ideal flow rate for a 15-minute period. Figure II-23 depicts two important characteristics:

1. There is a substantial range of flow over which speed is relatively insensitive to flow: this range extends to fairly high flow rates.
2. As flow approaches capacity, speed drops off at a sharp rate.

The data for Figure II-23 are taken from the *Highway Capacity Manual* (HCM) (11).

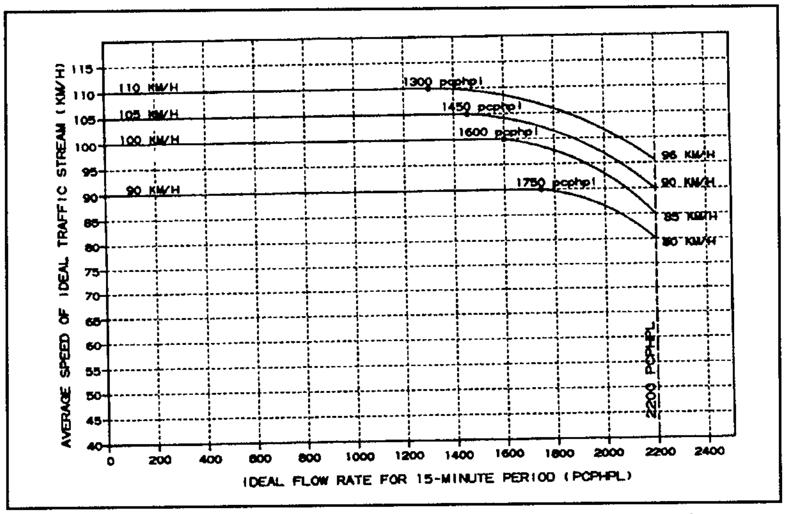


Figure II-23. Speed flow characteristics for basic freeway sections (for ideal conditions) (11).

GREEN BOOK CHAPTER III (Elements of Design)

The following text shows the recommended revisions to the sections on sight distance (pages 117-125 and 136-141), sight distance on horizontal curves (pages 219-223), and vertical curves (pages 279-286 and 288-293) that appear in Chapter III of the 1994 Green Book. The recommended changes are intended primarily to incorporate the research findings.

SIGHT DISTANCE

General Considerations

The ability to see ahead is of the utmost importance in the safe and efficient operation of a vehicle on a highway. On a railroad, trains are confined to a fixed path, yet a block signal system and trained operators are necessary for safe operation. On the other hand, the path and speed of motor vehicles on highways and streets are subject to the control of drivers whose ability, training, and experience are quite varied. For safety on highways the designer must provide sight distance of sufficient length that drivers can control the operation of their vehicles to avoid striking an unexpected object on the traveled way. Certain two-lane highways should also have sufficient sight distance to enable drivers to occupy the opposing traffic lane for passing overtaken vehicles without risk of accident. Two-lane rural highways should generally provide such passing sight distance at frequent intervals and for substantial portions of their length. Conversely, it normally is of little practical value to provide passing sight distance on two-lane urban streets or arterials. The length and interval of passing sight distance should be compatible with the criteria established in the chapter pertaining to that specific highway or street classification.

Sight distance is discussed in four steps: (1) the distances required for stopping, applicable on all highways; (2) the distances required for the passing of overtaken vehicles, applicable only on two-lane highways; (3) the distances needed for decisions at complex locations; and (4) the criteria for measuring these distances for use in design. The design of alinement and profile to provide these distances and to meet these criteria are described later in this chapter. The special conditions related to sight distances at intersections are discussed in Chapter IX.

Stopping Sight Distance

Sight distance is the length of roadway ahead visible to the driver. The minimum sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although the greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average operator or vehicle to stop in this distance.

Stopping sight distance is the sum of two distances: the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied and the distance required to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively.

Brake Reaction Time

Brake reaction time is the interval between the instant that the driver recognizes the existence of an object or hazard on the roadway ahead and the instant that the driver actually applies the brakes. This interval includes the time required to make the decision that a stop is necessary. Under certain conditions, such as emergency conditions denoted by flares or flashing lights, operators accomplish these tasks almost instantly. Under most other conditions the operator must subconsciously associate the object ahead with stationary objects adjacent to the roadway, such as walls, fences, trees, poles, or bridges, to determine that the object is also stationary or moving at a slow speed. These determinations take time, the amount of which varies considerably depending on the distance to the object, the acuity of the operator, the natural rapidity with which the driver reacts, atmospheric visibility, the type and the condition of the roadway, and the type, color, and condition of the hazard. Vehicle speed and the roadway environment probably also influence reaction time. Normally, an operator traveling at or near the design speed is more alert than one traveling at a lesser speed. An operator on an urban facility confronted by innumerable possibilities for conflicts from parked vehicles, driveways, and cross streets is also likely to be more alert than the same operator on a limited-access facility where such conditions should be almost nonexistent.

The study (1) referred to in Chapter II was based on data from 321 drivers who expected to apply their brakes. The median reaction-time value for these drivers was 0.66 s with 10 percent requiring 1.5 s or longer. These findings correlate with those of earlier studies in which alerted drivers were also used. Another study (2) gives 0.64 s as the average value; 5 percent of the drivers required over 1 s. In a third study (3) reaction-time values ranged from 0.4 to 1.7 s. In the Johansson and Rumar study (1), when the signal was unexpected, the drivers' responses were found to increase by approximately 1 s or longer; some reaction times being 1.5 s or more. This increase substantiated earlier laboratory and road tests in which the conclusion was drawn that the driver who required 0.2 to 0.3 s under alert conditions required 1.5 s under normal conditions.

Minimum reaction times thus could be at least 1.64 s; 0.64 s for alerted drivers plus 1 s for the unexpected signal. Because the studies used simple prearranged signals, they represent the least complex of roadway conditions. Even under these simple conditions it was found that some operators may take over 3.5 s to respond. Because actual conditions on the highway are generally more complex than those of the studies and because there is wide diversity in the reaction times required, it is evident that the value adopted should be greater than 1.64 s. In determination of sight distance for design, the reaction time ~~should be larger than the average for all drivers under normal conditions. It~~ should be large enough to include the reaction time required for nearly all drivers under most highway conditions. For approximately 90 percent of the drivers in the first study mentioned, a reaction time of 2.5 s was found to be adequate. A reaction time of 2.5 s has thus been assumed in the development of Table III-1.

A reaction time of 2.5 s is considered adequate for more complex conditions than those of the various studies, but it is not adequate for the most complex conditions encountered by the driver. Additional consideration of the most complex conditions such as those found at multiphase at-grade intersections and ramp termini at through roadways can be found later in this chapter in the section "Decision Sight Distance."

Braking Distance

The approximate braking distance of a vehicle on a level roadway may be determined by the use of the standard formula:

$$\cancel{d = \frac{V^2}{254f}}$$

$$\mathbf{d = 0.039V^2/a}$$

where: d = braking distance, m;
V = initial speed, km/h; and
~~f = coefficient of friction between tires and roadway.~~
a = driver deceleration, m/s²

~~In this formula for braking distance the f factor is used as an overall or a single value that is representative for the whole of the speed change. Measurements show that f is not the same for all speeds. It decreases as the initial speed increases. It varies considerably because of many physical elements such as air pressure of tires, composition of tires, tire tread pattern and depth of tread, type and condition of the pavement surface, and the presence of moisture, mud, snow or ice. The braking distance also depends on the braking system of the vehicle. The several variables are allowed for if f is computed for each test from the standard formula. It thus represents the equivalent constant friction factor. The values of f in Figure III-1 (A and B) were calculated on this basis for some of the curves that represent tests in which only speed and distance were recorded.~~

~~Figure III-1 A illustrates friction coefficients found by different investigators. Curves 1 to 6 are from a study (4) in which more than 1,000 measurements of forward skidding were made on 32 pavements in both wet and dry conditions. Several types of tires were being used. Coefficients of friction were computed by using actual stopping distances in the standard stopping formula. Curves 7 and 8 are representative of several curves of a study (5) in which over 50 surfaces were tested in the dry condition by three different methods using three types of tires. Curves 9 and 10 from the same study are representative of wet pavements. Curve 11 is the calculated equivalent f value for stopping sight distances measured (3) on high-type pavement; these were the only tests that included stops from speeds of about 100 to 110 km/h. This curve is the average of all stops measured, but comparison with on-the-road samples (at low speeds) shows that the vehicles and drivers were somewhat better than the average on the highways.~~

~~Because of the lower coefficients of friction on wet pavements as compared with dry, the wet condition governs in determining stopping distances for use in design. The coefficients of friction used for design criteria should represent not only wet pavements in good condition but also surfaces approaching the end of their useful lives. The values should encompass nearly all significant pavement surface types and the likely field conditions. Figure III-1B summarizes the results of a series of 600 measurements made on modern pavements in Germany (6). Each road section was tested at 20, 40, 60, and 80 km/h by a locked-wheel trailer. Because the test methods differ, there is no direct correlation between these curves and those of Figure III-1A or Table III-1. However, Figure III-1B does indicate that there is a wide variation between the coefficients of friction for various pavements, which reflects the effect that surface texture has on stopping sight distance and, therefore, the importance of this factor.~~

~~The friction factor values used in calculating safe stopping distances should allow for worn tires as well as for new tires and for nearly all types of treads and tire compositions. The friction factor values used should also encompass the differences in vehicle and driver braking from various speeds. On the other hand, the values used need not be so low as to be suitable for pavements under icy conditions. Preferably, the f values used for design should be nearly all-inclusive, rather than average; however, available data are not fully detailed over the range for all these variables, and conclusions must be made in terms of the safest reported average values. The coefficients of friction in Table III-1 have been selected based on these criteria as being appropriate for the calculation of stopping sight distances. Comparison with Figure III-1A shows that the range of values in Table III-1 is generally conservative; the upper limit reflects the concern for the more recent measurements such as those of Figure III-1B, which show a few coefficients for wet pavements with values near or less than the values used for developing the lower limits for stopping sight distance.~~

~~The average running speed for low-volume conditions rather than design speed is used in formulating the limiting values for minimum stopping distance. This speed is the initial value given in the second column of Table III-1. Studies show that many operators drive just as fast on wet pavements as they do on dry. To account for this factor, design speed in place of average running speed is used to formulate stopping distance values, as shown by the higher values in the second column of Table III-1.~~

Studies documented in the literature show that most drivers choose decelerations greater than 4.5 m/s^2 when confronted with the need to stop for an unexpected object in the roadway. Approximately 90 percent of all drivers choose decelerations that are greater than 3.4 m/s^2. These decelerations are within the driver's capability to stay within his or her lane and maintain steering control during the braking maneuver on wet surfaces.

Thus, 3.4 m/s^2 (a comfortable deceleration for most drivers) is recommended as the deceleration threshold for determining required stopping sight distance. Implicit in this deceleration threshold is the requirement that the vehicle braking system and pavement friction values are at least equivalent to 3.4 m/s^2 (0.34 g). Skid data show that most wet pavement surfaces on state maintained roadways exceed this threshold. Braking data show that most vehicle braking systems can exceed the skidding friction values for the pavement.

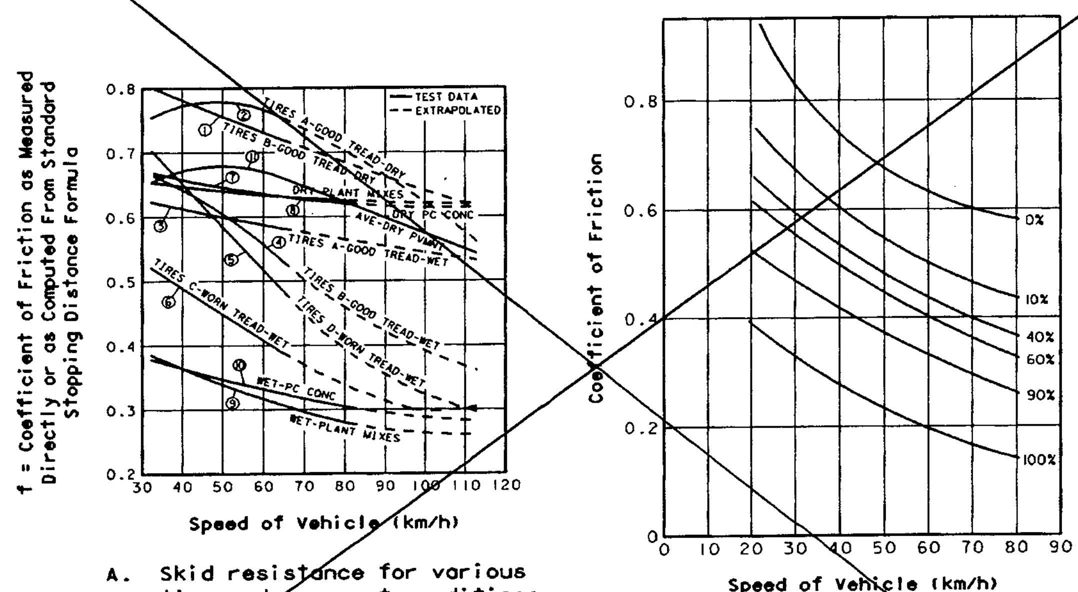
~~Design Speed (km/h)~~	~~Assumed Speed for Condition (km/h)~~	~~Brake Reaction~~ ~~Time (s)~~	~~Brake Reaction~~ ~~Distance (m)~~	~~Coefficient of Friction[a] f~~	~~Braking Distance on Level (m)~~	~~Stopping Sight Distance for Design (m)~~
~~30~~	~~30-30~~	~~2.5~~	~~20.8-20.8~~	~~0.40~~	~~8.8-8.8~~	~~29.6-29.6~~
~~40~~	~~40-40~~	~~2.5~~	~~27.8-27.8~~	~~0.38~~	~~16.6-16.6~~	~~44.4-44.4~~
~~50~~	~~47-50~~	~~2.5~~	~~32.6-34.7~~	~~0.35~~	~~24.8-28.1~~	~~57.4-62.8~~
~~60~~	~~55-60~~	~~2.5~~	~~38.2-41.7~~	~~0.33~~	~~36.1-42.9~~	~~74.3-84.6~~
~~70~~	~~63-70~~	~~2.5~~	~~43.7-48.6~~	~~0.31~~	~~50.4-62.2~~	~~94.1-110.8~~
~~80~~	~~70-80~~	~~2.5~~	~~48.6-55.5~~	~~0.30~~	~~64.2-83.9~~	~~112.8-139.4~~
~~90~~	~~77-90~~	~~2.5~~	~~53.5-62.5~~	~~0.30~~	~~77.7-106.2~~	~~131.2-168.7~~
~~100~~	~~85-100~~	~~2.5~~	~~59.0-69.4~~	~~0.29~~	~~98.0-135.6~~	~~157.0-205.0~~
~~110~~	~~91-110~~	~~2.5~~	~~63.2-76.4~~	~~0.28~~	~~116.3-170.0~~	~~179.5-246.4~~
~~120~~	~~98-120~~	~~2.5~~	~~68.0-83.3~~	~~0.28~~	~~134.9-202.3~~	~~202.9-285.6~~

[a] ~~Values of coefficient of friction generally approximate curves 9 and 10 (coefficient of friction for wet-PC concrete and wet-plant mixes) shown in Figure III-1A.~~

Table III-1. Stopping sight distance (wet pavements).

Initial Speed (km/h)	Perception-Brake Reaction		Driver Deceleration for Design (m/s^2)	Braking Distance on Level (m)	Stopping Sight Distance for Design (m)
	Time (s)	Distance (m)			
30	2.5	20.8	3.4	10.2	31.0
40	2.5	27.8	3.4	18.2	45.9
50	2.5	34.7	3.4	28.4	63.1
60	2.5	41.7	3.4	40.8	82.5
70	2.5	48.6	3.4	55.6	104.2
80	2.5	55.5	3.4	72.6	128.2
90	2.5	62.5	3.4	91.9	154.4
100	2.5	69.4	3.4	113.5	182.9
110	2.5	76.4	3.4	137.3	213.7
120	2.5	83.3	3.4	163.4	246.7

Table III-1. Recommended stopping sight distances for design



A. Skid resistance for various tire and pavement conditions.

B. Skid Resistance of Pavement - Percent of Pavements Which Exceed Coefficient of Friction.

Figure III-1. Variation in coefficient of friction with vehicular speed.

Design Values

The sum of the distance traversed during the brake reaction time and the distance to stop the vehicle is the minimum stopping sight distance. The computed distances for wet pavements and for various speeds at the assumed conditions are shown in Table III-1 and were developed using the following formula:

$$\cancel{d = (0.278)(t)(V) + \frac{V^2}{254f}}$$

$$\mathbf{d = (0.278)(t)(V)} \cancel{\frac{V^2}{254f}} + \mathbf{0.039V^2/a}$$

where: t = brake reaction time, generally assumed to be 2.5 s;
V = initial speed, km/h; and
~~f = coefficient of friction between tires and roadway.~~
a = driver decelerations m/s^2

~~Any length of stopping sight distance within the range of values established in Table III-1 is acceptable for a specific speed. However, values approaching or~~ **Values** exceeding the **required stopping sight distances** ~~upper limit of the range~~ should be used as the basis for design wherever conditions permit. Use of ~~the upper limit~~ **larger** values increases the margin of safety by providing for the drivers who operate at or near the design speed during wet weather. To ensure that new pavements will have initially, and will retain, coefficients of friction comparable to ~~those~~ **the deceleration values** given in the table, designs should meet the criteria established in the AASHTO *Guidelines for Skid Resistant Pavement Design* (7). ~~Although research data may demonstrate that proposed pavements will have initially, and will retain coefficients of friction greater than those given in Table III-1, this finding should not be considered justification for using stopping sight distance values less than the minimums contained in this table. Variations in results (particularly because of the methods of determining coefficients of friction) preclude such direct comparisons. If portions of existing pavements are to be retained, skid tests should be made. Although direct comparisons cannot be made, the need for increased lengths of stopping sight distance should be considered when tests yield coefficients of friction significantly below those in Table III-1.~~

~~Table III-1 includes data for the higher design speeds for wet conditions where no actual test data are available. Values for design speeds above 100 km/h are extrapolated from the curves for lower speeds in Figure III-1A.~~

Effect of Grade on Stopping

When a highway is on a grade, the standard formula for braking distance is the following:

$$\cancel{d = \frac{V^2}{254(f \pm G)}}$$

$$d = \frac{V^2}{254\ (a/9.81) \pm G)}$$

in which G is the percent grade divided by 100, and the other terms are as previously stated. The stopping distances on upgrades are shorter; those on downgrades are longer. The stopping sight distances on various grades are indicated in Table III-2. These corrections are computed for wet conditions, the assumed design criterion used in Table III-1. The brake reaction time is assumed to be the same as for level conditions. Design speed is used in calculating ~~downgrade~~ corrections. ~~average running speed in calculating upgrade corrections. The different criteria for descending and ascending grades are based on the effect grades have on the speed of individual vehicles, particularly trucks; the effect these vehicles have on the overall speed of the traffic stream; and the premise that many drivers, particularly those in automobiles, do not compensate completely for the changes in speed caused by grades.~~

On nearly all roads and streets the grade is traversed by traffic in both directions, but the sight distance at any point on the highway generally is different in each direction, particularly on straight roads in rolling terrain. As a general rule, the sight distance available on downgrades is larger than on upgrades, more or less automatically providing the necessary corrections for grade. This may explain why some design offices do not make corrections in stopping sight distance because of grade. Exceptions are one-way roads or streets, as on divided highways with independent design profiles for the two roadways. For these the separate grade corrections are in order and the refinement in design is in keeping with the overall standards used.

~~Design Speed km/h~~	~~Stopping Sight Distance (m) for Downgrades~~			~~Assumed Speed for Condition (km/h)~~	~~Stopping Sight Distance (m) for Upgraded~~		
	~~3%~~	~~6%~~	~~9%~~		~~3%~~	~~6%~~	~~9%~~
~~30~~	~~30.4~~	~~31.2~~	~~32.2~~	~~30~~	~~29.0~~	~~28.5~~	~~28.0~~
~~40~~	~~45.7~~	~~47.5~~	~~49.5~~	~~40~~	~~43.2~~	~~42.1~~	~~41.2~~
~~50~~	~~65.5~~	~~68.6~~	~~72.6~~	~~47~~	~~55.5~~	~~53.8~~	~~52.4~~
~~60~~	~~88.9~~	~~94.2~~	~~100.8~~	~~55~~	~~71.3~~	~~68.7~~	~~66.6~~
~~70~~	~~117.5~~	~~125.8~~	~~136.3~~	~~63~~	~~89.7~~	~~85.9~~	~~82.8~~
~~80~~	~~148.8~~	~~160.5~~	~~175.5~~	~~70~~	~~107.1~~	~~102.2~~	~~98.1~~
~~90~~	~~180.6~~	~~195.4~~	~~214.4~~	~~77~~	~~124.2~~	~~118.8~~	~~113.4~~
~~100~~	~~220.8~~	~~240.6~~	~~256.9~~	~~85~~	~~147.9~~	~~140.3~~	~~133.9~~
~~110~~	~~267.0~~	~~292.9~~	~~327.1~~	~~91~~	~~168.4~~	~~159.1~~	~~151.3~~
~~120~~	~~310.1~~	~~341.0~~	~~381.7~~	~~98~~	~~190.0~~	~~179.2~~	~~170.2~~

~~**Table III-2. Effect of grade on stopping sight distance - wet conditions.**~~

Design Speed km/h	Stopping Sight Distance (m) for Downgrades 3%	6%	9%	Stopping Sight Distance (m) for Upgrades 3%	6%	9%
30	32.0	33.2	34.6	30.2	29.5	28.9
40	47.7	49.8	52.3	44.5	43.3	42.2
50	65.8	69.1	73.1	60.9	58.9	57.3
60	86.4	91.1	96.9	79.3	76.5	74.1
70	109.5	115.9	123.8	99.8	96.1	92.8
80	135.1	143.5	153.8	122.5	117.5	113.3
90	163.2	173.8	186.8	147.2	140.9	135.5
100	193.8	206.8	222.9	174.0	166.3	159.6
110	226.9	242.6	262.0	202.9	193.6	185.5
120	262.4	281.2	304.3	233.9	222.8	213.2

Table III-2. Effect of grade on stopping sight distance - wet conditions.

Variation for Trucks

The ~~derived~~ **recommended** minimum stopping sight distances directly reflect passenger car operation and might be questioned for use in design for truck operation. Trucks as a whole, especially the larger and heavier units, require longer stopping distances from a given speed than passenger vehicles do. However, there is one factor that tends to balance the additional braking lengths for trucks for given speeds with those for passenger cars. The truck operator is able to see the vertical features of the obstruction substantially farther because of the higher position of the seat in the vehicle. Separate stopping sight distances for trucks and passenger cars, therefore, are not used in highway design standards.

There is one situation that should be treated with caution, in which every effort should be made to provide stopping sight distances greater than the minimum design value. When horizontal sight restrictions occur on downgrades, particularly at the ends of long downgrades, the greater height of eye of the truck operator is of little value, even when the horizontal sight obstruction is a cut slope, when (on long downgrades) truck speeds may closely approach or exceed those of passenger cars. Although the average truck operator tends to be more experienced than the average passenger car operator and quicker to recognize hazards, it is best under such conditions to supply a stopping sight distance that meets or exceeds the values in Table III-1.

Criteria for Measuring Sight Distance

Sight distance is the distance along a roadway that an object of specified height is continuously visible to the driver. This distance is dependent on the height of the driver's eye above the road surface, the specified object height above the road surface, and the height of sight obstructions within the line of sight.

Height of Driver's Eye

For sight distance calculations for passenger vehicles, the height of the driver's eye is considered to be ~~1070~~ **1080** mm above the road surface. This value is based on studies ~~**(14, 15, 16, 17)**~~ **(14)** which show that average vehicle heights decreased since 1960 to 1300 mm with a comparable decrease in average eye heights to ~~1070~~ **1080** mm. ~~The average vehicle heights decreased 66 mm in this period, which correlates well with the 53 mm reduction in average eye heights. In the same time period the minimum height of eye decreased 64 mm to 1000 mm. Because of this significant change in the minimum eye heights, the design eye height has been reduced from 1140 to 1070 mm.~~ This change in eye height has the effect of lengthening minimum crest vertical curves by approximately 5 percent, thereby providing about 2.5 percent more sight distance. Because of various factors that appear to place practical limits on any further decreases in passenger car heights and the relatively small increases that further change would mandate in lengths of vertical curves, ~~1070~~ **1080** mm is considered to be the height of driver's eye for measuring both stopping and passing sight distances. For large trucks the driver eye height ranges from ~~1.8 to 2.4 m~~, **2.3 to 2.6,** the most common being 2.4 m. For design 2.4 m is the assumed eye height for trucks.

Height of Object

For stopping sight distance calculations, the height of object is considered to be ~~150~~ **600** mm above the road surface. For passing sight distance calculations, the height of object is considered to be 1300 mm above the road surface.

Stopping sight distance object. ~~The object height of 150 mm was adopted for stopping sight distance calculation purposes in 1965.~~ The basis for ~~its~~ selection of a **600 mm** object height was largely an arbitrary rationalization of possible hazardous object size and a driver's ability to perceive and react to a hazardous situation. ~~If other vehicles were the only likely hazard to be encountered, the height of vehicle taillights, 460 mm to 600 mm, would be sufficient object height. Such a height, however, would preclude a driver's seeing small animals, rocks, or other debris that are likely to be encountered in the roadway.~~ It is considered that a ~~150~~ **600** mm-high object is representative of the ~~lowest~~ **realistic** object that can create a hazardous condition and be perceived as a hazard by a driver in time to stop before reaching it. Using object heights of less than ~~150~~ **600** mm for stopping sight distance calculations results in ~~considerably~~ longer crest vertical curves **without documented safety benefits.** ~~For example, if the roadway surface is used as the sighted object, crest vertical curves would have to be about 85 percent longer than when 150 mm is used as the object height.~~ ~~The~~ Object heights of less than ~~150~~ **600** mm could substantially increase construction costs because additional excavation would be required to provide the longer crest vertical curves. It is also doubtful that the driver's ability to perceive a hazardous situation would be increased **because recommended stopping sight distances for high speed design are beyond most driver capabilities to detect small objects.**

Passing sight distance object. The object height of 1300 mm is adopted for passing sight distance calculations, superseding the 1400-mm object height, which had been used since 1940. Because vehicles are the objects that must be seen when passing and because the height of the average passenger vehicle body has been reduced to its current 1300-mm height above the pavement, this height will be used for calculation purposes. Passing sight distances calculated on this basis are also considered adequate for night conditions because the beams of the headlights of an opposing vehicle generally are seen from a greater distance than its top could be seen in the daytime.

Sight Obstructions

On tangents the obstruction that limits the driver's sight distance is the road surface at some point on a crest vertical curve. On horizontal curves the obstruction that limits the driver's sight distance may be the road surface at some point on a crest vertical curve, or it may be some physical feature outside of the traveled way, such as a longitudinal barrier, a bridge-approach fill slope, a tree, foliage, or the backslope of a cut section. Accordingly, all highway construction plans should be checked in both the vertical and horizontal plane for sight distance obstructions.

Measuring and Recording Sight Distance on Plans

The design of horizontal alinement and vertical profile using sight distance and other criteria is covered later in this chapter, particularly the detail design of horizontal and vertical curves. Sight distance, however, should be considered in the preliminary stages of design when both the horizontal and vertical alinement are still subject to adjustment. By determining graphically the sight distances on the plans and recording them at frequent intervals, the designer can appraise the overall layout and effect a more balanced design by minor adjustments in the plan or profile. Methods for scaling sight distances are demonstrated in Figure III-3. The figure also shows a typical sight distance record that would be shown on the final plans.

Because the view of the highway ahead may change rapidly in a short distance, it is desirable to measure and record sight distance for both directions of travel at each station. Both horizontal and vertical sight distances should be measured and the shorter lengths recorded. In the case of two-lane highways, passing sight distance in addition to stopping sight distance should be measured and recorded.

Sight distance charts such as those in Figures III-39 through III-42 may be used to establish minimum lengths of vertical curves. Charts similar to Figures III-24A and III-24B are useful for determining the radius of horizontal curve or the lateral offset therefrom needed to provide the required sight distance. Once the horizontal and vertical alinements are tentatively established, the practical means of examining sight distances along the proposed highway is by direct scaling on the plans.

Horizontal sight distance on the inside of a curve is limited by obstructions such as buildings, hedges, wooded areas, highground, or other topographic features. These generally are plotted on the plans. Horizontal sight is measured with a straightedge, as indicated at the upper left in Figure III-3. The cut slope obstruction is shown on the worksheets by a line representing the proposed excavation slope at a point 600 mm (approximate average of ~~1070~~ **1080** mm and ~~150~~ **600** mm) above the road surface for stopping sight distance and at a point about 1100 mm above the road surface for passing sight distance. The position of this line with respect to the centerline may be scaled from the plotted highway cross sections. Preferably, the stopping sight distance should be measured between points on the one traffic lane, and passing sight distance from the middle of one lane to the middle of the other lane. Such refinement on two-lane highways generally is not necessary and measurement to the centerline or traveled way edge is suitable. Where there are changes of grade coincident with horizontal curves that have sight-limiting cut slopes on the inside, the line-of-sight intercepts the slope at a level either lower or higher than the assumed average height. In measuring sight distance the error in the use of the assumed 600- or 1100-mm height usually can be ignored.

Vertical sight distance may be scaled from a plotted profile by the method illustrated at the right center of Figure III-3. A transparent strip with parallel edges 1300 mm apart and with scratched lines ~~150~~ **600** mm and ~~1070~~ **1080** mm from the upper edge, in accordance with the vertical scale, is a useful tool. The ~~1070~~-**1080** mm line is placed on the station from which the vertical sight distance is desired, and the strip is pivoted about this point until the upper edge is tangent to the profile. The distance between the initial station and the station on the profile intersected by the ~~150~~ **600** mm line is the stopping sight distance. The distance between the initial station and the station on the profile intersected by the lower edge of the strip is the passing sight distance.

A simple sight distance record is shown in the lower part of Figure III-3. Sight distances in both directions are indicated by arrows and figures at each station on the plan and profile sheet of the proposed highway. To avoid the extra work of measuring the unusually long sight distances that may occasionally be found, a selected maximum value may be recorded. In the example shown, all sight distances of more than 1000 m are recorded as 1000+, and where this occurs for several consecutive stations, the intermediate values are omitted. Sight distances less than 500 m may be scaled to the nearest 10 m and those greater than 500 m to the nearest 50 m. The available sight distances along a proposed highway also may be shown by other methods. Several States use a sight distance graph, plotted in conjunction with the plan and profile of the highway, as a means of demonstrating sight distances. Sight distances can easily be determined also where plans and profiles are drawn using computer-aided design and drafting systems (CADD).

Sight distance records for two-lane highways may be used to advantage to tentatively determine the marking of no-passing zones in accordance with criteria given in the MUTCD **(8)**. Marking of such zones is an operational rather than a design problem. No-passing zones thus established serve as a guide for markings when the highway is completed; the zones so determined should be checked and adjusted by field measurements before actual markings are placed.

Sight distance records also are useful on two-lane highways for determining the percentage of length of highway on which sight distance is restricted to less than the passing minimum, which is important in evaluating capacity. With recorded sight distances, as in the lower part of Figure III-3, it is a simple process to determine the percentage of length of highway with a given sight distance or greater.

Sight Distances on Horizontal Curves

Another element of horizontal alinement is the sight distance across the inside of curves. Where there are sight obstructions (such as walls, cut slopes, buildings, and longitudinal barriers) on the inside of curves, a design to provide adequate sight

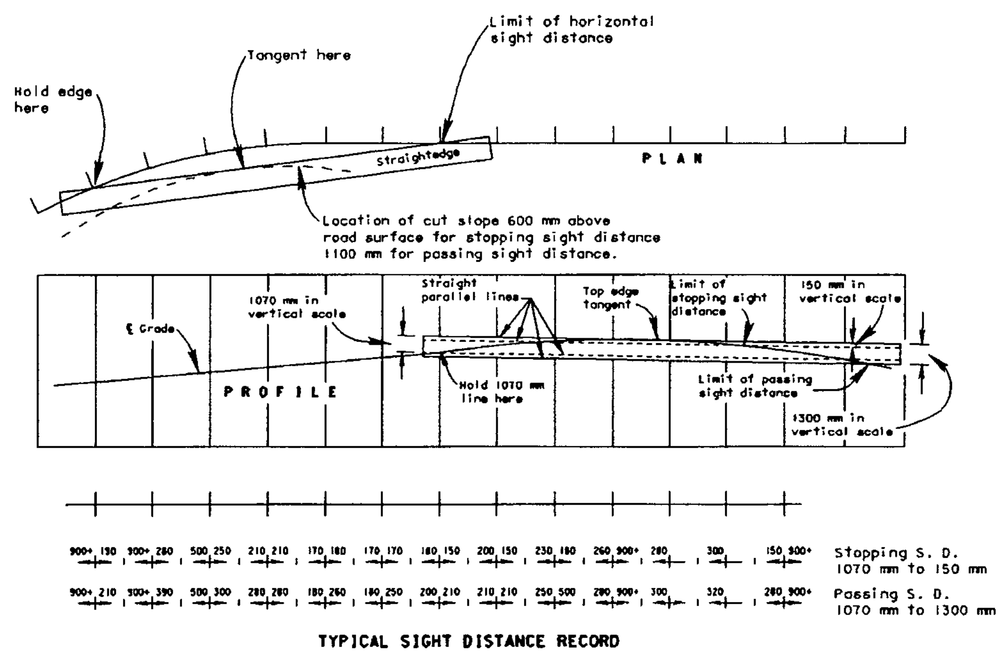


Figure III-3. Scaling and recording sight distances on plans.

distance may require adjustment in the normal highway cross section or change in alinement if the obstruction cannot be removed. Because of the many variables in alinement and cross sections and in number, type, and location of possible obstructions, specific study usually is necessary for each condition. Using design speed and a selected sight distance as a control, the designer should check the actual condition and make the necessary adjustments in the manner most fitting to provide adequate sight distance.

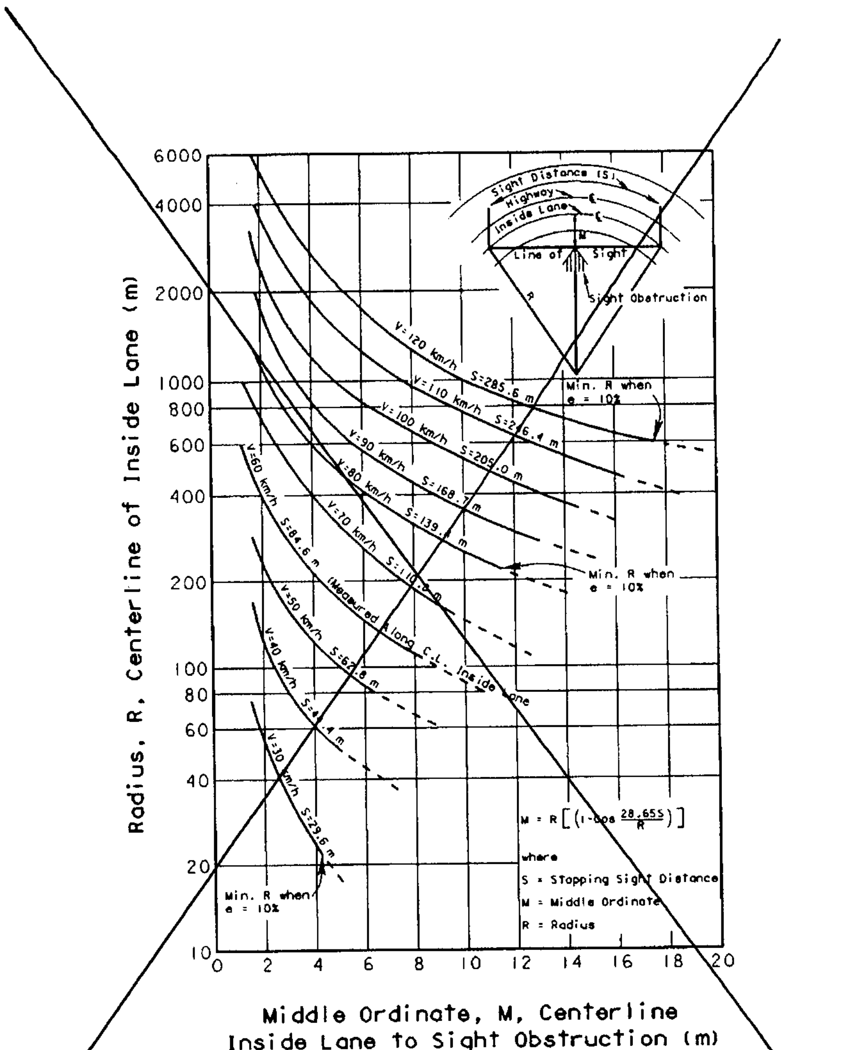
Stopping Sight Distance

For general use in design of a horizontal curve, the sight line is a chord of the curve, and the applicable stopping sight distance is measured along the centerline of the inside lane around the curve. Figure~~s~~ III-24~~A and III-24B are~~ **is a** design chart showing the required middle ordinates for clear sight areas to satisfy the ~~upper and lower values, respectively, of~~ stopping sight distance required for curves of various radii.

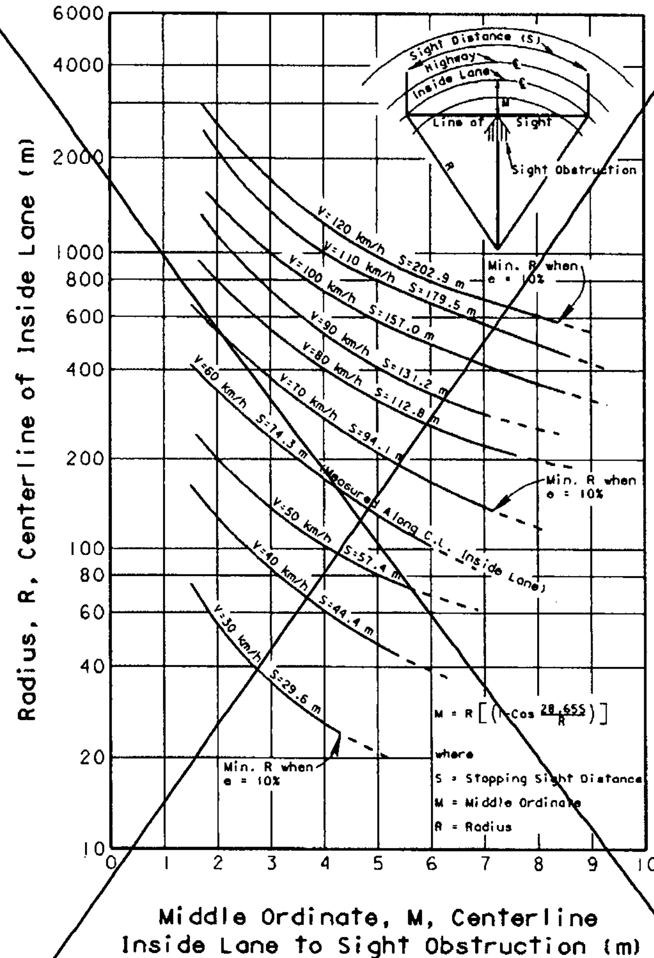
These design chart utilize the stopping sight distance values of Table III-1. ~~The overlap of the ranger at the higher design speeds in this table precludes development of a single design chart. However, use of the two charts in combination provides the same result; namely, a value that exceeds the minimum established in Figure III-24A, but not the value established in Figure III-24B, will provide acceptable stopping sight distance. As was the case with the stopping sight distances in Table III-1, a value at or approaching the upper limit~~ **These values** should be used as a minimum where conditions permit because of the increased safety that is provided.

The values ~~at or approaching the upper limit is~~ **in Figure III-24 are** an application of geometry for the several dimensions, as indicated in the diagrammatic sketch and formulas on the figures. These formulas apply only to circular curves longer than the sight distance for the pertinent design speed. For any design speed the relation of R to M is a straight line. Relations of R, M, and V in these chart forms can be quickly checked. For example, with an 80-km/h design speed and a curve with a 350 m radius, a clear sight distance with a middle ordinate **of 5.9m** ~~between 5.3 m (lower value) and 7.5 m (upper value)~~ is needed for stopping sight distance. ~~As another example, for a sight obstruction condition with M = 6.0 m on a curve with a 175 m radius, the resulting sight distance is approximately at the upper value of the range for a speed of 60 km/h.~~

Horizontal sight restrictions may occur where there is a cut slope on the inside of the curve. For the height criteria used for stopping sight distance of ~~1070~~ **1080**-mm height of eye and ~~150~~ **600**-mm height of object, a height of ~~600~~ **840**-mm may be used as the midpoint of the sight line where the cut slope usually obstructs sight. This assumes that there is little or no vertical curvature. For a highway with a 6.6-m traveled way, 1.8 m shoulders, 0.6-m ditch section, and 1:2 cut slopes, the sight obstruction is about 5.5 m outside the centerline of the inner lane. This is sufficient for adequate sight distance at 50 km/h when curves have a radius of about ~~80~~ **90** m or more and at 80 km/h when curves have a radius of about ~~300~~ **380** m or more. Curves sharper than these would require flatter slopes, benching, or other adjustments. At the other extreme, highways with normal lateral dimensions of more than ~~9~~ **14** m provide adequate stopping sight distances at curves over the entire range of design speeds and curves.



~~Figure III-24A. Range of uppervalues - relation between radius and value of middle ordinate necessary to provide stopping sight distance on horizontal curves.~~



~~Figure III-24(B). Range of lower values - relation between radius and value of middle ordinate necessary to provide stopping sight distance on horizontal curves.~~

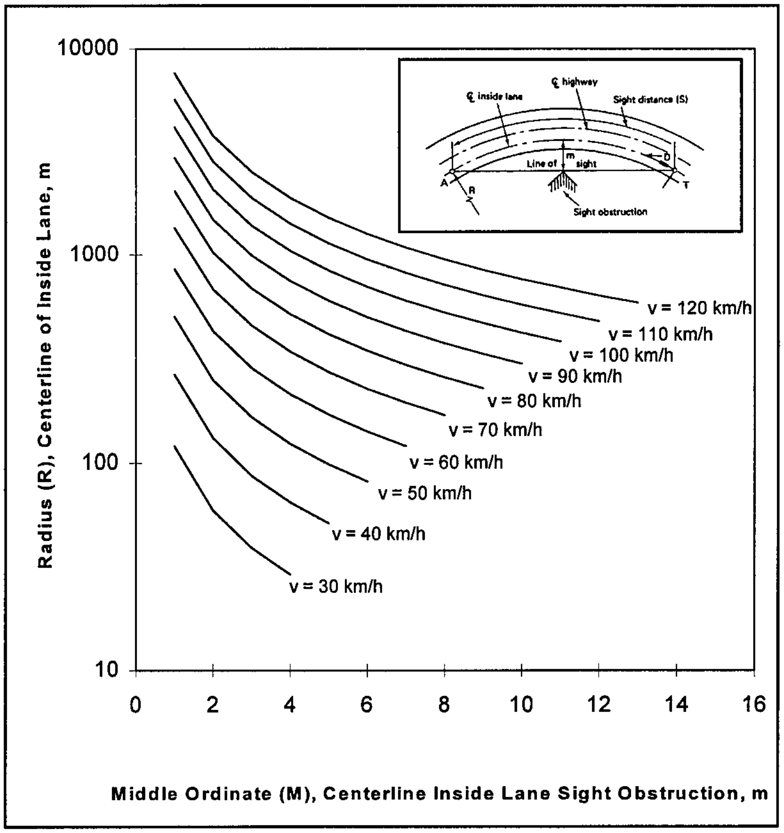


FIGURE III-24. Relationship between radius and value of middle ordinate necessary to provide stopping sight distance on horizontal curves.

In some instances retaining walls, concrete median safety barriers, and other similar features constructed on the inside of curves may be obstructions and must be checked for stopping sight distance adequacy. As an example, an obstruction of this type offset 1.2 m from the inside edge-of-traveled way has a middle ordinate of about 3.0 m. At 80 km/h this provides adequate sight distance when curves have radius of about ~~550~~ **685** m or more. If the offset width is increased to 3.3 m, a curve with a ~~320~~ **405** m (or more) radius provides adequate sight distance at the same 80-km/h speed. The same would be true for existing buildings or similar obstructions on the inside of curves.

When the needed stopping sight distance would not be available because the railing or a longitudinal barrier constitutes the obstruction, alternatives should be considered for both safety and economic reasons. The alternates are: increase the offset to the obstruction, increase the radius or reduce the design speed. However, any alternative selected should not require the width of the shoulder on the inside of the curve to exceed 3.6 m because the potential exists that drivers will use the shoulders in excess of that width as a passing or travel lane.

As can be seen from Figure III-24~~A and III-24B,~~ the method presented is only exact when both the vehicle and the sight obstruction are located within the limits of the simple horizontal curve. When either the vehicle or the sight obstruction is situated beyond the limits of the simple curve, the values obtained are only approximate. The same is true if either the vehicle or the sight obstruction, or both, is situated within the limits of spiral or a compound curve. In these instances, the value obtained would result in middle ordinate values slightly larger than those needed to satisfy the selected stopping sight distance. In many instances, the resulting additional clearance will not be significant. Whenever Figure~~s~~ III-24~~A and III-24B are~~ **is** not applicable, it is advisable to check the designs either by utilizing graphical procedures or by utilizing a computational method. Reference **(59)** provides a method for computing the needed values.

Vertical Curves

General Considerations

Vertical curves to effect gradual change between tangent grades may be any one of the crest or sag types depicted in Figure III-38. Vertical curves should be simple in application and should result in a design that is safe, comfortable in operation, pleasing in appearance, and adequate for drainage. The major control for safe operation on crest vertical curves is the provision of ample sight distances for the design speed. Minimum stopping sight distance should be provided in all cases. Wherever economically and physically feasible, more liberal stopping sight distances should be used. Further additional sight distance should be provided at decision points.

Consideration of motorists' comfort requires that the rate of change of grade be kept within tolerable limits. This consideration is most important in sag vertical curves where gravitational and vertical centrifugal forces act in the same direction. Appearance also should be considered. A long curve has a more pleasing appearance than a short one, which may give the appearance of a sudden break in the profile due to the effect of foreshortening.

Drainage of curbed roadways on sag vertical cures, Type III, Figure III-38, requires careful profile design to retain a gradeline of not less than 0.5 percent or, in some cases, 0.30 percent for the outer edges of the roadway. Although not desirable, flatter grades may be necessary in extenuating circumstances.

For simplicity the parabolic curve with an equivalent vertical axis centered on the vertical point of intersection (VPI) is usually used in roadway profile design. The vertical offsets from the tangent vary as the square of the horizontal distance from the curve end (point of tangency). The vertical offset from the tangent grade at any point along the curve is calculated as a proportion of the vertical offset at the VPI, which is AL/800, where the symbols are shown in Figure III-38. The rate of change of grade at successive points on the curve is a constant amount for equal increments of horizontal distance, and equals the algebraic difference between intersection tangent grades divided by the length of curve in meters, or A/L in percent per meter. The reciprocal L/A is the horizontal distance in meters required to effect a 1 percent change in gradient and is, therefore, a measure of curvature. The quantity L/A, termed "K," is useful in determining the horizontal distance from the vertical point of curvature (VPC) to the apex of Type I curves or to the low

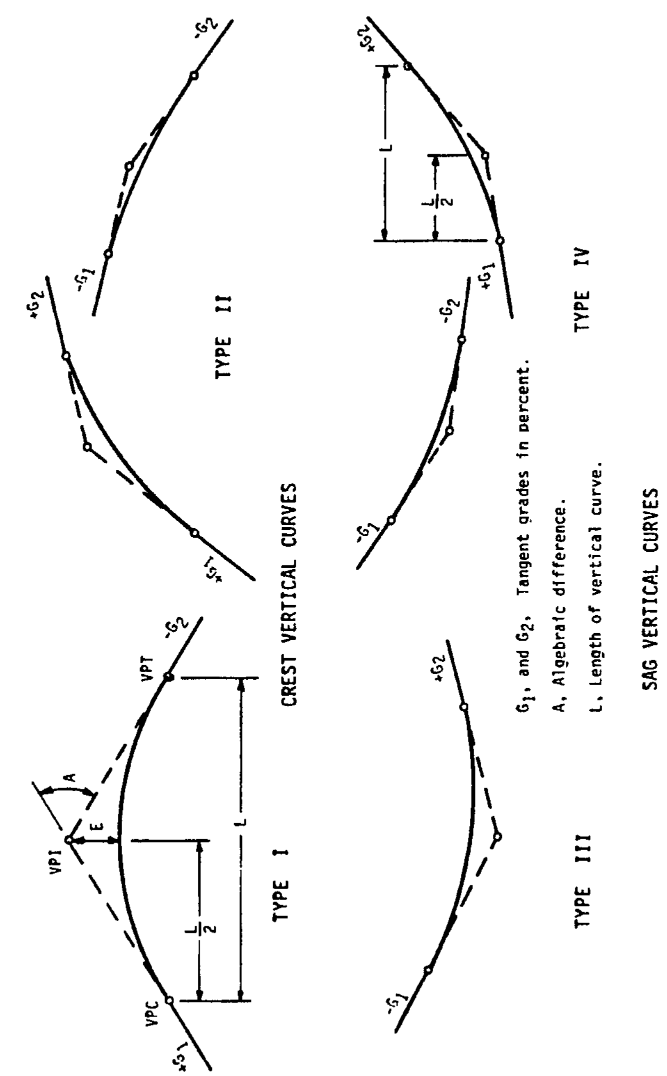


Figure III-38. Types of vertical curves.

point of Type III curves. This point where the slope is zero occurs at a distance from the VPC equal to K times the approach gradient. The K value is also useful in determining minimum lengths of vertical curves for various design speeds. Other details on parabolic vertical curves are found in textbooks on highway engineering.

On certain occasions, because of critical clearance or other controls, the use of unsymmetrical vertical curves may be required. Because the conditions dictating the need for these curves are infrequent, the derivation and use of the appropriate formulas have not been included herein. For use in such limited instances, refer to unsymmetrical curve data found in a number of highway engineering texts.

Crest Vertical Curves

Minimum lengths of crest vertical curves as determined by sight distance requirements generally are satisfactory from the standpoint of safety, comfort, and appearance. An exception may be at decision areas, such as sight distance to ramp exit gores, where longer lengths are necessary. Refer to the section in this chapter concerning decision sight distance.

The basic formulas for length of a parabolic vertical curve in terms of algebraic difference in grade and sight distance follow:

When S is less than L,

$$L = \frac{AS^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2} \qquad (1)$$

When S is greater than L,

$$L = 2S - \frac{200(\sqrt{h_1} + \sqrt{h_2})^2}{A} \qquad (2)$$

where: L = length of vertical curve, m;
S = sight distance, m;
A = algebraic difference in grades, percent;
h_1 = height of eye above roadway surface, m (normally ~~1070~~ **1080** mm/1000 mm/m)
h_2 = height object above roadway surface, m (normally ~~150~~ **600** mm/1000 mm/m)

When the height of eye and the height of object are ~~1070~~ **1080**mm and ~~150~~ **600** mm, respectively, as used for stopping sight distance.

When S is less than L,

$$\cancel{L = \frac{AS^2}{404}} \qquad \cancel{(3)}$$

$$L = \frac{\mathbf{AS^2}}{\mathbf{658}} \qquad (3)$$

When S is greater than L,

$$\cancel{L = 2S - \frac{404}{A}} \qquad \cancel{(4)}$$

$$L = 2S - \frac{658}{A} \qquad (4)$$

Design controls - stopping sight distance. The required lengths of vertical curves from formulas 3 and 4 for different values of A to provide the ~~upper value of the range of~~ stopping sight distances for each design speed are shown in Figure III-39. The solid lines give the required lengths, on the basis of rounded values of K as determined from these equations. ~~The dotted line for K = 39.39 gives unrounded values for 70 km/h for the comparison~~.

The short dashed curve at the lower left, crossing these lines, indicates where S = L. Note that to the right of the S = L line, the value of K, or length of vertical curve per percent change in A, is a simple and convenient expression of the design control. For each design speed this single value is a positive whole number that is indicative of the rate of vertical curvature. The design control in terms of K covers all combinations of A and L for any one design speed; thus, A and L need not be indicated separately in a design value tabulation. The selection of design curves is facilitated because the required length of curve in meters is equal to K times the algebraic difference in grades in percent, L = KA. Conversely, the checking of plans is simplified by comparing all curves with the design K value.

Table III-35 shows the computed K values for lengths of vertical curves as required for the ~~range of~~ values of stopping sight distances, Table III-1, for each design speed. For direct use in design, values of K are rounded as shown in the right column. The ~~upper~~, rounded values of K are plotted as the solid lines in Figure III-39. Rounded values of K are higher than computed values, but the differences are not significant.

Where S is greater than L (lower left in Figure III-39), computed values plot as a curve (as shown by the dashed line for 70 km/h) that bends to the left, and for small values of A the required lengths are zero because the sight line passes over the apex. This relation does not represent desirable design practice. Most of the States use a minimum length of vertical curve, expressed as either a single value, a range for different design speeds, or a function of A. Values now in use range form about 30 to 100 m. To recognize the distinction in design speed and to approximate the range of current practice, minimum lengths of vertical curves are expressed as about 0.6 times the design speed, or $L_{min} = 0.6V$ where V is in kilometers per hour and L is in meters. These terminal adjustments show as the vertical lines at the lower left of Figure III-39.

The above values of K derived when S is less than L also can be used without significant error where S is greater than L. As shown in Figure III-39, extension of the diagonal lines to meet the vertical lines for minimum lengths of vertical curves results in appreciable differences from the theoretical only where A is small and little or no additional cost is involved in obtaining longer vertical curves.

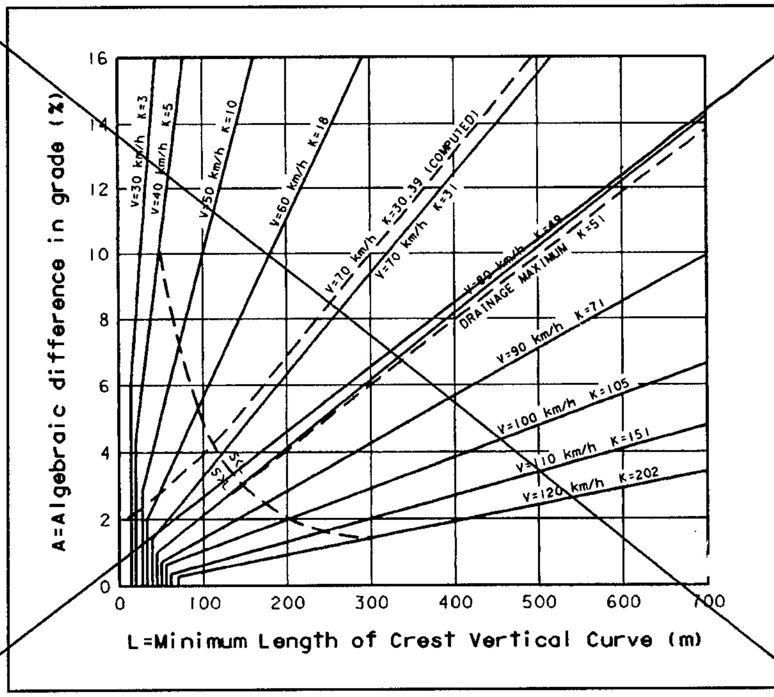
~~The lower K factors for each speed developed in Table III-35 were based on the assumption that most vehicles reduce their speed in inclement weather or on wet pavements. As discussed in greater detail in the section "Sight Distance" in this chapter, studies conducted in recent years of traffic on wet and dry pavements fail to support this stated hypothesis. Nevertheless, a range of values of stopping sight distance have been computed based on friction factors for wet pavements and on vehicular speeds equal to the average running speed and the design speed of the highway. Figure III-40 provides the required lengths of vertical curves for different values of A to provide the lower range of stopping sight distances for each design speed (based on the corresponding running speed).~~

~~Design Speed (km/h)~~	~~Assumed Speed for Condition (km/h)~~	~~Coefficient of Friction f~~	~~Stopping Sight Distance for Design (m)~~	~~Rate of Vertical Curvature, K (length (m) per % of A)~~	
				~~Computed~~	~~Rounded for Design~~
~~30~~	~~30-30~~	~~0.40~~	~~29.6-29.6~~	~~2.17-2.17~~	~~3-3~~
~~40~~	~~40-40~~	~~0.38~~	~~44.4-44.4~~	~~4.88-4.88~~	~~5-5~~
~~50~~	~~47-50~~	~~0.35~~	~~57.4-62.8~~	~~8.16-9.76~~	~~9-10~~
~~60~~	~~55-60~~	~~0.33~~	~~74.3-84.6~~	~~13.66-17.72~~	~~14-18~~
~~70~~	~~63-70~~	~~0.31~~	~~94.1-110.8~~	~~21.92-30.39~~	~~22-31~~
~~80~~	~~70-80~~	~~0.30~~	~~112.8-139.4~~	~~31.49-48.10~~	~~32-49~~
~~90~~	~~77-90~~	~~0.30~~	~~131.2-168.7~~	~~42.61-70.44~~	~~43-71~~
~~100~~	~~85-100~~	~~0.29~~	~~157.0-205.0~~	~~61.01-104.02~~	~~62-105~~
~~110~~	~~91-110~~	~~0.28~~	~~179.5-246.4~~	~~79.75-150.28~~	~~80-151~~
~~120~~	~~98-120~~	~~0.28~~	~~202.9-285.6~~	~~101.90-201.90~~	~~102-202~~

~~**Table III-35. Design controls for crest vertical curves.**~~

Initial Speed (km/h)	Stopping Sight Distance for Design (m)	Rate of Vertical Curvature, K [length (m) per % of A]	
		Crest Curves	Sag Curves
30	31.0	2	4
40	45.9	3	8
50	63.1	6	12
60	82.5	10	17
70	104.2	17	22
80	128.2	25	29
90	154.4	36	36
100	182.9	51	44
110	213.7	69	53
120	246.7	93	62

Table III-35. Recommended design controls for vertical curves.



~~Figure III-39. Design controls for crest vertical curves, for stopping sight distance - upper range.~~

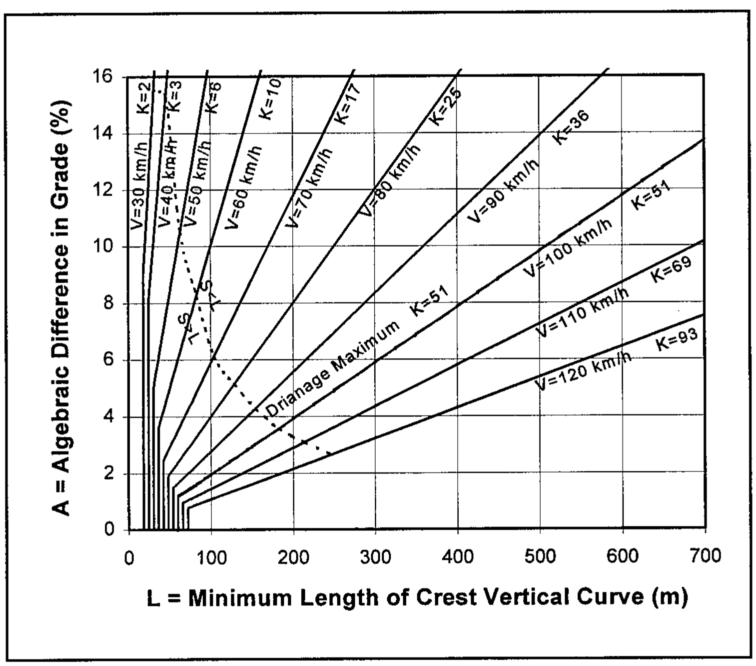
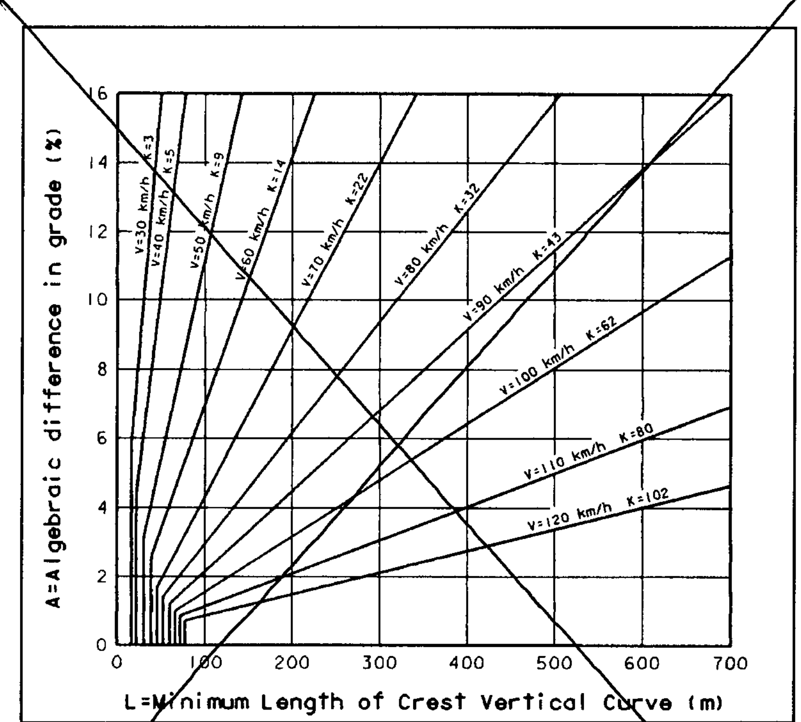


Figure III-39. Design control for crest vertical curves for stopping sight distance.

For night driving on highways without lighting, the length of visible roadway is that roadway which is directly illuminated by the headlights of the vehicle. For certain conditions, the minimum stopping sight distance values used for design exceed the length of visible roadway. First, vehicle headlights have limitations on the projection distance for the light intensity levels that are required for visibility purposes. When headlights are operated on low-beam, the reduced candlepower at the source plus the downward projection angle significantly restrict the length of visible roadway surface. Thus, particularly for high-speed conditions, stopping sight distance values exceed road-surface visibility distances afforded by the low-beam headlights regardless of whether the roadway profile is level or vertically curving. Secondly, for crest vertical curves the area forward of the headlight beam point of tangency with the roadway surface is shadowed and receives only indirect illumination. Since the headlight mounting height (typically about 600 mm) is lower than the driver eye height (~~1070~~ **1080** mm for design), the sight distance to an illuminated object ~~150~~ **600** mm in height is controlled by the height of the vehicle headlights rather than by the direct line of sight. Any object within the shadow zone must be high enough to extend into the headlight beam to be directly illuminated. On the basis of formula 1, the bottom of the headlight beam is about 400 mm above the roadway at a distance ahead of the vehicle equal to the low value of the range of stopping sight distance. Although the vehicle headlight system does limit roadway visibility length as mentioned above, there is some mitigating effect in that other vehicles, whose taillight height typically varies from 450 to 600 mm, and other sizable hazardous objects receive direct lighting from headlights at stopping sight distance values used for design. It also may be rationalized that drivers are aware that visibility at night is less than during the day regardless of road and street design features, and that vehicle operators are thus more attentive and alert.

There is a level point of minute length on a crest vertical curve of Type I (Figure III-38), but no difficulty with drainage on highways with curbs is experienced if the curve is sharp enough so that a minimum grade of 0.30 percent is reached at a point about 15 m from the crest. This corresponds to a 51-m per percent change in grade; this line is plotted in Figure III-39 as the drainage maximum. All combinations above or to the left of this line would satisfy this criterion for drainage. The combinations below and to the right of this line involve flatter vertical curves. Special attention is needed in these cases to ensure proper pavement drainage near the apex of crest vertical curves. It is not intended that a K value of 51 be considered a design maximum, but merely the value beyond which drainage must be more carefully designed.



~~Figure III-40. Design controls for crest vertical curves for stopping sight distance - lower range.~~

Sag Vertical Curves

At least four different criteria for establishing lengths of sag vertical curves are recognized to some extent. These are (1) headlight sight distance, (2) rider comfort, (3) drainage control, and (4) a rule-of-thumb for general appearance.

Headlight sight distance has been used directly by some authorities and for the most part is the basis for determining the length of sight distance used herein. When a vehicle traverses a sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. General use is being given to a headlight height of 600 mm and a 1° upward divergence of the light beam from the longitudinal axis of the vehicle. The upward spread of the light beam provides some additional visible length but this is generally ignored. The following formulas show the S, L, and A relation, using S as the distance between the vehicle and point where the 1° angle of light ray intersects the surface of the roadway:

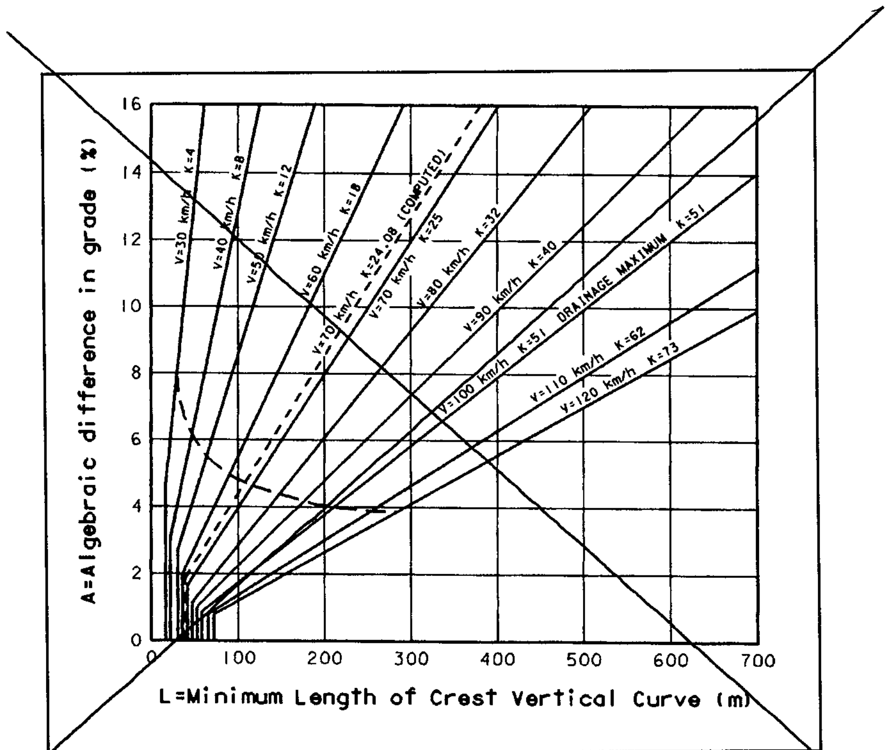
When S is less than L,

$$L = \frac{AS^2}{200[0.6 + S(\tan 1°)]} = \frac{AS^2}{120 + 3.5S} \qquad (7)$$

When S is greater than L,

$$L = 2S - \frac{200[0.6 + S(\tan 1°)]}{A} = 2S - \left(\frac{120 + 3.5S}{A}\right) \qquad (8)$$

where: L = length of sag vertical curve, m;
S = light beam distance, m; and
A = algebraic difference in grades, percent.



~~Figure III-41. Design controls for sag vertical curves - upper range.~~

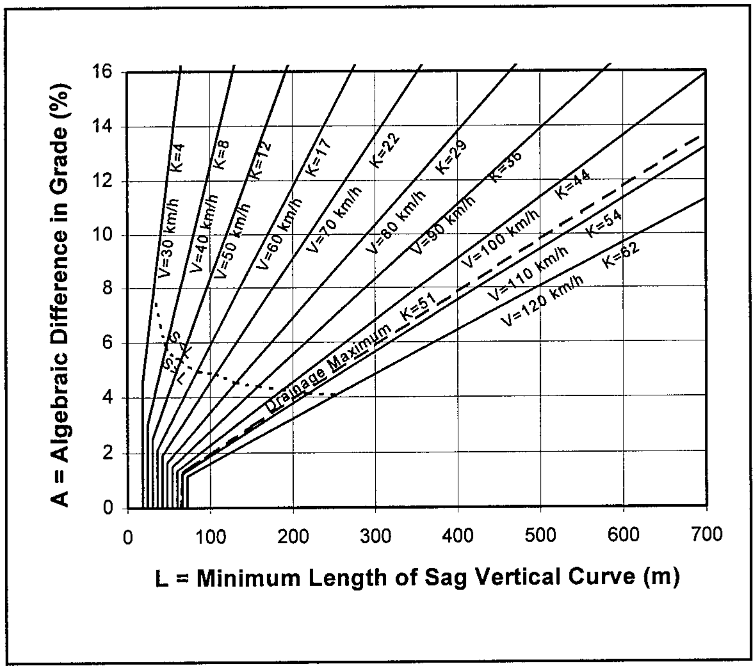
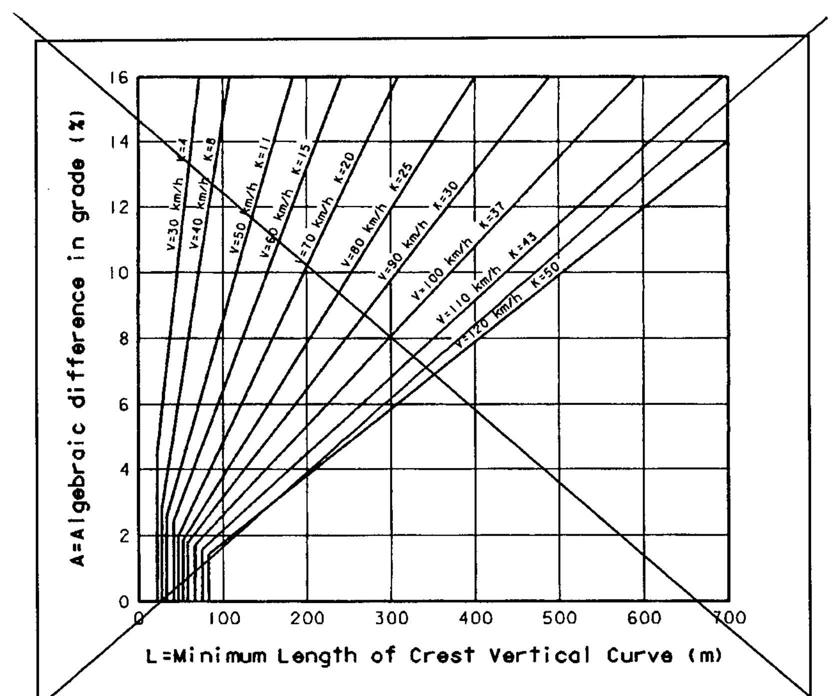


Figure III-41. Design controls for sag vertical curves.



~~Figure III-42. Design controls for sag vertical curves—lower range.~~

For overall safety on highways, a sag vertical curve should be long enough so that the light beam distance is nearly the same as the stopping sight distance. Accordingly, it is pertinent to use stopping distances for different design speeds as the S value in the above formulas. The resulting lengths of vertical curves for the ~~upper value of the range of~~ **recommended** stopping sight distances for each design speed are shown in Figure III-41 with solid lines using rounded K values as was done for crest vertical curves. ~~and the dotted line for K = 26.6 being an unrounded value for 70 km/h for comparison. Figure III-42 provides the lengths of sag vertical curves for various algebraic differences in grades for the lower range of stopping sight distance.~~

The comfort effect of change in vertical direction is greater on sag than on crest vertical curves because gravitational and centrifugal forces are combining rather than opposing forces. Comfort due to change in vertical direction is not measured readily because it is affected appreciably by vehicle body suspension, tire flexibility, mass carried, and other factors. The limited attempts at such measurements have led to the broad conclusion that riding is comfortable on sag vertical curves when the centrifugal acceleration does not exceed 0.3 m/s^2. The general expression for such a criterion is:

$$L = \frac{AV^2}{395}$$

where L and A are the same as in previous formulas, and V is the design speed, km/h.

The length of vertical curve required to satisfy this comfort factor at the various design speeds is only about 50 percent of that required to satisfy the headlight sight distance requirement for the normal range of design conditions.

Drainage affects design of vertical curves of Type III (Figure III-38) where curbed sections are used. An approximate criterion for sag vertical curves is the same as that expressed for the crest conditions, that is, providing a minimum grade of 0.30 percent within 15 m of the level point. This criterion plots the same or very close to the same as the line shown in Figure III-41 for the 100 km/h, K = 51. The drainage requirement differs from other criteria in that the length **of sag vertical curve determined for it is a maximum, whereas, the length** for any other criterion is a minimum. The maximum length of the drainage criterion is greater than the minimum length for other criteria up to 100 km/h and is nearly equal for other criteria up to 120 km/h for minimum-length vertical curves.

For general appearance, some use formerly was made of rule-of-thumb for length of sag vertical curves wherein the minimum value of L is 30A or, in Figure III-41, K = 30. This approximation is a generalized control for small or intermediate values of A. Compared with headlight sight distance, it corresponds to a design speed between 70 and 80 km/h. On high-type highways longer curves are deemed appropriate to improve appearance.

From the preceding it is evident that design controls for sag vertical curves differ from those for crests, and separate design values are needed. The headlight sight distance basis appears to be the most logical for general use, and the values determined for stopping sight distances are within the limits recognized in current practice. It is concluded to use this criterion to establish design values for a range of lengths of sag vertical curves. As in the case of crest vertical curves, it is convenient to express the design control in terms of the K rate for all values of A. This entails some deviation from the computed values for small values of A, but the differences are not significant. Table III-37 shows the ~~range of~~ computed values and the rounded values of K selected as design controls. The lengths of sag vertical curves on the basis of the design speed values of K are shown by the solid lines in Figure III-41. It is to be emphasized that these lengths are ~~minimum values~~ based on design speed; longer curves are desired wherever feasible, but special attention to drainage must be exercised where a K value in excess of 51 is used.

~~Assumed Design Speed (km/h)~~	~~Speed for Condition (km/h)~~	~~Coefficient of Friction f~~	~~Stopping Sight Distance for Design (m)~~	~~Rate of Vertical Curvature, K (length (m) per % of A~~ Computed	~~Rate of Vertical Curvature, K~~ ~~Rounded for Design~~
~~30~~	~~30-30~~	~~0.40~~	~~29.6-29.6~~	~~3.88-3.88~~	~~4-4~~
~~40~~	~~40-40~~	~~0.38~~	~~44.4-44.4~~	~~7.11-7.11~~	~~8-8~~
~~50~~	~~47-50~~	~~0.35~~	~~57.4-62.8~~	~~10.20-11.54~~	~~11-12~~
~~60~~	~~55-60~~	~~0.33~~	~~74.3-84.6~~	~~14.45-17.12~~	~~15-18~~
~~70~~	~~63-70~~	~~0.31~~	~~94.1-110.8~~	~~19.62-24.08~~	~~20-25~~
~~80~~	~~70-80~~	~~0.30~~	~~112.8-139.4~~	~~24.62-31.86~~	~~25-32~~
~~90~~	~~77-90~~	~~0.30~~	~~131.2-168.7~~	~~29.62-39.95~~	~~30-40~~
~~100~~	~~85-100~~	~~0.29~~	~~157.0-205.0~~	~~36.71-50.06~~	~~37-51~~
~~110~~	~~91-110~~	~~0.28~~	~~179.5-246.4~~	~~42.95-61.68~~	~~43-62~~
~~120~~	~~98-120~~	~~0.28~~	~~202.9-285.6~~	~~49.47-72.72~~	~~50-73~~

~~**Table III-37. Design controls for sag vertical curves.**~~

Initial Speed (km/h)	Stopping Sight Distance for Design (m)	Rate of Vertical Curvature, K [length (m) per % of A] Crest Curves	Sag Curves
30	**31.0**	**2**	**4**
40	**45.9**	**3**	**8**
50	**63.1**	**6**	**12**
60	**82.5**	**10**	**17**
70	**104.2**	**17**	**22**
80	**128.2**	**25**	**29**
90	**154.4**	**36**	**36**
100	**182.9**	**51**	**44**
110	**213.7**	**69**	**53**
120	**246.7**	**93**	**62**

Table III-37. Recommended design controls for vertical curves.

Minimum lengths of vertical curves for flat gradients also are recognized for sag conditions. The values determined for crest conditions appear to be generally suitable for sags. Lengths of sag vertical curves, shown as vertical lines in Figure III-41, are equal to 0.6 times the design speed.

Sag vertical curves shorter than the length computed from Table III-37 may be justified for economic reasons in cases where an existing element, such as a structure not ready for replacement, controls the vertical profile. In certain cases ramps may also be designed with shorter sag vertical curves. Fixed source lighting is desirable in these cases. For street design, some engineers accept design of a sag or crest where A is about 1 percent or less without a length of calculated vertical curve. However, field modifications during construction usually result in constructing the equivalent to a vertical curve, even if short.

TABLE I-1. NECESSARY REVISIONS TO THE AASHTO GREEN BOOK

Page	Description
28	Paragraph 4 to be revised to include the use of deceleration rates used to calculate stopping sight distances
62	Section entitled "Operating Speed" to be revised as indicated in Appendix I (pg. I-2).
62	First paragraph of section entitled "Design Speed" to be revised as indicated in Appendix I (pg. I-2).
63	Third paragraph to be revised as indicated in Appendix I (pg. I-3).
63	Fifth paragraph to be revised as indicated in Appendix I (pg. I-3).
65	Fifth paragraph to be revised as indicated in Appendix I (pg. I-5).
66	Second paragraph to be revised as indicated in Appendix I (pg. I-5).
67	Second paragraph to be revised as indicated in Appendix I (pg. I-6).
67	Fourth paragraph to be revised as indicated in Appendix I (pg. I-6).
68	Fifth paragraph to be revised as indicated in Appendix I (pg. I-7).
69	Second paragraph to be revised as indicated in Appendix I (pg. I-8).
119	Second paragraph to be revised as indicated in Appendix I (pg. I-11).
119	Formula to be revised as indicated in Appendix I (pg. I-12).
119-123	Text from final paragraph on page 120 to second paragraph on page 123 to be deleted and replaced with text as indicated in Appendix I (pg. I-13).
120	Table III-1 to be replaced with Table 58 as indicated in Appendix I (pg. I-15).
122	Figure III-1 to be deleted. Revise remaining figure numbers after deletion of this figure.
123	Formula to be revised as indicated in Appendix I (pg. I-17).
123	Final paragraph to be revised as indicated in Appendix I (pg. I-17).
124	Second paragraph to be revised as indicated in Appendix I (pg. I-17).
124	Formula to be revised as indicated in Appendix I (pg. I-17).
124	Third paragraph to be revised as indicated in Appendix I (pg. I-18).

Page	Description
125	Table III-2 to be revised.
125	First paragraph to be revised as indicated in Appendix I (pg. I-18).
127	Table III-3 values to be revised based on revised object and eye heights.
127	Third paragraph to be revised to represent revised object and eye heights.
136	Final paragraph to be revised as indicated in Appendix I (pg. I-19).
137	Paragraph entitled " Stopping sight distance object" to be revised as indicated in Appendix I (pg. I-19).
138	Final paragraph to be revised as indicated in Appendix I (pg. I-21).
139	Second paragraph to be revised as indicated in Appendix I (pg. I-21).
140	Figure III-3 to be revised to indicate the revised object and eye heights for both stopping and passing sight distances.
219-223	Section entitled "Stopping sight distance" to be revised as indicated in Appendix I (pg. I-23 to I-26).
220	Figure III-24 to be revised.
221	Figure III-25 to be deleted.
223	Section entitled "Passing sight distance" to be revised to indicate the revised driver eye height.
282	Revise values of h_1 and h_2 in equations (1) and (2). In the paragraph that follows revise the object and driver eye heights as indicated in Appendix I (pg. I-29).
282-283	Revise formulae (3) and (4) as indicated in Appendix I (pg. I-29).
283-288	Section entitled "Design Controls-stopping sight distance" to be revised as indicated in Appendix I (pg. I-30 to I-33)
284	Table III-35 to be revised as indicated in Appendix I (pg. I-31).
284	Figure III-39 to be revised.
287	Formulae (5) and (6) must be revised to incorporate new driver eye height.
287	Table III-36 to be revised to incorporate new formulae for equations (5) and (6).
288-293	Section entitled "Sag vertical curves" to be revised as indicated in Appendix I (pg. I-34 to I-37).

Page	Description
289	Figure III-41 to be revised.
290	Figure III-42 to be deleted.
292	Table III-37 to be replaced by Table 60 from "Final Draft Report".
418	Section entitled "Sight Distance"to be revised to incorporate revised object and eye heights.
419	Table V-2 to be revised to reflect revised object and eye heights.
420	Table V-3 to be revised to reflect revised eye height.
427	Table V-9 to be revised to reflect revised eye height
429	Revise stopping sight distance range referred to in paragraph entitled "Sight Distance".
446	Section entitled "Sight Distance" to be revised to incorporate revised object and eye heights.
446	Table V-11 to be revised to incorporate revised object and eye heights.
447	Table V-12 to be revised to incorporate revised driver eye height.
461	Section entitled "Sight Distance" to be revised to incorporate revised object and eye heights.
462	Table VI-2A to be revised to incorporate revised object and eye heights.
462	Table VI-2B to be revised to incorporate revised driver eye height.
472	Review stopping sight distance range referred to in paragraph entitled "Sight Distance"
490	Table VII-3 to be revised to incorporate revised eye and objects heights. Check legend of last column (PSD?)
710	Second paragraph to be revised to incorporate revised driver eye height.
710	Figure IX-38 to be revised.
715	Figure IX-41 to be revised.
722	Section entitled "General Considerations" to be revised to replace coefficients of friction formulation with revised deceleration rate formulation.
722	Table IX-10 to be revised. (SSD values)

Page	Description
722	Table IX-11 to be revised.(SSD values)
723	Section entitled "General Considerations" to be revised to incorporate new driver eye and object heights.
724	Figure IX-44 to be revised. (SSD values)
725	Figure IX-45 to be revised. (SSD values)
797-801	Formulae, discussion, Table IX-21 and Figures IX-78,79 regarding sight distance calculation to be reviewed due to the change in the formulation of the calculation of Stopping sight distance.