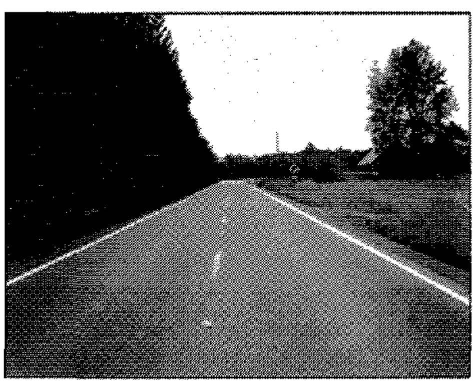
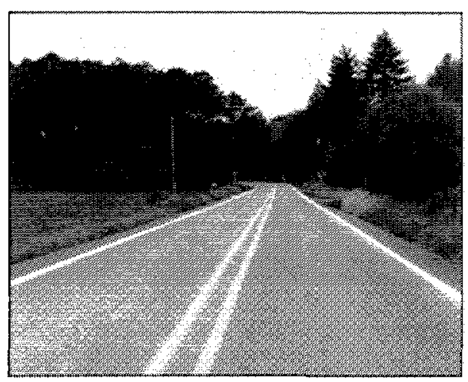
Horizontal Alignment Design Consistency for Rural Two-Lane Highways

Publication No. FHWA-RD-94-034

January 1995





U.S. Department of Transportation
Federal Highway Administration

Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

FOREWORD

U.S. policy for designing rural horizontal alignments relies on the selection and application of design speeds to achieve consistency. However, since drivers typically select their speeds on long tangents oblivious to the design speeds on downstream horizontal curves, large speed reductions approaching and entering curves - and curve operating speeds in excess of curve design speeds - can result when the designer or design policy underestimates or disregards driver speed preferences. The resulting operating speed inconsistencies are believed to be a common cause of curve-related accidents.

The research documented in this report confirms that accident rates are higher at horizontal curves that experience greater tangent-to-curve speed reductions. Models and related computer software were developed to assist engineers in plotting expected profiles of operating speed and driver workload as a function of horizontal alignment. User's manuals for that software are being published separately as Report Nos. FHWA-RD-94-038 (English units) and FHWA-RD-94-039 (metric units).

This study was the first in a new series involving design consistency. The models developed in this and subsequent related efforts will undergo additional validation testing and will then be incorporated into the Design Consistency Module of the Interactive Highway Safety Design Model (IHSDM). The IHSDM will consist of a series of interactive computer programs enabling roadway designers and design reviewers to assess the potential safety effects of specific geometric design decisions. A conceptual plan for the development of the IHSDM was recently published as Report No. FHWA-RD-93-122.

Sufficient copies of this report are being distributed to provide a minimum of two copies for each Regional office and three copies to each Division office and State highway agency. Direct distribution is being made to the Division offices. Additional copies for the public are available from the National Technical Information Service (NTIS), Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161. A small charge will be imposed by NTIS.

Lyle Saxton, Director
Office of Safety and Traffic
Operations Research and Development

NOTICE

Technical Report Standard Title Page

1. Report No.
FHWA-RD-94-034

2.
PB95-170338

3. Recipient's Catalog No.

4. Title and Subtitle

HORIZONTAL ALIGNMENT DESIGN CONSISTENCY FOR RURAL TWO-LANE HIGHWAYS

5. Report Date

6. Performing Organization Code

7. Author(s)

R.A. Krammes, R.Q. Brackett, M.A. Shafer, J.L. Ottesen, I.B. Anderson, K.L. Fink, K.M. Collins, O.J. Pendleton, C.J. Messer

8. Performing Organization Report No.

Research Report 7193-5

9. Performing Organization Name and Address

Texas Transportation Institute
The Texas A&M University System
College Station, Texas 77843

10. Work Unit No. (TRAIS)
3A5A

11. Contract or Grant No.
DTFH61-91-C-00050

12. Sponsoring Agency and Address

Office of Safety and Traffic Operations R&D
Federal Highway Administration
U.S. Department of Transportation
6300 Georgetown Pike
McLean, VA 22101-2296

13. Type of Report and Period Covered
Final Report
Oct. 1991-Sept. 1993

14. Sponsoring Agency Code

15. Supplementary Notes

Contracting Officer's Technical Representative (COTR): Justin True, HSR-20

16. Abstract

The state of the practice in highway geometric design consistency was determined through a review of U.S. and foreign geometric design policy, practice, and research. Models, and a menu-driven microcomputer procedure for their use, were developed for operating-speed and driver-workload consistency evaluations of rural two-lane highway horizontal alignments. The operating-speed model was calibrated based upon speed and geometry data for 138 horizontal curves and 78 of their approach tangents in 5 States. The driver workload model was calibrated based upon 2 occluded vision test studies on a total of 55 subjects. The operating-speed data suggest that 85th percentile speeds generally exceed the design speed of horizontal curves whose design speed is less than drivers' desired speed (i.e., 85th percentile speed on long tangents). A preliminary evaluation comparing model-estimated operating-speed reductions versus degree of curvature as predictors of accident experience was conducted using a data base of 1,126 curve sites in 3 States. The evaluation suggests that accident experience increases as the required speed reduction from an approach tangent to a horizontal curve increases.

17. Key Words
Rural two-lane highway, horizontal curves, accident analysis, operating speed, driver workload

18. Distribution Statement
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	137	

Form DOT F 1700.7 (8-72) Reproduction of form and completed page is authorized.

ACKNOWLEDGMENTS

The authors acknowledge two groups of individuals who made important contributions to the research. First, the following individuals provided valuable information and insights on geometric design policy and practices in their respective countries:

- Great Britain: R. Chaplin and M. Garnham, The Department of Transport; A. Whittingham, Wootton Jeffreys Consultants Limited.

- France: T. Brenac, Service d'Etudes Techniques des Routes et Autoroutes.

- Germany: R. Lamm, University of Karlsruhe; C. Levin and H.V. Matthess, Technical University of Darmstadt.

- Sweden: K. Renström, Swedish National Road Administration; A. Carlsson and U. Hammarström, Swedish Road and Traffic Research Institute.

- Switzerland: E. Boppart, Swiss Federal Institute of Technology; A. Herman, Federal Office of Highways.

Second, the following individuals coordinated the assistance of their respective State departments of transportation in identifying study sites and obtaining highway geometry and accident data:

- New York: C. Torre, M. Traver.

- Oregon: D. Greenberg.

- Pennsylvania: B. Snyder.

- Texas: M. Marek.

- Washington: D. Gripne, L. Hinson, L. Messmer.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

TABLE OF CONTENTS

TABLE OF CONTENTS (CONTINUED)

TABLE OF CONTENTS (CONTINUED)

LIST OF FIGURES

LIST OF FIGURES (CONTINUED)

LIST OF TABLES

LIST OF TABLES (CONTINUED)

1. INTRODUCTION

BACKGROUND

U.S. policy for the design of rural highway alignments is stated in *A Policy on Geometric Design of Highways and Streets* by the American Association of State Highway and Transportation Officials (AASHTO).[1] The policy relies on the selection and application of a design speed to achieve consistency among individual alignment elements.

Growing numbers of geometric design researchers and practitioners recognize that the design-speed concept as applied in the United States is not able to guarantee consistent alignments. Australia and several European countries have revised their implementation of the design-speed concept for rural two-lane highway horizontal alignments to address consistency questions more explicitly. Several procedures for evaluating alignment consistency have been proposed in the United States, but to date none have been adopted.

Therefore, the study documented herein was undertaken to evaluate the state of the practice in geometric design consistency and to develop models and procedures for evaluating the consistency of alternative designs. The scope was limited to rural two-lane highway horizontal alignments, which have been the focus of most previous work on design consistency because they are the most common source of consistency problems. This report documents the methodology, analysis results, key findings, conclusions, and recommendations of the study.

ORGANIZATION OF THE REPORT

The report is organized into six chapters. This introductory chapter describes the research scope and objectives, background, and the research philosophy and approach. Chapter 2 provides a critical review of current U.S. design policy related to horizontal-alignment consistency. Chapter 3 presents the analysis methodology and results, speed-profile model, and microcomputer procedure for operating-speed consistency evaluation. Chapter 4 presents the analysis methodology and results, workload-profile model, and microcomputer procedure for driver-workload consistency evaluation. Chapter 5 compares speed-reduction estimates and degree of curvature as predictors of accident experience on horizontal curves. Chapter 6 summarizes the study effort and findings and provides conclusions and recommendations.

OBJECTIVES

This study, "State-of-the-Practice Geometric Design Consistency," is the first major research on design consistency that has been sponsored by the Federal Highway Administration since the work by Messer, Mounce, and Brackett in the late 1970's.[2-5] The research objectives are to:

- Evaluate U.S., European, Australian, and Canadian geometric design consistency policy, practice, and research.

- Collect and analyze traffic operations, geometry, and accident data to evaluate the nature, magnitude, and consequences of operating-speed inconsistencies on rural two-lane highways.

- Collect and analyze data on driver workload at horizontal curves using the occluded vision test method.

- Develop models—and procedures for their use—to evaluate the consistency of alternative horizontal alignment designs for rural two-lane highways.

LIMITATIONS OF CURRENT U.S. DESIGN POLICY

The design-speed concept as the basis for alignment consistency on rural highways originated in the 1930's in response to increasing accident rates at horizontal curves. Many of the rural highways in use at that time had been designed and/or constructed for horse-drawn vehicles whose low operating speeds were not a controlling design issue. Many of the existing curves were not safe for the speeds at which motor vehicles could operate on their approach tangents. Therefore, as motor vehicles became predominant, accidents on these horizontal curves increased. The design-speed concept for alignment design was adopted to overcome this accident problem.

As originally conceived, the design-speed concept had two fundamental principles:

- All curves along an alignment should be designed for the same speed.

- The design speed should reflect the uniform speed at which a high percentage of drivers desire to operate.

The design-speed concept was intended to ensure alignment consistency—measured with respect to the uniformity of operating speeds along the alignment. A consistent alignment would allow most drivers to operate safely at their desired speed along the entire alignment, whereas an inconsistent alignment would require most drivers to decelerate from their desired speed in order to safely traverse certain alignment elements.

In many respects, the nature of the safety problem posed by horizontal curves today is the same as in the 1930's. That is, many of the older alignments in current use have horizontal curves whose design speeds are lower than the desired speeds of the majority of today's drivers. A consequence of this inconsistency between design and desired speeds is accident rates on horizontal curves that are 1.5 to 4.0 times greater than on tangents.[6]

The design-speed concept can offer acceptable uniform operating speeds only for drivers whose desired speeds do not exceed the design speed. Unfortunately, our existing design-speed-based alignment policy permits the selection of a design speed that is less than the desired speeds of a majority of the drivers, but it neither recognizes nor compensates for the operating-speed inconsistencies that inevitably result. Therefore, adherence to current U.S. design policy no longer guarantees that the resulting alignments will be consistent with respect to the uniformity of operating speeds. Chapter 2 provides a more detailed critique of the design-speed concept.

CONCEPTUAL FRAMEWORK

The causes and consequences of alignment inconsistencies are best explained within the context of driver-vehicle-roadway interactions. Figure 1 illustrates these interactions. It also depicts two candidate measures of consistency: operating speed and driver mental workload.

The driving task is principally an information-processing and decision-making task. The roadway geometry and other factors (including the roadside environment, weather, traffic control devices, traffic conditions, etc.) are the primary inputs to the driving task. The outputs are control actions that translate into vehicle operations. The operation of the vehicle can be observed and characterized by traffic measurements (including operating speed, lateral placement, etc.). Understanding how driver characteristics (particularly expectancy and attention level) affect driver information processing is a key to understanding how roadway geometry influences vehicle operations and safety.

Driver workload is a principal measure of driver information processing. It is defined as "the time rate at which drivers must perform a given amount of work or driving task."[(4)] The work is mental (i.e., information processing) rather than physical. Driver workload requirements increase with increasing geometric complexity. Driver workload also increases as the time available to process a given amount of information decreases due to increases in speed and/or reductions in sight distance.

Expectancy influences a driver's attention level and, consequently, the rate at which a driver processes the information necessary to perform the driving task (i.e., driver workload). Expectancy is defined as "an inclination, based upon previous experience, to respond in a set manner to a roadway or traffic situation."[(7)] It represents drivers' tendencies to react to what they expect rather than to the roadway or traffic situation as it actually exists.

Two basic forms of driver expectancy are a priori and ad hoc.[(8)] A priori expectancies are long-held expectancies that drivers bring to the driving task based upon their collective previous experience. Unusual geometric features (e.g., a one-lane bridge), features with unusual dimensions (e.g., a very long and/or very sharp horizontal curve), and features combined in unusual ways (e.g., an intersection hidden beyond a crest vertical curve) may violate a priori expectancies.

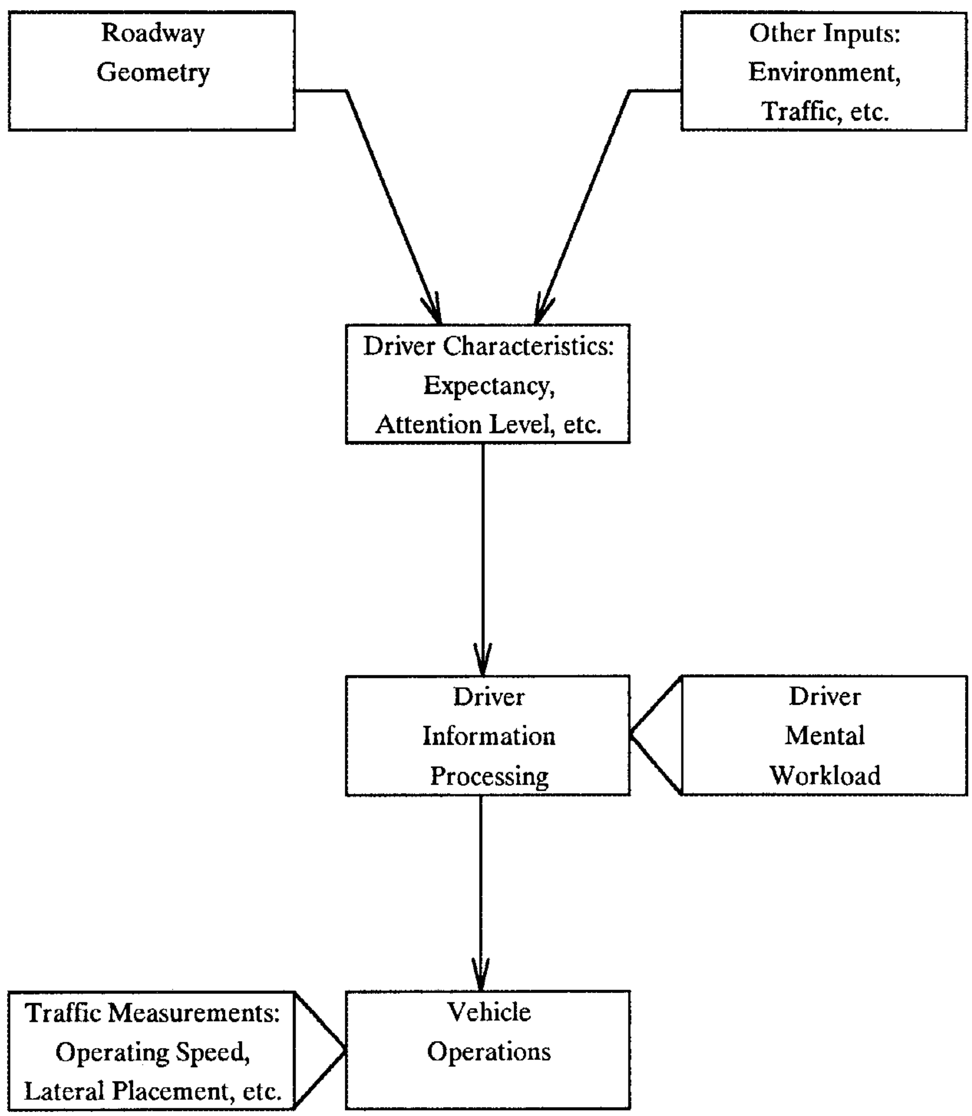


Figure 1. Flowchart of driving task inputs and outputs.

Ad hoc expectancies, on the other hand, are short-term expectations that drivers formulate during a particular trip on a particular roadway; they are based upon site-specific practices and situations encountered in transit. Geometric features whose dimensions differ significantly from upstream features (e.g., a horizontal curve significantly sharper than preceding curves) may violate ad hoc expectancies.

A driver's attention level refers to the proportion of information-processing capacity allocated to the driving task. "Drivers allocate sufficient attention to maintain a perceived level of driving safety."[(4)] Most rural highways have relatively low workload demands and, therefore, drivers often have relatively low attention levels on them. Geometric inconsistencies, however, demand more attention than typically required and, therefore, than drivers expect. If sight distance to an unexpectedly demanding feature is adequate, then drivers should have sufficient time to increase their attention level and process the required information at the rate necessary to select and complete the appropriate vehicle control actions; if sight distance is not adequate, then some drivers may not be able to process the required information as quickly as necessary. Even with ample sight distance, however, the feature must be sufficiently recognizable to prompt the driver to attend to it.

Geometric inconsistencies may violate a priori and/or ad hoc expectancies. These violations result from a disparity between drivers' expected and actual workload requirements. Drivers who recognize this disparity increase their attention level and adjust their speed and/or path. Drivers who fail to recognize the disparity or who take too long to react, may make speed and/or path errors that increase the likelihood of accidents. Therefore, abrupt speed and/or path changes are common manifestations of the unexpectedly high workload demands associated with geometric inconsistencies.

RESEARCH PHILOSOPHY AND APPROACH

The basic philosophy of the research documented herein is that geometric design consistency can be best explained within the context of the interactions among the three components of the driver-vehicle-roadway system. Figure 2 illustrates the research approach that follows logically from this philosophy. Roadway geometry influences driver workload, vehicle operations, and accident experience. A fundamental hypothesis is that changes in driver workload and vehicle operating speeds, which evaluate a geometric feature within the context of the preceding alignment, should better explain accident experience than measures of the geometric feature in isolation (e.g., degree of curvature).

In figure 2, the legs extending from roadway geometry to vehicle operations and to driver workload represent models for predicting operating speed and driver workload as a function of horizontal curve geometry. These models are presented in chapter 2 for operating speed and chapter 3 for driver workload. The leg between driver workload and vehicle operations represents the intent to integrate the models for the two measures. The legs to accident experience represent the evaluation of each measure as a predictor of accident experience. Analysis of accident experience at horizontal curves is discussed in chapter 5.

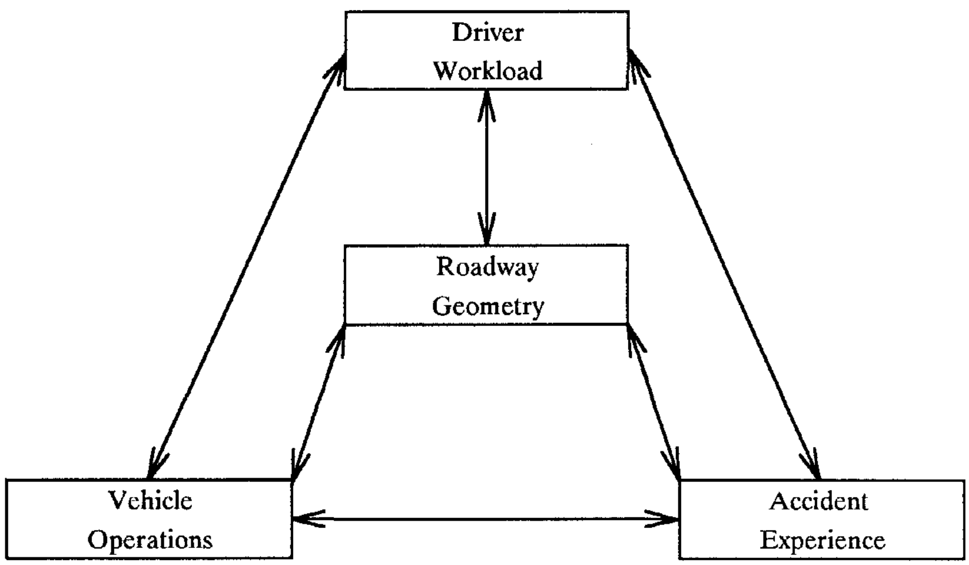


Figure 2. Schematic of the research approach for this study.

2. CURRENT U.S. RURAL ALIGNMENT DESIGN POLICY AND PRACTICE

Vehicle speed is a key control parameter in geometric design. "Without question, the 'design equation' is most sensitive to vehicle speed—not only because the ability to stop or corner is a function of the square of speed but also because the impact forces of a collision are also a function of the square of speed."[9] A fundamental question in design policy and practice is the speed upon which the alignment of a roadway should be based. In the United States, the answer is the design speed as defined by AASHTO.[1]

There is growing concern, however, about the selection and application of design speeds in the United States. The principal concerns relate to disparities between design speeds and drivers' desired speeds, and the resulting need for drivers to reduce their operating speed below their desired speed on certain alignment features, particularly horizontal curves.

In this report, operating speed refers to the speed at which drivers are observed operating their vehicles. The 85th percentile of the distribution of observed speeds is used as the principal measure of the operating speeds associated with a particular location or geometric feature. This definition differs from the classical definition of operating speed, which 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 at any time exceeding the safe speed as determined by the design speed on a section-by-section basis."[1] The classical definition is difficult to interpret and apply, and is rarely used in practice. In recent years, particularly when discussed relative to design speed, the term "operating speed" has had the meaning used in this report.

ORIGINS OF DESIGN-SPEED-BASED RURAL ALIGNMENT DESIGN

This section draws from several authors who have provided interesting and insightful reviews of the history and evolution of the design-speed concept.[10-14] The design-speed concept was developed in the 1930's as a mechanism for designing rural highway alignments that permitted the majority of drivers to operate uniformly at their desired speed. As design practice and driver behavior have evolved, however, the concept has lost effectiveness at producing consistent alignments.

When highway transportation in the United States changed from non-motorized to motorized vehicles in the early 1900's, horizontal curves designed and/or constructed for the low operating speeds of non-motorized vehicles became the site of large numbers of accidents by motorized vehicles being operated at much higher speeds.[15] Good attributes the first statements advocating a design-speed concept to Young who argued in 1930 that "... roads ... should be planned on a miles-per-hour basis—that is, sections of highways, preferably between towns, should have all curves superelevated for the same theoretical speed."[12,16]

The basis for selecting a design speed that first appeared in AASHTO policy reflected the work by Barnett, who recommended that "The assumed design speed of a highway should be the maximum reasonably uniform speed which would be adopted by the faster driving group of vehicle operators, once clear of urban areas."(17) Barnett's statements about the intent of the design-speed concept remain relevant today:(17)

> ...the aim in designing any section of highway should be a balance in design. All features should be safe for the assumed design speed. The unexpected is always dangerous so that if a driver is encouraged to speed up on a few successive comparatively flat curves the danger point will be the beginning of the next sharp curve.

The following two key facets of the design-speed concept are evident in the statements by Young and Barnett:

- Selection of the appropriate design speed.
- Application of that speed in design.

Young emphasized the application of a uniform speed to the superelevation design of all horizontal curves along a rural highway. Barnett emphasized the selection of the appropriate design speed based upon a high-percentile value from the distribution of operating speeds.

CURRENT U.S. POLICY ON DESIGN SPEED

The 1990 AASHTO policy 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."(1) Although this definition is abstract, AASHTO provides clarification on both the selection and application of the design speed.

With respect to the selection of an appropriate design speed, AASHTO states:(1)

- "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."

- "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 for a secondary road 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. ... A cumulative distribution of vehicles 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, where this is feasible."

- "A pertinent consideration in selecting design speeds is the average trip length. The longer the trip, the greater the desire for expeditious movement."

Table 1 summarizes the 1990 AASHTO recommendations on minimum design speeds for rural highways. The recommendations are based on functional classification and terrain. For collectors and locals, additional guidance is given based on traffic volumes.

Table 1. AASHTO guidelines on minimum design speed (km/h) for rural highways.[1]

Functional Classification	Terrain		
	Level	Rolling	Mountainous
Arterial	96.6-112.7	80.5-96.6	64.4-80.5
Collector	64.4-96.6	48.3-80.5	32.2-64.4
Local	48.3-80.5	32.2-64.4	32.2-48.3
Conversion: 1 km/h = 0.621 mi/h			

With respect to the application of the selected design speed, the 1990 AASHTO policy provides the following guidance:[1]

- "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."

- "Although the selected design speed establishes the maximum degree of curvature and minimum sight distance necessary for safe operation, 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 economical design. Even in rugged terrain an occasional tangent or flat curve may be fitting. 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 is also 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."

- "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 affected abruptly but should be affected 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, 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."

AASHTO's guidance is reasonable, and it accurately reflects the original intent and spirit of the design-speed concept. Compliance with this intent and spirit should minimize consistency problems. Quantitative guidance is not provided, however, and compliance is therefore difficult to ensure. For example, quantitative guidance is lacking on the percentile value in the speed distribution that should be used as the design speed, the length of tangent or number of flat curves in succession that encourage drivers to speed up, the distance that is sufficient to effect a gradual change in design speeds, etc.

CURRENT U.S. PRACTICE FOR ENSURING ALIGNMENT CONSISTENCY

The geometric design policies and practices of nine State highway agencies were reviewed, i.e., California, Colorado, Florida, Illinois, New York, Oregon, Pennsylvania, Texas, and Washington.

Review Procedure

Each State's design manual and policy on resurfacing, restoration, rehabilitation (3R) projects were reviewed. In addition, telephone interviews were conducted with the individuals responsible for the State's geometric design policy and manual. The interview topics are summarized in figure 3.

Summary of Findings

The States' design policies and practices for new construction are closely patterned after the AASHTO policy.[1] The discussions in the States' design manuals of the selection and application of design speeds for rural highway alignments are almost identical to AASHTO policy.

Most of the States' policies on alignment design for 3R projects are patterned after Transportation Research Board Special Report 214, *Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation.*[18] Recommendations in Special Report 214 for both horizontal and vertical curvature are based upon the disparity between a curve's design speed and the 85th percentile running (i.e., operating) speed approaching the curve. Some States modify these guidelines to emphasize proven accident experience. That is, if a feature is deficient but does not have a proven accident problem, then an improvement would not be mandated.

For horizontal curvature, Special Report 214 recommends:[18]

> Highway agencies should increase the superelevation of horizontal curves when the design speed of an existing curve is below the running speeds of approaching vehicles and the existing superelevation is below the allowable maximum specified by AASHTO new construction policies. Highway agencies should evaluate reconstruction of horizontal curves when the design speed of the existing curve is more than 15 mph below the running speeds of approaching vehicles (assuming the improved superelevation cannot reduce this difference below 15 mph) and the average daily traffic volume is greater than 750 vehicles per day.

For vertical curvature, Special Report 214 recommends:[18]

> Highway agencies should evaluate the reconstruction of hill crests when (a) the hill crest hides from view major hazards such as intersections, sharp horizontal curves, or narrow bridges; (b) the average daily traffic is greater than 1,500 vehicles per day; and (c) the design speed of the hill crest (based on the minimum stopping sight distance provided) is more than 20 mph below the running speeds of vehicles on the crest.

REVIEW OF STATE HIGHWAY AGENCY PRACTICES

1. Geometric Design Policies, Procedures, Standards, and Guidelines:

   - What documents contain your geometric design policies, procedures, standards, and guidelines for new location and reconstruction projects? For 3R projects? For safety projects?
   - Who (residency/district/region/central office) is responsible for the prioritization, geometric design, and review of new location and reconstruction projects? Of 3R projects? Of safety projects?
   - What are your most common types of design work now? During the next 10 years?

2. Geometric Design Consistency Problems in the State:

   - For which geometric features do you have the greatest difficulty in complying with existing design standards for new location and reconstruction projects? For 3R projects? For safety projects?
   - On what types of roads and in which parts of your State are geometric inconsistencies most likely to be found?
   - What are the most common causes of geometric inconsistencies in your State?

3. Procedures for Evaluating Geometric Design Consistency:

   - Do you currently use quantitative procedures to evaluate the consistency of existing and/or proposed alignments?
   - Is there a need for quantitative procedures for evaluating alternative alignments and determining whether or not design exceptions are justified? If so, how and where in your project development process could such procedures best be incorporated?

4. Possible Future Cooperation:

   - Could you help us identify study sites for consistency evaluation and speed data collection? Obtain roadway inventory and plan-profile sheets? Obtain accident data? Obtain county of residence information from license plate data? Obtain approvals from the appropriate Department personnel in the local area?

Figure 3. Questionnaire on State highway agency practices.

Several States have other policies or programs that also address consistency issues. For example, California uses a design checklist as a formal tool for documenting that the project review has considered important issues and that written justifications for exceptions are provided where needed. Questions on the checklist include: "Do horizontal curves lead vertical curves so that horizontal curves are not hidden by vertical curves?" and "Have ghost spots been considered and eliminated?"

New York has design policies that: (1) consider adjacent roadway sections in determining bridge widths for bridge replacement projects, (2) use the 85th percentile speed as the design speed even if it is less than the posted speed limit, and (3) consider compatibility with the adjacent section of the roadway in making decisions about a design exception.

Several years ago, the Pennsylvania Department of Transportation started a safety corridor initiative. Traditionally, safety projects were chosen based upon a benefit-cost ratio and performed on a spot basis. Safety problems persisted between the improved sections, however. The Department implemented the safety corridor initiative to evaluate longer stretches of roadway, and they performed projects in those longer lengths. This initiative incorporates the systemwide evaluation of features that is an important characteristic of consistency concepts.

The most common types of design work in the States reviewed are 3R projects on two-lane rural highways and freeways, bridge rehabilitation and replacement projects, and safety projects. Only one State identified new alignment projects as a major component of their work during the next 10 years. Several States indicated that there would be some, but not an extensive amount of, rural highway upgrades from two to four lanes.

The responses to the question regarding the most common types of consistency problems were directed toward the standards with which the States have the most difficulty complying. Most States identified horizontal curves and crest vertical curves first or second. The terrain type (level, rolling, or mountainous) influences whether the problem is more likely to be horizontal or vertical. Other features identified were side slopes and lane and shoulder widths. Difficulties in complying with standards are most common on two-lane highways in rolling and mountainous terrain. One State indicated that problems were more common on low-volume roads because the most severe alignment features on the higher volume roads had already been fixed. High construction costs, difficulties in acquiring right-of-way, public concerns, environmental constraints, and the requirements for justifying exceptions sometimes lead to specification of a design speed lower than actual operating speeds would suggest.

Several States indicated that this research contract would be useful if it could help determine when and where alignment improvements should be made. In some cases, designers need justification to convince decision-makers that the costs of improvements are justified. In other cases, designers need help in justifying design exceptions. The States urged that the procedures developed be easy to use, but not necessarily "cookbook" procedures, and involve little, preferably no, new data requirements.

EVIDENCE OF A DISPARITY BETWEEN DESIGN AND OPERATING SPEEDS

The design-speed concept originated in the 1930's as a response to the safety problems resulting from the growing disparity between the speeds for which horizontal curves were designed and/or constructed and the speeds at which drivers chose to operate their vehicles. Two recent studies indicate a current disparity between design and operating speeds on rural two-lane highways.

Speed data collected in 1978 on 12 rural two-lane highways in 3 States (Arkansas, Illinois, and Texas) are summarized in table 2.[4] The speed data were collected at random points along both tangents and curves of roadways with 80.5-km/h (50-mi/h), 96.6-km/h (60-mi/h), and 112.7-km/h (70-mi/h) design speeds. Whereas the 112.7-km/h (70-mi/h) design speed encompasses operating speeds almost to the 95th percentile, the 96.6-km/h (60-mi/h) design speed represents only the average operating speed, and the 80.5-km/h (50-mi/h) design speed was exceeded by the majority of drivers.

Table 2. 1978 Operating-speed statistics for rural two-lane highways by design speed.[4]

Design Speed (km/h)	Sample Size	Average Speed (km/h)	85th Percentile Speed (km/h)	95th Percentile Speed (km/h)
80.5	2,646	90.0	101.3	107.9
96.6	2,446	96.0	107.1	113.5
112.7	3,568	95.5	107.4	114.3
Conversion: 1 km/h = 0.621 mi/h				

Speed data reported in 1991 for 28 horizontal curves in 3 States (Maryland, Virginia, and West Virginia) are summarized in figure 4.[19] Each data point on the figure represents the measured 85th percentile speed and inferred design speed for a horizontal curve. The diagonal line indicates equal 85th percentile and design speeds. The inferred design speed was calculated using the standard superelevation equation given the degree of curvature and measured superelevation rate near the midpoint of the curve. The inferred design speed is the maximum speed, in 10-km/h (6.21-mi/h) increments, whose corresponding side-friction factor did not exceed the maximum side-friction factors recommended in an addendum to the 1990 AASHTO policy on "Interim Selected Metric Values for Geometric Design."[1] One curve had a design speed of 100 km/h (62.1 mi/h), and the other 27 curves had design speeds ranging between 30 km/h (18.6 mi/h) and 80 km/h (49.7 mi/h). All 27 of the curves with design speeds $\leq$ 80 km/h (49.7 mi/h) had 85th percentile speeds that exceeded the

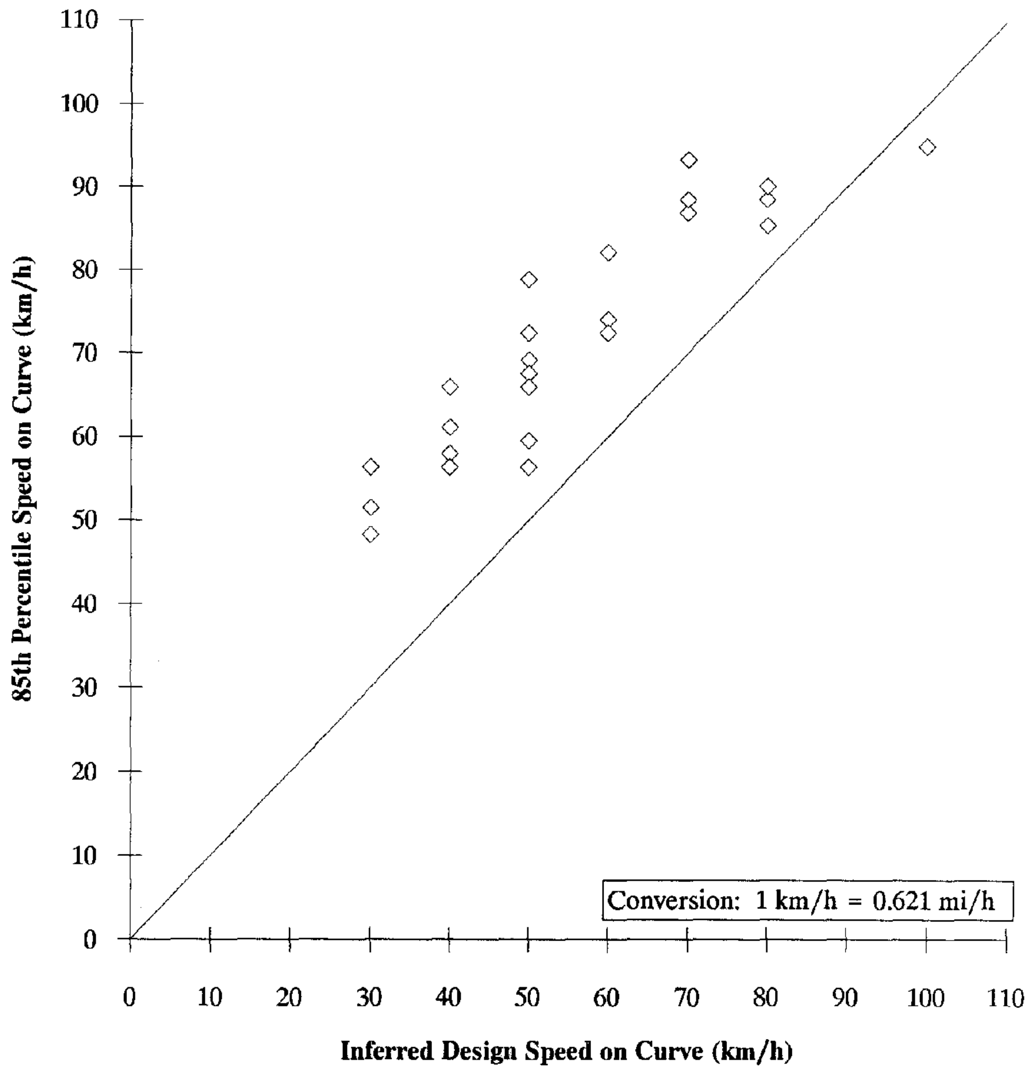


Figure 4. The 85th percentile speed versus inferred design speed for 28 curves in 3 States.[19]

design speed. Only the single 100-km/h (62.1-mi/h) design-speed curve had an observed 85th percentile speed less than the design speed.

Figure 4 is remarkably consistent with a similar figure published by McLean for rural two-lane highways in Australia.[20] McLean found that horizontal curves with design speeds less than 90 km/h (55.8 mi/h) had 85th percentile speeds that were consistently faster than the design speed, whereas curves with design speeds greater than 90 km/h (55.8 mi/h) had 85th percentile speeds that were consistently slower than the design speed.

These data suggest that 85th percentile speeds exceed design speeds less than 100 km/h (62.1 mi/h) at horizontal curves on rural two-lane highways. McLean's findings prompted a revision of the Australian design procedures for roadways with lower design speeds. The question for U.S. geometric design policy-makers is whether the similar disparities in the United States should also prompt revisions to U.S. procedures for selecting and applying design speeds.

CRITIQUE OF THE U.S. DESIGN-SPEED-BASED ALIGNMENT DESIGN POLICY

There are several weaknesses in the design-speed concept as it is applied in the United States to the design of rural horizontal alignments. These weaknesses have reduced the effectiveness of the concept to the point that adherence to it no longer guarantees consistent alignments. Weaknesses exist in the procedures for both selecting and applying the design speed in rural horizontal alignment design.

Design-Speed Selection Process

AASHTO suggests that the design speed "should be consistent with the speed a driver is likely to expect" and "should fit the travel desires and habits of nearly all drivers."[1] Australia's McLean argues, however, that "Design speed is no longer the speed adopted by 'the faster driving group of vehicle operators,' but has become a design procedural value used for the 'design and correlation of design elements' which is also a 'maximum safe speed.'"[13] Good suggests, "... there seems to have been a change in emphasis from design speed as a speed which might be expected from driver behaviour, to a speed which is 'safe' from the designer's point of view."[12]

AASHTO guidelines on minimum design speeds permit the selection of design speeds as low as 48.3 km/h (30 mi/h) on rural collector highways.[1] The data in table 2 and figure 4 suggest that these guidelines underestimate drivers' desired speeds and that U.S. design policy-makers should reconsider these recommended minimum values for new construction.

Design-Speed Application Process

There are several fundamental flaws in the logic and assumptions underlying how the design speed is applied in U.S. design practice. These flaws are likely to create problems only if the selected design speed is less than drivers' desired speeds. Table 2 and figure 4 indicate that the disparity between design and operating speeds is restricted almost entirely to horizontal curves with design speeds less than 100 km/h (62.1 mi/h).

As applied in the United States, the design-speed concept presumes that a design will be consistent if the individual alignment features share the same design speed. Unfortunately, the concept, as implemented in the United States, cannot guarantee an alignment that promotes uniform operating speeds not exceeding the design speed. A fundamental limitation is that the design speed applies only to horizontal and vertical curves, and not to the tangents that connect those curves. Design speed has no practical meaning on horizontal tangents. It provides no basis for establishing maximum tangent lengths to promote consistency by controlling the maximum operating speeds that can be attained. AASHTO's statements encouraging the use of above-minimum values also may have a negative effect on consistency among alignment elements by facilitating operating speeds that exceed the design speed of the controlling element.

Although AASHTO suggests general controls that address qualitatively the coordination of and consistency among horizontal and vertical alignment elements, quantitative guidance to help ensure alignment consistency is not provided. The design-speed concept does not provide sufficient coordination among individual geometric features to ensure consistency. It controls only minimum values and encourages the use of above-minimum values. For example, a highway with an 80-km/h (49.7-mi/h) design speed could have only one curve with a maximum safe speed of 80-km/h (49.7 mi/h) and all other features with maximum safe speeds of 120-km/h (74.5 mi/h). As a result, operating speeds approaching the critical curve are likely to exceed the 80-km/h (49.7-mi/h) design speed. Such an alignment would comply with an 80-km/h (49.7-mi/h) design speed but may violate drivers' ad hoc expectancy and exhibit undesirable operating-speed profiles.

AASHTO policy on the distribution of superelevation on curves less sharp than the maximum degree of curvature, which is the principal mechanism for ensuring horizontal alignment consistency in the design-speed concept, also has a flawed assumption that limits the extent to which it can be relied upon to promote uniform operating speeds. Hayward notes, "The method used by most states to distribute the maximum superelevation throughout the range of intermediate curve radii has weakened the relationship between design speed and the limiting speeds suggested through the laws of physics."[21] The recommended superelevation rate for curves increases parabolically from zero for zero degree of curvature (i.e., a tangent) to the selected maximum superelevation rate for the maximum degree of curvature at a given design speed. The side-friction factor at the design speed for a curve with a given degree of curvature and recommended superelevation rate also increases parabolically from zero for zero degree of curvature to the assumed maximum value corresponding to the design speed at the maximum degree of curvature. The assumption required by this distribution of side-friction factors is that drivers operate uniformly at the design speed (be it 50 or 120 km/h [31.1 or 74.5 mi/h]) even on curves at which they could operate at higher speeds without exceeding the assumed maximum side-friction factor. Clearly, that assumption is unreasonable.

Hayward points out another limitation on the ability of AASHTO to ensure consistent alignment design throughout the United States: "Because different states employ differing rates of maximum superelevation, the same curve can have different design-speed values in different states."[21] The result is a lack of consistency among States. Curves of the same degree of curvature having different superelevation rates in different States complicate the drivers' task of selecting the appropriate speed on a curve.

Because AASHTO policy incorrectly presumes that drivers operate uniformly at the selected design speed, the rural alignment design process lacks a feedback loop in which the driver speed behavior resulting from the designed alignment is estimated and compared with the assumed design speed. Given the almost inevitable disparity between design and operating speeds on low design-speed alignments, there is a need to check for and resolve disparities between design speed and estimated operating speed on individual curves and between the operating speeds on successive alignment features. As will be discussed in chapter 3, several countries have introduced such a feedback loop in their design procedures, especially for roadways with low design speeds.

McLean provides a good summary of the limitations of the design-speed concept:[13]

> Design speed ... only really has meaning in the presence of physical roadway characteristics which limit the safe speed of travel. This is not the case for level, tangent sections. Even for physical features that limit safe speed of travel, the design speed only specifies minimum values; above minimum values are recommended wherever terrain and economy permit. Thus, a road can be designed with a constant design speed as conceived by the designer, yet have considerable variation in speed standard and, to a driver, appear to have a wide variation in design speed.

CURVE SIGNING PRACTICES

Curve signing practices are intended to influence driver speed behavior and, therefore, merit consideration relative to curve design practices. AASHTO notes that, "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."[1]

Advisory-speed plates on curve-warning signs should help mitigate the disparity between design and operating speeds. A 1989 attitudinal survey of motorists indicates, however, that advisory-speed plates have one of the lowest compliance rates of all traffic control devices.[22] A related speed data collection effort at 113 curves in 4 States (California, New York, Texas, and Virginia) found, "In an overwhelming majority of the observations, the vehicle speeds in the curve were above the posted advisory speed."[22] The 1991 study whose data for 28 curves in 3 States were illustrated in figure 4 produced a similar finding: "At most curves, posted advisory speeds were well below the prevailing traffic speed."[19] Figure 5 summarizes the observed 85th percentile speed versus the posted

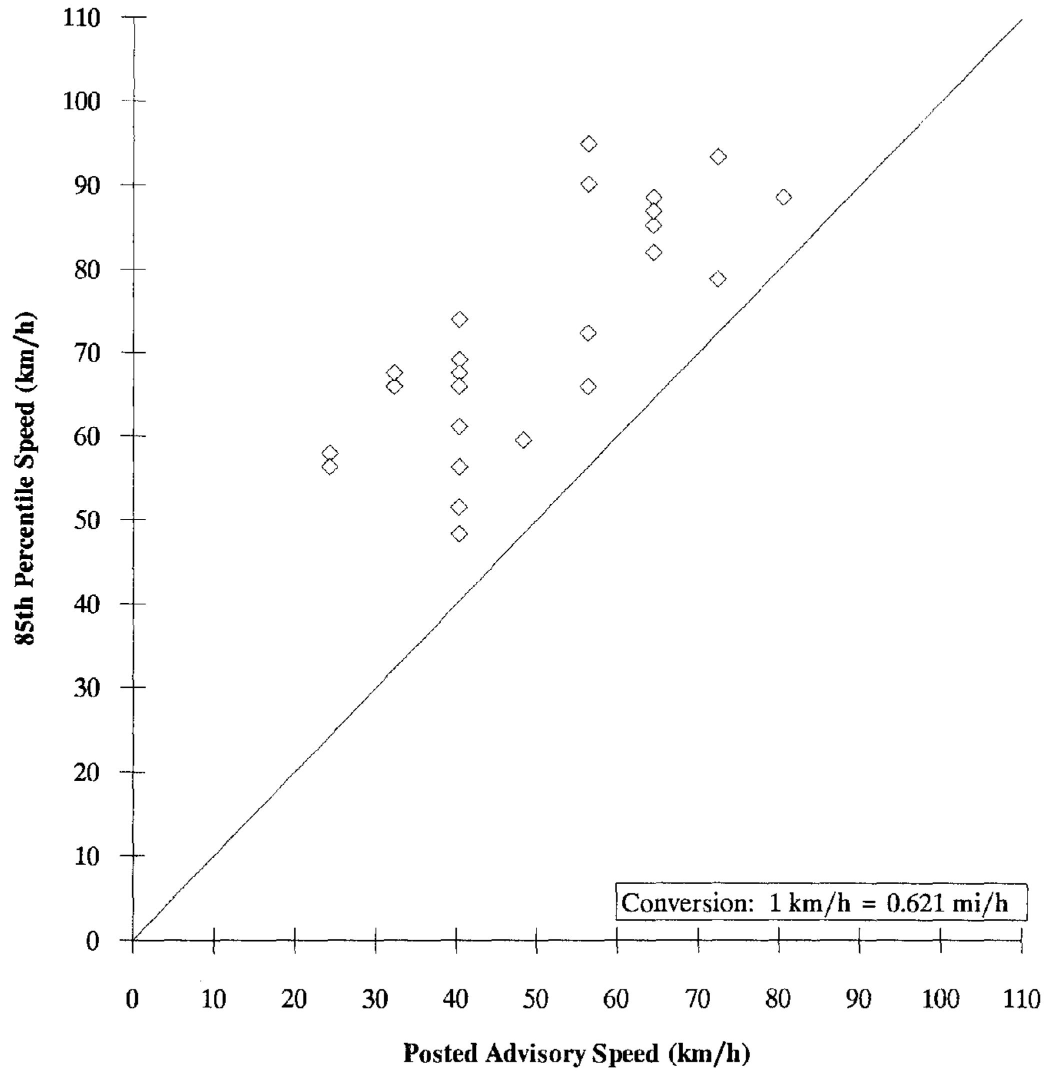


Figure 5. The 85th percentile speed versus posted advisory speed for 28 curves in 3 States.(19)

advisory speed for the same 28 curves. Both studies suggested that the observed disparity between operating speeds and posted advisory speeds is due in part to a lack of uniformity in the selection of posted speeds.[19,22]

Merritt surveyed States on their method of determining the advisory speed that should be posted at a horizontal curve.[15] The two basic methods involve AASHTO maximum side-friction factors or ball-bank indicator measurements (which correspond to similar side-friction factors). Data indicating that drivers consistently exceed posted advisory speeds on curves suggest that drivers tolerate greater lateral acceleration (or side friction) than assumed for design and signing purposes.[19,22] Figure 6 plots the side-friction factor corresponding to the 85th percentile speeds from figure 4 versus the inferred design speed. Also shown are AASHTO's maximum side-friction factors by design speed. The factors are based on passenger comfort levels measured during the 1940's. These comfort levels may be outdated by changes in vehicle design and driver preferences and tolerances and, therefore, may not reflect today's drivers and vehicles. When applied to advisory-speed signing, these side-friction factors yield unrealistically low advisory speeds and deny drivers the meaningful curve information they need.

Unfortunately, little information is available on how drivers judge horizontal curvature and select an appropriate speed. One study found that drivers begin decelerating only a short distance before a curve requiring a speed reduction and continue to decelerate between the beginning and midpoint of the curve.[23] These data suggest that drivers have difficulty judging the sharpness of curvature and appropriate speed before entering a curve.

The experience of driving through several thousand curves during the data collection effort documented in chapter 3, although not scientific, leads the authors to similar observations:

- Drivers' judgment of the sharpness of curvature from an approach tangent is difficult and complicated by variations in the length of curvature, vertical alignment, and background environment. For example, the longer a curve of a given degree of curvature, the sharper it appears from the approach tangent.

- Only after entering the curve and experiencing the lateral acceleration is it possible to determine the appropriate operating speed with accuracy.

These observations, if correct, add to the importance of both consistency in rural horizontal alignment design and meaningful curve signing.

SUMMARY

U.S. policy on the design of rural alignment relies on the selection and application of a design speed to achieve consistency with respect to operating speeds. AASHTO indicates that the selected design speed should be "nearly all inclusive of the typically desired speeds of drivers."[1] Recent empirical data, however, suggest that AASHTO's recommended minimum design speeds underestimate desired speeds.

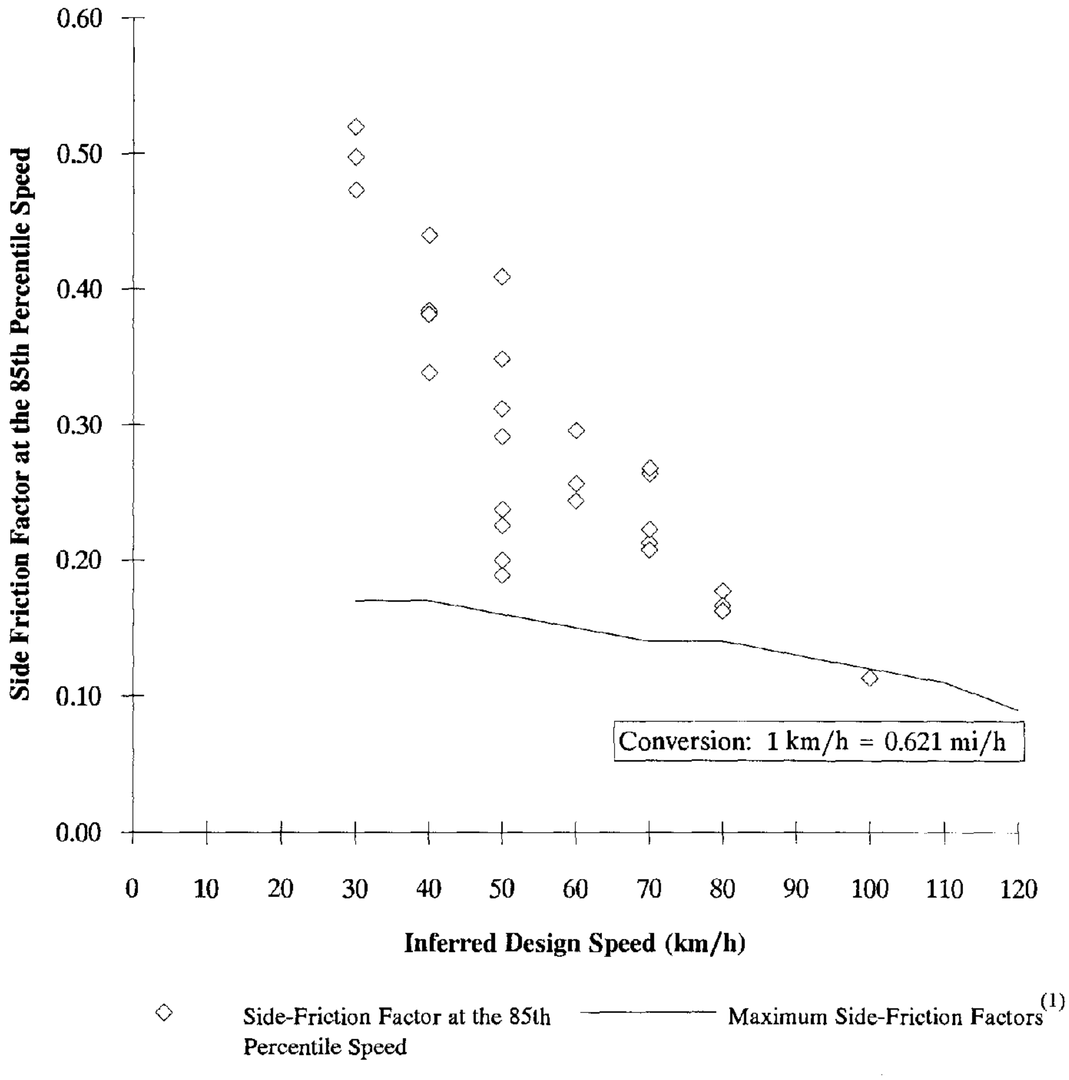


Figure 6. Side-friction factor at the 85th percentile speed versus inferred design speed for 28 curves in 3 States.[19]

The design-speed-based rural alignment design policy in the United States can facilitate uniform operating speeds along an alignment only for drivers whose desired speed is less than or equal to the design speed. Table 2 and figure 4 suggest that adverse disparities between design and operating speeds are restricted almost exclusively to alignments with design speeds less than 100 km/h (62.1 mi/h).

U.S. policy presumes that drivers will not exceed the design speed even though U.S. practice permits the selection of design speeds less than drivers' desired speeds. U.S. policy provides no quantitative guidance on mitigating the disparities between design and operating speeds (illustrated in table 2 and figure 4) because it incorrectly presumes that such disparities do not occur.

U.S. policy on superelevation design may promote, rather than mitigate, operating-speed inconsistencies. Policies on maximum superelevation rates and on the distribution of superelevation lead to superelevation rates on curves with a given radius that may vary from State to State and within a State from roadway to roadway depending on the State's maximum superelevation rate and the roadway's design speed. These variations in superelevation rates complicate drivers' speed selection task on horizontal curves and may exacerbate the disparity between design and operating speeds.

Advisory-speed signing practices on horizontal curves are generally based upon AASHTO maximum side-friction factors, but also lack uniformity nationwide. These side-friction factors may not reflect current vehicle design and driver tolerances. The lack of uniformity and use of values that do not reflect current driver behavior reduce the effectiveness of current curve signing practices at mitigating disparities between design and operating speeds.

3. OPERATING-SPEED-BASED MEASURES OF CONSISTENCY

One of the principal objectives of this study was to develop an operating-speed-based consistency evaluation procedure for rural two-lane highways in the United States. This chapter presents the results of the evaluation of operating-speed-based measures of consistency and documents the development of an evaluation procedure in the following order:

1. Operating-speed considerations in European, Australian, and Canadian alignment design practices are reviewed.

2. Previously developed U.S. operating-speed-based consistency evaluation procedures are discussed.

3. Previous research characterizing driver speed behavior on rural horizontal alignments is reviewed.

4. This study's data collection effort to characterize U.S. driver speed behavior is documented, and the statistical analysis results are presented.

5. The development of a speed-profile model for evaluating operating-speed consistency on rural two-lane highway horizontal alignments is described.

6. Finally, a menu-driven microcomputer procedure for evaluating operating-speed consistency is introduced.

EUROPEAN, AUSTRALIAN, AND CANADIAN ALIGNMENT DESIGN PRACTICES

Design consistency research, policies, and practices in Europe (Great Britain, France, Germany, Sweden, and Switzerland), Australia, and Canada were evaluated. The evaluation included a review of research literature and selected design manuals and guidelines as well as interviews with researchers and government officials. Canada implements the design-speed concept in essentially the same way as the United States. Since practices in Europe and Australia differ from the United States, the discussion focuses on these practices. The countries are presented in an order that facilitates the discussion.

Germany

In Germany, design guidelines are prepared by committees of the German Road and Transportation Research Association (Forschungsgesellschaft für Strassen- und Verkehrswesen) consisting of both researchers and practicing engineers. A design consistency evaluation procedure was first included in the 1973 edition of German alignment design guidelines. The motivation was a highway safety problem in Germany that prompted stronger safety criteria. The latest update to the alignment design guidelines in 1984 included only minor revisions to the consistency procedure.[24] Lamm and Cargin prepared an English translation of the 1984 guidelines that is quoted in this report.[25]

The German guidelines use both design and 85th percentile operating speeds for alignment design of rural roadways. The design speed is used, as in the United States, to determine minimum radii of horizontal curves, maximum grades, and minimum K-values for crest vertical curves. The estimated 85th percentile speed, however, is used to evaluate and design superelevation rates and sight distances.

The 85th percentile speed on rural two-lane highways is estimated using empirical relationships based upon the curvature change rate and the pavement width. The curvature change rate is computed for roadway sections as the sum of the angular change in direction divided by the length of the segment. For a simple circular curve without spiral transitions, the curvature change rate is equivalent to the degree of curvature, except for different units.

The German guidelines indicate that the design speed and expected 85th percentile speed on two-lane highways should be well-balanced. The expected 85th percentile speed should not exceed the design speed by more than 20 km/h (12.4 mi/h). If it does, then the guidelines require that either the design speed be increased or the design be modified to reduce the expected 85th percentile speed.

The German guidelines provide several instructions for achieving consistency in the alignment design. First, the design speed "shall remain constant for longer road sections so that the road characteristic is well-balanced for a road operator over the course of the road section. If, in the course of a longer road section—for example, by definite changes in topography—a change in the road characteristic and a corresponding change of the design speed is necessary, then in the transition section the design elements must be carefully tuned to each other so that they change only gradually."[25]

Second, the German guidelines specify that, "The 85th percentile speed shall be consistent for the duration of the road section."[25] Acceptable ranges are specified for the radii of successive curves; these are similar in nature to the U.S. guidelines for the radii of compound curves. Minimum radii following a tangent are also specified, as summarized in table 3. The table is drawn from the German guidelines, but the approximate U.S. equivalents have been substituted for the German roadway categories.

Third, the German guidelines specify, "If the determined values for the 85th percentile speed between successive road sections differ by more than 10 km/h, the speed values between the two sections should be adjusted to allow for a gradual transition of the speed."[25] If the speed differential between sections exceeds 10 km/h, a transition section should be designed with an intermediate value for curvature change rate and, therefore, expected 85th percentile speed.

Table 3. German guidelines for minimum radius following a long tangent.[25]

Road Category	Tangent Length (m)	Minimum Radius (m)
Rural Principal Arterials	$L \geq 600$ m	min R > 600 m
	$L < 600$ m	min R > L
Rural Minor Arterials	$L \geq 500$ m	min R > 500 m
	$L < 500$ m	min R > L
Conversion: 1 m = 3.28 ft		

Switzerland

Geometric design policy in Switzerland is embodied in a series of Swiss norms. The policies are developed by committees of the Swiss Association of Road Specialists (Vereinigung Schweizerisher Straßenfachleute) consisting of both researchers and practitioners.

The Swiss operating-speed consistency procedure originated more than 30 years ago and is detailed in Swiss Norm 640 080b.[26] The procedure estimates the speed profile along an alignment and identifies excessive speed differentials between successive elements. The procedure is applied to rural highways only. The original procedure considered the effects of both horizontal curve radius and vertical grade. Research in the late 1970's indicated, however, that grade up to 6 to 7 percent had no influence on passenger-car operating speeds.[27] Therefore, in the current version of the procedure, speeds are estimated based upon only the horizontal alignment.

The speed profile is estimated based upon three pieces of information:

- Speed on horizontal curves.
- Maximum speed on tangents.
- Deceleration and acceleration rates entering and exiting horizontal curves.

Originally, the speed profile represented observed 85th percentile speeds. Table 4 summarizes the speed estimates currently used for various curve radii. The most recent data, however, indicate that speeds have increased on sharper curves (i.e., radii less than 400 m [1,312 ft]), and there has been a corresponding increase in accident experience.[28] Instead of modifying the speed-radius relationship to reflect these new data, the Swiss decided to retain the old relationship and use it as a standardized speed that they consider safe rather than as the 85th percentile speed.

Table 4. Swiss standardized speeds for various curve radii.[25]

Road Type	Speed (km/h)	Radius (m)
Urban Roads	40 45 50	45 60 75
Rural Roads	55 60 65 70 75 80	95 120 145 175 205 240
Freeways	85 90 95 100 105 110 115 120	280 320 370 420 470 525 580 ≤650
Conversions: 1 km/h = 0.621 mi/h; 1 m = 3.28 ft		

The speeds used for long tangents in the speed profile are the national speed limits for different classes of roadways. The national speed limits were recently reduced. This change was incorporated into the current version of the procedure, which made it necessary to change the definition of an excessive speed differential (since they changed the assumed speed on tangents but not on curves).

Prescribed acceleration and deceleration rates (0.8 m/s^2 [2.6 ft/s^2]) are used to estimate the speed profile on the tangents between curves. It is assumed that acceleration occurs immediately upon departing a curve and that deceleration commences a sufficient distance in advance of a curve so that vehicles can decelerate at the prescribed rate to the prescribed speed on curves.

The speed profile for a roadway should satisfy three conditions:[26]

1. If the preceding element is a tangent or a large-radius curve, i.e., ≥ 420 m (1,378 ft), then the speed differential to the succeeding curve should not exceed 5 km/h (3.1 mi/h). This condition is new to the latest version of the procedure. It was introduced because more problems have been observed at curves following a long tangent than at curves following a sequence of curves.

2. In a sequence of curves, the speed differential should be ≤ 10 km/h (6.2 mi/h). Differentials ≥ 20 km/h (12.4 mi/h) must be avoided.

3. The existing sight distance should equal or exceed the length of transition required to change speed at a rate of 0.8 m/s^2 (2.6 ft/s^2) between successive curves.

There is little new roadway construction in Switzerland; therefore, the procedure is applied primarily to existing roadways. If any of the three conditions are violated, accident experience on the roadway is checked. If there is an accident problem, action is taken to correct the violation.

France

Service d'Etudes Techniques des Routes et Autoroutes (SETRA), a service agency of the French Ministry of Transport (Ministère de L'Èquipement, du Logement, et de l'Espace), develops design standards for national roads and motorways in France. The existing design policy "Instruction sur les Conditions Techniques D'Aménagement des Routes Nationales" was last revised in 1975.[29] SETRA recently drafted new guidelines for national roads, entitled "Amenagement des Routes Principales en Dehors des Agglomérations: Recommandations techniques pour la conception générale et la géométrie de la route."[30] Designers will be asked to apply these guidelines for national roads. They will not be compulsory for lower-class roadways, although they were developed to apply to these roads, too. The new guidelines are the product of a comprehensive review of the safety and operational effects of roadway geometry and a reassessment of the basis for design.

The new French guidelines are considerably different in concept from the old "Instruction," which was classical in approach and reliance on the design-speed concept. The new guidelines depart from the design-speed concept and emphasize that the driver-roadway interaction influences speed behavior and must be taken into account.

The old "Instruction" had five road categories, and each had a design-speed range of 40 km/h (24.8 mi/h) to 120 km/h (74.5 mi/h). All alignment features were related to the design speed. SETRA observed that this approach does not consider the effect of alignment on the actual speed behavior of drivers; it disregards the fact that the design may permit higher speeds. Furthermore, it may encourage the designer to use larger radii, which is not desirable if a sharper curve that requires slower speeds exists downstream. In the new guidelines, therefore, the correlation between roadway type and design speed is stronger, i.e., the new guidelines specify only a 20-km/h (12.4-mi/h) range of design speeds for each of the three new road categories.

For new roads, the new French guidelines consider lateral acceleration, consistency, and visibility. Levels of comfort are suggested for lateral acceleration, e.g., 3.5 m/s^2 (11.5 ft/s^2) for a 60-km/h (37.3-mi/h) design speed, 1.9 m/s^2 (6.2 ft/s^2) for 80 km/h (49.7 mi/h), and 0.8 m/s^2 (2.6 ft/s^2) for 100 km/h (62.1 mi/h). These values result in minimum radii based upon actual speeds. The values are not presumed to ensure safety, and they are not compulsory. The rationale is that the safety effects of lateral acceleration are not significant, hence the designer should try, but not be required, to provide a desirable level of comfort. For existing roads, however, the French guidelines do not consider it necessary to have such a constraint on lateral acceleration. Instead, accident experience should be considered (i.e., if the accident experience is not serious, then an improvement is not required).

Consistency is important in the new French guidelines, and it is considered in several areas. One area is related to roadway type (similar to U.S. functional classification). Route consistency is considered very important. For example, on multilane divided highways, at-grade intersections have high accident probabilities because they are not expected by the driver. Interestingly, the same consistency problem was cited during several of the U.S. State highway agency reviews.

A second area in which consistency is considered important is horizontal alignment. The Institut National de Recherche sur les Transports et Leur Sécurité (INRETS) performed a study on 800 km (497 mi) of national roads with widths greater than 6 m (19.7 ft). The study concluded: (1) curves with radii less than 150 m (492 ft) were a problem if preceded by a tangent longer than 400 to 500 m (1,312 to 1,640 ft), and (2) curves with different radii (i.e., compound curves) were a problem. SETRA has reflected these findings in the new guidelines with the criteria for new roads summarized in table 5.

Table 5. French guidelines for minimum radius following a long tangent.[30]

Preceding Tangent Length (m)	Minimum Radius (m)
> 500 m	200 m
> 1000 m	300 m
Conversion: 1 m = 3.28 ft	

A horizontal curve accident study is underway in France that is analyzing about 3,000 curves using a computerized accident data base. The scope is limited to national roads with two or three lanes. The study is considering the following curve characteristics: radius, number of radii (i.e., simple or compound curve), and length of "easy" alignment upstream/downstream of the curve. Two definitions of easy alignment are being considered: (1) tangent or curve with radius greater than 500 m (1,640 ft), and (2) tangent or curve with radius greater than 1000 m (3,280 ft).

SETRA is developing microcomputer software for the estimation of 85th percentile operating speeds. The software would be used early in the design process. The input data include: (1) the general cross-section (i.e., number of lanes, number of carriageways, lane widths), (2) horizontal alignment, and (3) longitudinal profile. The output is an 85th percentile speed profile along the proposed alignment. A next step will be to add an expert system to look at consistency problems and to provide recommendations on passing-sight distance. The procedure requires an estimated sight distance that is calculated assuming a lateral sight restriction 3 m (9.8 ft) from the pavement edge. The speeds on curves are based upon a study by Gambard and Louah, who developed a regression model that explained 70 percent of the variability in observed speeds based on three variables describing local conditions: (1) pavement width, (2) radius, and (3) grade.[31] Speed data were collected at 230 sites on national roads. The speed-estimation procedure starts by calculating sight distance. Then, at each point, it looks downstream the maximum visible distance within which the software identifies the most constraining point (e.g., horizontal curve or vertical grade) with respect to operating speed. The software estimates the necessary acceleration/deceleration rate to arrive at an appropriate speed at that constraining point. If the deceleration rate exceeds a maximum acceptable value, then the software gives a warning. The software uses the necessary deceleration rate even if it exceeds the maximum, in order to reduce speeds to the appropriate value at the constraining point. The software also considers maximum acceleration and deceleration rates on grades. Since the constraints on deceleration rate related to horizontal and vertical alignment control at different places, the software chooses the controlling deceleration rate.

Sweden

Geometric design policy in Sweden is developed and maintained by the Swedish National Road Administration (Vägverket) with research support from the Swedish Road and Traffic Research Institute (Väg-och Trafik-Institutet [VTI]). The Administration prepared an English translation of the current edition of their geometric design guidelines, entitled "Standard Specifications for Geometric Design of Rural Roads" (Trafikleder på Landsbygd), which is dated 1986.[32]

For the most part, Swedish design policy follows the classical approach to the use of design speed. They do not have an explicit, quantitative design-consistency evaluation procedure. They do, however, give consideration to expected operating speeds.

The appropriate design speed is based upon land-use intensity (urban or rural) and the roadway classification (national roads, principal, secondary, or tertiary country roads). However, "In determining geometrical minimum elements, it has been taken into consideration that speeds higher than the actual design speed often occur. Therefore the speed selected for the respective design speeds is that which survey results indicate 85 percent of drivers can be expected to be under."[32] This consideration is reflected in the design guidelines through two sets of standards: regular minimum and exceptional minimum. The regular minimum is based upon the 85th percentile speed associated with the design speed, whereas the exceptional minimum is based upon driving at the speed limit.

There is a Swedish procedure for estimating the median speed profile of a roadway based upon the horizontal curve radii and vertical grades. Its primary application related to geometric design is evaluating the need for and benefits of climbing lanes. The profile may be estimated manually or using the VTI traffic simulation model.[32,33]

Great Britain

Geometric design standards for national roadways in Great Britain (and Northern Ireland) are developed and maintained by the Department of Transport. The current Departmental Standard on "Highway Link Design" is dated 1981.[34] A Departmental Advice Note provides additional guidance.[35]

British standards prior to 1981 promoted flowing alignments for rural two-lane highways. In 1981, the standards were revised to their current form for the following reasons: (1) design speeds were not well-defined, (2) rigid adherence to the standards led to high construction and environmental costs that may not be justified by the minimal safety effects of minor departures from the standards, and (3) the flowing-alignment designs were not providing adequate guidance to drivers because they permitted passing over much of their length, but with only marginally adequate passing-sight distance.[36,37]

Current British standards provide a structured system of design speeds and departures from standards. They emphasize that sections of two-lane rural highways should have either clearly adequate or clearly inadequate passing-sight distance and that sections with marginally adequate passing-sight distance should be avoided. The standards permit curves with large enough radii to provide adequate passing-sight distance or curves with radii small enough that passing-sight distance is clearly inadequate. However, curves with intermediate radii that drivers might incorrectly judge as having adequate passing-sight distance are not recommended.

Current British standards have some unique features concerning how the design-speed concept is applied. As in most countries, the British standards emphasize that the design speed should be "consistent with the anticipated vehicle speeds on the road."[34] In the United States (and much of Europe), the design speed is based upon the functional classification of the roadway, the land-use environment, and the terrain. The British do not employ functional classification concepts. Instead, they emphasize the effects of alignment and layout (cross-section and access control) constraints on operating speeds in selecting a design speed. The alignment constraint for two-lane highways is a function of the "bendiness" of the alignment, which is defined as the total degree of curvature per kilometer, and the harmonic mean of available sight distance. The layout constraint is a function of the road type (two-lane or multilane divided), cross-section width, and access density. The roadway type and cross-section are selected based upon anticipated demand volumes. An iterative approach is suggested for the selection of design speed and design of the alignment. In this approach, an initial alignment is designed to a trial design speed. Then, the alignment and layout constraint factors are computed "to identify locations where elements of the initial trial alignment may be relaxed to achieve cost or environmental savings, or conversely, where design should be upgraded, according to the calculated design speed."[34] This approach attempts to balance design and operating speeds.

The structured system of design speeds provides an objective basis for evaluating departures from standards (i.e., design exceptions). The design speed represents the 85th percentile speed of vehicles. Design speeds are structured in steps such that for a given design speed the next higher step represents the 99th percentile speed and the next lower step represents the 50th percentile speed. Desirable and absolute minimum values for sight distance and horizontal and vertical curvature are specified for each design speed such that the desirable minimum for one design speed is the absolute minimum for the next higher design-speed step. Departures from standards by one design-speed step are generally accepted if significant cost or environmental constraints exist. Two-step departures require formal review.

British standards focus on passing (overtaking) behavior as the principal safety concern on two-lane rural highways. In both horizontal and crest vertical curve design, consideration is given to the passing-sight distance that is provided. The philosophy is that curvature should either have clearly adequate passing-sight distance or be designed such that no-passing markings are required. Crest vertical curves not in passing sections should have near minimum K-values. The standards encourage "a 'go with the ground' approach, incorporating climbing lanes at hills combined with short non-overtaking crests."[36] Horizontal curvature should be either flat enough for passing-sight distance to be provided with lines of sight within the right-of-way or near minimum radii. As summarized in table 6, British horizontal curve design standards for two-lane rural highways specify a range of radii with marginal passing-sight distance that are not recommended. Whereas the coordination of horizontal and vertical alignment is encouraged, the need to provide passing zones at adequate intervals is of overriding concern.

Table 6. British guidelines on horizontal curve radius for passing-sight distance.[34]

Design Speed (km/h)	Range of Radii (m) for Curve Categories			
	A	B	C	D
50	>2040	720-2040	255-720	127-255
60	>2880	1020-2880	360-1020	180-360
70	>4080	1440-4080	510-1440	255-510
85	>5760	2040-5760	720-2040	360-720
100	>8160	2880-8160	1020-2880	510-1020

Curve Categories:

A—Straight and Nearly Straight Overtaking Sections (both directions)
B—Right-Hand Curve Overtaking Sections
C—Radii Not Recommended
D—Non-Overtaking Sections with Warning Lines

Conversions: 1 km/h = 0.621 mi/h; 1 m = 3.28 ft

Australia

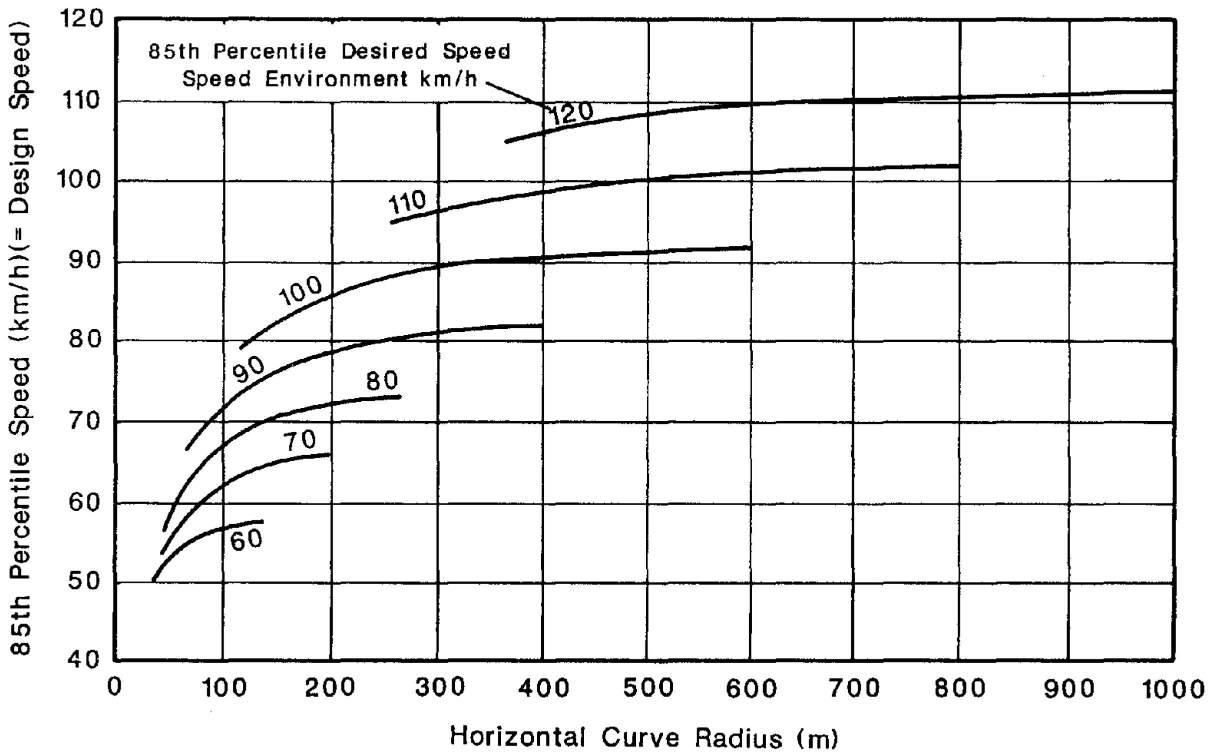
As part of a review of Australia's roadway design standards, the Australian Road Research Board undertook studies of the relationship between design speeds and actual operating speeds. The study, which was conducted by McLean, focused on driver speed behavior on horizontal curves. The study included a review of previous research, speed data collection at 120 curves, an analysis of side friction and superelevation, and an evaluation of the design-speed concept.[13,20,38-40] The study identified disparities between design speeds and actual operating speeds on low-speed (i.e., ≤ 90 km/h [55.9 mi/h]) alignments, pinpointed weaknesses in the design-speed concept, and led to revised design procedures for low-speed alignments that incorporate consideration of operating speeds in order to improve alignment consistency.

McLean noted that for curves with design speeds ≥ 100 km/h (62.1 mi/h), the design speed was greater than the 85th percentile speeds; but for curves with design speeds ≤ 90 km/h (55.9 mi/h), the design speed was less than the 85th percentile speeds.[38] It was concluded that for high-speed (i.e., ≥ 100 km/h [62.1 mi/h]) alignments, the classical design-speed concept was appropriate, because for such alignments, 85th percentile speeds were less than the design speed. For low-speed (i.e., ≤ 90 km/h [55.9 mi/h]) alignments, however, the predicted 85th percentile speed should be used as the design speed.

The prediction of 85th percentile speeds was based upon McLean's analysis of speeds on curves, which found them to be influenced primarily by the desired speed on the roadway and the horizontal curve radius. Figure 7 plots the relationship between 85th percentile speed and radius for various desired speeds (or speed environments). The desired speed (speed environment) was measured as the 85th percentile speed of free-flowing passenger cars on long tangents and is related to the terrain and the range of horizontal curve radii on the roadway section, as presented in table 7.

Table 7. Australian speed environment guidelines.[41]

Approximate Range of Horizontal Curve Radii (m)	Terrain Type			
	Flat	Undulating	Hilly	Mountainous
Less than 75			75 km/h	70 km/h
75 - 300		90 km/h	85 km/h	
150 - 500		100 km/h	95 km/h	
Over 300 - 500	115 km/h	110 km/h		
Over 600 - 700	120 km/h			
Conversions: 1 m = 3.28 ft; 1 km/h = 0.621 mi/h				



Conversion: 1 km/h = 0.621 mi/h 1 m = 3.28 ft

Figure 7. Australian procedure for estimating 85th percentile speeds on curves.[41]

The fact that the 85th percentile driver was observed to operate at speeds in excess of the design speed of a low-design-speed curve indicated that drivers were actually accepting side-friction factors greater than the design values. Therefore, new maximum side-friction factors were computed for low-speed alignments based upon the existing superelevation rate and curve radius and the observed 85th percentile speed. These factors are summarized in table 8.

Table 8. Australian side-friction guidelines.[41]

Design Speed (km/h)	Coefficient of Side Friction
50	0.35
60	0.33
70	0.31
80	0.26
90	0.18
100	0.12
110	0.12
120	0.11
130	0.11
Conversion: 1 km/h = 0.621 mi/h	

The speed-environment values in table 7, the 85th percentile speeds on curves in figure 7, and the side-friction factors in table 8 form the basis for the low-speed alignment design procedure and, according to McLean, "... place bounds on the variation of curve speed standard within the road segment. That is, the design procedure sets standards on alignment consistency rather than minimum standards for individual curves."[42]

Canada

The Transportation Association of Canada publishes the *Manual of Geometric Design Standards for Canadian Roads.*[43] The current version is the 1986 edition. The manual functions in a manner similar to the AASHTO design policy: it provides a basis for uniformity of design standards, but does not serve directly as the official design manual in all parts of the country. Many provinces have their own manual.

The design-speed concept is applied in the classical manner in Canada. In most respects, the Canadian policy on rural alignment design is similar to U.S. policy.

Summary

Whereas the United States and Canada continue to adhere to the design-speed concept as classically applied, five European countries and Australia have enhanced their use of design speed to incorporate explicit consideration of actual driver speed behavior in terms of 85th percentile operating speeds. Three countries (France, Germany, and Switzerland) have speed-profile estimation techniques for evaluating speed consistency along an alignment. Superelevation design is based on estimated 85th percentile speeds if they exceed the design speed of the roadway.

U.S. OPERATING-SPEED-BASED CONSISTENCY EVALUATION PROCEDURES

Two research teams in the United States have developed operating-speed-based rural alignment consistency evaluation procedures: Leisch and Leisch, and Lamm et al.[44,45] Although these procedures have not been widely used in the United States, they are a significant part of the foundation upon which the current research is built and are discussed in turn.

Leisch and Leisch

In 1977, Leisch and Leisch published an operating-speed-based consistency evaluation procedure.[44] The procedure estimated an operating-speed profile using techniques that are similar to the Swiss method.

The Leisch and Leisch procedure has several unique features. It estimates average speeds for both passenger cars and trucks, whereas other procedures consider only passenger car speeds. It considers the effect of both horizontal and vertical alignment, whereas other procedures consider horizontal alignment only and are limited in applicability to grades that do not affect passenger car speeds (i.e., less than 6 percent). It also has separate procedures for estimating deceleration and acceleration distances entering and departing a curve, whereas other procedures assume equal acceleration and deceleration rates. Leisch and Leisch estimate deceleration rates as a function of the approach tangent speed and the magnitude of the required speed reduction. They estimate acceleration rates departing a curve as a function of the speed reduction approaching the curve, the speed on the curve, the "distance and degree of restrictiveness of the geometry in sight" departing the curve, and the average grade beyond the curve.[44]

The speed estimates are derived from AASHTO design policies of 1965 and 1973.[46,47] Therefore, they are not likely to represent current driver speed behavior.

Leisch and Leisch recommend a three-part, 16.1-km/h (10-mi/h) rule:[44]

1. Average automobile speeds along an alignment should vary by no more than 16.1 km/h (10 mi/h).

2. Design-speed reductions should not exceed 16.1 km/h (10 mi/h).

3. Average truck speeds should differ from average automobile speeds by no more than 16.1 km/h (10 mi/h).

Lamm et al.

Lamm et al. adapted and refined the German procedure for U.S. use.[45] They estimate the change in 85th percentile speed from a tangent to a horizontal curve or between successive horizontal curves on two-lane rural highways, depending on whether the tangent is independent or non-independent. The procedure is based upon an analysis of 261 horizontal curves in New York State.[48,49] The determination of whether a tangent is independent or non-independent is based upon the length of the tangent. A tangent is independent if it is long enough for traffic to accelerate over some part of its length.[50] A speed profile is developed using a procedure similar to the Swiss. Lamm et al. rate horizontal alignment consistency in terms of the change in degree of curvature (ΔD) and corresponding change in 85th percentile operating speeds ($\Delta V85$) between successive horizontal elements, as follows:[45]

- Good: $\Delta D \leq 5°$ or $\Delta V85 \leq 9.7$ km/h (6 mi/h).

- Fair: $5° < \Delta D \leq 10°$ or 9.7 km/h (6 mi/h) $< \Delta V85 \leq$ 19.3 km/h (12 mi/h).

- Poor: $\Delta D > 10°$ or $\Delta V85 > 19.3$ km/h (12 mi/h).

PREVIOUS RESEARCH ON DRIVER SPEED BEHAVIOR

The principal operating-speed-based measure of alignment consistency is the change in 85th percentile speed from an approach tangent to a horizontal curve. A review of U.S. and foreign research and design practice indicates that two approaches have been taken to estimate operating-speed reductions from a tangent to a horizontal curve:

- Estimating operating speed reduction as a function of the degree of curvature of the curve.

- Estimating operating speeds on the horizontal curve and on the approach tangent and computing the speed reduction as the difference between these two speed estimates.

A limitation of the first approach is that it does not explicitly account for factors that affect speeds on the approach tangent, including the degree of curvature of the preceding curve and the length of the tangent. Therefore, the second approach is preferred and is the focus of discussion.

Estimating Speeds on Horizontal Curves

Two theories have been proposed to predict operating speeds on horizontal curves:

- Speeds are a function only of local characteristics (e.g., degree of curvature, grade, and cross-section within the curve).

- Speeds are a function of both local characteristics and the general character of the alignment.

In general, 85th percentile speeds on horizontal curves have been estimated with reasonable accuracy (R^2-values between 0.647 and 0.84) using models based on local characteristics alone. The general character of the alignment has been reflected in drivers' desired speed that has been estimated as a function of the terrain and overall character of the alignment. Incorporating the desired speed into the regression models for speeds on curves increases the proportion of explained variability. R^2-values of approximately 0.92 were reported for models reflecting both local characteristics and desired speeds. The desired speed for a particular highway, however, is not easily defined and is difficult to measure.

Table 9 lists regression equation forms that have been developed to model 85th percentile speeds on horizontal curves. These equations represent the models developed by Glennon et al., Lamm and Choueiri, and Taragin in the United States; McLean in Australia; Emmerson in England; Gambard and Louah in France; Lamm et al. in Germany; Kanellaidis et al. in Greece; and Lindenmann and Ranft in Switzerland.[23,28,31,39,51-55] Each equation uses the 85th percentile speed at the curve midpoint (V85) as the dependent variable and degree of curvature (D) as the independent variable.

Table 9. Regression equation forms for 85th percentile speeds on horizontal curves.

Equation Form	Regression Equation: V85 =
Linear	$\beta_0 + \beta_1 D$
Exponential	$EXP\ (\beta_0 + \beta_1 D)$
Inverse	$(\beta_0 + \beta_1 D)^{-1}$
Polynomial	$\beta_0 + \ \beta_i D^i$
Where: V85 = 85th percentile speed β_i = regression coefficient for D to the ith power D = degree of curvature	

Estimating Speeds on Tangents

The maximum speed achieved on a tangent section depends largely upon the tangent length, the sharpness of the curves on either end of the tangent, and drivers' desires. Figure 8 shows schematically three different cases that are typically used in speed-profile estimation techniques.[26,45] For each case, a horizontal alignment consisting of two curves with an intervening tangent is given. A representative 85th percentile speed profile is presented below each alignment. The alignments in the three cases differ only by the length of the tangent between the two curves.

Speeds on curves are assumed constant, and acceleration and deceleration are assumed to occur only on tangent sections. Furthermore, acceleration and deceleration rates are assumed to be equal in magnitude.[26,45] Several studies support these assumptions. Good cites Holmquist of Sweden, who found that "...the deceleration before, and the acceleration after, the curve were mirror reflections of each other."[12] Lamm et al. found that acceleration and deceleration rates departing and approaching curves were almost equal at 0.85 m/s^2 (2.8 ft/s^2).[50] This value is comparable to the 0.8-m/s^2 (2.6-ft/s^2) rate used in Switzerland.[26] Other studies, however, have reported that some deceleration and acceleration occur within the horizontal curve.[23,56,57]

In case 1 of figure 8, the driver accelerates (or decelerates) uniformly between the curves and, therefore, the maximum 85th percentile tangent speed occurs at one end of the tangent. In case 2, the tangent is long enough for some acceleration, but it is not sufficiently long for drivers to attain their desired speed at normal acceleration and deceleration rates. In case 3, the tangent is long enough for a driver to accelerate to and, for some distance, maintain a desired speed. Thus, the tangent length and adjoining curves determine the maximum 85th percentile speed attained on tangents. Drivers are assumed to have adequate sight distance to begin decelerating a sufficient distance upstream of a curve so that at the normal deceleration rate, they can reach the 85th percentile curve speed at the beginning of the curve.

The desired speed on long tangents has been either given as a standard assumed value or estimated as the intercept of a linear regression equation for speeds on curves.[26,41,45] Australia uses assumed standard values that represent the speed environment for different terrain types (flat, undulating, hilly, mountainous) and ranges of horizontal curve radii.[41] Switzerland assigns the posted speed limit as the desired speed.[26] Lamm et al. use the intercept of their linear regression equation for speeds on curves.[45]

Speed-Profile Models

A model of the form illustrated in figure 8 was deemed appropriate for estimating operating-speed profiles along rural two-lane highway horizontal alignments. Lamm et al., the Swiss, and Leisch and Leisch developed models of this general form.[26,45] None of these models was considered suitable for adoption as a U.S. procedure, however, primarily because of limitations in the data with which they were calibrated.

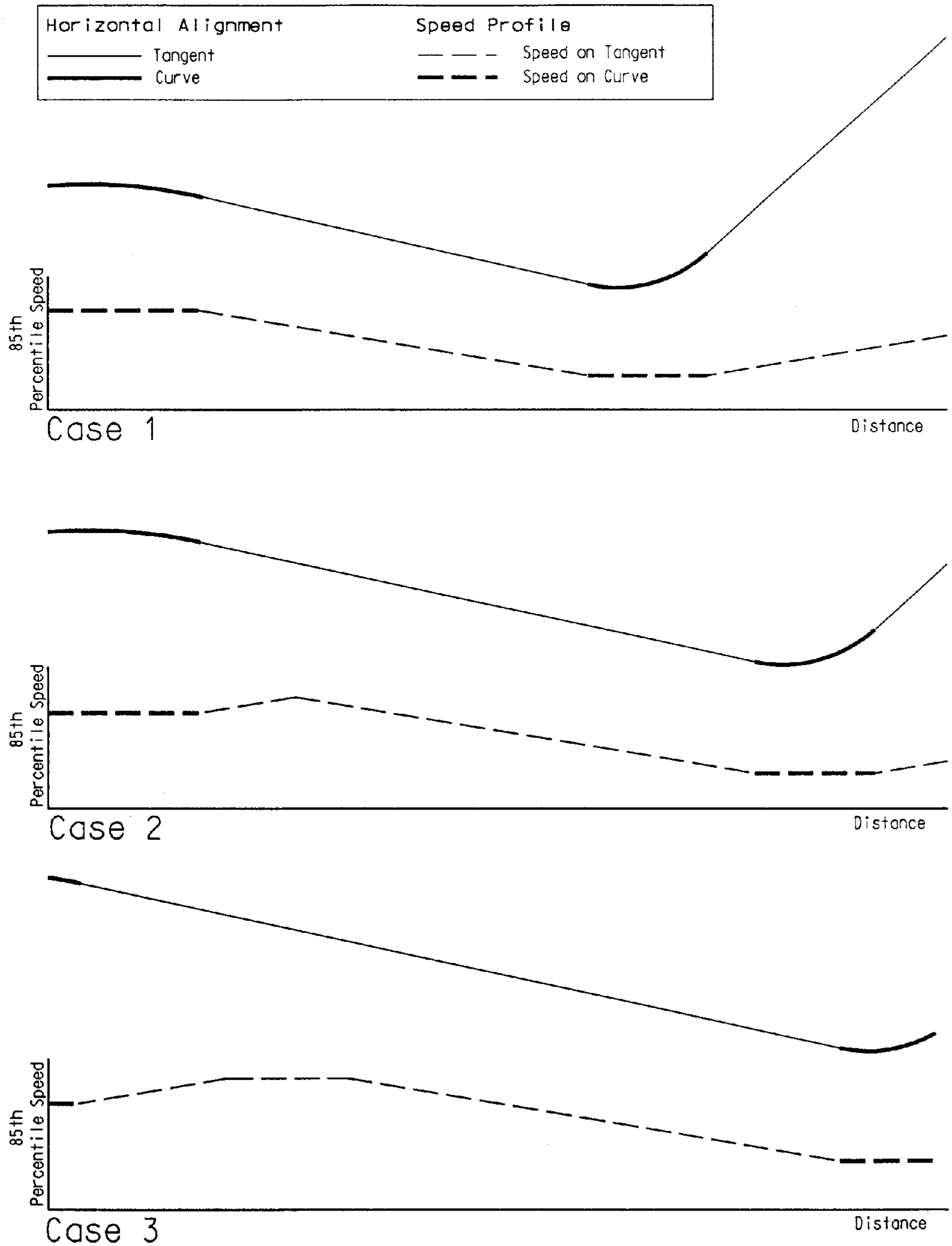


Figure 8. Schematic of speed-profile model.

Lamm et al. calibrated their model based upon data from New York State only.[48,49] The Swiss model is based upon Swiss data.[28] The Leisch and Leisch model uses speed estimates based upon the 1965 and 1973 AASHTO policies, which are outdated.[46,47]

SPEED DATA COLLECTION AND ANALYSIS

The inputs to the speed-profile model illustrated in figure 8 are horizontal alignment data, including the degree of curvature of each curve and the stationing of the beginning and end of each curve, from which curve and tangent lengths can be calculated. The output is the 85th percentile speed at each point along the horizontal alignment, which is used to estimate the speed reduction from the approach tangent to the curve.

The data required to calibrate the model include:

- 85th percentile speeds on horizontal curves.

- Desired speeds on long tangents.

- Deceleration and acceleration rates entering and departing curves.

Given available funding, data collection efforts were focused on the first two items. The acceleration and deceleration rate of 0.85 m/s^2 (2.8 ft/s^2) that was estimated by Lamm et al. was adopted without validation.[45]

Data Collection

Spot speed data were collected at a sample of horizontal curves and their approach tangents on rural two-lane highways in three geographic regions of the United States: the East (New York and Pennsylvania), West (Washington and Oregon), and the South (Texas). Data were collected during June through September 1991.

Speeds were measured using radar guns that function at a higher frequency than standard-issue radar equipment. Consequently, the radar guns are undetectable by radar detection devices. The radar guns were factory-calibrated. A tuning-fork test was performed at each data collection site to ensure that the gun was operating at the proper frequency and, therefore, giving accurate speed measurements.

Speed data were collected only during daylight hours. During all data collection periods, the weather was clear and the pavement was dry.

The speeds of free-flowing passenger cars were measured. Neither trucks nor recreational vehicles were included in the analysis. Data collectors concealed themselves as much as possible when taking speed readings to avoid influencing the measured speeds.

Speeds were measured at the midpoint of the selected curves, and (at a subset of curve locations) near the midpoint of long tangent sections where desired speeds were believed to be attained. A minimum tangent length of 244 m (800 ft) was specified.

A minimum of 50 speeds were measured in each direction of each curve. At the subset of sites where speeds were measured on both the approach tangent and curve, a minimum of 50 paired speed measurements were obtained.

Table 10 summarizes the site selection controls and criteria. The design speeds of the 29 rural two-lane highways on which data were collected ranged between 40 km/h (24.8 mi/h) and 100 km/h (62.1 mi/h). The highways were located in level-to-rolling terrain and had good pavement conditions, 3.05- to 3.66-m (10- to 12-ft) lane widths, and 0 to 2.44-m (0 to 8-ft) shoulders. Grades on the curves and approach tangents studied were less than 5 percent. No significant improvements to the pavement or the geometry had been made on the selected highways within the past 5 years (for accident analysis purposes). Curves at least 61 m (200 ft) in length and with degrees of curvature between 1° and 20° were selected. Two 23° curves and one 30° curve, which otherwise met the selection criteria, were also studied to increase the range of degrees of curvature covered. Sites were selected so that the curves would fall in case 3 of figure 8 (approach tangent lengths at least 244 m [800 ft]) in order to observe the maximum speed difference from tangent to curve.

Table 10. Speed-data collection site-selection controls and criteria.

Control	Criteria
Area Type	Rural
Administrative Classification	State
Functional Classification	Collector and Minor Arterial
Design Speed	≤ 100 km/h (62.1 mi/h)
Posted Speed Limit	80.5-88.5 km/h (50-55 mi/h)
Terrain	Level to Rolling
Grade	≤ 5 percent
Traffic Volumes	750-2500 vehicles/day
Lane Widths	3.05-3.66 m (10-12 ft)
Shoulder Widths	0-2.44 m (0-8 ft)
Alignment Data	Available
Degree of Curvature	1-20° (Primarily 3-12°)
Length of Curve	≥ 61 m (200 ft)
Tangent Length	≥ 244 m (800 ft)
Sight Distance to Curves	≥ 122 m (400 ft)

All of the roadways on which data were collected had 80.5-km/h (50-mi/h) or 88.5-km/h (55-mi/h) posted regulatory speed limits. Most of the sharper curves had advance curve/turn warning signs; some also had advisory speed plates. Speed limit and curve signing were recorded on the data collection form.

Table 11 summarizes by State, the number of different roadways, and the number of curves and tangents on which speed data were collected. The number of curve and tangent lanes are also reported. At all but two of the curves, speeds were measured in both lanes. The study sites are evenly distributed among the three regions of the United States. In total, 22,740 usable speed observations were obtained.

Table 11. Number of speed study sites by State.

State	Number of Roadways	Number of Curves			Number of Tangents		
		Total	Inside Lane	Outside Lane	Total	Inside Lane	Outside Lane
New York	8	43	43	43	23	18	16
Oregon	1	4	4	4	2	0	4
Pennsylvania	1	4	4	4	4	2	2
Texas	10	45	45	43	15	10	15
Washington	9	42	42	42	22	18	18
TOTAL	29	138	138	136	66	48	55

Statistical Analysis Methodology

Three data bases were created: (1) a by-lane data base that contained speed and geometry data separated for each lane of each curve and tangent, (2) a by-site data base that combined the speed data for both lanes and represented the curve or tangent as a whole, and (3) a paired data base that included only the subset of curve and approach tangent lanes for which paired speed measurements were obtained. For each data base, separate regression equations were developed to estimate: (1) the 85th percentile speed at the curve midpoint, and (2) the 85th percentile speed on long tangent sections (i.e., drivers' desired speed). The model forms summarized in table 9 were evaluated. Table 12 lists the independent variables considered when developing models for the 85th percentile speeds on both curves and tangents.

Table 12. Independent variables considered in modeling 85th percentile speeds on horizontal curves and tangents.

Independent Variable	Source of Data	Curve	Tangent
Degree of Curvature	Plans	X	
Curve Length	Plans	X	
Deflection Angle	Plans	X	
Superelevation Rate	Field	X	
Travel-Way Width	Field	X	X
Total Pavement (Lane & Shoulder) Width	Field	X	X
Sight Distance to Curve	Field, Plans, and Video Logs	X	X
Speed on Preceding Tangent	Field	X	
Tangent Length	Plans	X	X
Speed on Preceding Curve	Field	X	X
Lane (Inside or Outside)	Field	X	X
Terrain Type (Level or Rolling)	Plans		X
Annual Average Daily Traffic (AADT)	Traffic Reports	X	X
Geographic Region	Field		X

All statistical analyses were conducted at the 0.05 significance level (α) with the Statistical Analysis Software (SAS) microcomputer package.[58] The P-value reported in SAS is the probability of erroneously rejecting the null hypothesis based upon the sample data. Therefore, rejection of the null hypotheses presented herein required a P-value < 0.05.

The principal techniques for analyzing 85th percentile speeds on curves and long tangents were regression and analysis of variance. These techniques assume the data are normally distributed, which was verified using a Wilk-Shapiro test.

The statistical results for 85th percentile speeds on curves and long tangents were combined with the assumed acceleration and deceleration rates to create speed profiles as shown in figure 8. These profiles, although simplified, represent the expected 85th percentile speeds along a horizontal alignment and can be used to check for operating-speed inconsistencies.

DRIVER SPEED BEHAVIOR ON U.S. RURAL TWO-LANE HIGHWAYS

Results are presented first for observed speeds on long tangents. Then, observed speed behavior on horizontal curves is described.

85th Percentile Speeds on Long Tangents

The 85th percentile speeds on the long tangents studied ranged from 85.3 to 112.7 km/h (53 to 70 mi/h). The mean of the 85th percentile speeds was 97.9 km/h (60.8 mi/h).

Among the independent variables tested, the only statistically significant results related to geographic region and terrain. Table 13 summarizes the results. The East and West were combined because sample sizes for level terrain were too small to make statistical comparisons otherwise. The only statistically significant difference was between the means of the 85th percentile speeds on long tangents for level terrain in the South (Texas)—102.4 km/h (63.6 mi/h)—and for rolling terrain in the East and West—95.5 km/h (59.3 mi/h). The means for all of the other terrain and region categories fell within ± 1.9 km/h (1.2 mi/h) of the overall mean of the 85th percentile speeds on long tangents.

Table 13. Means of the 85th percentile speeds (km/h) on long tangents by region and terrain.

Region	Terrain		
	Level	Rolling	All Terrain
South	102.4*	99.2	99.8
East and West	97.9	95.5*	96.0
All Regions	99.8	96.6	97.9
* Difference is statistically significant at α=0.05.			
Conversion: 1 km/h = 0.621 mi/h			

The following independent variables were not useful in explaining the variability among the 85th percentile speeds on long tangents:

- Travel-way width.
- Total pavement width.
- Characteristics of the preceding curve (degree of curvature and 85th percentile speed).

- Characteristics of the succeeding curve (degree of curvature, length of curve, superelevation rate, and sight distance to the curve).

- Tangent length.

- Annual average daily traffic.

Two previous studies found that travel-way width was a statistically significant independent variable for predicting 85th percentile speeds on tangents.[31,45] These analyses do not support that finding, possibly because of the narrow range of widths in the data base. Travel-way widths varied only between 5.79 to 7.63 m (19 to 25 ft), with most between 6.10 to 7.32 m (20 to 24 ft). Total pavement widths varied between 6.10 to 12.2 m (20 to 40 ft), with most between 7.32 to 9.76 m (24 to 32 ft). These widths are typical of State-maintained, rural, two-lane highways. This finding should not be extrapolated to the full range of roadway widths, however.

The finding that neither the characteristics of the preceding and succeeding curves nor the length of tangent were significant suggests that the tangents studied were sufficiently long that the speeds measured were desired speeds and were not constrained by preceding alignment features. The finding that AADT was not significant is consistent with the intent to measure free-flow speeds.

85th Percentile Speeds on Horizontal Curves

The evaluation of 85th percentile speeds on curves had two objectives:

- Compare 85th percentile speed and design speed on horizontal curves.

- Develop a regression equation for 85th percentile speeds on horizontal curves.

First, the relationship between 85th percentile speeds and design speeds is discussed. Then, the development of the regression equation is documented.

Relationship Between 85th Percentile Speeds and Design Speeds on Horizontal Curves

Chapter 2 presented evidence from previous research of disparities between 85th percentile speeds and the design speeds of horizontal curves. The data collected during this study were also evaluated for evidence of such disparities.

Figure 9 summarizes the comparison of 85th percentile speeds and design speeds at the 138 horizontal curves studied. The data indicate that the 85th percentile speed exceeded the design speed on a majority of curves in each 10-km/h (6.2-mi/h) design-speed increment less than or equal to 100 km/h (62.1 mi/h). The 85th percentile speed was less than the design speed for all curves whose design speed was 110 km/h (68.3 mi/h) or higher.

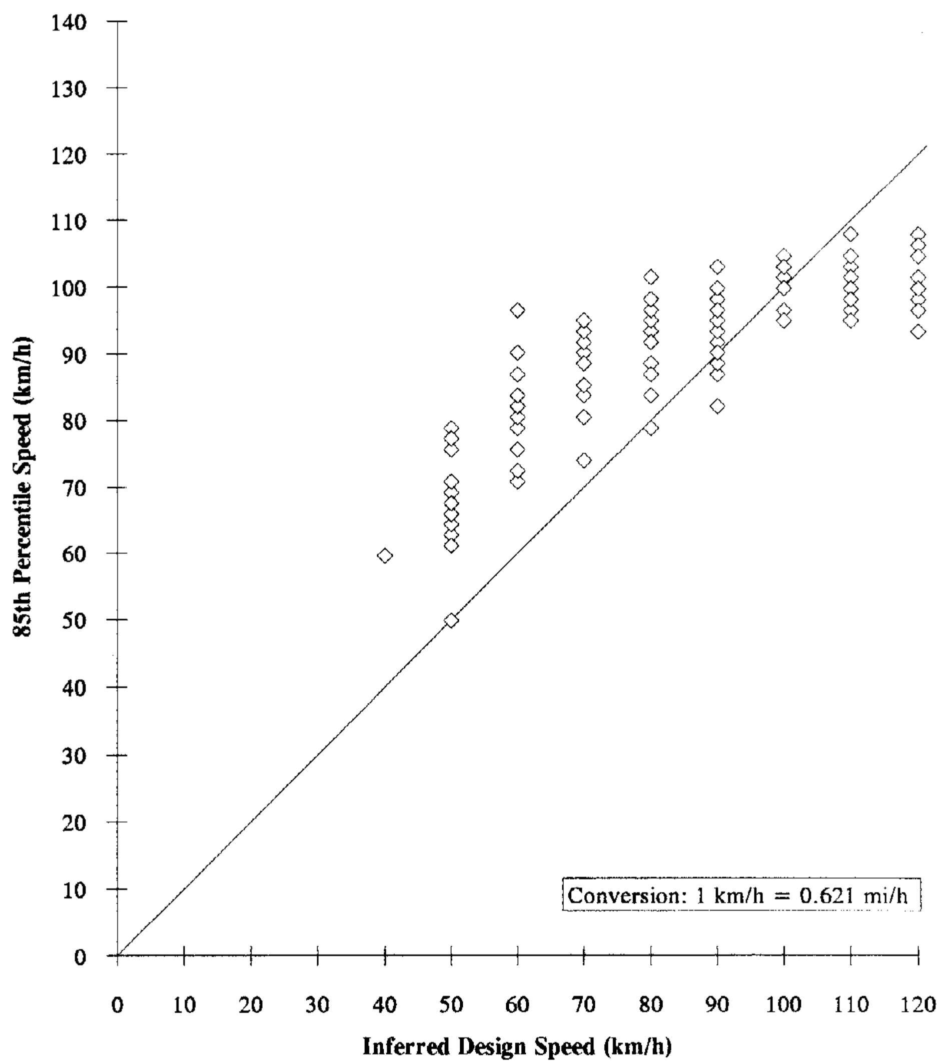


Figure 9. The 85th percentile speed versus inferred design speed for 138 curves in 5 States.

Figure 10 shows the magnitude of the disparity between the 85th percentile speeds and design speeds. The disparity is greatest for the lowest design speeds. For curves with design speeds between 40 and 60 km/h (24.8 and 37.3 mi/h), the 85th percentile speeds average approximately 20 km/h (12.4 mi/h) faster than the design speed. The disparity decreases approximately linearly as the design speed increases from 60 to 120 km/h (37.3 to 74.5 mi/h).

Two of the criticisms in chapter 2 of the U.S. design-speed-based policy for rural horizontal alignment design were related to the following provisions: (1) the distribution of superelevation on curves less sharp than the maximum degree of curvature, and (2) the use of different maximum superelevation rates in different States. These provisions permit curves with the same degree of curvature to have different superelevation rates, because the design superelevation rates for a given degree of curvature vary according to the design speed and the specified maximum superelevation rate. It was hypothesized that this variation in superelevation rates complicates the driver's task of selecting the appropriate speed on curves. Therefore, to examine the extent of this variation, the measured superelevation rate was plotted versus the degree of curvature for the 138 horizontal curves at which speeds were observed in this study. Figure 11 confirms the considerable variability among the superelevation rates for a given degree of curvature.

Another way of evaluating the reasonableness of U.S. horizontal curve design assumptions relative to actual driver speed behavior is to compare the side-friction factor implied by the 85th percentile speed observed on curves to AASHTO's recommended maximum side-friction factors, which are a function of design speed. This comparison is illustrated in figure 12. The results indicate that the side-friction factors at the observed 85th percentile speeds are greater than the AASHTO maximum side-friction factors at a majority of curves with design speeds $\leq$ 90 km/h (55.9 mi/h) and are less than the AASHTO maximum factors at all of the curves with design speeds $>$ 100 km/h (62.1 mi/h).

In summary, there is an undesirable disparity between 85th percentile speeds and design speeds for most curves with design speeds $\leq$ 100 km/h (62.1 mi/h). The 85th percentile driver generally exceeds the design speed and corresponding AASHTO maximum side-friction factor on curves with design speeds $\leq$ 100 km/h (62.1 mi/h).

Regression Equation for 85th Percentile Speeds on Horizontal Curves

The analysis of 85th percentile speeds on horizontal curves was conducted in three steps: (1) for each basic form in table 9, the best-fitting regression equation containing degree of curvature only was identified, (2) other characteristics of the curve that might improve the explanatory power of the relationships were evaluated, and (3) the benefits of adding the observed 85th percentile speed on the approach tangent (representing the desired speed along the roadway) were evaluated.

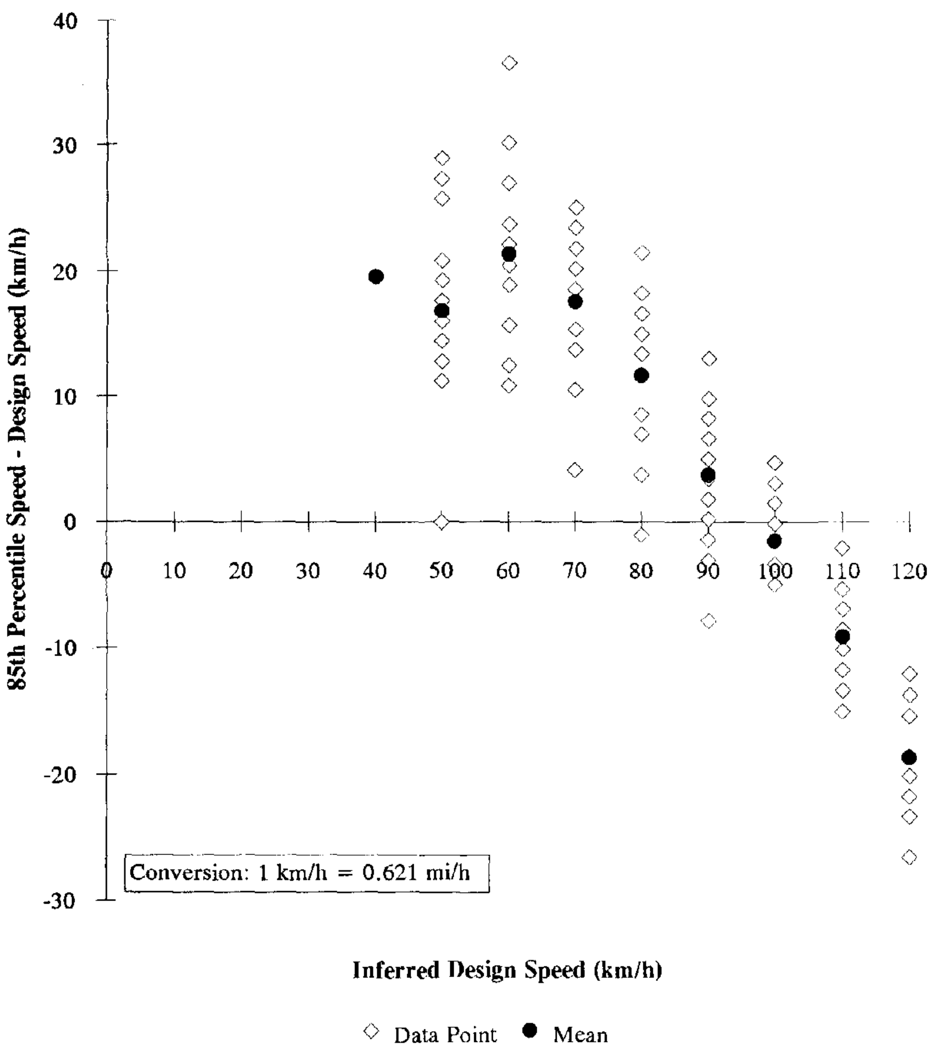


Figure 10. Disparity between inferred design speed and 85th percentile speed for 138 curves in 5 States.

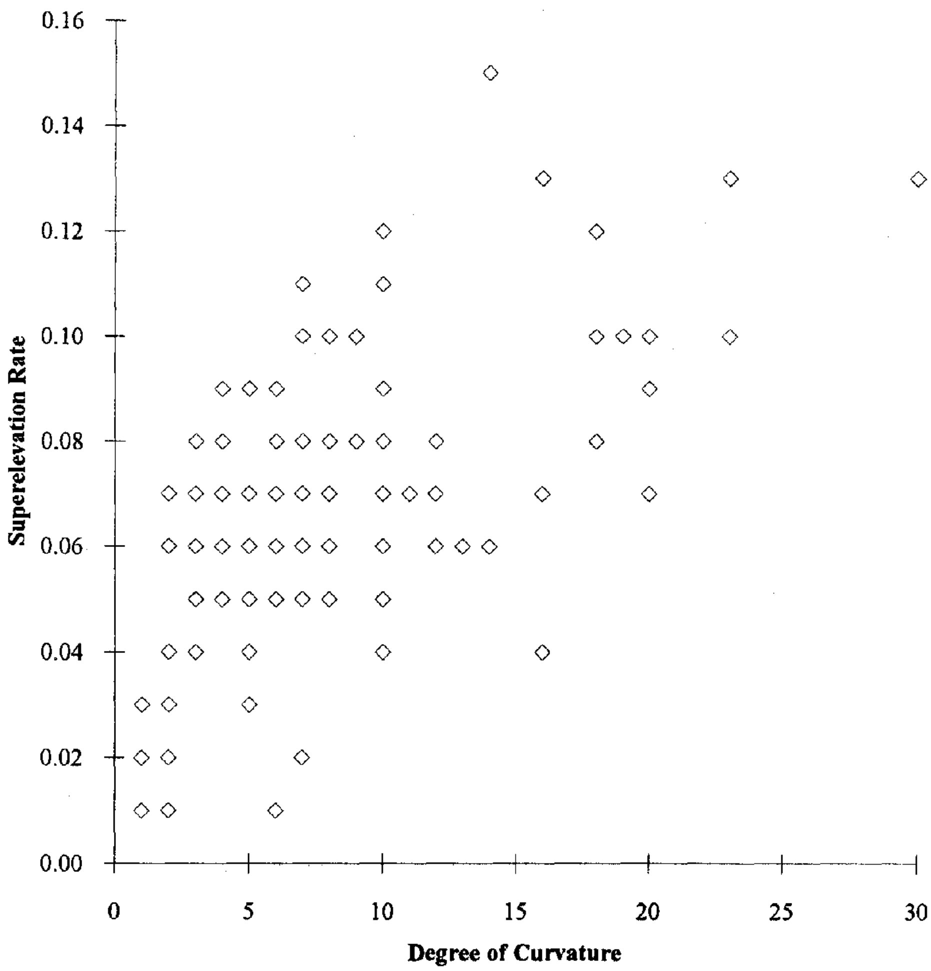


Figure 11. Superelevation rate versus degree of curvature for 138 curves in 5 States.

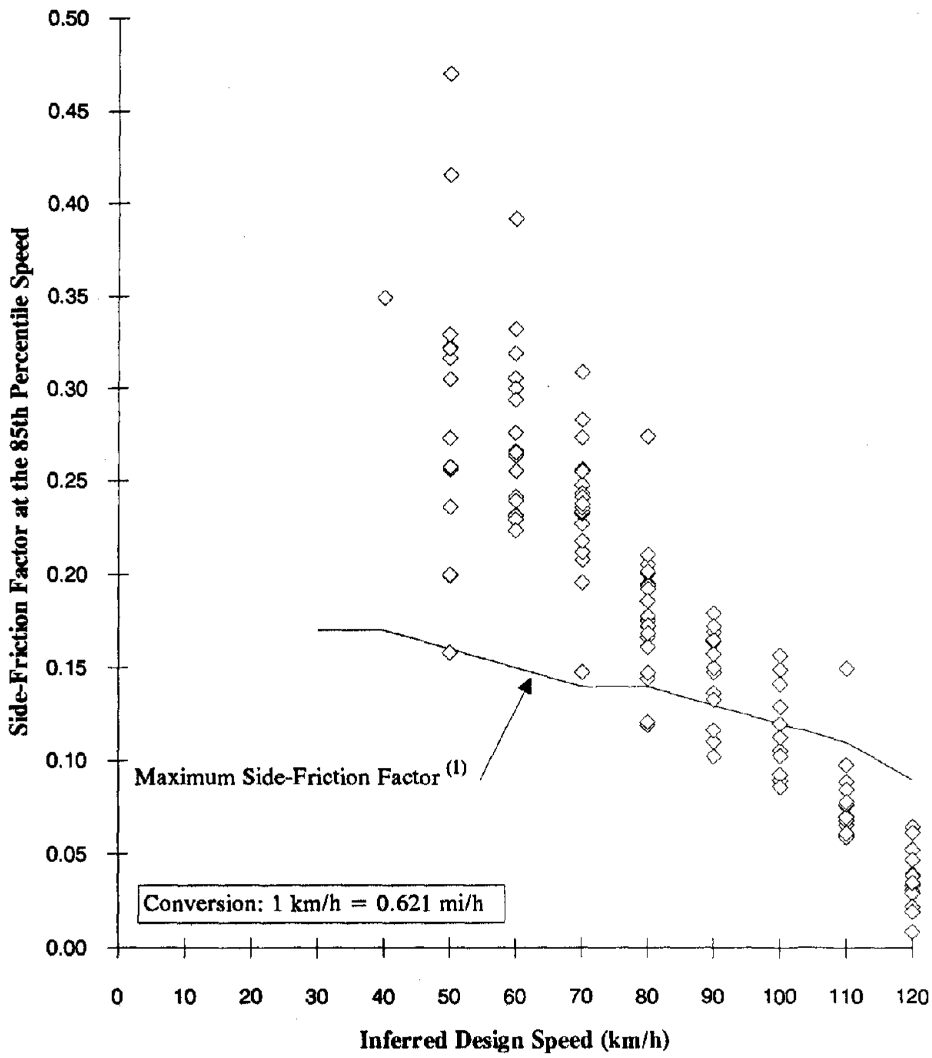


Figure 12. Side-friction factor at the 85th percentile speed versus inferred design speed for 138 curves in 5 states.

Scatter Plot of Data. Figure 13 is a scatter plot of 85th percentile speed versus degree of curvature for the 138 curves in the data base. The scatter plot indicates that 85th percentile speeds decrease approximately linearly as degree of curvature increases.

The scatter plot appears relatively flat through 4°. Therefore, statistical tests of the data in this range were conducted. It was found that the means of the 85th percentile speeds on curves $\leq 4°$ were not significantly different from each other and were not significantly different from the mean of the 85th percentile speeds on long tangents. It was hypothesized that the speeds on both long tangents and curves $\leq 4°$ may be constrained by the 88.5-km/h (55-mi/h) speed limit.

Equation Form with Respect to Degree of Curvature. The equation that provides the generally accepted relationship between design speed and degree of curvature is an inverse-root function:

$$V = \left[\frac{222{,}040\ (e\ +\ f)}{D}\right]^{\frac{1}{2}} \tag{1}$$

where:
- V = design speed (km/h) [1 km/h = 0.621 mi/h]
- D = degree of curvature (°)
- e = superelevation rate
- f = side-friction factor.

The exponential and inverse equation forms in table 9 possess the same general characteristics as this inverse-root relationship. The linear form simplifies this relationship.

The regression results for each of the basic equation forms in table 9 (linear, exponential, inverse, and third-order polynomial) are summarized in table 14. Figure 14 superimposes the regression equations on the scatter plots of the data. Many other equation forms were tested, but since none produced better results, the discussion is restricted to these four.

The equations were compared with respect to the following criteria: goodness of fit, simplicity and practicality of the equation form, conformity to the scatter plot, and reasonableness of the intercept term (i.e., speed at 0 degrees of curvature as an estimate of the speed on tangents). Each form yielded similar goodness-of-fit measures: R^2-values between 0.80 and 0.82 and P-values (corresponding to the calculated F-statistic for the regression equation) of 0.0001 or less. All of the β's in all four regression equations were statistically significant at $\alpha = 0.05$. The linear and polynomial forms produced intercept values closest to the observed mean of the 85th percentile speed on long tangents (97.9 km/h [60.8 mi/h]). The linear equation, which was comparable or superior with respect to the other criteria, was deemed the preferred form due to its simplicity.

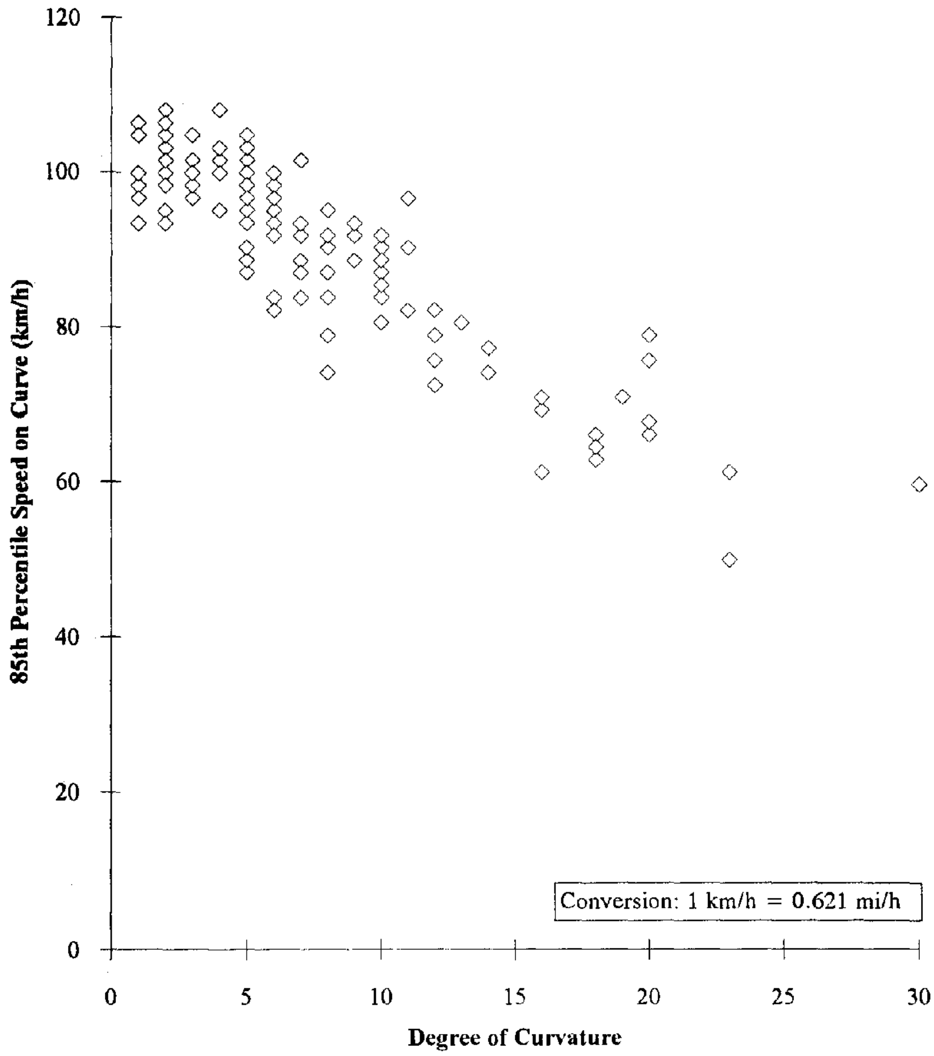
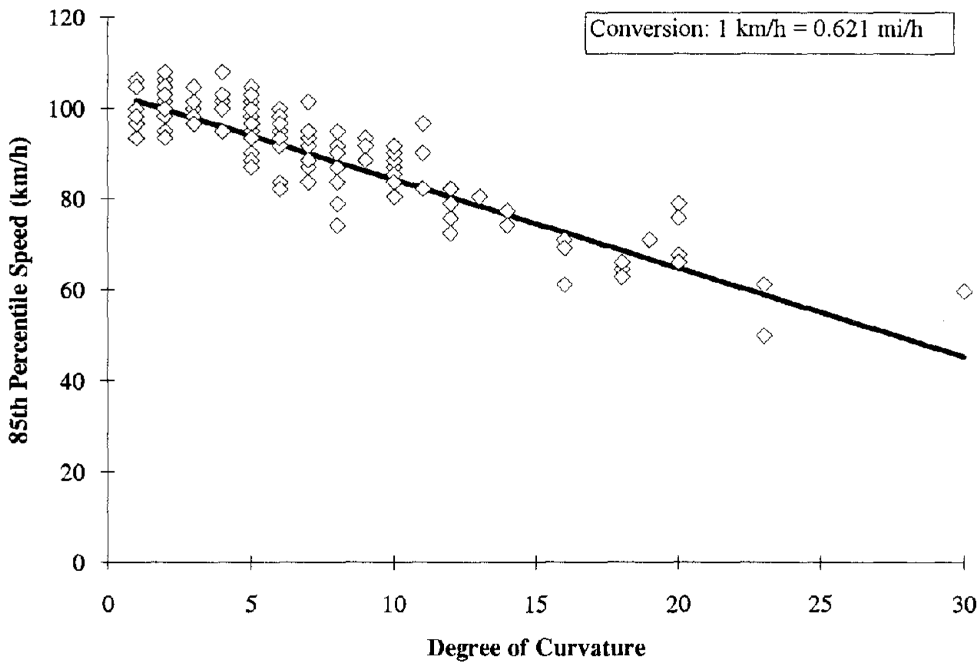
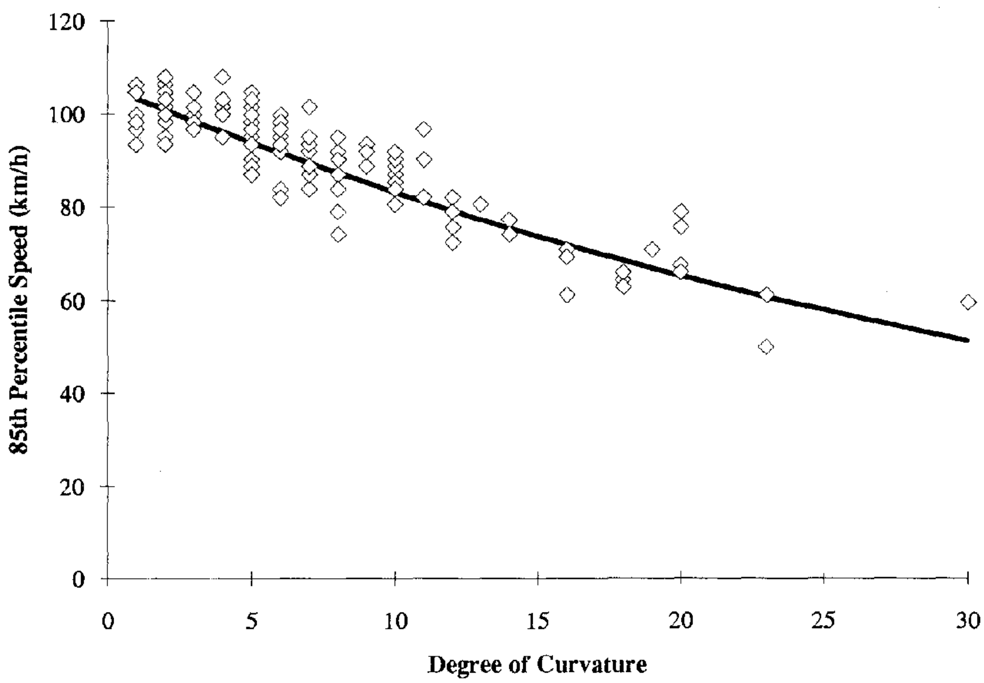


Figure 13. Scatter plot of 85th percentile speed versus degree of curvature.

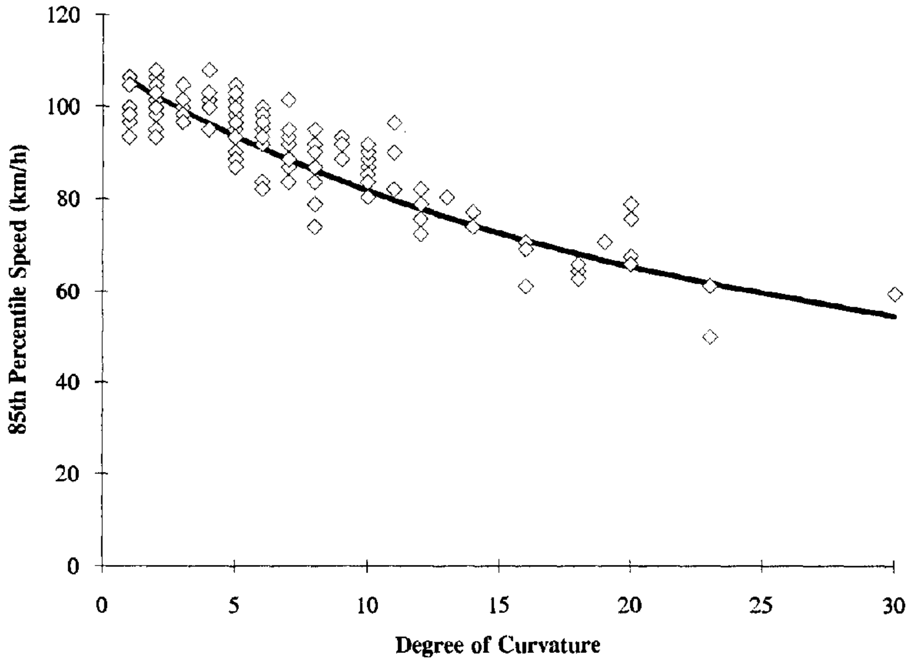


(a) Linear

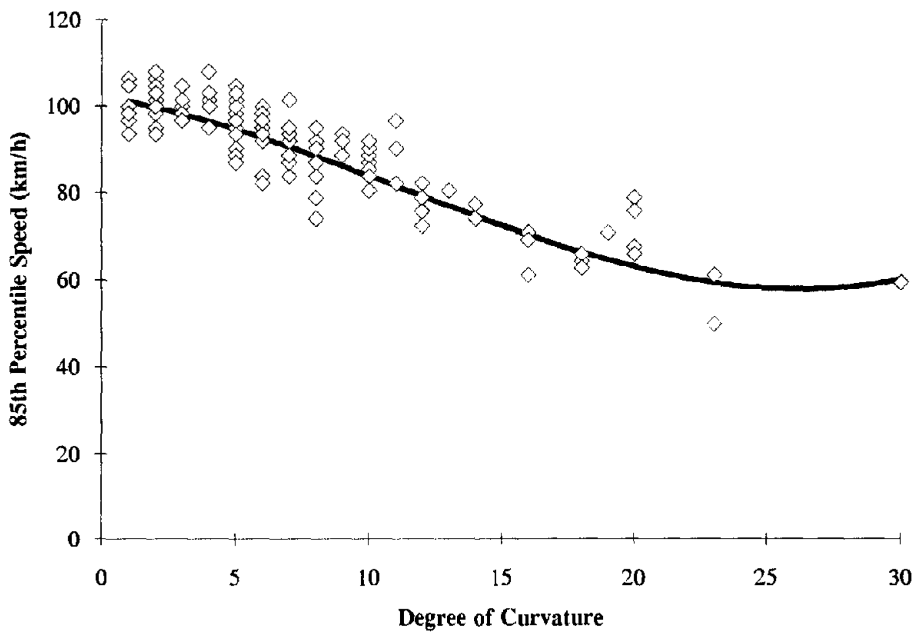


(b) Exponential

Figure 14. Regression equations for 85th percentile speed versus degree of curvature.



(c) Inverse



(d) Third-Order Polynomial

Figure 14. Regression equations for 85th percentile speed versus degree of curvature (continued).

Table 14. Regression analysis results for 85th percentile speed versus degree of curvature.

Regression Estimate	Equation Form from Table 9			
	Linear	Exponential	Inverse	Third-Order Polynomial
β_0 (km/h)	103.66	4.66	0.0092	102.19
β_1 (km/h/°)	-1.95	-0.02	0.0003	-1.05
β_2 (km/h/°^{2})	--	--	--	-0.11
β_3 (km/h/°^{3})	--	--	--	0.0034
F-Statistic	556.89	585.07	540.97	204.06
P-Value	0.0001	0.0001	0.0001	0.0001
R^2-Value	0.80	0.81	0.80	0.82
Conversion: 1 km/h = 0.621 mi/h				

The linear regression equation for 85th percentile speed on curves is as follows:

$$V85 = 103.66 - 1.95\,D \qquad (2)$$

where: $V85$ = 85th percentile speed on the curve (km/h) [1 km/h = 0.621 mi/h]
D = degree of curvature (°).

The only serious concern about the linear equation is that the intercept term—103.66 km/h (64.4 mi/h)—overestimates the mean of the 85th percentile speeds on long tangents—97.9 km/h (60.8 mi/h). The intercept might be interpreted as an estimate of what 85th percentile speeds on long tangents would be if they were not constrained by the 88.5-km/h (55-mi/h) speed limit on this class of roadway.

Evaluation of Other Independent Variables. The other independent variables listed in table 12 were tested for their statistical significance and contribution to the explanatory power of the regression equation for 85th percentile speeds on horizontal curves. The variables that were directional in nature (i.e., variables including sight distance to the curve whose values were different for each lane of the curve) were evaluated with the by-lane data base that distinguished the inside and outside lanes of each curve. The variables that were not directional (e.g., length of curve or deflection angle) were evaluated using both the by-lane and by-site data bases.

For each of the equation forms, the same independent variables were statistically significant:

- Degree of curvature.
- Length of curve.
- Deflection angle, which is the product of degree of curvature and length of curve divided by 100.

No additional variables in table 12 were statistically significant at $\alpha = 0.05$. Because only the characteristics of the curve itself were significant, and no significant difference was found between the mean of the 85th percentile speeds for the inside and outside lanes, a single regression equation containing only information relative to the curve is considered sufficient for both travel directions.

Among the multiple-variable regression equations, the linear form continued to perform as well as the more complex forms. Therefore, the recommended multiple-variable form is the following multiple-linear regression equation, which had an R^2-value of 0.82 and a root mean square error of 5.1 km/h (3.1 mi/h):

$$V85 = 102.45 - 1.57\,D + 0.0037\,L - 0.10\,I \tag{3}$$

where:

$V85$	=	85th percentile speed on the curve (km/h) [1 km/h = 0.621 mi/h]
D	=	degree of curvature (°)
L	=	length of curve (m) [1 m = 3.28 ft]
I	=	deflection angle (°).

Table 15 summarizes the analysis results for the regression parameters in the multiple-linear equation. Figure 15 provides a three-dimensional representation of the equation.

The effect of the length of curve and deflection angle terms is evident. For curves $\leq 4°$, 85th percentile speeds increase as the length of curve increases. For curves $> 4°$, 85th percentile speeds decrease as the length of curve increases. Two different hypotheses may explain the effect of curve length on the 85th percentile speeds at the midpoint of sharp curves. The first hypothesis is that the shorter the sharp curve, the more likely it is that drivers flatten their path and, therefore, the faster the speed drivers are able to maintain through the curve. The second hypothesis is that the longer the sharp curve, the greater the distance over which drivers can decelerate and, therefore, the more likely it is that drivers reach their desired speed before the midpoint of the curve. A corollary of the second hypothesis is that speeds measured at the midpoint of longer sharp curves more accurately reflect drivers' desired speed than speeds measured at the midpoint of a shorter curve with the same degree of curvature. Another implication of the second hypothesis is that drivers decelerate between the beginning and midpoint of sharp curves.

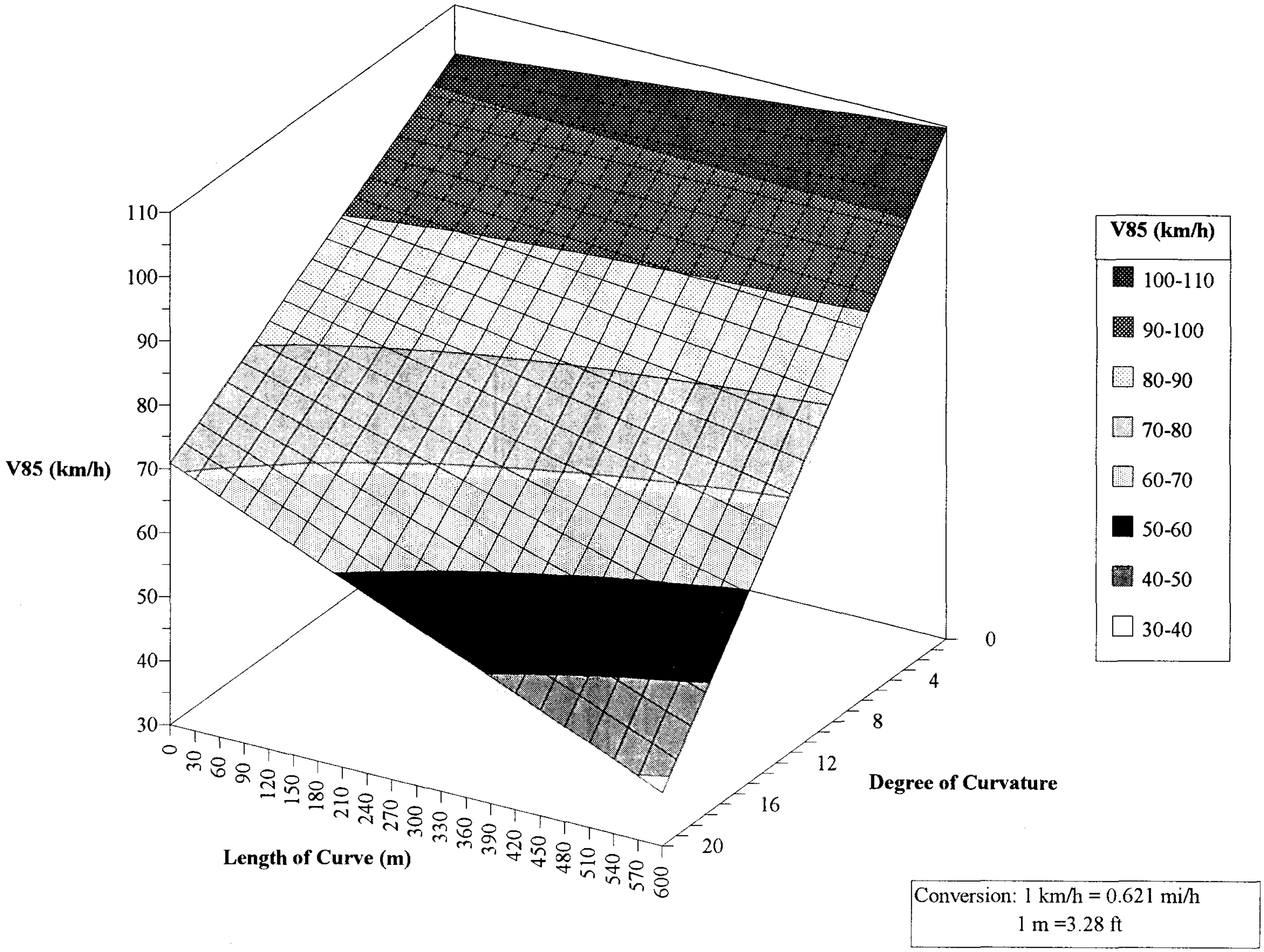


Figure 15. Multiple-linear regression equation for 85th percentile speeds on curves.

Table 15. Regression analysis results for multiple-linear equation for speeds on curves.

	β_0	β_1	β_2	β_3
Parameter Estimate	102.43 km/h	-1.58 km/h/°	0.0037 km/h/m	-0.10 km/h/°
Standard Error	1.23 km/h	0.13 km/h/°	0.0014 km/h/°	0.03 km/h/°
P-value for t-test	0.0001	0.0001	0.0093	0.0005
$V85 = \beta_0 + \beta_1 D + \beta_2 L + \beta_3 I$				
Conversions: 1 km/h = 0.621 mi/h; 1 m = 3.28 ft				

Approach Tangent Speed in the Regression Equation. The final step in the analysis was to evaluate the benefits of adding to the multiple-linear equation, the 85th percentile speed on the approach tangent to the curve as an independent variable. The resulting equation had an R^2-value of 0.90 and root mean square error of 4.2 km/h (2.6 mi/h):

$$V85 = 41.62 - 1.29\,D + 0.0049\,L - 0.12\,I + 0.95\,V_t \tag{4}$$

where:
- $V85$ = 85th percentile speed on the curve (km/h) [1 km/h = 0.621 mi/h]
- V_t = 85th percentile speed on approach tangent (km/h) [1 km/h = 0.621 mi/h]
- D = degree of curvature (°)
- L = length of curvature (m) [1 m = 3.28 ft]
- I = deflection angle (°).

The statistical fit measures for equation 4 cannot be compared directly to the measures for equations 2 and 3, because equation 4 was fit to the paired data base of 78 curves, whereas equations 2 and 3 were fit to the by-site data base of 138 curves. The statistical results suggest that knowing the approach tangent speed improves the prediction of 85th percentile speeds on the curve. However, because the analysis of 85th percentile speeds on long tangents did not yield a statistically significant prediction equation, it was not possible to expand this equation for use on all curves by allowing the use of an estimated, rather than observed, 85th percentile speed on the approach tangent. Therefore, this equation would be useful only if approach tangent speeds were actually measured.

SPEED-PROFILE MODEL

A speed-profile model for estimating the reduction in 85th percentile speeds from an approach tangent to a horizontal curve was constructed using the statistical analysis results for speeds on horizontal curves and long tangents. Both equations 2 and 3 were considered for estimating 85th percentile speeds on curves. Since the data did not suggest a useful predictive equation for 85th percentile speeds on long tangents, consideration was given to the approach used by Lamm et al. in which the intercept term of the regression equation for curves was used as the estimate of the 85th percentile speed on long tangents.[45] Since the intercept terms of equations 2 and 3 overestimated the observed speeds on tangents, however, it was concluded that the mean of the observed 85th percentile speeds on long tangents—97.9 km/h (60.8 mi/h)—should be used in the speed-profile model as the desired speed on long tangents. Furthermore, for curves whose estimated 85th percentile speed is greater than 97.9 km/h (60.8 mi/h), the speed on the curve is set equal to 97.9 km/h (60.8 mi/h).

The model assumes that speeds are constant through the horizontal curve. All deceleration and acceleration occur on the tangents approaching and departing the curve. Acceleration and deceleration rates are assumed to be equal. The 0.85-m/s^2 (2.8-ft/s^2) rate reported by Lamm et al. was used in the model.[45]

Basic Form of the Model

Figure 16 illustrates the variables that define the speed-profile model. Table 16 lists the equations for those variables. The equations, which were drawn from Lamm et al., were re-derived from basic equations of motion in order to verify their accuracy, and then converted to metric units.[45]

The tangent classification (cases 1, 2, or 3 from figure 16) is determined by comparing the actual tangent length (TL) to the critical tangent length (TL_c). The critical tangent length is calculated as follows:

$$TL_c = \frac{2V_f^2 - V85_1^{\,2} - V85_2^{\,2}}{25.92\ a} \tag{5}$$

where:

TL_c	=	critical tangent length (m) [1 m = 3.28 ft]
V_f	=	85th percentile desired speed on long tangents (km/h) [1 km/h = 0.621 mi/h]
$V85_n$	=	85th percentile speed on curve n (km/h) [1 km/h = 0.621 mi/h]
a	=	acceleration/deceleration rate = 0.85 m/s^2 [1 m/s^2 = 3.28 ft/s^2].

Alternative Form of the Model

The basic form illustrated in figure 16 assumes that deceleration begins where required, even if the beginning of the curve is not yet visible. An alternative form of the speed-profile model was developed to consider the effect of limited sight distance in a manner similar to the Swiss and French methods. The alternative form did not permit deceleration to occur before the point at which the curve comes into view or, in other words, required the deceleration distance to be less than or equal to the available sight distance to the curve. This approach assumes no reaction time between when the curve comes into view and when deceleration begins.

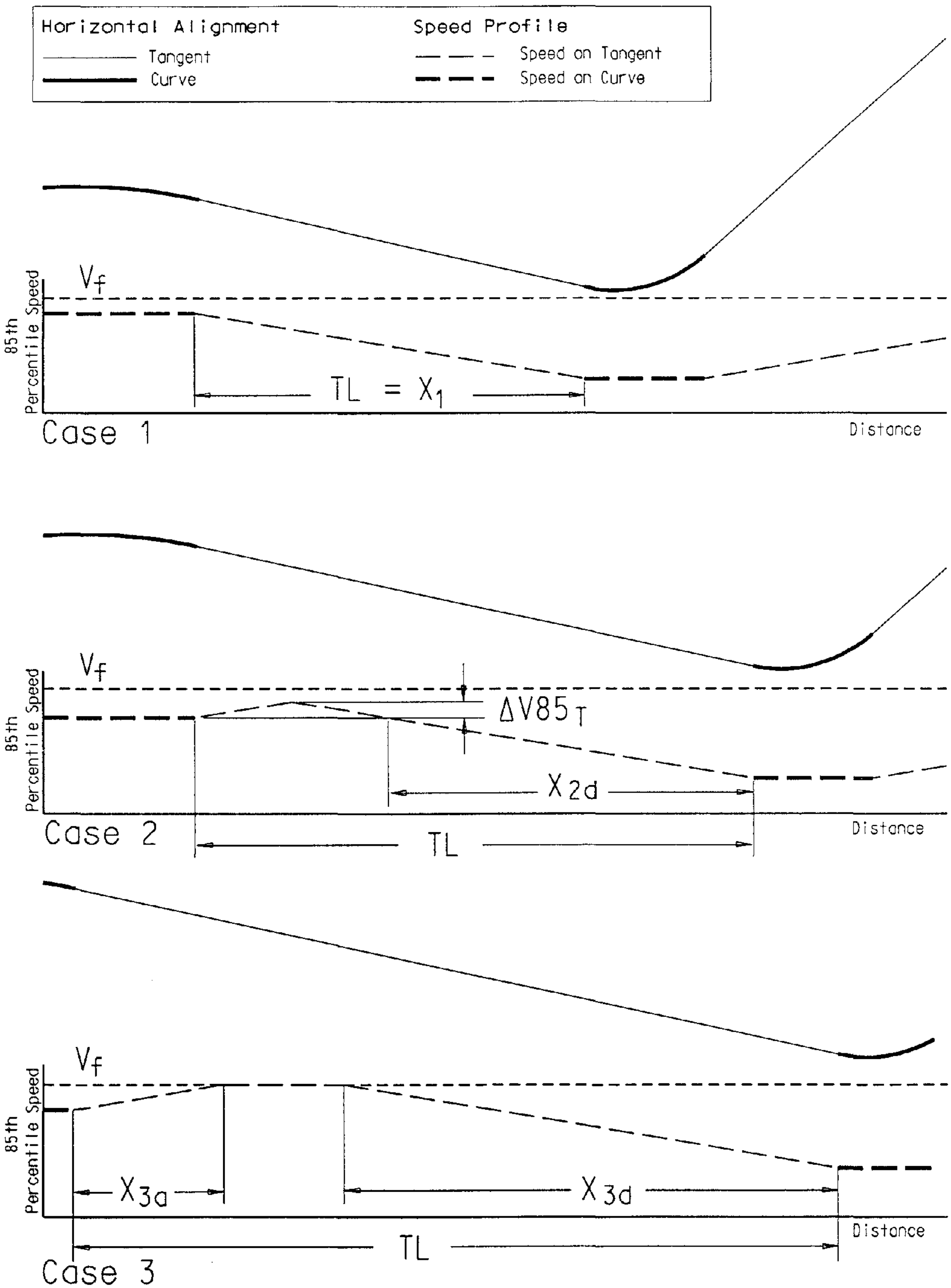


Figure 16. Variable definitions for speed-profile model.

Table 16. Equations for constructing 85th percentile speed profile.[45]

Case	Condition	Equation	V_f Reached?
1	$TL = X_1$	$$X_1 = \frac{V85_1^2 - V85_2^2}{25.92\ a}$$	No
2.1	$TL < TL_c$	$$X_{2d} = \frac{V85_1^2 - V85_2^2}{25.92\ a}$$ $$Max(V85)^*_{Tan} = V85_1 + \Delta V85_{Tan}$$ $$\Delta V85_{Tan} = \frac{-2V85_1 + [4V85_1^2 + 44.06(TL - X_{2d})]^{\frac{1}{2}}}{2}$$ *Note that when calculating ***Max(V85)$_{Tan}$***, the curve with the lower degree of curvature must be selected.	No
2.2	$TL = TL_c$	$$X_{2a} = X_{3a} = \frac{V_f^2 - V85_1^2}{25.92\ a}$$	Yes, reached but not sustained
3	$TL > TL_c$	$$X_{3d} = \frac{V_f^2 - V85_2^2}{25.92\ a}$$ $$X_{3a} = \frac{V_f^2 - V85_1^2}{25.92\ a}$$	Yes, reached and sustained

Where:
- $X_{i,(a,d)}$ = distance traveled for case i during acceleration (a) or deceleration (d) (km/h)
- $V85_n$ = 85th percentile speed on curve n (km/h)
- $\Delta V85$ = difference between the 85th percentile speeds (km/h)
- a = acceleration/deceleration rate = 0.85 m/s^2 (2.8 ft/s^2)
- TL = tangent length (m)
- TL_c = critical tangent length (m)
- V_f = 85th percentile desired speed on long tangents (km/h)

Considering sight distance in this manner affects the speed-reduction estimates only for those curves for which the preceding deceleration distance was greater than the available sight distance to the curve. For all three cases illustrated in figure 16, the speed-reduction estimate either remains the same or increases when the effect of limited sight distance is considered. In case 3, the reduction in the deceleration distance increases the length of tangent over which the desired speed is sustained and increases the deceleration rate so that the estimated speed on the horizontal curve is reached. However, since the maximum speed attained on the approach tangent does not change, the estimated speed reduction remains the same. Curves in case 1 change to case 2 when the effect of limited sight distance is considered and, therefore, the speed-reduction estimate increases. In case 2, the reduction in the deceleration distance increases the maximum speed attained on the approach tangent and, therefore, increases the estimated speed reduction.

Speed-Profile Model Validation

A preliminary validation study was conducted to evaluate the accuracy of the speed-reduction estimates from the speed-profile model. The speed-reduction estimates were compared to measured speed reductions at 78 sites where speed data had been collected both on the approach tangent and at the midpoint of the curve.

The results are presented in table 17, which reports the mean of the absolute value of the difference between the estimated and measured speed reductions for 78 approach tangent-curve pairs for each form of the speed-profile model. All model forms produced reasonably accurate estimates. The forms that did not consider sight distance had smaller mean absolute differences than the forms that did consider sight distance. The forms that used the multiple-linear regression equation for 85th percentile speeds on horizontal curves had smaller mean absolute differences than the forms that used the simple-linear regression equation.

Table 17. Mean absolute difference between estimated and measured speed reductions.

Form of Speed-Profile Model		Mean Absolute Difference (km/h)
Sight Distance Effects Considered?	Equation for 85th Percentile Speed on Curves	
No	Simple Linear	3.8
No	Multiple Linear	2.8
Yes	Simple Linear	4.2
Yes	Multiple Linear	3.9
Conversion: 1 km/h = 0.621 mi/h		

Figure 17 is a scatter plot of the differential between the estimated and measured speed reduction versus degree of curvature for 78 approach tangent-curve pairs for the speed-profile model form that used the multiple-linear regression equation for 85th percentile speed on curves and that did not consider sight-distance effects. The majority of estimates (79 percent) are within 5 km/h (3.1 mi/h) of the measured speed reduction.

The preliminary validation study results are promising. They must be viewed with caution, however, since the actual speed reductions were based upon the same data (in paired form) as were used (in unpaired form) to calibrate the speed-profile model from which the estimated speed reductions were obtained. Furthermore, the data represent curves that met site-selection criteria controlling the tangent length and sight distance to the curve; most of the approach tangent-curve pairs fall in case 3 (in figure 16) and most have more than 122 m (400 ft) of sight distance to the curve. Therefore, the preliminary validation study does not test the accuracy of the model at estimating speed reductions for all approach tangent-curve conditions.

A comprehensive validation study is recommended before the speed-profile model is adopted for use in design practice. An independent data base should be developed to test the validity of the speed-profile model. The validation study should evaluate the individual components and assumptions of the model as well as the resulting speed-reduction estimates. The estimated values or regression equations for speeds on horizontal curves, desired speeds on long tangents, and deceleration and acceleration rates approaching and departing curves should be validated individually. The key assumptions to be validated are that deceleration and acceleration rates are equal and that all deceleration and acceleration occur on the approach and departure tangents. The appropriate assumption about the effect of limited sight distance should also be evaluated.

It is recommended that the validation data base include the following subsets:

- A sample of curves similar to those with which the model was calibrated, i.e., satisfying the criteria in table 10 and from one or more of the States in which the calibration data were collected. These data would be used to test whether the model represents the population of curves for which the model was developed.

- A sample of curves that do not have long approach tangents, but that otherwise satisfy the criteria in table 10. These data would be used to test whether the model predictions are equally valid for cases 1, 2, and 3 in figure 16.

- A sample of curves from States different from those in which the calibration data were collected. These data would be used to test whether the model predictions are valid nationwide.

- A sample of curves should be selected for more detailed speed measurements that would permit the determination of the rates and locations relative to the curves that deceleration and acceleration actually occur. These data would be used to test whether the assumptions about acceleration and deceleration are appropriate.

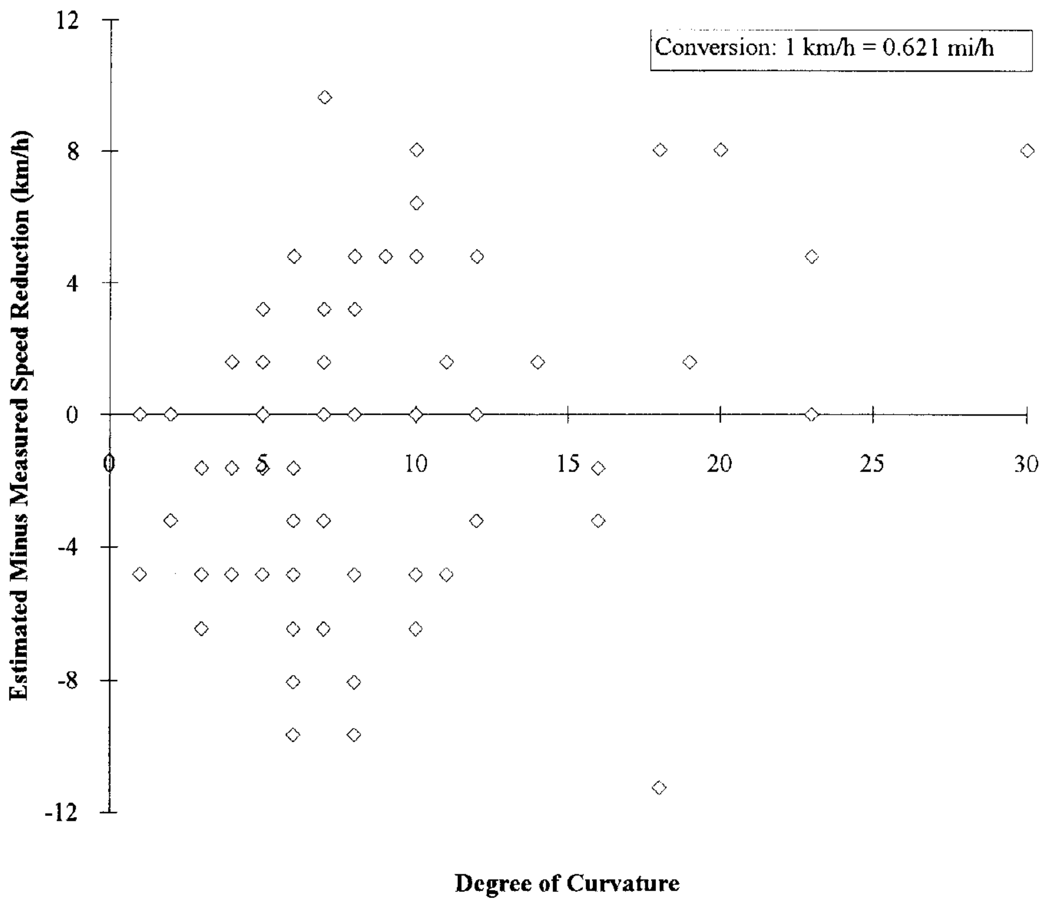


Figure 17. Preliminary validation results for speed-profile model.

MICROCOMPUTER PROCEDURE FOR USING THE SPEED-PROFILE MODEL

A menu-driven, microcomputer procedure was developed to simplify the use of the speed-profile model. The procedure requires only alignment data available on standard plan-profile sheets: degree of curvature, and stationing of the beginning and end of horizontal curves. A data entry table is provided for entering the required alignment data. The procedure performs all necessary calculations. Output is available in both tabular and graphical formats. Software and users manuals for the microcomputer procedure (in both English and metric units) are provided as separate reports.[59,60]

SUMMARY

This chapter documents the development of a speed-profile model for estimating reductions in 85th percentile operating speeds from approach tangents to horizontal curves on rural two-lane highways. The speed-profile model was calibrated using speed and geometry data collected at 138 horizontal curves and 78 approach tangents on 29 rural highways in 3 regions of the United States.

The analysis produced several important findings regarding speeds on horizontal curves and on long tangent sections. With respect to 85th percentile speeds on long tangents, it was not possible to develop a useful statistical equation. The only significant difference identified was that tangent speeds on level roadways in Texas were significantly faster than on roadways in rolling terrain in the East and West. The overall 97.9-km/h (60.8-mi/h) mean of the 85th percentile speeds on all long tangent sections is a reasonable estimate of drivers' desired speed on this type of highway, although it is probably constrained by the posted speed limit.

With respect to 85th percentile speeds on horizontal curves, a model that is linear with respect to degree of curvature was selected as the most appropriate. In addition to degree of curvature, the length of curve and deflection angle also had statistically significant effects on curve speeds. Neither approach characteristics (sight distance, approach tangent length, and preceding degree of curvature) nor the cross-section through the curve (travel-way width and total pavement width) were statistically significant predictors. The 85th percentile speeds on the inside and outside lanes of curves were not significantly different. The 85th percentile speeds on curves with degrees of curvature $\leq 4°$ did not differ significantly from 85th percentile speeds on long tangents.

The observed driver speed behavior on horizontal curves confirms the results of previous studies that revealed a disparity between 85th percentile speeds and design speeds. The data collected during this study indicate that the 85th percentile speed exceeds the design speed on most curves whose design speed is less than the desired speed on long tangents—97.9 km/h (60.8 mi/h). Considerable variability was observed among the superelevation rates used on curves with a given degree of curvature. These findings suggest several recommendations for improving U.S. rural horizontal alignment design policy: (1) AASHTO should review recent data on driver speed behavior and consider revisions to the guidelines on minimum design speed, (2) the speed-profile model developed during this study should serve as the basis for a feedback loop, incorporated into U.S. design procedures, to check for operating-speed inconsistencies on rural highway horizontal alignments with design speeds $\leq$ 100 km/h (62.1 mi/h), and (3) AASHTO should consider revising its policies to promote nationwide uniformity of horizontal curve design, including adopting a nationwide maximum superelevation rate and revising the method for selecting the superelevation rate for a particular curve. Alternative methods are to: (1) select the superelevation rate for a curve based upon the estimated 85th percentile speed approaching the curve (which is used in several European countries), or (2) specify a unique superelevation rate for each degree of curvature.

Several forms of the speed-profile model (using either a simple-linear or multiple-linear regression equation for estimating 85th percentile speeds on curves, with or without considering the effect of limited sight distance) were developed. A preliminary validation study suggests that the model produces reasonable speed-reduction estimates. The model form that did not consider the effect of sight distance and that used the multiple-linear regression equation produced the most accurate speed-reduction estimates. Since this validation study used a subset of the same data used to calibrate the speed-profile model, albeit in different forms, a comprehensive validation study using an independent data base is recommended.

4. DRIVER-WORKLOAD-BASED MEASURES OF CONSISTENCY

Driver-workload-based measures of geometric design consistency have conceptual appeal. Their development, however, has been limited by the difficulty in measuring workload.

Messer, Mounce, and Brackett developed a procedure for evaluating geometric design consistency based upon a model that estimates driver workload as a function of roadway geometry.[2-5] This procedure has not been extensively used, however. Its principal criticism is that it is based upon subjective appraisals rather than objective measurements of workload.

Therefore, this study focused on obtaining objective measurements of driver workload, with the scope limited to horizontal curves and tangents. This chapter presents an evaluation of driver-workload-based measures of consistency and documents the development of a driver-workload model based upon occluded vision test measurements. The chapter is organized as follows:

- Previous research on three topics is reviewed: driver workload; the Messer, Mounce, and Brackett model; and alternative methods for measuring driver workload.

- This study's methodology is described for the objective measurement of driver workload on horizontal curves using occluded vision tests.

- The statistical analysis results are presented.

- Finally, a menu-driven microcomputer procedure for driver-workload-based consistency evaluation is introduced.

REVIEW OF PREVIOUS RESEARCH

Driver Workload

Driver workload is "the time rate at which drivers must perform a given amount of work or driving task."[4] The workload is associated with the information-processing demands of the driving task. To reduce the probability of speed and/or path errors in the driving task, the workload demands should be neither high enough to exceed drivers' information-processing capacity nor low enough to induce inattention.

Drivers divide their attention (i.e., the proportion of information-processing capacity allocated to driving) among three tasks: control, guidance, and navigation. Highway geometry primarily influences the guidance task (i.e., guiding the vehicle at an appropriate speed along the path defined by the roadway). Therefore, this study focuses on the information-processing demands of the guidance task.

Drivers extract information from the roadway environment in order to guide the vehicle along the roadway. The information pertains to the directional path of the roadway and the position of the vehicle relative to the path. Directional information can be provided by the roadway itself, delineation devices along the roadway, traffic on or near the roadway, and the roadside environment. Position information is provided by the hood of the vehicle viewed in relation to the edge(s) of the roadway.

Drivers allocate information-processing capacity to the guidance task based upon their expectations about the workload that will be imposed by the roadway ahead. The higher the estimated workload demand, the greater the attention level allocated to the task. Workload estimates for a given driver vary as a function of vehicle characteristics, vehicle speed, traffic, ambient lighting and weather conditions, and roadway geometry. These estimates will also vary as a function of driver ability, expectancy, experience, and physiological and psychological state.

Underestimating the workload demands of the guidance task can result in errors and, possibly, accidents. Workload demands may be underestimated when a driver—through lack of experience—fails to recognize characteristics of the roadway that require greater attention; when an experienced driver expects a lower workload than is required; when a driver's information-processing ability is impaired; or when a driver's attention is distracted from the guidance task such that the stream of information upon which workload projections are based is interrupted.

Drivers manage guidance workload in one of two ways: (1) by increasing their attention level (i.e., allocating more information-processing capacity to the driving task) by fixating on the road geometry, particularly during curve negotiation, or (2) by reducing speed, thereby reducing the rate at which information is received and increasing the time available to process information.[61,62] Both of these actions require the driver to recognize the workload demand imposed by the roadway ahead. Accurate recognition is facilitated by consistently designed roadway geometry. This study focuses on the effect of roadway geometry on the workload demands of the guidance task.

Estimating Driver Workload from Roadway Geometry

Messer, Mounce, and Brackett developed a driver-workload-based procedure to evaluate geometric design consistency on rural highways (both two-lane and multilane).[2-5] The procedure includes: (1) a model for estimating driver workload as a function of the roadway geometry, and (2) level of consistency criteria for evaluation purposes. The model estimates the driver workload associated with a geometric feature based upon the criticality of and sight distance to the feature, and its consistency and spacing relative to preceding features. The scope of the model includes 10 geometric features: horizontal curves, crest vertical curves, bridges, divided-highway transitions, lane drops, intersections, railroad grade crossings, shoulder-width changes, lane-width reductions, and crossroad overpasses.

The model for estimating driver workload is represented by the following equation:

$$WL_n = R_f \times S \times E \times U + C \times WL_{n-1} \tag{6}$$

where:
- WL_n = workload value for feature n
- R_f = workload potential rating for feature n
- S = sight-distance factor
- E = feature expectation factor
- U = driver unfamiliarity factor
- C = feature carryover factor
- WL_{n-1} = workload value for preceding feature n-1.

The equation consists of two terms that are added to compute the workload value for a particular feature. The first term indicates that the workload value (WL_n) for feature n is a function of: (1) the criticality of the feature, (2) the sight distance to the feature, (3) the similarity between the feature and the preceding feature, and (4) the familiarity of drivers with the feature. The second term accounts for the effect of preceding features on the workload value for feature n. Numerical values for each of the factors in the model are presented in tables and figures.

The workload potential rating (R_f) quantifies the criticality of a feature. The rating was based upon subjective appraisals of features by design, traffic, and human factors engineers on a 7-point scale (from 0 = "no problem" to 6 = "big problem"). Feature criticality depends upon the feature type and its relative frequency of occurrence, basic operational complexity, and overall accident experience. For example, workload potential ratings for horizontal curves are provided for various combinations of degree of curvature and deflection angle. The ratings for two-lane highways range from 0.5 for a 1° curve with a 10° deflection angle to a 7.7 for an 8° curve with a 120° deflection angle. The workload value is computed by modifying the workload potential rating by factors that account for the sight distance to the feature and the feature's similarity and proximity to the preceding feature.

The sight-distance factor (S) becomes larger as the sight distance to the feature decreases. Therefore, for a feature with a given criticality, the workload value increases as the sight distance to the feature decreases. As sight distance decreases, there is a decrease in the time available at a given speed for drivers to process the information about the feature. As a result, there is an increase in the workload (i.e., the time rate at which the fixed amount of information associated with the feature must be processed).

The feature expectation factor (E) decreases the workload value for the feature if it is similar to the preceding feature. The similarity of the preceding feature creates an ad hoc expectancy that reduces the effort required to process the information associated with the feature.

The driver unfamiliarity factor (U) adjusts the workload value based upon the percentage of drivers familiar with the roadway. Familiar drivers have a priori expectancies (i.e., prior knowledge of the roadway and, therefore, the particular feature) that reduce the effort required to process the information associated with the feature. The other factors in the equation were calibrated based upon unfamiliar drivers. Therefore, the driver unfamiliarity factor reduces the workload value as the percentage of familiar drivers increases.

The feature carryover factor (C) represents the proportion of the workload from the preceding feature that carries over to the feature when the features are closely spaced. The contribution of preceding feature n-1 to the workload value for feature n increases as the separation distance between the features decreases. That is, when features are closely spaced, there is less driving time between the features to shed the information associated with preceding feature n-1 and to process the information about feature n. Therefore, the workload value for feature n increases as the distance from the preceding feature decreases.

Level-of-consistency criteria are provided to assess the likely driver response associated with a workload value of a given magnitude. These criteria are similar in concept to the level-of-service criteria used in highway capacity analysis. Table 18 defines the levels.

Table 18. Level-of-consistency criteria based upon driver workload.[4]

Level of Consistency	Workload Value	Likely Driver Response
A	≤1	No Problem Expected
B	≤2	
C	≤3	Small Surprises Possible
D	≤4	
E	≤6	
F	>6	Big Problem Possible

Two studies have evaluated the relationship between the workload values estimated using this model and accident experience on rural two-lane highways.[63,64] The first study was limited to 5 rural two-lane highways in Texas; the second study included 19 Texas highways. The results of the first study indicate that accidents are more likely to occur on features with high workload values than on features with low workload values.[63] The results of the second study indicate that the average accident rate for features with level-of-consistency F is double the rate for levels A through E; furthermore, the accident rate for level-of-consistency F is approximately 1.5 times the average accident rate for rural two-lane highways in Texas, whereas the accident rates for levels A through E are below the average.[64] These results suggest that there may be a threshold workload value below which accident rates are not significantly affected by variations in workload, but above which accident rates increase significantly.

Previous research suggests that driver workload is a promising measure of geometric design consistency. Preliminary evaluations indicate that the workload values estimated using the model developed by Messer, Mounce, and Brackett may be good indicators of high accident locations on rural two-lane highways. One strength of driver workload as a measure of consistency is that, in theory, it can be applied to any geometric feature—unlike operating-speed reduction, which is limited in application to horizontal, and possibly vertical, alignment. The principal weakness of driver workload is that it is difficult to measure. The Messer, Mounce, and Brackett model is based upon subjective appraisals rather than objective measurements, which makes it difficult to validate and, therefore, limits it credibility.

Alternative Methods for Measuring Driver Workload

In order to address the criticism about the subjective basis of the driver workload estimates from the Messer, Mounce, and Brackett model, a review of the human factors literature was conducted to evaluate objective methods for measuring driver workload. The four potential methods that were identified are the primary task, secondary task, information storage, and vision occlusion methods.

Primary Task Method

The primary task method involves the direct measurement of the operator's performance on the primary task. This method assumes that the human operator is a single-channel information-processing device.[65] Measurements are made by increasing the information flow rate for the primary task until performance decreases. Primary task methods can identify information-overload conditions, but they generally lack the desired sensitivity to discriminate relative workload levels when information demands are less than the operator's mental capacity, which generally would be the case for the guidance level of the driving task.[66]

Secondary Task Method

The secondary task method determines "how much additional work the operator can undertake while still performing the primary task to meet system criteria."[66] The secondary task is performed both in the absence of and in conjunction with the primary task. The reduction in performance on the secondary task when performed in conjunction with the primary task represents the workload demanded by the primary task.[67] This method has the following drawbacks: the secondary task might distract the operator from the primary task, the secondary task might inflate estimates of workload due to the additional information-processing effort required to transition between the primary and secondary tasks, and the operator might become absorbed in the primary task and neglect the secondary task.

Information Storage Method

In the information storage method, the human operator voluntarily controls the presence or absence of primary task information.[68] When not receiving information about the primary task, the operator relies on the integration of information previously received to make predictions concerning future performance requirements. Assuming error-free performance, the workload can be estimated by the amount of time the operator requires access to information as a proportion of the total time the task is being performed.

Vision Occlusion Method

The vision occlusion method is a form of the information storage method that has been applied to the driving task. In this method, drivers voluntarily occlude their vision, opening their eyes only when they think it necessary to extract information for the guidance task.[69-71] According to Williges and Wierwille, in this approach, "The assumption is made that the driver's attention is only intermittently on the road. Between observations of the road, the driver approaches a threshold of uncertainty concerning the placement of his/her own, as well as other, vehicles on the road. When this threshold of uncertainty is reached, the driver once again observes the road."[68]

If vehicle speed is constant and lane integrity is not violated, then the amount of time that drivers are unwilling to have their vision occluded, over a fixed length of roadway, represents the mental workload required for the guidance task for that particular section of roadway. The lower the information-processing demands for guiding the vehicle along the roadway, the longer the time drivers will voluntarily keep their vision occluded. Conversely, the greater the information-processing demands, the greater the amount of time a driver will need to look at the roadway and the higher the mental workload.

Farber and Gallagher evaluated the use of a vision interruption apparatus by having subjects drive a slalom course at a controlled speed without hitting any of the traffic cones that delineated the course.[70] The apparatus allowed subjects to request 0.5-s glimpses of visual information by pressing a foot switch located to the left of the brake. They concluded that the apparatus "is applicable as a basic measuring tool to any research in which driver performance provides the criterion measure."[70]

Hicks and Wierwille used a vision occlusion device on a driving simulator to examine the differences among workload measurement techniques.[72] The video signal on the driving simulator was blanked until the subject vocally requested visual information. The research showed that the simulated nature of the driving task in the experiment affected the results by allowing subjects to accept larger lane deviations than usually would be accepted. In addition, the research showed that subjects need time to become familiar with the vision occlusion device. If subjects are not given sufficient time, the workload measurements will be inflated by the additional workload associated with learning to use the device.

Another application of the vision occlusion method by Godthelp involved a study of the behavior of drivers immediately before the beginning of a horizontal curve.[69] Driver vision was occluded 0.5 s before the beginning of the curve, and the occlusion lasted 1.5 s. The results of the study indicated that drivers take an anticipatory steering action before the beginning of the curve and that the error in the steering action increases as the degree of curvature increases.

The vision occlusion method was also discussed as part of a model for driving behavior introduced by Godthelp, Milgram, and Blaauw in 1984.[73] In the results reported, voluntary occlusion times for drivers varied as a function of vehicle speed from an average of more than 5 s at 20 km/h (12.4 mi/h) to less than 2.5 s at 100 km/h (62.1 mi/h).

In order for workload to remain constant at different demand levels, either speeds or occlusion time must change. Van Der Horst and Godthelp reported the results in table 19, which indicate that as lane widths were reduced from 3.55 m (11.6 ft) to 2.05 m (6.7 ft), median voluntary occlusion time decreased.[74] Furthermore, as vehicle speed increased for any given lane width, occlusion time decreased.

Table 19. Median occlusion times (s) for given lane widths and speeds.[74]

Speed (km/h)	Lane Width (m)			
	2.05	2.55	3.05	3.55
20	2.4	3.6	3.8	4.3
60	1.4	2.0	2.5	2.9
100	0.9	1.4	1.9	2.1
Conversions: 1 km/h = 0.621 mi/h; 1 m = 3.28 ft				

In summary, previous research suggests that voluntary vision occlusion time is a sensitive measure of the workload demands of roadway geometry and that the vision occlusion method is the most promising alternative for the objective measurement of driver workload in the guidance task.

OCCLUDED VISION TEST STUDIES

A two-part experiment consisting of a series of occluded vision tests was designed and conducted to obtain objective measures of driver workload on horizontal curves. The experimental design, vision occlusion apparatus, subjects, test procedure, and test courses are described in this section. The tests were conducted on horizontal curves laid out on former airport runways at the Texas A&M Proving Ground Research Facility. Tests were conducted during daylight conditions.

Experimental Design

The experimental design for the occluded vision test studies involved one dependent variable and two independent variables. The dependent variable was driver workload. The independent variables were degree of curvature and deflection angle. Two studies were conducted to collect occluded vision test data. Originally, only the first study was planned. The second study was conducted to fill gaps in and expand the independent variable levels tested. Table 20 summarizes the levels of degree of curvature and deflection angle in the two studies. The data from the two studies were combined for the statistical analysis of the relationship between driver workload and the two independent variables.

Table 20. Independent variables for occluded vision tests.

Degree of Curvature	Deflection Angle			
	Study 1		Study 2	
	20°	45°	45°	90°
3°	X	X		
6°	X	X	X	X
9°	X	X	X	X
12°			X	X

Vision Occlusion Apparatus

A computerized vision occlusion apparatus was installed in a Ford Taurus station wagon to control and measure the visual input to the test subjects. The apparatus consisted of a pair of goggles, a microcomputer system, and a foot switch.

The goggles were wrap-around plastic safety goggles on which a thin liquid crystal film was affixed. The natural state of the goggles was opaque, but when an AC voltage was applied, the goggles instantaneously became transparent. The transparency of the goggles could be controlled by the experimenter using the keyboard of the microcomputer, or by the subject tapping a switch located on the floorboard with his or her left foot. The foot switch relayed the subject's request to the onboard microcomputer that then provided a 0.5-s interval of vision to the subject. The microcomputer recorded the times at which each request for vision was made and the times at which the vehicle passed the beginning and ending points of each curve. The beginning and ending points were entered by the experimenter through a keyboard.

For safety, the research vehicle was equipped with an additional brake in the front passenger area for the experimenter to activate if deemed necessary. The positions of the experimenter and equipment allowed the experimenter to simultaneously activate the brake, seize control of the steering wheel, and clear the goggles.

Subjects

Subjects were recruited using pre-existing subject lists compiled from previous research efforts. Subjects were financially compensated for their time.

The first study involved 40 subjects. The second study involved 15 subjects, some of whom had participated in the first study. Since approximately 1 year separated the two studies, prior experience was not expected to bias the results of the second study.

Subjects were selected to ensure a balance of age and gender. The age and gender distribution of the subjects in the two studies is summarized in table 21. The older age group was intentionally overrepresented in the first study.

Table 21. Age and gender distribution of subjects.

Age (Years)	Study 1		Study 2	
	Male	Female	Male	Female
16 - 24	3	4	2	3
25 - 54	10	8	3	2
55 +	8	7	3	2

Test Procedure

Although there were minor differences between the two studies, the basic test procedures were the same. First, the experimenter explained the occluded vision test, and the subjects completed consent forms. Next, during an orientation period, the experimenter provided detailed instructions about the test procedure and the use of the vision occlusion apparatus, and the subjects familiarized themselves with the vehicle and apparatus. Then, when the subjects were comfortable with the vehicle and the apparatus, the occluded vision test data were collected.

The instructions to the subjects were to request visual information as many times as perceived to be absolutely necessary to guide the vehicle along the simulated path without deviating from the lane. A deviation from the path was defined as two tires running on or beyond the edges of the path that were delineated with raised pavement markers. It was emphasized that the first priority was to maintain lane integrity. If a deviation from the path occurred, the test would be halted and that segment of the course would be repeated.

During the orientation period, subjects were encouraged to take as much time as they needed to become familiar and comfortable with the vehicle, the vision occlusion apparatus, and the test course. The orientation included several stages. First, subjects became familiar with the instrumented vehicle by driving it on portions of the runway network that were not part of the delineated test course. Second, subjects drove at a speed of their own choosing through the test course without the vision occlusion apparatus. Third, subjects drove through the test course without the vision occlusion apparatus with the cruise control set at 72.5 km/h (45 mi/h), which was the speed at which data would be collected. Finally, subjects drove through the test course with the vision occlusion apparatus with the cruise control set at 72.5 km/h (45 mi/h). Subjects were allowed to repeat any of these stages as many times as they wished.

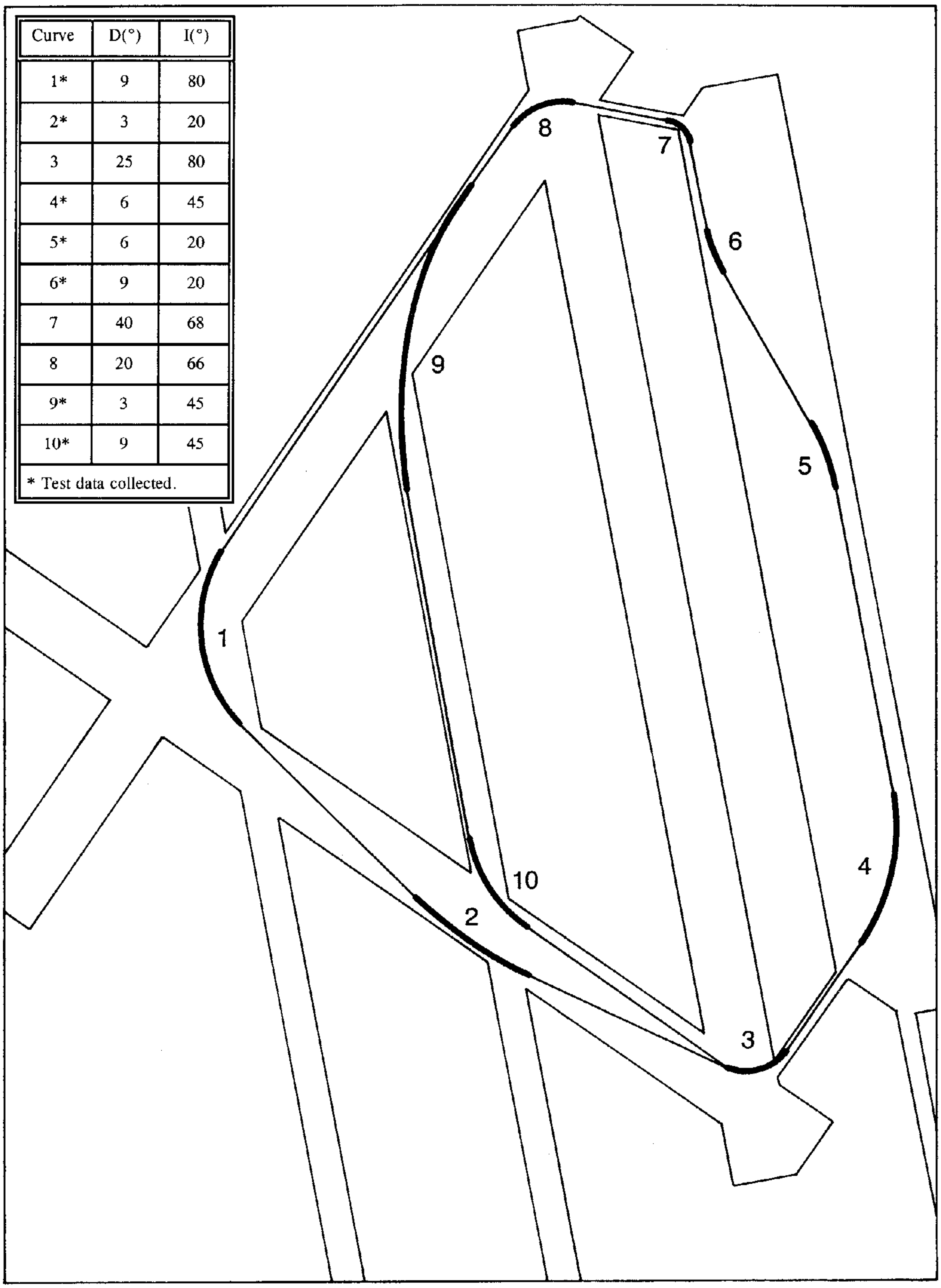
After the orientation period, data were collected. Each subject completed the test course four times, changing direction after each completion, so that each direction of curve (left and right) was presented twice. The test procedures for the two studies differed slightly due to the layout of the test courses.

Test Courses

In the first study, the test course consisted of a series of curves and connecting tangents, as shown in figure 18. None of the curves were superelevated. Subjects made an almost-continuous run through the closed-loop test course. Data were collected on only seven curves and their approach tangents, as noted in figure 18. (The data on the 9° curve with an 80° deflection angle were not used.) The remaining curves, which were included to close the loop, were sharper than the test curves. Therefore, the cruise control and vision occlusion apparatus were disengaged as the subjects drove through these curves. The starting location on the test course and direction of travel were randomized among the subjects.

In the second study, the test course consisted of a series of isolated curves with approach and departure tangents. This study was divided into two parts. The test course for each part is illustrated in figure 19. The first part involved three curves with 90° deflection angles and degrees of curvature of 6°, 9°, and 12°. The second part involved three curves with the same degrees of curvature, but with 45° deflection angles. None of the curves were superelevated. Approximately 2 months separated the conduct of the two parts of the study. The instruction, consent, orientation, and data collection process was repeated for each part. During data collection, subjects made separate runs through the series of three isolated curves and associated tangents. The starting curve and direction were randomized among the subjects. Occluded vision test data were also collected separately on another long tangent segment approximately 600 m (1,968 ft) in length.

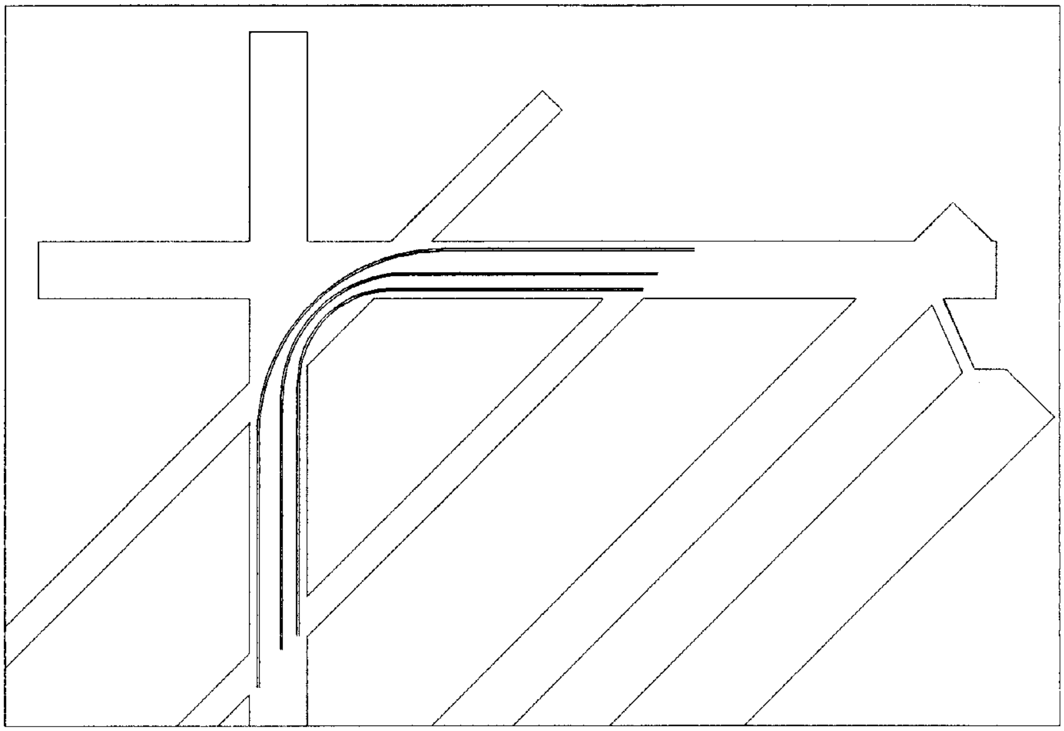
White ceramic raised pavement markers were used to delineate the test courses. The pavement markers were placed every 6.1 m (20 ft) in the tangent sections and every 3.05 m (10 ft) in the curved sections.



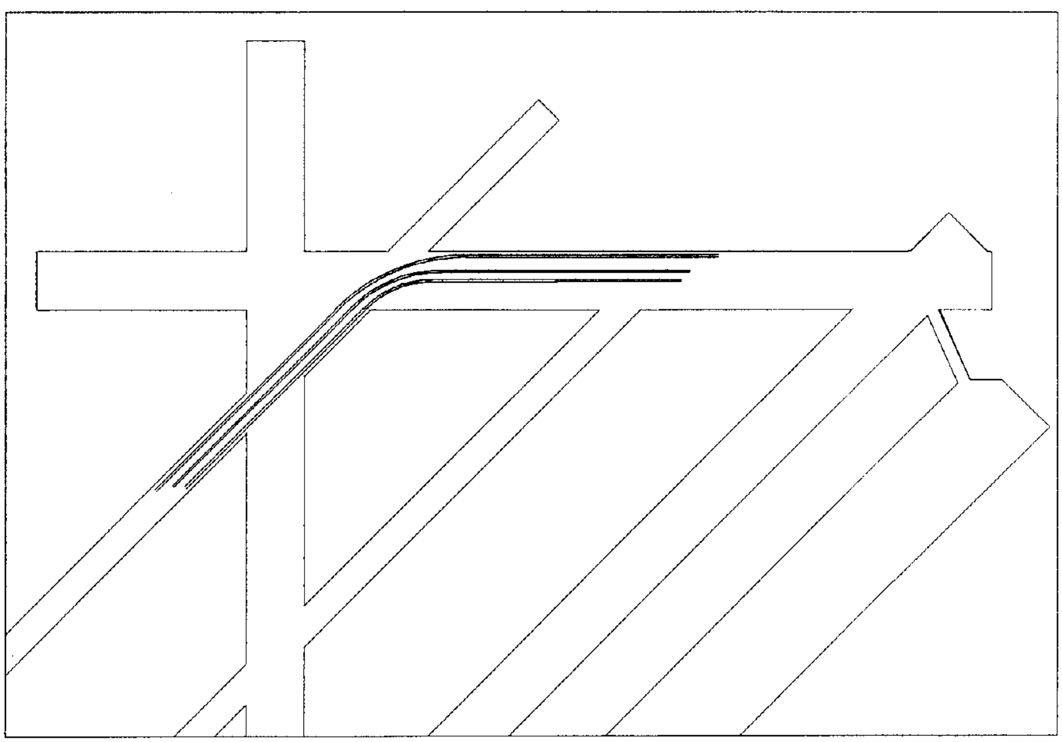
Curve	D(°)	I(°)
1*	9	80
2*	3	20
3	25	80
4*	6	45
5*	6	20
6*	9	20
7	40	68
8	20	66
9*	3	45
10*	9	45

* Test data collected.

Figure 18. Test course for occluded vision tests—study 1.



(a) Part 1 — 90° deflection angle



(b) Part 2 — 45° deflection angle

Figure 19. Test course for occluded vision tests — study 2.

OCCLUDED VISION TEST RESULTS

First, the calculation of workload values is described. Then, the statistical analysis of degree of curvature and deflection angle as predictors of driver workload on horizontal curves is discussed.

Workload Calculations

A workload observation was calculated each time a subject requested vision. These individual workload observations were plotted to study workload profiles in the vicinity of curves. Then, the individual workload observations were averaged to obtain a driver workload estimate for each subject on each curve.

Individual Workload Observations

The individual workload observations represent the workload during the time interval between requests for vision. The equation for calculating a workload observation is:

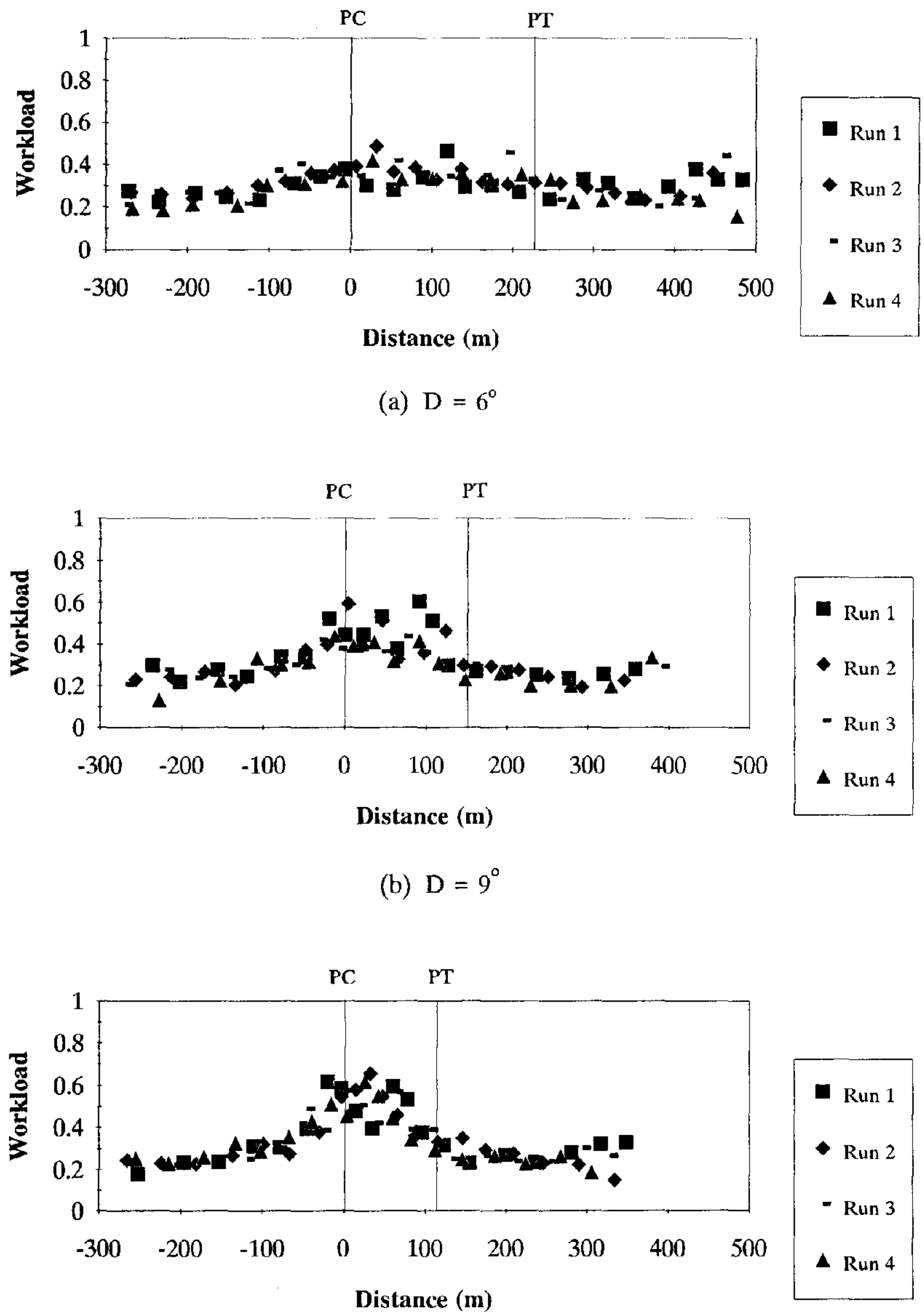
$$WL_i = \frac{0.5}{t_2 - t_1} \tag{7}$$

where: WL_i = workload over time interval i from t_1 to t_2
t_2 = clock reading at current information request (s)
t_1 = clock reading at previous information request (s)
0.5 = time increment during which subject has vision (s).

When a subject needed more information to guide the vehicle along the path without error, the time between requests was shorter. This shorter interval indicated a higher workload.

Workload Profiles

Figure 20 illustrates the workload observations for a representative subject. The values are for 6°, 9°, and 12° curves with a 45° deflection angle (from the second part of the second study). The workload observations for each of the four runs through the curves are plotted versus distance relative to the point of curvature (PC) of the curve. Therefore, negative distances represent locations on the approach tangent upstream of the PC. The plots for each curve show a similar bell-shaped pattern of workload values on the approach tangent, curve, and departure tangent. The values are low at the beginning of the approach tangent, increase toward the end of the approach tangent, peak near the PC, decrease throughout the curve, level off by the point of tangency (PT), remain almost constant through the departure tangent, and increase slightly at the end of the test course (which was marked with traffic cones). The peak workload values increase with increasing degree of curvature.



(a) D = 6°

(b) D = 9°

(c) D = 12°

Figure 20. Scatter plot of workload observations for a representative subject on 6°, 9°, and 12° curves with a 45° deflection angle.

Figures 21 and 22 show the average workload profiles for the curves with 45° and 90° deflection angles from the second study. Figure 21 for the 45°-deflection-angle curves covers the approach tangent and curve. Figure 22 for the 90°-deflection-angle curves covers only the curve. (The data for 90°-deflection-angle curves, which were collected first, did not permit the workload observations for the approach tangents to be averaged and plotted.) The workload observations for all subjects were grouped by the 30.5-m (100-ft) interval relative to the PC in which the request for vision was made. The averages of the workload observations within each interval were computed and plotted in figures 21 and 22.

The average workload profiles in figure 21 for the 45°-deflection-angle curves have the same bell-shaped pattern described in figure 20. The profiles are almost identical until approximately 75 m (246 ft) before the PC, where they begin diverging to reach different peak workload values either immediately before the PC (for the 6° curve) or after the PC (for the 9° and 12° curves). The average workload profiles in figure 22 for the 90°-deflection-angle curves are lower at the beginning of the curve and decrease less through the curve than for the 45°-deflection-angle curve with the same degree of curvature.

Unfortunately, without the average workload values for the approach tangents to the 90°-deflection-angle curves, it is impossible to determine whether figure 22 captures the peak workload values. It is hypothesized, however, that the peak workload occurred before the PC of the 90°-deflection-angle curves for essentially the same reason that it did for the 6° curve with the 45° deflection angle. It was noted in chapter 2 that the length of curve influences a driver's perception of the sharpness of curvature. A longer curve appears sharper to a driver from the approach tangent than a shorter curve with the same degree of curvature. It is hypothesized that longer curves command more of a driver's attention before entering the curve, a driver's information-processing rate before entering a longer curve is more likely to be sufficient and, therefore, workload is more likely to peak before entering the curve. It is further hypothesized that at shorter curves, drivers are more likely to underestimate the sharpness of the curve, devote less attention to the curve than they should before entering a short curve, and increase their information-processing rate after entering the curve; therefore, workload is more likely to peak after entering shorter curves with a given degree of curvature.

Driver Workload Estimates for Horizontal Curves

Two methods of calculating a driver-workload estimate for each subject at each curve were considered. The first method was to average all workload observations between the PC and PT as the measure of a subject's workload on a curve. The second method was to average only the workload observations between the PC and the midpoint of a curve. The second method produced higher workload values that were considered more representative, because they better reflected the more critical, peak workload values near the beginning of a curve.

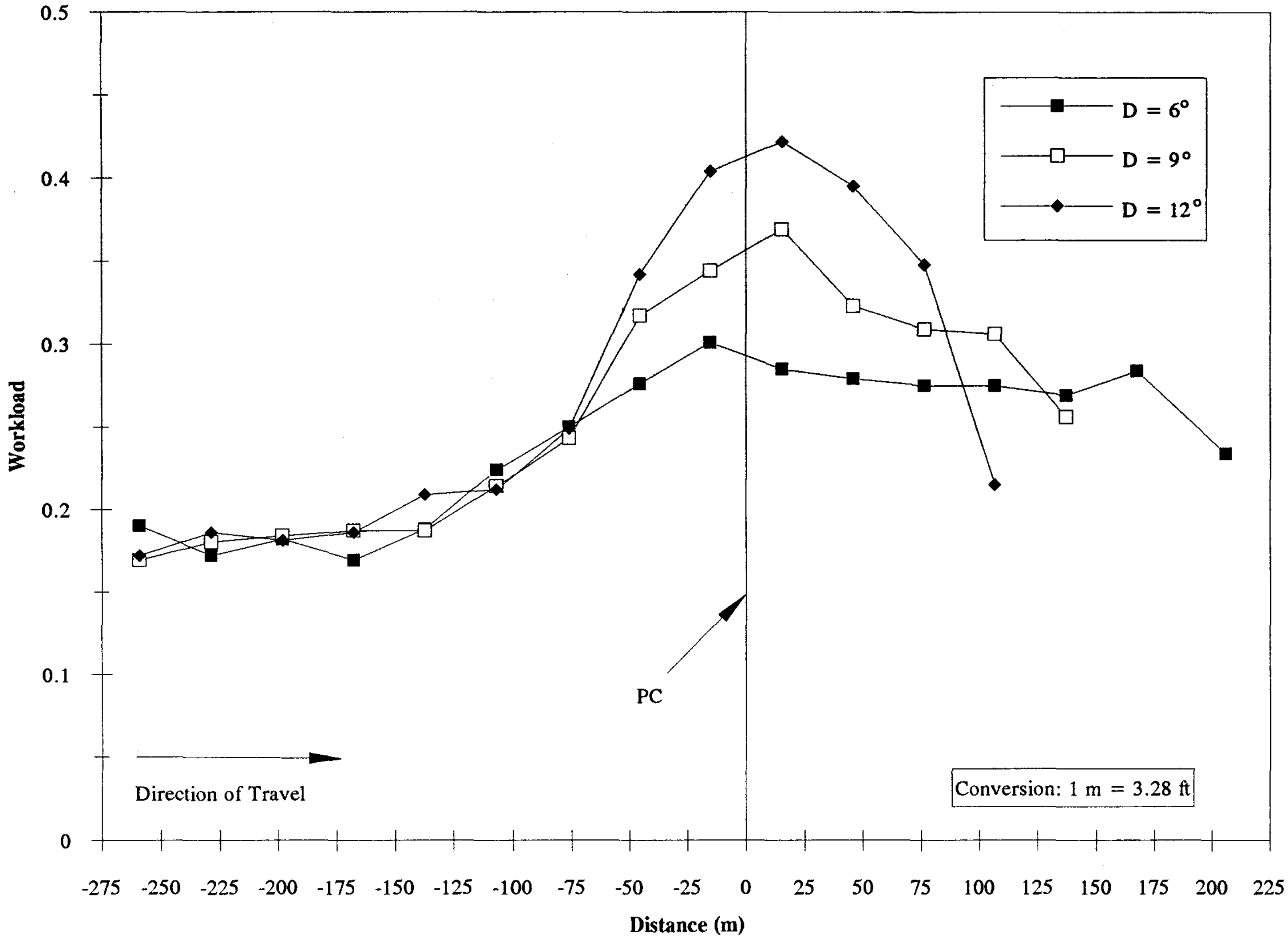


Figure 21. Average workload profile for 6°, 9°, and 12° curves with a 45° deflection angle.

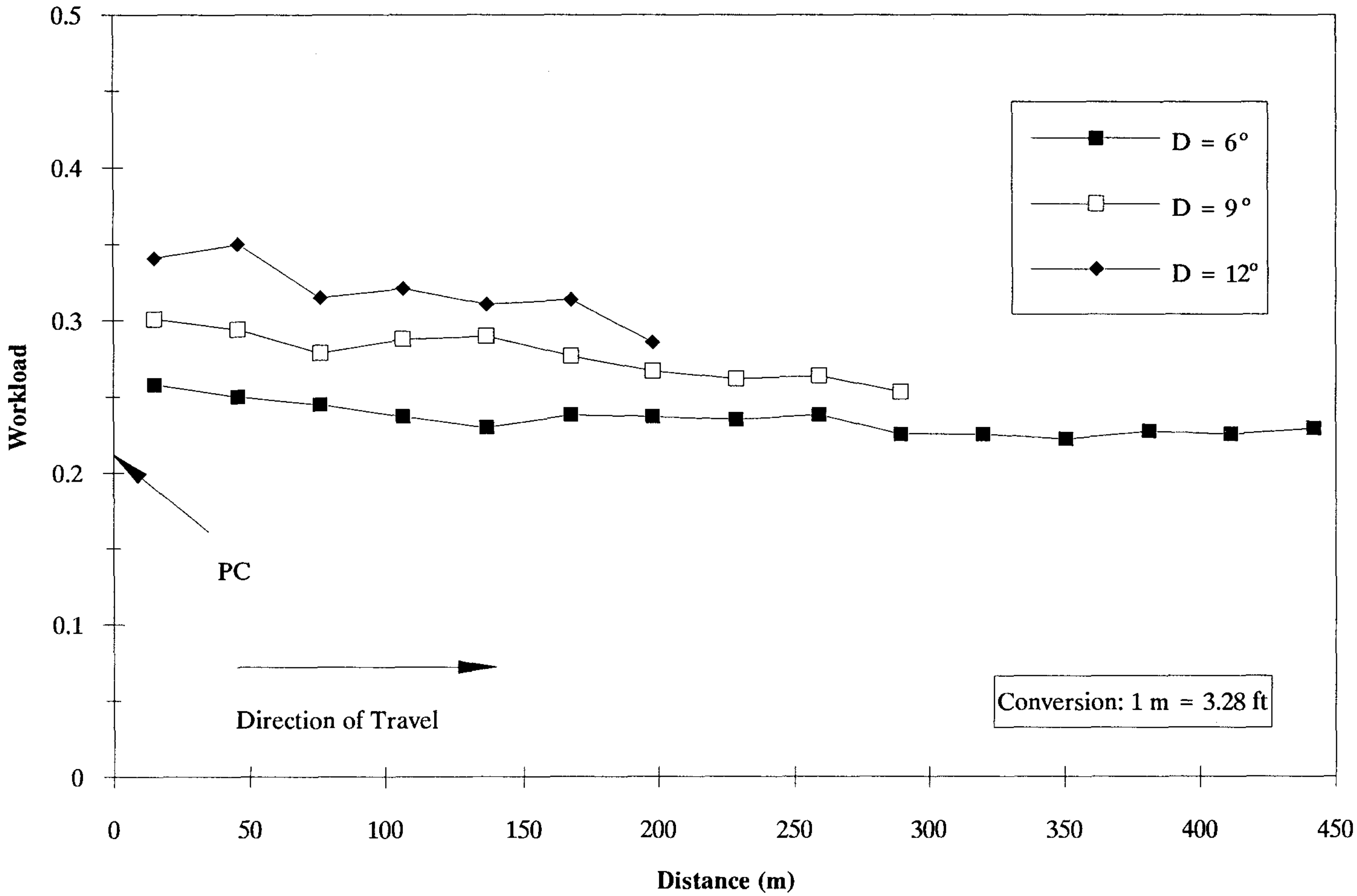


Figure 22. Average workload profile for 6°, 9°, and 12° curves with a 90° deflection angle.

The analysis of variance reported later in this chapter was performed using the estimates from both methods. The estimates from the half-curve averaging method yielded models with better statistical fit than the full-curve averaging method. Therefore, only the results from the half-curve averaging method are reported in this chapter.

Table 22 summarizes the workload estimates for each curve, averaged across all subjects. As expected, for each deflection angle, the workload value increases as degree of curvature increases. The effect of deflection angle is more complex. For the curves in the first study, the workload values on the 45°-deflection-angle curves were 2 to 10 percent higher than on the 20°-deflection-angle curve with the same degree of curvature. For the curves in the second study, the curves with workload values on the 45°-deflection-angle curves were 16 to 24 percent higher than on the corresponding 90°-deflection-angle curve. The statistical significance of these differences is discussed in more detail later in this chapter.

Table 22. Average workload values from the occluded vision test studies.

Degree of Curvature	Deflection Angle			
	20° *	45° *	45° **	90° **
3°	0.306	0.339	---	---
6°	0.346	0.352	0.279	0.241
9°	0.404	0.422	0.340	0.285
12°	---	---	0.412	0.332

* Study 1
** Study 2

Both of the occluded vision test studies measured workloads on 6° and 9° curves with a 45° deflection angle. The average workload values for the curves from the first study fell outside a 95-percent confidence interval for the averages from the second study. The differences between these values can be attributed to two factors. First, the curves in the first study were part of a closed-loop test course; whereas, the curves in the second study were independent. As a result, subjects in the first study experienced more sustained workload demands and carryover between curves than in the second study in which subjects could relax between curves. Second, the age distributions of the subjects in the two studies were different. As summarized in table 20, the older driving population was intentionally overrepresented in the first study; whereas, equal numbers of subjects were selected from the three age groups in the second study. Older drivers in the first study had significantly higher workload values than younger drivers; therefore, the overrepresentation of older drivers in the first study yielded higher workload values than the second study.

Driver Workload Estimates for Tangents

As a baseline measure of driver workload, workload data were collected on a tangent section of the test course in each study. The average driver workload on a tangent was 0.176. This value indicates that, on average, the subjects required vision only 17.6 percent of the time in order to drive without path errors on the tangent sections of the test courses.

Statistical Analysis of Driver Workload on Horizontal Curves

In order to develop a workload-profile model similar to the speed-profile model described in chapter 3, it was necessary to determine the statistical relationship between driver workload and the two independent variables (degree of curvature and deflection angle). First, for each of the two occluded vision test studies, analysis of variance was performed to test for statistically significant differences (at $\alpha = 0.05$) among the mean workload values. Second, the data from the two studies were combined in order to develop a regression equation for driver workload on horizontal curves.

Analysis of Variance

The analysis of variance for driver workload on horizontal curves included two factors (degree of curvature and deflection angle) and the interaction between those factors. The subjects were treated as replications.

For the first study, degree of curvature had three levels (3°, 6°, and 9°) and deflection angle had two levels (20° and 45°). The analysis-of-variance results indicated that there were statistically significant differences among the workload values for the three degrees of curvature (with a P-value less than 0.0001). However, neither deflection angle (with a P-value of 0.215) nor the interaction between degree of curvature and deflection angle (with a P-value of 0.834) were significant. A multiple comparison test using the studentized range distribution indicated that the difference between 3° and 6° curves was not statistically significant, but the differences between 3° and 9° curves and between 6° and 9° curves were statistically significant.

For the second study, degree of curvature had three levels (6°, 9°, and 12°) and deflection angle had two levels (45° and 90°). The analysis-of-variance results indicated that both degree of curvature (with a P-value less than 0.0001) and deflection angle (with a P-value of 0.001) were statistically significant, but the interaction was not significant (with a P-value of 0.607). A multiple comparison test indicated that the differences between the mean workload values were significant for all three pairs of degree-of-curvature levels (i.e., between 6° and 9°, 6° and 12°, and 9° and 12°). The curves with a 90° deflection angle had significantly lower mean workload values than those with a 45° deflection angle. Although this result may be valid, it is counterintuitive. It is hypothesized that the result is an anomaly and that the workload values for the 90° deflection angle curves are artificially low because the workload estimate failed to capture the peak workload before the beginning of the curve.

Combined Workload Estimates from the Two Occluded Vision Test Studies

In order to develop a regression equation for driver workload on horizontal curves based upon the workload estimates from both studies, two adjustments were made to the estimates from the first study to account for the differences identified in table 22. First, since the three age groups in the second study had an equal number of subjects, the mean workload value for each curve in the first study was recalculated to weigh equally the workload values for the three age groups. After making this adjustment, the two curves in common between the two studies (6° and 9° curves with a 45° deflection angle) were compared. The average workload values for the first study were still greater than the values for the second study by a factor of 1.26. Therefore, the second adjustment was to divide the values from the first study by a scaling factor of 1.26. The combined workload estimates based upon the data from the two occluded vision test studies are presented in table 23.

Table 23. Combined workload estimates from the two occluded vision test studies.

Degree of Curvature	Deflection Angle		
	20°	45°	90°
3°	0.241	0.270	---
6°	0.273	0.279	0.241
9°	0.322	0.340	0.285
12°	---	0.412	0.332

Regression Equation for Driver Workload on Horizontal Curves

The basic form of the regression equation for driver workload was as follows:

$$WL = \beta_0 + \beta_1 D + \beta_2 I \qquad (8)$$

where:
WL = average workload over the first half of curve
D = degree of curvature (°)
I = deflection angle (°).

The equation of this form fit to the data in table 23 had an R^2-value of 0.70 and a model P-value of 0.002. The regression parameter estimates are summarized in table 24. The parameter estimate for deflection angle was not significant at $\alpha = 0.05$; the sign suggests that workload decreases as deflection angle increases, which is hypothesized to result from underestimating the workload values for curves with a 90° deflection angle. Therefore, deflection angle was eliminated as an independent variable in the model.

Table 24. Regression analysis results for driver workload on horizontal curves versus degree of curvature and deflection angle.

	β_0	β_1	β_2
Parameter Estimate	0.216	0.016	-0.001
Standard Error	0.022	0.002	0.0003
P-Value	0.0001	0.0003	0.0534
$WL = \beta_0 + \beta_1 D + \beta_2 I$			

An evaluation of the 563-curve accident data base that is described in chapter 5 indicates that a 20° deflection angle is the 54th percentile deflection angle; 45° is the 87th percentile value; and 90° is the 98th percentile value. It was concluded that retaining the data for 90°-deflection-angle curves would underestimate driver workload for horizontal curves with more common deflection angles. Therefore, these data were removed, and the model was rerun with only the data for curves with 20° and 45° deflection angles in order to obtain more representative driver workload estimates. The resulting regression was as follows:

$$WL = 0.193 + 0.016 * D \tag{9}$$

where: WL = average workload over the first half of curve
D = degree of curvature (°).

Equation 9 has an R^2-value of 0.90 and root mean square error of 0.020. The P-value for the degree-of-curvature parameter estimate is 0.0001. The standard error of the parameter estimate is 0.0025. Figure 23 presents the equation graphically.

WORKLOAD-PROFILE MODEL

A workload-profile model was developed using the mean workload value on tangents of 0.176 and equation 9 to estimate driver workload on horizontal curves. The measure of consistency for horizontal curves was defined as the difference between the driver workload on the curve and the driver workload on the approach tangent.

The workload-profile model has been incorporated into the menu-driven, microcomputer procedure described in chapter 3 that also estimates speed profiles. The procedure requires only alignment data available on standard plan-profile sheets: degree of curvature and stationing of the beginning and end of each horizontal curve. A data entry table prompts the user for the required alignment data. Output is available in both tabular

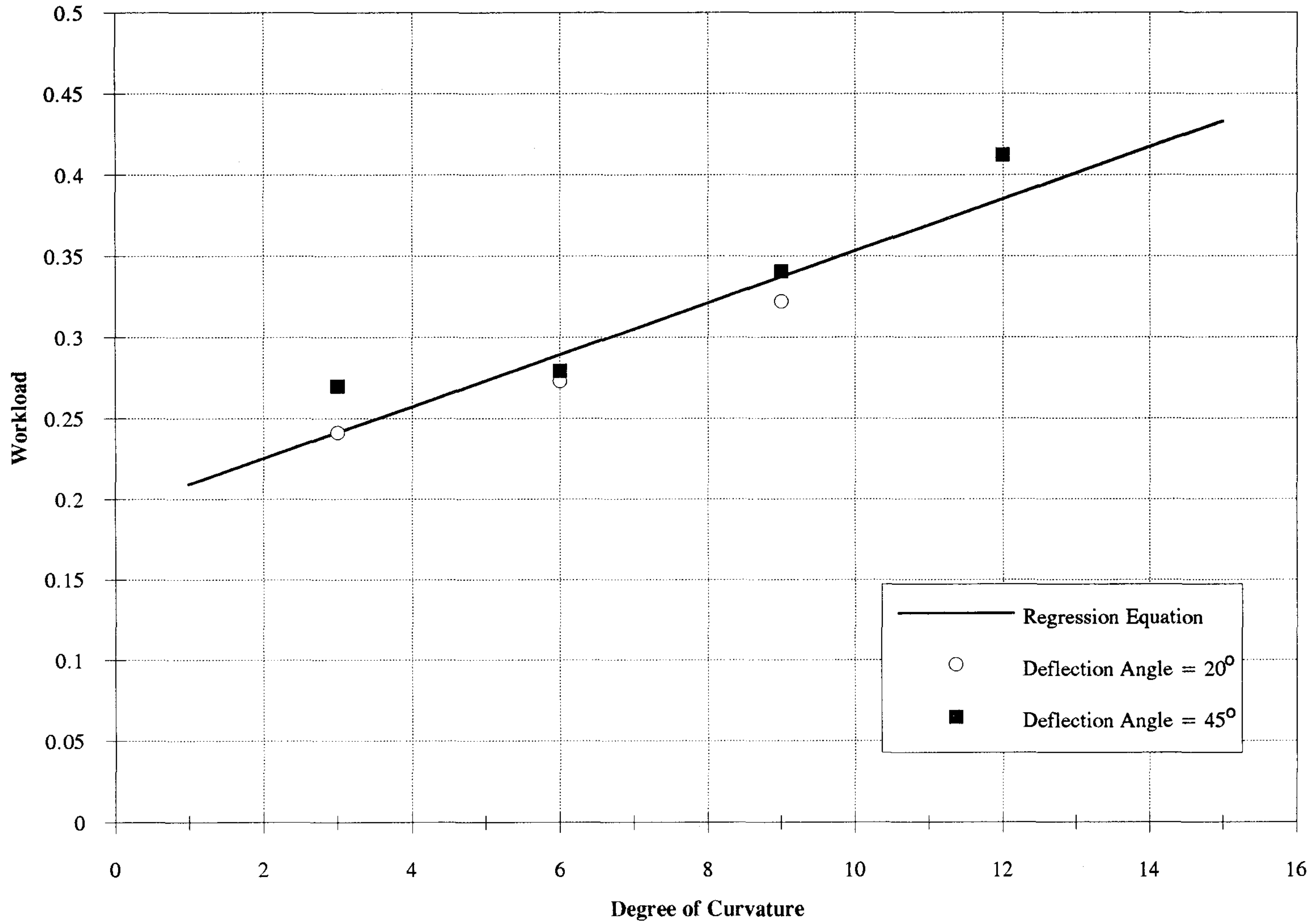


Figure 23. Driver workload versus degree of curvature.

and graphical formats. Software and users manuals for the procedure (in both English and metric units) are provided as separate reports.[59,60]

CONCLUSIONS

Several conclusions can be drawn from the occluded vision tests. The first conclusion is that occluded vision tests are a reasonable method for obtaining objective measurements of driver workload on horizontal curves. The second conclusion is that driver workload on a horizontal curve increases as degree of curvature increases. The third conclusion is that driver workload does not differ significantly on horizontal curves with 20° versus 45° deflection angles. The fourth conclusion is that driver workload peaks near the beginning of horizontal curves.

5. ACCIDENT EVALUATION

A fundamental hypothesis of this study is that changes in driver workload and operating speed, which evaluate a geometric feature within the context of the preceeding alignment, should be better predictors of accident experience than measures of the feature in isolation. The statistical analysis to evaluate this hypothesis focuses on estimated speed reduction and degree of curvature as predictors of accident rates at horizontal curves. Change in driver workload was not evaluated because it is estimated as a linear function of degree of curvature and, therefore, would produce results identical to degree of curvature.

First, previous research on accident experience at horizontal curves on rural two-lane highways is reviewed. Second, the analysis methodology used in this study is discussed. Finally, the analysis results are presented.

PREVIOUS HORIZONTAL CURVE ACCIDENT RESEARCH

The 1992 compendium on *Safety Effectiveness of Highway Design Features*, Volume II, *Alignment* lists the factors that previous research has suggested are related to accident experience on horizontal curves:[6]

- Traffic volume on the curve and traffic mix (e.g., percent trucks).
- Curve features (degree of curve, length of curve, central angle, superelevation, presence of spiral or other transition curves).
- Cross-sectional curve elements (lane width, shoulder width, shoulder type, shoulder slope).
- Roadside hazard on the curve (clear zone, sideslope, rigidity and type of obstacles).
- Stopping sight distance on curve (or on curve approach).
- Vertical alignment on horizontal curve.
- Distance to adjacent curves.
- Presence/distance from curve to the nearest intersection, driveway, bridge, etc.
- Pavement friction.
- Presence and type of traffic control devices (signs and delineation).
- Others.

Preceding page blank

Most of these factors—curve features, cross-sectional elements, roadside hazard on the curve, stopping-sight distance on the curve, and vertical alignment on the curve—relate to conditions local to curves. Several other factors—sight distance on the curve approach; distance to adjacent curves; and distance from the curve to the nearest intersection, driveway, bridge, etc.—relate to the geometric context within which the curve is located; these factors are incorporated into various consistency evaluation procedures and distinguish measures of consistency from measures of curves in isolation.

The importance of these contextual factors in accident experience at horizontal curves largely determines the incremental accident-prediction benefits of measures of consistency over measures of conditions local to curves. For example, the principal operating-speed-based measure of consistency—the reduction in 85th percentile speed from the approach tangent to the midpoint of the horizontal curve—is a function of some or all of the following variables (depending on the form of the speed-profile model that is used): degree of curvature and length of the curve in question, length of preceding tangent, degree of curvature and length of the preceding curve, and sight distance to the curve in question. The driver workload model developed by Messer, Mounce, and Brackett estimates driver workload as a function of the following factors: degree of curvature, deflection angle, sight distance to the curve, and distance from preceding geometric feature.[4] If the contextual factors explain a significant amount of the variability in accident experience left unexplained by measures of curves in isolation, and if these contextual factors are properly modeled in consistency evaluation procedures, then measures of consistency should be better predictors of accident experience on curves than measures of curves in isolation.

The remainder of this review focuses on previous research results that provide insight into the significance of these contextual factors. The discussion is divided into two sections: (1) geometic measures correlated with accident experience on curves, and (2) operating-speed-based surrogate measures for accident experience.

Geometric Measures

Four studies considered both local and contextual geometric measures in modeling accident experience at horizontal curves on rural two-lane highways. The studies include three of the most recent efforts in the United States as well as one study in New Zealand. Both the variables considered and the model forms used were of interest.

Glennon, Neuman, and Leisch conducted analysis of covariance and discriminant analysis of a data base of 3,304 sites in four States.[23] The sites were 0.97-km (0.60-mi) sections of rural two-lane highway that each contained a single horizontal curve. As a result, the data base excluded curves with short tangents and represents isolated curves. In the analysis of covariance, the dependent variable was the accident rate per million vehicle miles, and the independent variables were State, degree of curvature, length of curve, roadway width, and shoulder width. The resulting model explained only 19 percent of the variance in accident rates among sites. "State, degree of curve, and their two-way interactions with the other variables accounted for most of the explained variance."[23] Among the geometric variables, degree of curve and shoulder width had the greatest effect on accident rates, and roadway width and length of curve had a relatively small effect. The discriminant analysis considered 12 variables, which included 10 local factors and 2 contextual factors. The local factors were: degree of curvature, length of curve, maximum superelevation, ratio of superelevation at the beginning of the curve to the maximum superelevation, rate of change of superelevation, roadway width, shoulder width, shoulder type, roadside hazard rating, and pavement skid-resistance factor. The contextual factors were advance sight distance and approach alignment; the values for these variables represented composite values for both directions approaching a site. Sites were categorized as high- or low-accident sites based upon the number of accidents per segment. The five most discriminating variables were: roadside rating, shoulder width, length of curve, degree of curve, and pavement rating. Neither of the contextual factors were useful discriminating variables.

Transportation Research Board Special Report 214, *Designing Safer Roads—Practices for Resurfacing, Restoration, and Rehabilitation*, reports an accident model that was calibrated using the Glennon, Neuman, and Leisch data base.[18,23] The model had the following form:

$$A_c = AR_s\ (L)\ (V) + \beta\ (D)\ (V) \tag{10}$$

where: A_c = number of accidents on curved segment
AR_s = accident rate on straight segment
L = length of curve
V = traffic volume
β = regression parameter
D = degree of curvature.

Zegeer et al. analyzed a data base of 10,900 horizontal curves on rural two-lane highways in Washington.[75] They considered linear-regression models of two forms:

$$\log\ (A) = \beta_o + \beta_1 \log\ (V) + \beta_2 \log\ (L) + \beta_3\ (D) + \beta_i\ (\textit{other variables}) \tag{11}$$

$$\log\ (AR) = \beta_0 + \beta_1\ (D) + \beta_i\ (\textit{other variables}) \tag{12}$$

where: A = number of accidents on curve
AR = accident rate on curve
β_i = regression parameters
V = traffic volume
L = length of curve
D = degree of curvature.

The models differ with respect to the dependent variable definition (accident frequency or accident rate per unit of exposure) and the treatment of the traffic volume and length of curve variables (as independent variables or as an exposure measure in the denominator of the dependent variable). The latter form was fit using a weighted least squares procedure in which the weight was the product of average daily traffic volume and length of curve. Both local geometric and contextual factors were considered as independent variables. The local geometric variables included: maximum grade on curve, maximum superelevation, roadside recovery area, roadside rating scale, outside shoulder width, inside shoulder width, outside shoulder type, inside shoulder type, surface width, surface type, and an indicator (0, 1) for the presence of spirals. The contextual factors included: maximum distance to adjacent curve, minimum distance to adjacent curve, and terrain type. The following local geometric variables, in addition to degree of curvature, were significant: total roadway width (surface width plus outside shoulder width plus inside shoulder width), and presence of spiral. None of the contextual factors were statistically significant. Both model forms were successful at describing the relationship between accidents on curves and roadway characteristics. Because the authors needed a model form that could be used to calculate accident reduction factors, however, they selected a third model form:

$$A = [\beta_1\ (L \times V) + \beta_2\ (D \times V) + \beta_3\ (S \times V)]\ (\beta_4)^W \tag{13}$$

where:

- A = number of accidents on curve
- L = length of curve
- V = traffic volume
- D = degree of curvature
- S = presence of spiral (0 if no spiral, 1 if spiral exists)
- W = total roadway width (lane and shoulder widths combined).

Zegeer et al. performed some additional analysis of the minimum and maximum distances to adjacent curves.[75] They reported the following:

> When tested separately in various models as continuous variables, no significant effects were found at the 5 percent level for either variable. However, when the maximum distance to adjacent curve was expressed as a categorical variable for several distances (e.g., maximum tangent distance greater than 0.3 mi (0.5 km)), it was marginally significant (p = 0.06). Further analyses were not conducted, although there appears to be some evidence that tangents above a certain length may result in some increase in accidents on the curve ahead.

Matthews and Barnes developed a data base of 4,666 horizontal curves on the rural two-lane portions of a 2,000-km State highway in New Zealand to evaluate the relationship between roadway geometry and horizontal curve accidents.[76] The evaluation considered the following variables: annual average daily travel (AADT), direction of curve, radius, approach tangent length, gradient, and prior roadway curvature (i.e., the sum of the deflection angles of curves within the preceding 2 km [1.24 mi]). The statistical analysis methodology involved dividing the independent variables into five or more categories, computing the mean value of the independent variable and the mean accident rate for each category, and regressing the means. Accident rates were computed using both 100 million vehicles and million vehicle-kilometers as exposure terms. The results for both accident rate measures suggested that accident risk is much higher than average on short radius curves located at the end of long tangents, on steep downgrades, and on relatively straight sections of road (i.e., for which the prior roadway curvature was small). The authors note, "It is likely that excessive speed was a major factor contributing to the high accident rate on sharp curves as the typical geometry prior to curve entry (e.g., long tangents) would have placed little physical restriction on travel speed."[76]

In summary, previous research on geometric measures as predictors of accident experience on horizontal curves has focused primarily on measures of curves in isolation, although several measures of the geometric context of curves have also been studied. Degree of curvature is consistently the best predictor. Shoulder width or total roadway width have also been significant. Contextual variables have shown promise, but have not been strong enough to be included in the final models of the three most recent U.S. studies. Similar statistical conclusions result from the use of accident rates with respect to both million vehicle-kilometers and million vehicles entering curves.

Operating-Speed-Based Surrogate Measures

Several studies have evaluated operating-speed-based measures as predictors of accident rates at horizontal curves on rural two-lane highways. Most, however, have been limited to small data bases, and the findings have not been consistent.

Taylor et al. evaluated several operational measures: the variance of the lateral placement distribution at the midpoint of a curve; the mean, variance, and skewness of the speed distribution at the midpoint of a curve; and the deceleration rate and speed change from the beginning to the midpoint of a curve.[77] The evaluation consisted of regression analyses of accident rate versus these operational measures for a sample of nine horizontal curves. The variance of the lateral placement distribution was the only measure with a statistically significant correlation with the accident rate on curves. The speed-based measure with the best, albeit not statistically significant, correlation with accident rate was the deceleration rate from the beginning to the midpoint of the curve.

Stimpson et al. performed correlation and stepwise multiple-linear regression analyses of accident surrogates for isolated horizontal curves as well as tangent and winding alignments.[78] Their data base of 78 accidents at 20 isolated horizontal curves was too small to produce conclusive results. On tangent and winding alignments, however, measures of lateral placement were the best predictors of accident potential. They also indicated that, "The skewness index of the speed distribution appears to qualify as an acceptable indicator of hazardous operation."[78]

Datta et al. studied surrogate measures for isolated horizontal curves on two-lane rural highways.[79] Isolated meant that at least 0.4 km (0.25 mi) separated the curve from the preceding traffic event that required the driver to change speed and/or path (i.e., another curve, railroad-grade crossing, or controlled intersection). The study considered both operational measures (encroachments and speed differential) and non-operational measures (degree of curvature; grade; shoulder width; distance since last traffic event requiring the driver to adjust speed or path; superelevation error (i.e., the difference between actual and required superelevation rates); roadside slope; and type, location, and frequency of fixed objects). Several measures of speed differential were considered: from the approach to the beginning of the curve, from the approach to the midpoint of the curve, and from the beginning to the midpoint of the curve. The data base contained 25 sites in Michigan. The AADT at the sites ranged from 1,000 to 8,000 vehicles per day. The degree of curvature ranged from 5° to 24°. Grades ranged from 0 to 5 percent. The researchers analyzed the entire data base as well as subsets of sites with similar sight distance, grade, driveway density, and posted speed limits. The inside and outside lanes were analyzed both separately and in combination. Dependent variables included accident rates per million vehicles for total, opposite direction, and run-off-road accidents. Degree of curvature was the only statistically significant predictor of total accident rate at all sites. Models with R^2-values greater than 0.65 included multiple-linear regression models for: (1) outside-lane accident rate as a function of distance since last traffic event and speed differential from the beginning to the midpoint of the curve, and (2) run-off-road accident rate as a function of degree of curvature and superelevation error.

Terhune and Parker evaluated the surrogate measures identified by Datta et al. on a larger data base consisting of 78 horizontal curves in New York.[79,80] Their best-fitting models for total accidents and total road-departure accidents per million vehicles included AADT, degree of curvature, and distance to the last major traffic event in the outside lane. The regression coefficient for distance to the last major traffic event suggested that those accident rates decreased as the distance increased, which was opposite of the effect observed by Datta et al.[79] Attempts to validate these best-fitting models using a data base of 40 curves from Ohio and 41 curves from Alabama suggested that the distance to the last major traffic event was not significant and, therefore, it was eliminated. Terhune and Parker concluded that the best predictors of accident experience on rural horizontal curves were degree of curvature and AADT.

Zegeer et al. studied police accident reports for 104 fatal and 104 non-fatal accidents on horizontal curves on two-lane rural highways in North Carolina.[75] They found that "the estimated speed prior to fatal crashes was much higher than to the non-fatals" and concluded that "...speed is a definite factor, perhaps in both the occurrence of and also the severity of crashes on curves."[75]

Zegeer et al. also reexamined the Terhune and Parker data base.[75,80] They summarized the results of their operational and accident analyses as follows: "...degree of curve is clearly the geometric feature which most affects accidents and vehicle operations on horizontal curves, where sharper curves result in significantly increased rates of accidents, as well as high rates of speed reduction and vehicle encroachments."[75]

Lamm et al. analyzed 85th percentile speeds and accident rates on horizontal curves as a function of degree of curvature based upon data from 261 curved segments of rural two-lane highways in New York.[45] They concluded that linear functions of degree of curvature were sufficient for predicting 85th percentile speeds and accident rates. Their linear-regression equation for accident rate as a function of degree of curvature had an R^2-value of 0.43. They inferred a relationship between 85th percentile speed reductions and accident rate since both were a function of degree of curvature.

ANALYSIS METHODOLOGY

In order to evaluate speed reduction—estimated using the speed-profile model described in chapter 3—as a predictor of accident rates at horizontal curves, a data collection and analysis effort was conducted. The analysis methodology included collecting data for a representative sample of rural two-lane highways from three regions of the United States, using the speed-profile model to estimate speed reductions, and analyzing the statistical relationship between estimated speed reduction and accident experience on these curves. The accident evaluation must be considered a preliminary—rather than definitive—assessment of the merits of the measures of consistency, which is compatible with the scope of the subject contract.

Data Collection

Alignment and accident data were obtained from New York, Texas, and Washington for 21 roadway segments totaling 399 km (248 mi) of rural two-lane highway. These were the same highways from which speed study sites were selected based upon the controls and criteria summarized in table 10. The highways were minor arterials and collectors in level-to-rolling terrain. They had 80.5- to 88.5-km/h (50- to 55-mi/h) posted speed limits. Their cross-sections consisted of 3.05- to 3.66-m (10- to 12-ft) travel-lane widths and 0 to 2.44-m (0 to 8-ft) shoulders. No improvements had been made to alignment, cross-section, or pavement since 1987. Traffic volumes ranged from 280 to 4,500 vehicles/day, with 95 percent of the roadways having less than 3,500 vehicles/day. All horizontal curves along these sections were included in the data base unless they: (1) were within 0.8 km (0.5 mi) of a town or the end of the roadway, or (2) had a public roadway intersection within the curve.

Alignment Data

Alignment data were extracted from plan-profile sheets (for New York and Texas sites) or computerized alignment data and video logs (for Washington sites). The alignment data included degree of curvature, length of curve, approach tangent length, sight distance to the beginning of the curve, degree of curvature and length of preceding curve, travel-way width, and total pavement width.

Accident Data

Accident summaries encompassing 5 years (1987 through 1991) were obtained from each State. Police accident reports were requested for those segments of roadway used in the accident analysis. In New York, however, the police accident reports through October 1988 had been destroyed. Since these reports were necessary to determine the nature of the accidents, the analysis included only 3 years and 2 months of New York accident data.

The police reports were used to eliminate accidents that were not roadway-related or that involved vehicles other than passenger cars. Specifically, accidents were excluded from the analysis if they involved one of the following: (1) driver asleep, (2) animal in the roadway, (3) parked, turning, or passing vehicle, (4) mechanical defect in the vehicle, or (5) bicycle, motorcycle, or truck. The 235 accidents involving passenger cars that were retained in the analysis included: single-vehicle run-off-road accidents, multiple-vehicle collisions in opposing directions, and multiple-vehicle collisions in the same direction. These are the same accident types that Zegeer et al. had identified as being "overrepresented on curves when compared to tangents."[(75)]

Sample Size

The data base consists of 563 curves. Since the speed reduction entering a curve is a function of approach conditions, each lane (or direction) of a curve was treated as a separate site, resulting in a total of 1,126 curve sites (i.e., 2 directions for each of 563 curves). The data base includes 78 curves at which speed data had also been collected. Table 25 summarizes the distribution of curve sites and accidents by State.

Table 25. Number of curve sites and accidents by State.

State	Number of Curve Sites	Number of Accidents
New York	376	43
Texas	238	70
Washington	512	122
Total	1,126	235

Estimation of Speed Reduction

Speed reductions were estimated using the speed-profile model described in chapter 3. The speed reduction associated with a horizontal curve was computed as the difference between the estimated maximum 85th percentile speed along the approach tangent and estimated 85th percentile speed at the midpoint of the curve.

A key component of the speed-profile model is a regression model for estimating 85th percentile speeds on horizontal curves. Two alternative regression equations that differ somewhat in accuracy and complexity were identified in chapter 3; a simple-linear function of degree of curvature only, and a multiple-linear function of degree of curvature, length of curve, and deflection angle. The speed-profile model was run using both equations to determine whether one produced speed-reduction estimates better correlated with accident experience.

Chapter 3 also identified two forms of the speed-profile model: one that considered the effect of limited sight distance to the beginning of a horizontal curve and one that did not. Both forms were evaluated in the accident analysis to provide some additional information to help judge the best balance of accuracy and complexity in estimating operating-speed reductions.

Statistical Analysis Methods

Three characteristics of the data base influenced the selection of statistical analysis methods: the paucity of accident data (235 accidents); the large percentage of sites with no accidents (85 percent); and the unequal distribution of sites by speed reduction and degree of curvature. Figure 24 illustrates the number of curve sites by degree of curvature. Figure 25 summarizes the number of accidents by degree of curvature.

Categorical analysis techniques were considered but rejected. It was concluded that these techniques were not consistent with the level of precision or strength of conclusions that could be obtained from the size of data base that was available. Traditional linear regression was selected as a more appropriate analysis technique.

In using traditional linear regression, it was necessary to compensate for the limitations of the data base and avoid violating the assumptions of the technique. The analysis method involved grouping the curve sites into speed-reduction intervals, treating the intervals (rather than individual sites) as the observations, computing the mean accident rate and mean speed reduction for each interval, and regressing mean accident rate versus mean speed reduction. For comparison purposes, an identical method was used to group the data into degree-of-curvature intervals and regress mean accident rate versus mean degree of curvature. Several different groupings of the sites into speed-reduction (and degree-of-curvature) intervals were studied to ensure that the statistical results were not dependent upon a particular grouping of the data.

In order to satisfy the normality assumption of linear regression by invoking the Central Limit Theorem, the curve sites were grouped such that each speed-reduction interval contained at least 50 curve sites. The Central Limit Theorem states that as long as the sizes of samples drawn from a population are sufficiently large, the means of such samples are approximately normally distributed, even if the sampled population is not normal. The rule of thumb for the minimum sample size sufficiently large to invoke the Central Limit Theorem is 30.[81] The minimum sample size of 50 was selected to further compensate for the small number of accidents in the data base and the high percentage of sites with no accidents.

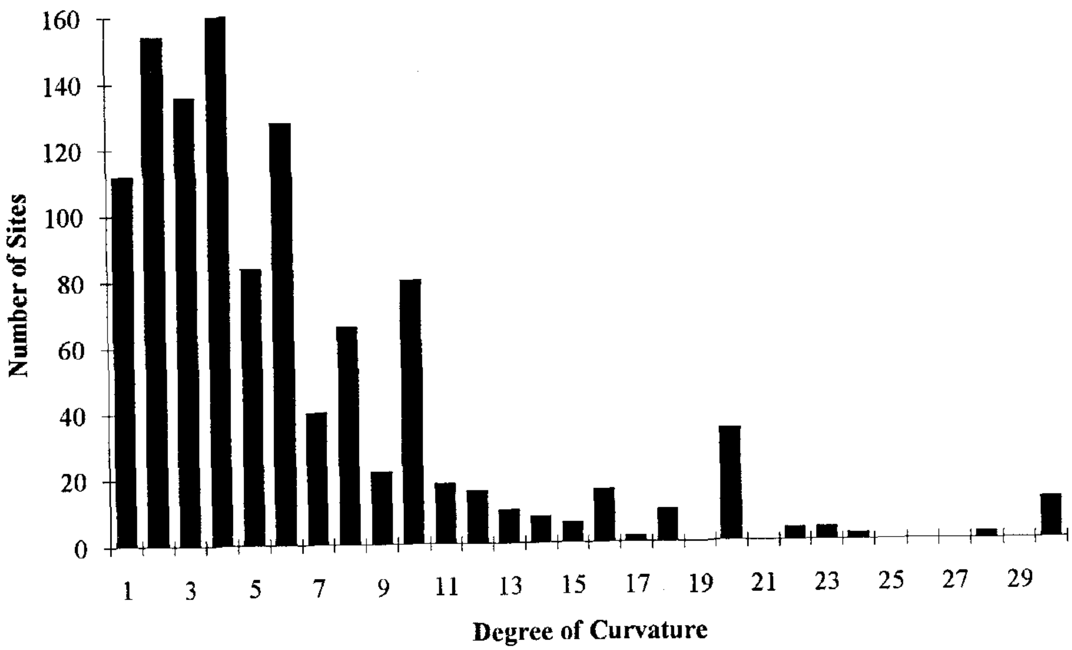


Figure 24. Number of curve sites in accident data base by degree of curvature.

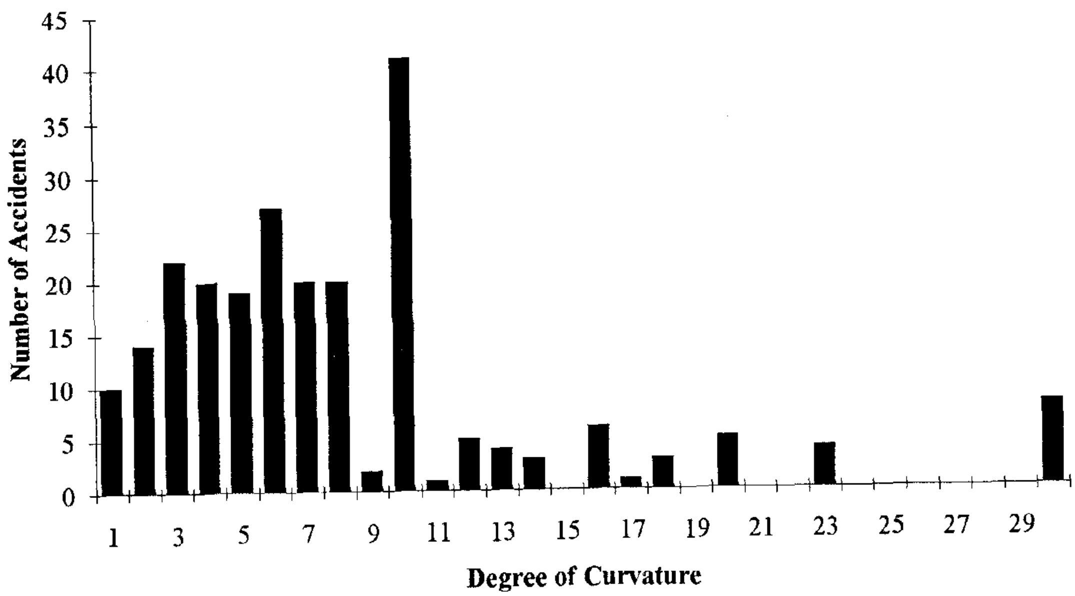


Figure 25. Number of accidents in data base by degree of curvature.

One drawback of aggregating the data into intervals is that it reduces the ability to discriminate between: (1) degree of curvature and speed-reduction estimates, (2) speed reductions estimated using different regression equations for speeds on horizontal curves, and (3) speed reductions estimated with and without considering the effect of limited sight distance to horizontal curves. This limitation is unfortunate, but the limitation associated with the analysis method is not deemed significantly more restricting than the limitation associated with the size of the data base. The analysis should be considered a preliminary assessment of the merits of measures of consistency rather than a definitive comparison of the three alternatives listed above.

Linear regression analysis was performed to evaluate the statistical relationship between the mean speed reduction (or mean degree of curvature) and the mean accident rate in each interval. The regression equation forms were as follows:

$$Mean\ AR = \beta_0 + \beta_1\ (Mean\ \Delta V85) \tag{14}$$

$$Mean\ AR = \beta_0 + \beta_1\ (Mean\ D) \tag{15}$$

where: AR = accident rate
$\Delta V85$ = difference between estimated maximum 85th percentile speed on approach tangent and estimated 85th percentile speed at the midpoint of the horizontal curve
D = degree of curvature.

Two different exposure terms were considered for the denominator of the accident rate: (1) million vehicle-kilometers traveled on the curve sites in a given interval, and (2) million vehicles entering the curve sites. Statistical analyses were performed using both accident rates. For the accident rate per million vehicles entering the curve sites, the length of curve was accounted for by breaking the sites into deflection-angle categories and developing separate regression equations for mean accident rate versus mean speed reduction (or mean degree of curvature). Almost identical conclusions were drawn using both accident rates. Since the accident rate per million vehicle-kilometers simplified the analysis and provided greater flexibility, it was preferred.

Therefore, results are presented in this report only for the model form in equation 14 with accident rate per million vehicle-kilometers. The mean accident rate in each speed-reduction (or degree-of-curvature) interval was computed as follows:

$$Mean\ AR = \frac{\sum (A_i)}{\sum ((V_i/2)\times(Y_i)\times(365\ days)\times(L_i))} \tag{16}$$

where: AR = accident rate
A_i = number of accidents at curve i during Y_i years

V_i = annual average daily traffic volume at curve i, averaged over Y_i years
Y_i = number of years for which accident data were available at curve i
L_i = length of curve i
Σ = summation over i for all curves in a given category.

Several other independent variables were also considered for incorporation in the final model: State, deflection angle, travel-way width, and total pavement width (i.e., sum of travel-way and shoulder widths). State was evaluated because of concerns about the effect of differences among the States in accident reporting procedures and in the time period of the accident data. Deflection angle was of interest because of its observed effect on 85th percentile speed and the concern that accident frequency might not be linearly related to curve length as implied by an accident rate per million vehicle-kilometers. Travel-way and total pavement width were studied because previous research has suggested that the cross-section on a curve significantly affects accident rates.[6,23,75]

These variables were analyzed by dividing the data into three levels of each variable, as summarized in table 26. Each State was treated as a separate level. Deflection angle, travel-way width, and total pavement width were divided into the smallest, middle, and largest thirds.

Table 26. Levels of State, deflection angle, travel-way width, and total pavement width.

Variable	Levels		
	Smallest Third	Middle Third	Largest Third
State	WA	NY	TX
Deflection Angle (°)	≤13	>13-27	>27
Travel-Way Width (m)	6.10-6.41	>6.41-7.02	>7.02
Total Pavement Width (m)	6.10-7.62	>7.62-8.54	>8.54
Conversion: 1 m = 3.28 ft			

Each variable was evaluated independently by fitting regression models of the form:

$$Mean\ AR = \beta_0 + \beta_1 Mean \Delta V85 + \beta_2 L_m + \beta_3 L_l + (\beta_4 L_m + \beta_5 L_l) \times Mean \Delta V85 \quad (17)$$

$$Mean\ AR = \beta_0 + \beta_1 Mean D + \beta_2 L_m + \beta_3 L_l + (\beta_4 L_m + \beta_5 L_l) \times Mean D \quad (18)$$

where: AR	=	accident rate (accidents per million vehicle-kilometers)
$\Delta V85$	=	reduction in 85th percentile speed from approach tangent to horizontal curve (km/h)
D	=	degree of curvature (°)
β_0	=	intercept for the smallest-third level
β_1	=	slope for the smallest-third level
L_m	=	indicator that equals 1 for the middle-third level and 0 otherwise
L_l	=	indicator that equals 1 for the largest-third level and 0 otherwise
β_2	=	difference between intercepts for smallest-third and middle-third levels
β_3	=	difference between intercepts for smallest-third and largest-third levels
β_4	=	difference between slopes for smallest-third and middle-third levels
β_5	=	difference between slopes for smallest-third and largest-third levels.

The statistical significance of the variable represented by the indicators L_m and L_l was evaluated by t-tests of whether β_2 through β_5 were significantly different from 0 at α = 0.05. The signs of the parameter estimates for β_2 through β_5 indicate whether the slope/intercept for the corresponding level is greater or less than the parameter estimate for the smallest-third level.

STATISTICAL ANALYSIS RESULTS

First, results are presented for the speed-reduction measure of consistency. Then, the results for degree of curvature are presented.

Mean Accident Rate Versus Mean Speed Reduction

Table 27 summarizes the regression analysis results for four alternative forms of the speed-profile model presented in chapter 3: simple-linear and multiple-linear regression equation for the 85th percentile speed on curves, with and without considering the effect of limited sight distance. Each speed-profile model form yielded different speed-reduction estimates and different groupings of sites. Therefore, only general rather than precise comparisons could be made regarding the predictive ability of the speed reductions estimated with each speed-profile model.

The speed-profile model performed better with the multiple-linear regression equation (both with and without sight distance considered) than with the simple-linear regression equation for speeds on curves. The speed-profile model also performed better in the form that considered sight distance than in the form that did not consider sight distance.

Use of the multiple-linear regression equation for 85th percentile speeds on curves is recommended because: (1) it yields slightly better predictions of 85th percentile speeds on curves than the simple-linear equation, (2) it yields speed-reduction estimates that are slightly better predictors of mean accident rates than the simple-linear equation, and (3) data requirements for the speed-profile model are identical with the simple-linear and multiple-linear regression equations.

Table 27. Regression analysis results for mean accident rate versus mean speed reduction for alternative forms of the speed-profile model.

Equation for V85 on Curve	Simple Linear		Multiple Linear	
Sight Distance Considered?	Yes	No	Yes	No
β_0	0.72	0.95	0.47	0.54
β_1	0.24	0.25	0.27	0.27
Standard Error of β_1	0.04	0.06	0.04	0.05
Root Mean Square Error	0.66	0.97	0.58	0.69
F-Statistic	31.99	17.45	42.46	28.51
P-Value	0.0013	0.0058	0.0006	0.0018
R^2-Value	0.84	0.74	0.88	0.83
Mean AR = $\beta_0 + \beta_1\ \Delta V85$				

Although the speed-profile model forms that consider sight distance have better statistical fit, including sight distance considerations in the model is not recommended for practical reasons. Data on the sight distance to horizontal curves are difficult to obtain, and requiring such data would be a significant impediment to using the speed-profile model.

Therefore, the recommended speed-profile model form uses the multiple-linear regression equation for 85th percentile speeds on curves and does not consider the effect of limited sight distance. Figure 26 includes a scatter plot of the mean accident rates and mean speed reductions as well as a line representing the fitted regression equation for this model form.

Table 28 summarizes the characteristics of the sample of curves in each speed-reduction interval for the recommended speed-profile model form. The intervals with the larger speed reductions have generally shorter mean curve lengths and larger mean deflection angles. These intervals also have smaller mean AADT's, but the range of mean AADT's is reasonably narrow. The range of mean travel-way widths and mean total pavement widths is also narrow.

The effects of State, deflection angle, travel-way width, and total pavement width were evaluated using regression models of the form in equation 17 with speed-reduction estimates using the recommended speed-profile model. The three levels of each variable are summarized in table 26.

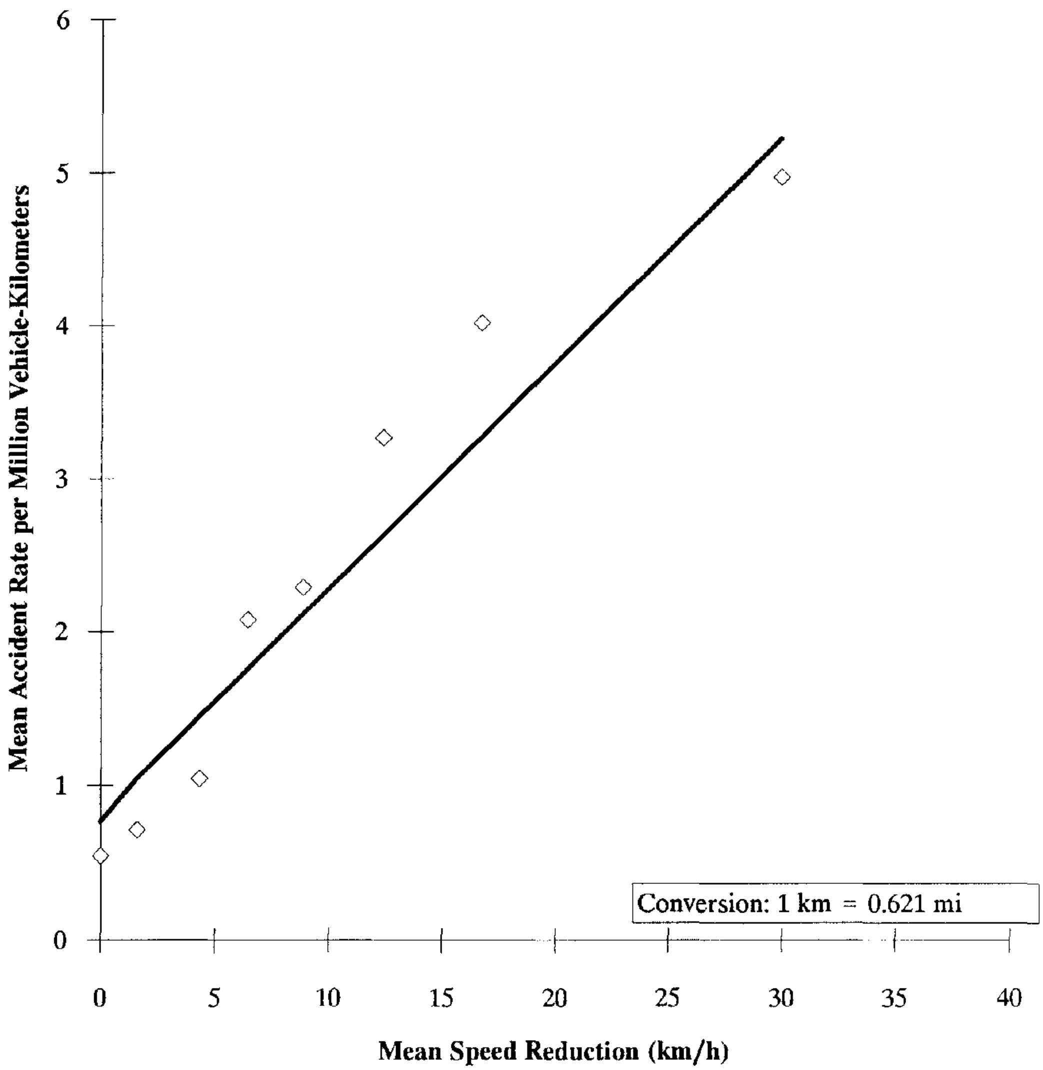


Figure 26. Mean accident rate versus mean speed reduction.

Table 28. Characteristics of the sample of curve sites in each speed-reduction interval.

Speed-Reduction Interval	Number of Curve Sites	Mean Speed Reduction (km/h)	Mean Curve Length (m)	Mean Deflection Angle (°)	Mean Travel-Way Width (m)	Mean Total Pavement Width (m)	Mean AADT (v/d)
1	484	0.00	175.6	14.5	6.88	8.55	1,567
2	160	1.61	161.5	21.4	6.69	8.42	1,595
3	53	3.22	121.9	21.7	6.79	8.62	1,369
4	118	4.83	139.3	25.9	6.73	8.38	1,457
5	92	7.30	146.2	33.5	6.89	8.73	1,848
6	64	10.31	150.5	40.6	6.62	8.20	1,378
7	58	13.21	88.3	31.8	6.92	8.21	1,239
8	97	25.23	110.4	57.6	6.76	8.35	1,255
Conversions: 1 km/h = 0.621 mi/h; 1 m = 3.28 ft							

Effect of State

Table 29 summarizes the regression analysis for the effect of State, including the parameter estimates, standard errors, and P-values for the t-test of the null hypothesis that the parameter equals zero. The results indicate that there are no statistically significant differences among the States in either the slopes or intercepts of the regression equation for mean accident rate versus mean speed reduction. These results allayed concerns about effects of possible differences among these States in accident reporting procedures and in the time periods covered by the accident data.

Effect of Deflection Angle

Table 30 summarizes the regression analysis for the effect of deflection angle, including the parameter estimates, standard errors, and P-values for the t-test of the null hypothesis that the parameter equals zero. The results indicate that there are no statistically significant differences among the deflection angle levels in either the slopes or intercepts of the regression equation for mean accident rate versus mean speed reduction. This result supports the assumption implicit in the accident rate per million vehicle-kilometers that accident frequency is linearly related to curve length.

TABLE 29. Regression analysis results for the effect of State.

Parameter	Estimate	Standard Error	P-Value
β_0	0.35	1.21	0.7768
β_1	0.28	0.18	0.1252
β_2	0.28	1.68	0.8699
β_3	0.52	1.72	0.7641
β_4	-0.10	0.24	0.6837
β_5	0.27	0.26	0.3079
Mean AR $= \beta_o + \beta_1$ Mean ΔV85 $+ \beta_2 L_m + \beta_3 L_l + (\beta_4 L_m + \beta_5 L_l)$ Mean ΔV85			

Table 30. Regression analysis results for the effect of deflection angle.

Parameter	Estimate	Standard Error	P-Value
β_0	0.78	0.80	0.3454
β_1	0.25	0.18	0.1671
β_2	0.15	1.04	0.8897
β_3	-0.41	1.04	0.6962
β_4	-0.09	0.20	0.6741
β_5	0.04	0.20	0.8558
Mean AR $= \beta_o + \beta_1$ Mean ΔV85 $+ \beta_2 L_m + \beta_3 L_l + (\beta_4 L_m + \beta_5 L_l)$ Mean ΔV85			

Effect of Travel-Way Width

Table 31 summarizes the regression analysis for the effect of travel-way width, including the parameter estimates, standard errors, and P-values for the t-test of the null hypothesis that the parameter equals zero. The results indicate that there are no statistically significant differences among the travel-way width levels in either the slopes or intercepts of the regression equation for mean accident rate versus mean speed reduction.

Table 31. Regression analysis results for the effect of travel-way width.

Parameter	Estimate	Standard Error	P-Value
β_0	0.80	1.07	0.4618
β_1	0.28	0.15	0.0718
β_2	0.45	1.51	0.7672
β_3	-0.56	1.53	0.7193
β_4	0.005	0.21	0.9799
β_5	0.02	0.22	0.9245
Mean AR $= \beta_o + \beta_1$ Mean ΔV85 $+ \beta_2 L_m + \beta_3 L_l + (\beta_4 L_m + \beta_5 L_l)$ Mean ΔV85			

Effect of Total Pavement Width

Table 32 summarizes the regression analysis for the effect of total pavement width, including the parameter estimates, standard errors, and P-values for the t-test of the null hypothesis that the parameter equals zero. The results indicate that there are no statistically significant differences among the total pavement-width levels in either the slopes or intercepts of the regression equation for mean accident rate versus mean speed reduction. Previous research has found that total pavement width significantly affects accident rates on curves.[75] The non-significance of the results in this study may be attributable to the relatively narrow range of widths at the curve sites.

Table 32. Regression analysis results for the effect of total pavement width.

Parameter	Estimate	Standard Error	P-Value
β_0	1.50	0.90	0.1108
β_1	0.37	0.12	0.0077
β_2	-1.20	1.29	0.3651
β_3	-1.09	1.22	0.4080
β_4	-0.12	0.19	0.5217
β_5	-0.17	0.18	0.3688
Mean AR $= \beta_o + \beta_1$ Mean ΔV85 $+ \beta_2 L_m + \beta_3 L_l + (\beta_4 L_m + \beta_5 L_l)$ Mean ΔV85			

Summary of Statistical Analysis of Mean Accident Rate Versus Mean Speed Reduction

The statistical analyses suggest that a simple-linear equation appropriately represents the relationship between mean accident rate and mean speed reduction when curve sites are grouped into speed-reduction intervals. Mean accident rate increases approximately linearly with mean speed reduction, and there are no obvious breakpoints in the relationship. None of the other variables tested (State, deflection angle, travel-way width, and total pavement width) were statistically significant at $\alpha = 0.05$.

Mean Accident Rate Versus Mean Degree of Curvature

The statistical analysis of mean accident rate versus mean degree of curvature started with a linear-regression analysis of the model form in equation 15. Curve sites were ordered by increasing degree of curvature and then grouped in 1° increments from 1° through 6° and in larger increments—as necessary to obtain a minimum sample size of 50—for the larger degrees of curvature.

Figure 27 is the scatter plot of mean accident rate versus mean degree of curvature for the resulting 11 intervals. The relationship appears linear, and the statistical fit is good (P-value = 0.0001, R^2-value = 0.92). Several other groupings of curve sites into different degree-of-curvature intervals were tested, and the results were similar.

Another observation from figure 27 was that the mean accident rates for the intervals through approximately 4° were similar. To test the hypothesis that the mean accident rates for the first four intervals (which included degrees of curvature from 0.25° to 4.00°) were significantly different, a linear-regression model was fit, and the slope was tested. The results indicated that the slope was not significantly different from 0, i.e., the mean accident rates were not significantly different (P-value = 0.0704).

This result suggests a breakpoint between 4° and 5°. That is, mean accident rates are approximately equal for curves with degrees of curvature $\leq$ 4°, but mean accident rates increase approximately linearly with mean degree of curvature for degrees of curvature $\geq$ 5°. This breakpoint corresponds to the breakpoint in the 85th percentile speed versus degree of curvature relationship presented in chapter 3. That is, 85th percentile speeds are approximately equal for curves with degrees of curvature $\leq$ 4°, but 85th percentile speeds decrease approximately linearly with increasing degree of curvature for curves with a degree of curvature $\geq$ 5°. This breakpoint corresponds to the maximum degree of curvature for a 100-km/h (62.1-mi/h) design speed (4.4° for a 0.08 superelevation rate). This design speed corresponds with the 97.9-km/h (60.8-mi/h) mean of the 85th percentile speeds for long tangents that represents drivers' desired speed on the class of roadway studied.

For comparison with the results for the eight speed-reduction intervals, the curve sites with degrees of curvature $\leq$ 4° were combined into a single interval to correspond approximately with the 0-km/h (0-mi/h) speed-reduction interval. The characteristics of the resulting eight degree-of-curvature intervals are summarized in table 33.

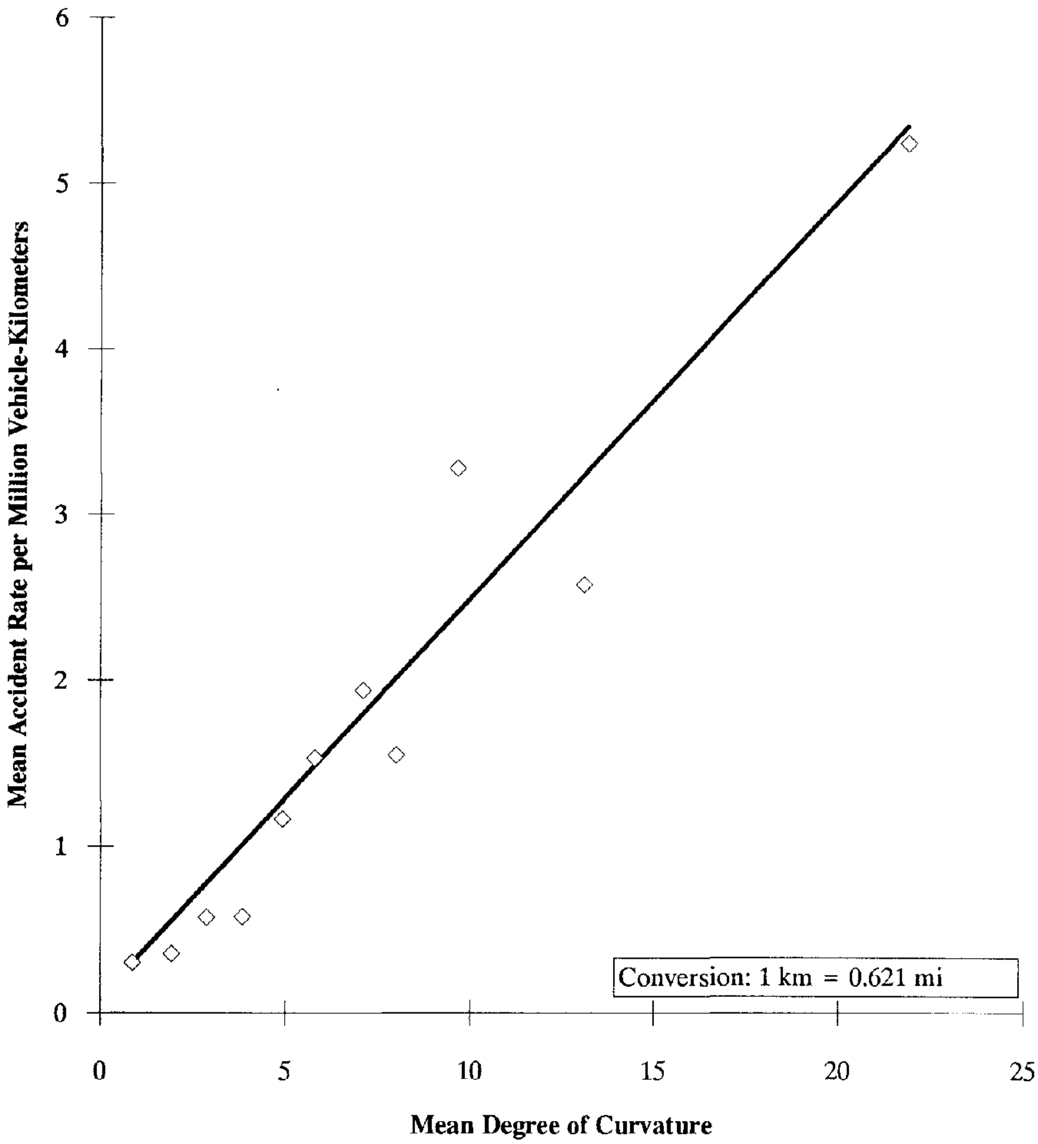


Figure 27. Mean accident rate versus mean degree of curvature for 11 intervals.

Table 33. Characteristics of the sample of curve sites in each degree-of-curvature interval.

Degree-of-Curvature (D) Interval	Number of Curve Sites	Mean D (°)	Mean Curve Length (m)	Mean Deflection Angle (°)	Mean Travel-Way Width (m)	Mean Total Pavement Width (m)	Mean AADT (v/d)
1	562	2.5	183.2	14.7	6.8	8.4	1,600
2	84	4.9	162.8	26.2	6.8	8.7	1,379
3	128	5.8	134.3	25.5	6.7	8.2	1,414
4	56	7.1	171.5	39.9	6.8	8.7	2,069
5	50	8.0	114.7	30.1	7.0	8.4	1,414
6	102	9.7	94.6	29.9	6.9	8.6	1,572
7	74	13.1	115.5	48.7	6.7	8.6	1,235
8	70	21.9	74.5	52.2	6.8	8.4	1,081
Conversion: 1 m = 3.28 ft							

Figure 28 is a scatter plot of the mean accident rate versus mean degree of curvature in the eight intervals. Table 34 summarizes the results of the regression analysis for the eight degree-of-curvature intervals. The mean accident rate increases approximately linearly with increasing mean degree of curvature, and the statistical fit is good. These results indicate that when curves whose 85th percentile speeds are approximately equal to the 85th percentile speed on long tangents (degrees of curvature $\leq 4^{\circ}$) are grouped, the statistical relationship between mean degree of curvature and mean accident rate for the resulting intervals is similar to the relationship between mean speed reduction and mean accident rate.

The effect of State, deflection angle, travel-way width, and total pavement width were evaluated using regression models of the form in equation 18. The levels of the variable summarized in table 26 were used. The only statistically significant variable was total pavement width. Table 35 summarizes the results, which indicate that the smallest total pavement-width level had a significantly steeper slope than either the middle or highest levels. This result indicates that at curves sites with smaller total pavement widths, mean accident rates increase more rapidly with increasing mean degree of curvature than at sites with larger total pavement widths. As was true with the analyses involving mean speed reduction, none of the other variables (State, deflection angle, and travel-way width) were statistically significant.

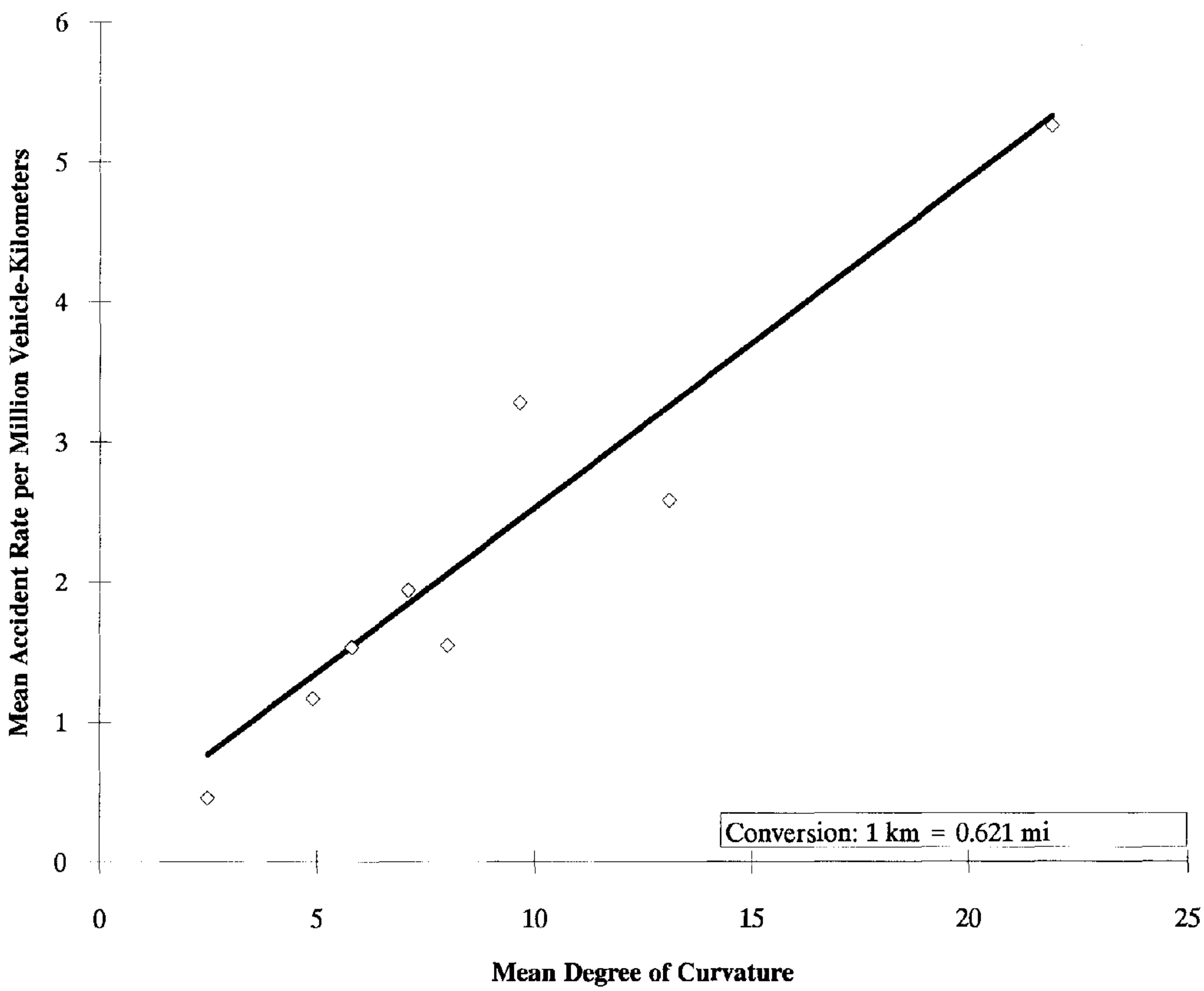


Figure 28. Mean accident rate versus mean degree of curvature for eight intervals.

Table 34. Regression analysis results for mean accident rate versus mean degree of curvature.

Regression Parameter	Estimate
β_0	0.182
β_1	0.234
Standard Error of β_1	0.0308
Root Mean Square Error	0.4937
F-Statistic	58.110
P-Value	0.0003
R^2-Value	0.91
Mean AR = $\beta_0 + \beta_1$ Mean D	

Table 35. Regression analysis results for the effect of total pavement width.

Parameter	Estimate	Standard Error	P-Value
β_0	0.6716	0.7217	0.3644
β_1	0.3765	0.0665	0.0001
β_2	-0.5227	1.0269	0.6169
β_3	-0.1907	1.0385	0.8564
β_4	-0.1724	0.0954	0.0874
β_5	-0.2435	0.0966	0.0214
Mean AR = $\beta_o + \beta_1$ Mean D + $\beta_2 L_m + \beta_3 L_1 + (\beta_4 L_m + \beta_5 L_1)$ Mean D			

SUMMARY

Due to the paucity of accident data, curve sites were grouped into intervals of speed reduction or degree of curvature. The mean accident rate and mean speed reduction (or mean degree of curvature) in each interval were computed. The interval—rather than an individual curve site—the unit of observation. The regression analysis used the mean accident rate as the dependent variable and the mean speed reduction (or mean degree of curvature) as the independent variable.

Speed reductions using four alternative forms of the speed-profile model were evaluated (i.e., using either a simple-linear or multiple-linear regression model for 85th percentile speeds on curves, with and without considering the effect of sight distance). The speed-profile model that uses the multiple-linear regression equation for 85th percentile speeds on curves and that does consider the effect of sight distance is the recommended model form.

Both mean speed reduction and mean degree of curvature were good predictors of the mean accident rate on horizontal curves (R^2-values of 0.91 and 0.83, respectively). The high R^2-values are attributable to the use of curve-site intervals as the unit of observation.

A breakpoint (between 4° and 5°) in the mean accident rate versus mean degree of curvature relationship occurred at the same place as in the 85th percentile speed versus degree of curvature relationship. This breakpoint corresponds to the maximum degree of curvature at a 100-km/h (62.1-mi/h) design speed, which, in turn, corresponds to the 97.9-km/h (60.8-mi/h) mean of the 85th percentile speed on long tangents (i.e., drivers' 85th percentile desired speed on the class of roadway studied).

It is concluded, therefore, that speed-reduction estimates from the speed-profile model described in chapter 3 help explain the relationship between degree of curvature and accident experience on horizontal curves. Horizontal curves that do not require speed reductions (generally, curves with degrees of curvature $\leq 4°$) have similar mean accident rates that are lower than the mean accident rates for curves that do require speed reductions. When curves are grouped into speed-reduction intervals, the mean accident rate on horizontal curves increases approximately linearly with mean speed reduction.

6. SUMMARY, FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

This study involved four major efforts:

- Review of U.S. and foreign geometric design consistency policy, practice, and research.
- Operating speed and highway geometry data collection and analysis, and development of a model—and a menu-driven microcomputer procedure for its use—for operating-speed-based consistency evaluation.
- Driver workload data collection and analysis using the occluded vision test method, and development of a model—and a menu-driven microcomputer procedure for its use—for driver-workload-based consistency evaluation.
- Accident and highway geometry data collection, and a preliminary evaluation of the reduction in 85th percentile operating speeds from an approach tangent to a horizontal curve as a predictor of accident experience on curves.

The scope of the data collection and model development was limited to rural two-lane highway horizontal alignments in level and rolling terrain.

The review of U.S. geometric design consistency policy and practice included a critical assessment of the AASHTO policy on rural alignment design, which is based upon the design-speed concept. An evaluation of design practices in nine States—California, Colorado, Florida, Illinois, New York, Oregon, Pennsylvania, Texas, and Washington—was conducted to identify practices related to geometric design consistency. For comparison purposes, geometric design consistency policies and practices were evaluated in seven other countries: Australia, Canada, France, Germany, Great Britain, Sweden, and Switzerland.

Operating speed and highway geometry data were collected for 138 horizontal curves and 78 of their approach tangents on 29 roadways in 5 States: New York, Oregon, Pennsylvania, Texas, and Washington. In total, 22,740 usable speed observations were obtained. Design speeds were inferred from the degree of curvature and measured superelevation rates and compared to the observed 85th percentile speeds. The roadways' inferred design speeds ranged between 40 km/h (24.8 mi/h) and 100 km/h (62.1 mi/h). Regression analysis and analysis of variance were used to identify statistically significant predictors of 85th percentile speeds on long tangents and on horizontal curves. A speed-profile model similar to those used in Switzerland and France and based upon equations developed by Lamm et al. was calibrated using a regression model for 85th percentile speeds on horizontal curves, the mean of the observed 85th percentile speeds on long tangents, and the acceleration and deceleration rates reported by Lamm et al.[45] An alternative form of the speed-profile model, which was inspired by the review of French design procedures, was also developed; it considered the effect of limited sight distance to curves on the magnitude of the speed reduction from the approach tangent to the curve. A menu-driven microcomputer procedure was developed to simplify the use of the speed-profile model.

Driver workload on horizontal curves was measured using the occluded vision test method on a series of test curves laid out on the former airport runways at the Texas A&M Proving Ground Research Facility. Vision occlusion goggles and a microcomputer-based data collection system were developed and installed in a test vehicle. Two parameters of curve geometry were evaluated: degree of curvature (including 3°, 6°, 9°, and 12°) and deflection angle (20°, 45°, and 90°). Two separate studies were conducted, and measurements were obtained for a total of 55 subjects. Workload was computed as the proportion of total driving time that drivers required vision in order to stay on a prescribed path while traveling with their speed fixed by cruise control. Regression analysis and analysis of variance were used to determine the relationship between driver workload on curves and degree of curvature and deflection angle. A regression equation for driver workload as a function of degree of curvature was developed and incorporated into a menu-driven microcomputer procedure that estimates the increase in workload from an approach tangent to a horizontal curve.

Accident and highway geometry data were collected for 1,126 curve sites (2 directions for each of 563 curves) in 3 States: New York, Texas, and Washington. Police accident reports were used to identify and locate accidents at the curve sites. Accident reports were available for 5 years (1987-1991) in Texas and Washington, and for 3 years and 2 months in New York. In total, 235 accidents—consisting of single-vehicle run-off-road accidents and multiple-vehicle same-direction and opposite-direction collisions—were identified. Regression analysis was used to perform a preliminary evaluation of the accident-prediction power of speed-reduction estimates based upon alternative forms of the speed-profile model. Due to the paucity of data, curve sites were grouped into speed-reduction intervals, and the intervals were used as the observations. The mean accident rate was regressed against the mean speed reduction. Accident rate was defined per million vehicle-kilometers traveled on the curves. Similar analyses were performed using degree of curvature. Other independent variables whose effect on mean accident rate was also analyzed, included State, deflection angle, travel-way width, and total pavement (lane plus shoulder) width.

FINDINGS

The review of U.S. and foreign geometric design consistency policy, practice, and research produced the following findings:

- U.S. rural alignment design practice closely follows AASHTO policy. That policy relies on the design-speed concept to provide operating-speed consistency.

- The design-speed concept as implemented in the United States can ensure operating-speed consistency only when the design speed exceeds the desired speed of a high percentile of drivers. U.S. alignment design policy lacks the checks necessary to avoid operating-speed inconsistencies when the design speed is less than the desired speed.

- Previous research on driver speed behavior on rural two-lane highways in the United States suggests that AASHTO policy permits the selection of design speeds less than drivers' desired speeds, particularly on collector highways in level and rolling terrain.

- The lack of uniformity in U.S. practice relative to superelevation design complicates the driving task.

- The lack of uniformity in U.S. practice reduces the credibility and effectiveness of advisory-speed signing at horizontal curves.

- In several European countries and Australia, design policy for rural two-lane highways incorporates a feedback loop to estimate 85th percentile speeds on horizontal alignments, to check for large disparities between design and 85th percentile speeds on a particular curve and between the 85th percentile speeds of successive horizontal alignment elements, and to modify the alignment design to reduce the disparities.

The evaluation of current U.S. driver speed behavior on rural two-lane highway horizontal alignments yielded the following findings:

- Drivers' desired speed on the class of rural two-lane highways studied—low-to-moderate volume (less than 3,500 v/d), minor arterial and collector highways with design speeds of 100 km/h (62.1 mi/h) or less in level and rolling terrain—was estimated using the mean of the observed 85th percentile speeds on long tangents. The estimated value is 97.9 km/h (60.8 mi/h).

- On curves whose inferred design speeds were less than drivers' desired speeds, observed 85th percentile speeds generally exceeded the respective design speeds; whereas on curves whose design speeds were greater than drivers' desired speeds, the observed 85th percentile speeds were generally less than the respective design speeds.

- On curves whose inferred design speeds were greater than drivers' desired speeds (i.e., degrees of curvature $\leq 4°$), 85th percentile speeds were not significantly different from the mean of the observed 85th percentile speeds on long tangents. On curves with degrees of curvature $> 4°$, 85th percentile speeds decreased approximately linearly as degree of curvature increased.

- All previously reported regression equation forms for 85th percentile speed on curves as a function of degree of curvature (linear, exponential, inverse, and polynomial) provided similar goodness-of-fit measures for the data collected during this study. The linear form was selected as the preferred equation due to its simplicity.

- Two independent variables, in addition to degree of curvature, were statistically significant in the regression equations for 85th percentile speeds on curves: length of curve and deflection angle. The significance of deflection angle indicates the interaction between length and degree of curvature. On curves $\leq 4°$, 85th percentile speeds increased as length increased; whereas on curves $> 4°$, 85th percentile speeds decreased as length increased. The multiple-linear equation with three independent variables—degree of curvature, length of curve, and deflection angle—explained only 1.5 percent more of the variability among 85th percentile speeds on curves than the simple-linear model with degree of curvature as the only independent variable. Since the multiple-linear equation has exactly the same data requirements as the simple-linear equation, however, it is preferred.

The evaluation of driver workload on horizontal curves using the occluded vision test method yielded the following findings:

- A linear relationship exists between driver workload, measured by occluded vision tests, and degree of curvature. Driver workload increases as degree of curvature increases.

- Driver workload on curves does not differ significantly between 20° and 45° deflection angles, which encompass a large percentage of typical deflection angles.

- Driver information demands begin increasing on approach tangents and peak near the beginning of horizontal curves.

The accident analysis provided the following preliminary evaluation of speed-reduction estimates as a predictor of accident experience on horizontal curves:

- When curve sites are grouped into categories with similar speed-reduction estimates, the mean accident rate increases approximately linearly as the mean speed reduction increases.

- Speed-reduction estimates based upon the multiple-linear regression equation for 85th percentile speeds on curves provided better prediction of mean accident rates than the simple-linear equation.

- Considering the effect of limited sight distance to curves in the speed-profile model improved the prediction of mean accident rates. Since data on sight distance to curves are difficult to obtain, however, including sight distance in the speed-profile model is not considered practical.

- For intervals of curve sites whose design speeds were greater than drivers' desired speeds (i.e., degree of curvature $\leq 4°$), mean accident rates did not differ significantly. For intervals of curve sites whose design speeds were less than drivers' desired speeds (i.e., degree of curvature $> 4°$), mean accident rates increased approximately linearly as mean degree of curvature increased.

CONCLUSIONS

The following general conclusions seem warranted based upon the findings of the study:

- Horizontal curves whose design speeds are less than drivers' desired speed on long tangents exhibit operating-speed inconsistencies that increase accident potential.

- U.S. rural alignment design policy lacks the ability to identify and address operating-speed inconsistencies.

- The speed-profile model calibrated in this study provides a mechanism for evaluating operating-speed consistency on rural alignments with design speeds $<$ 100 km/h (62.1 mi/h) that is comparable to procedures that have been adopted in Europe and Australia for similar purposes.

- The occluded vision tests conducted during this study produced reasonable driver workload measurements.

RECOMMENDATIONS

The following recommendations are made based upon the findings and conclusions of this study:

- AASHTO should review recent data on the distribution of today's drivers' desired speeds on rural highways, collect additional data as necessary, and consider revisions to its recommended minimum design speeds that represent, in AASHTO's own words, a "high-percentile value in this speed distribution." The exact percentile should be the subject of technical debate beyond the scope of this study. International practice is to use the 85th percentile, but a higher percentile might be considered. The decision must be made within the context of the overall design process with respect to where margins of safety are introduced, i.e., whether the margin of safety is introduced in the design speed and/or in other design values (e.g., the maximum side-friction factor for horizontal curve design).

- AASHTO should consider incorporating a feedback loop that identifies and addresses alignment inconsistencies in the alignment design process for rural highways with design speeds less than drivers' desired speeds. The speed-profile model presented herein provides a basis for this feedback loop that reflects international practice and current U.S. driver speed behavior.

- FHWA should consider followup research to further develop and validate the speed-profile model documented herein. Data should be collected to validate the assumed rates and locations relative to curves at which acceleration and deceleration actually occur. The follow-up research should also check the reasonableness of the model for alignment conditions and geographic regions of the United States different from those represented by the data collected during this study.

- AASHTO should consider changes to its policies on superelevation design, including adopting a nationwide maximum superelevation rate and revising the method for selecting the superelevation rate for a particular curve. Two alternatives that should be considered for the latter method would be to: (1) select the superelevation rate for a curve based upon estimated 85th percentile speeds approaching the curve, or (2) specify a unique superelevation rate for each degree of curvature.

- A new curve information system should be developed or the existing system should be modified to improve the accuracy, uniformity, and effectiveness of the available information upon which drivers select an appropriate speed and path through a horizontal curve. As a basis for the information system development/ modification, additional research should be conducted to clarify how drivers judge horizontal curvature and select the appropriate speed and path through a curve.

- FHWA should consider additional research to further develop the driver workload concepts and occluded vision test measurement methods developed during this study. These concepts and methods hold considerable promise for providing valuable fundamental knowledge on drivers' information requirements and performance of the guidance task, and for serving as a generic basis for measurement of the magnitude of operational and safety problems experienced by drivers.

REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*. Washington, DC: American Association of State Highway and Transportation Officials, 1990, with an addendum on "Interim Selected Metric Values for Geometric Design," 1993.

2. C. J. Messer, J. M. Mounce, and R. Q. Brackett. *Highway Geometric Design Consistency Related to Driver Expectancy*, Vol. I, *Executive Summary*. Report No. FHWA/RD-81/035. Washington, DC: Federal Highway Administration, 1981.

3. C. J. Messer, J. M. Mounce, and R. Q. Brackett. *Highway Geometric Design Consistency Related to Driver Expectancy*, Vol. II, *Research Report*. Report No. FHWA/RD-81/036. Washington, DC: Federal Highway Administration, 1981.

4. C. J. Messer, J. M. Mounce, and R. Q. Brackett. *Highway Geometric Design Consistency Related to Driver Expectancy*, Vol. III, *Methodology for Evaluating Geometric Design Consistency*. Report No. FHWA/RD-81/037. Washington, DC: Federal Highway Administration, 1981.

5. C. J. Messer, J. M. Mounce, and R. Q. Brackett. *Highway Geometric Design Consistency Related to Driver Expectancy*, Vol. IV, *Appendices*. Report No. FHWA/RD-81/038. Washington, DC: Federal Highway Administration, 1981.

6. C. V. Zegeer, J. M. Twomey, M. L. Heckman, and J. C. Hayward. *Safety Effectiveness of Highway Design Features,* Volume II, *Alignment*. Report No. FHWA-RD-91-045. Washington, DC: Federal Highway Administration, 1992.

7. N. J. Rowan, D. L. Woods, V. G. Stover, D. A. Anderson, and J. H. Dozier. *Safety Design and Operational Practices for Streets and Highways*. Technology Sharing Report No. 80-228. Washington, DC: Federal Highway Administration, 1980.

8. H. Lunenfeld and G. J. Alexander. *A Users' Guide to Positive Guidance*. 3rd Edition. Report No. FHWA-SA-90-017. Washington, DC: Federal Highway Administration, 1990.

9. J. C. Glennon and D. W. Harwood. "Highway Design Consistency and Systematic Design Related to Highway Safety." *Transportation Research Record* 681, 1978, pp. 77-88.

10. F. W. Cron. "Highway Design for Motor Vehicles–A Historical Review. Part 5: The Dynamics of Highway Curvature." *Public Roads*, Vol. 39, No. 4, March 1976, pp. 163-171.

11. F. W. Cron. "Highway Design for Motor Vehicles–A Historical Review. Part 6: Development of a Rational System of Geometric Design." *Public Roads*, Vol. 40, No. 1, June 1976, pp. 9-18.

12. M. C. Good. *Road Curve Geometry and Driver Behaviour.* ARRB Special Report No. 15. Victoria, Australia: Australian Road Research Board, 1978.

13. J. McLean. "An Alternative to the Design Speed Concept for Low Speed Alignment Design." *Transportation Research Record 702*, 1979, pp. 55-63.

14. K. Fitzpatrick and K. Kahl. *A Historical and Literature Review of Horizontal Curve Design*. Report No. TX-92/1949-1. Austin, TX: Texas Department of Transportation, 1992.

15. D. R. Merritt. "Safe Speeds on Curves: A Historical Perspective of the Ball Bank Indicator." *ITE Journal*, Vol. 58, No. 9, September 1988, pp. 15-19.

16. W. R. Young. "Road Design for Uniform Speed." *Engineering News Record*, Vol. 105, p. 539.

17. J. Barnett. "Safe Side Friction Factors and Superelevation Design." *Proceedings of the Highway Research Board*, Vol. 16, 1936, pp. 69-80.

18. *Designing Safer Roads: Practices for Resurfacing, Restoration, and Rehabilitation.* Special Report 214. Washington, DC: Transportation Research Board, 1987.

19. M. A. Chowdhury, D. L. Warren, and H. Bissell. "Analysis of Advisory Speed Setting Criteria." *Public Roads*, Vol. 55, No. 3, December 1991, pp. 65-71.

20. J. R. McLean. *Speeds, Friction Factors, and Alignment Design Standards*. Research Report ARR 154. Victoria, Australia: Australian Road Research Board, 1988.

21. J. C. Hayward. "Highway Alignment and Superelevation: Some Design-Speed Misconceptions." *Transportation Research Record 757*, 1980, pp. 22-25.

22. M. T. Pietrucha, K. S. Opiela, R. L. Knoblauch, and K. L. Crigler. *Motorist Compliance with Standard Traffic Control Devices*. Report No. FHWA-RD-89-103. Washington, DC: Federal Highway Administration, 1989.

23. J. C. Glennon, T. R. Neuman, and J. E. Leisch. *Safety and Operational Considerations for Design of Rural Highway Curves*. Report No. FHWA/RD-86/035. Washington, DC: Federal Highway Administration, 1985.

24. *Richtlinien für die Anlage von Straßen (RAS); Teil: Linienführung (RAS-L); Abschnitt 1: Element der Linienfürung (RAS-L-1).* Cologne, Germany: Forschungsgesellschaft für Strassen- und Verkehrswesen Arbietsgruppe Strasenentwurf, 1984.

25. R. Lamm and J. G. Cargin. Translation of the "Guidelines for the Design of Roads (RAS-L-1)," Edition 1984, Federal Republic of Germany and "The Swiss Norm SN 640080a, Highway Design, Fundamentals, Speed as a Design Element" as discussed by K. Dietrich, M. Rotach, and E. Boppart in *Road Design*, Zurich, Switzerland: ETH, Institute of Traffic Planning and Transport Technique, Edition 1983. May 1985.

26. *Geschwindigkeit als Projektierungselement*. Swiss Norm 640 080b. Zurich, Switzerland: Vereingung Schweizerischer Strassenfachleute, 1992.

27. E. Boppart, H. P. Lindenmann, and B. Ranft. *Geschwindigkeit in Steigungen und Gefällen*. Zurich, Switzerland: Eidgenössische Technische Hochschule, Institut für Verkehrsplanung und Transporttechnik, October 1979.

28. H. P. Lindemann and B. Ranft. *Geschwindigkeit in Kurven*. Zurich, Switzerland: Eidgenössische Technische Hochschule, Institut für Verkehrsplanung und Transporttechnik, 1978.

29. *Instruction sur les Conditions Techniques D'Aménagement des Routes Nationales*. Paris, France: Ministère de L'Èquipement et du Logement, Direction des Routes et de la Circulation Routière, 1975.

30. *Amenagement des Routes Principales en dehors des Agglomerations: Recommandations Techniques pour la Conception Génèrale et la Géomètrie de la Route*. DRAFT. Paris, France: Service d'Etudes Techniques des Routes et Autoroute, 1991.

31. J.-M. Gambard and G. Louah. *Vitesses Practiqués et Géométrie de la Route*. Paris, France: Service d'Etudes Techniques des Routes et Autoroute, 1986.

32. *Standard Specifications for Geometric Design of Rural Roads*. English Translation of *Trafikleder på landsbygd*. Borlänge, Sweden: Swedish National Road Administration (Vägverket), 1986.

33. A. Brodin and A. Carlsson. *The VTI Traffic Simulation Model: A Description of the Model and Programme System*. VTI Meddelande 321A. Linköping, Sweden: Swedish Road and Traffic Research Institute (Väg-och Trafik-Institutet), 1986.

34. *Road Layout and Geometry: Highway Link Design*. Departmental Standard TD 9/81. London, England: Department of Transport, Reprinted 1986.

35. *Highway Link Design*. Departmental Advice Note TA 43/84. London, England: Department of Transport, 1984.

36. D. Simpson and J. A. Kerman. "The Research and Development Background to 'Highway Link Design'." *Traffic Engineering & Control*, September 1982, pp. 414-421.

37. D. Simpson. "System Development and Monitoring Systems." In *Current Issues in Highway Design*. Proceedings of the Conference on Highway Design–Today and Tomorrow, Institution of Civil Engineers, London, England, February 6, 1985.

38. J. R. McLean. "Driver Behaviour on Curves–A Review." *Proceedings of the Australian Road Research Board*, Vol. 7, Part 5, 1974, pp. 129-147.

39. J. R. McLean. "Driver Speed Behaviour and Rural Road Alignment Design." *Traffic Engineering & Control*, Vol. 22, No. 4, 1981, pp. 208-211.

40. J. R. McLean. *Speeds on Curves: Side Friction Factor Considerations*. Australian Road Research Board Research Report No. 126. Victoria, Australia: Australian Road Research Board, 1983.

41. *Rural Road Design: Guide to the Geometric Design of Rural Roads*. Haymarket, New South Wales, Australia: AUSTROADS, 1989.

42. J. R. McLean. "The Principles of Geometric Road Design." 21st AARB Regional Symposium, Darwin, Northern Territory, Australia, 1984, pp. 1-21.

43. *Manual of Geometric Design Standards for Canadian Roads*. Ottawa, Ontario: Transportation Association of Canada, 1986.

44. J. E. Leisch and J. P. Leisch. "New Concepts in Design-Speed Application." *Transportation Research Record* 631, 1977, pp. 5-14.

45. R. Lamm, E. M. Choueiri, J. C. Hayward, and A. Paluri. "Possible Design Procedure to Promote Design Consistency in Highway Geometric Design on Two-Lane Rural Roads." *Transportation Research Record* 1195, 1988, pp. 111-121.

46. *A Policy on Geometric Design of Rural Highways*. Washington, DC: American Association of State Highway Officials, 1965.

47. *A Policy on Geometric Design of Urban Highways and Arterial Streets*. Washington, DC: American Association of State Highway Officials, 1973.

48. R. Lamm and E. M. Choueiri. *A Design Procedure to Determine Critical Dissimilarities in Horizontal Alignment and Enhance Traffic Safety by Appropriate Low-Cost or High-Cost Projects*. Washington, DC: National Science Foundation, 1987.

49. E. M. Choueiri and R. Lamm. *Operating Speeds and Accident Rates on Two-Lane Rural Highway Curved Sections—Investigations about Consistency and Inconsistency in Horizontal Alignment.* Albany, NY: State University of New York Research Foundation, 1987.

50. R. Lamm, E. M. Choueiri, and J. C. Hayward. "The Tangent as an Independent Design Element." *Transportation Research Record* 1195, 1988, pp. 123-131.

51. R. Lamm and E. M. Choueiri. "Recommendations for Evaluating Horizontal Design Consistency Based on Investigations in the State of New York." *Transportation Research Record* 1122, 1987, pp. 68-78.

52. A. Taragin. "Driver Performance on Horizontal Curves." *Public Roads.* Vol. 28, No. 2, June 1954, pp. 21-39.

53. J. Emmerson. "A Note on Speed-Road Curvature Relationships." *Traffic Engineering & Control,* November 1970, p. 369.

54. R. Lamm, A. K. Guenther, and E. M. Choueiri. "Safety Module for Highway Design: Applied Manually or Using CAD." Paper Presented at the 71st Annual Meeting of the Transportation Research Board. Washington, DC. January 1992.

55. G. Kanellaidis, J. Golias, and S. Efstathiadis. "Drivers' Speed Behaviour on Rural Road Curves." *Traffic Engineering & Control,* Vol. 31, No. 7/8, 1990, pp. 414-415.

56. C. H. Lee. "A Study into Driver-Speed Behaviour on a Curve by Using Continuous Speed Measurement Method." *Proceedings of the Australian Road Research Board,* Vol. 14, Part 4, September 1988, pp. 37-46.

57. G. Mintsis. "Speed Distributions of Road Curves." *Traffic Engineering & Control,* January 1988, pp. 21-27.

58. *SAS/STAT® User's Guide.* Version 6. Fourth Edition. Cary, NC: SAS Institute, Inc., 1990.

59. K. S. Rao, H. Oh, and R. Krammes. *Users Manual for the Highway Geometric Design Consistency Program (English Units).* Report No. FHWA-RD-94-038. Washington, DC: Federal Highway Administration, 1994.

60. K. S. Rao, H. Oh, and R. Krammes. *Users Manual for the Highway Geometric Design Consistency Program (Metric Units).* Report No. FHWA-RD-94-039. Washington, DC: Federal Highway Administration, 1994.

61. D. Shinar, E. D. McDowell, and T. H. Rockwell. "Eye Movements in Curve Negotiation." *Human Factors,* Vol. 19, No. 1, 1977, pp. 63-71.

62. R. R. Mourant, T. H. Rockwell, and N. J. Rackoff. "Drivers' Eye Movements and Visual Workload." *Highway Research Record* 292, 1969, pp. 1-10.

63. R. A. Krammes and S. W. Glascock. "Geometric Inconsistencies and Accident Experience on Two-Lane Rural Highways." *Transportation Research Record* 1356, 1992, pp. 1-10.

64. M. D. Wooldridge. *Design Consistency and Driver Error as Reflected by Driver Workload and Accident Rates*. Master of Science Thesis. College Station, TX: Texas A&M University, Civil Engineering Department, 1991.

65. J. W. Senders. "The Estimation of Operator Workload in Complex Systems." *Systems Psychology*, 1970, pp. 207-216.

66. W. B. Gartner and M. R. Murphy. *Pilot Workload and Fatigue: A Critical Survey of Concepts and Assessment Techniques*. Report No. NASA-TN-D-8365. Washington, DC: National Aeronautics and Space Administration, 1976.

67. W. B. Knowles. "Operator Loading Tasks." *Selected Readings in Human Factors*, 1990, pp. 205-206.

68. R. C. Willeges and W. W. Wierwille. "Behavioral Measures of Aircrew Mental Workload." *Human Factors*, Vol. 21, No. 5, 1979, pp. 549-574.

69. H. Godthelp. "Vehicle Control During Curve Driving." *Human Factors*, Vol. 28, No. 2, 1986, pp. 211-221.

70. E. Farber and V. Gallagher. "Attentional Demand as a Measure of the Influence of Visibility Conditions on Driving Task Difficulty." *Highway Research Record* 414, 1972, pp. 1-5.

71. D. Shinar, E. D. McDowell, N. J. Rackoff, and T. H. Rockwell. "Field Dependence and Driver Visual Search Behavior." *Human Factors*, Vol. 20, No. 5, 1978, pp. 553-559.

72. T. G. Hicks and W. W. Wierwille. "Comparison of Five Mental Workload Assessment Procedures in a Moving-Base Driving Simulator." *Human Factors*, Vol. 21, No. 2, 1979, pp. 129-143.

73. H. Godthelp, P. Milgram, and G. Blaauw. "The Development of a Time-Related Measure to Describe Driving Strategy." *Human Factors*, Vol. 26, No. 3, 1984, pp. 257-268.

74. R. Van Der Horst and H. Godthelp. "Measuring Road User Behavior with an Instrumented Car and an Outside-the-Vehicle Video Observation Technique." *Transportation Research Record* 1213, 1989, pp. 72-81.

75. C. V. Zegeer, R. Stewart, D. Reinfurt, F. Council, T. Neuman, E. Hamilton, T. Miller, and W. Hunter. *Cost-Effective Geometric Improvements for Safety Upgrading of Horizontal Curves*. Report No. FHWA-RD-90-021. Washington, DC: Federal Highway Administration, 1991.

76. L. R. Matthews and J. W. Barnes. "Relation Between Road Environment and Curve Accidents." *Proceedings*, 14th Australian Road Research Board Conference, Vol. 14, Part 4, 1988, pp. 105-119.

77. J. I. Taylor, H. W. McGee, E. L. Seguin, and R. S. Hostetter. *Roadway Delineation Systems*. NCHRP Report 130. Washington, DC: Highway Research Board, 1972.

78. W. A. Stimpson, H. W. McGee, W. K. Kittelson, and R. H. Ruddy. *Field Evaluation of Selected Delineation Treatments for Two-Lane Rural Highways*. Report No. FHWA-RD-77-118. Washington, DC: Federal Highway Administration, 1978.

79. T. K. Datta, D. D. Perkins, J. I. Taylor, and H. T. Thompson. *Accident Surrogates for Use in Analyzing Highway Safety Hazards*. Volume 2: *Technical Report*. Report No. FHWA/RD-82/104. Washington, DC: Federal Highway Administration, 1983.

80. K. W. Terhune and M. R. Parker. *An Evaluation of Accident Surrogates for Safety Analysis of Rural Highways*. Volume 2: *Technical Report*. Report No. FHWA/RD-86/128. Washington, DC: Federal Highway Administration, 1986.

81. J. L. Devore. *Probability and Statistics for Engineering and the Sciences*. Third Edition. Pacific Grove, CA: Brooks/Cole Publishing Company, 1991.