Speed Prediction for Two-Lane Rural Highways

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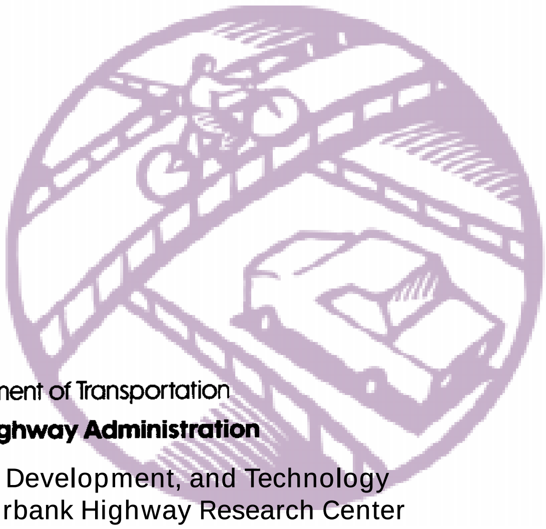

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FOREWORD

Current procedures for designing rural alinements rely on the selection and application of design speeds. U.S. highway geometric design researchers and practitioners generally recognize the need to supplement current design procedures for two-lane rural highways with reliable, quantitative safety-evaluation methods. To address this need, the Federal Highway Administration is developing the Interactive Highway Safety Design Model (IHSDM) as a framework for an integrated design process that systematically considers both the roadway and the roadside in developing cost-effective highway design alternatives. The focus of IHSDM is on the safety effects of design alternatives. Design consistency is one of several modules built around a commercial computer-aided design package in the current vision of IHSDM. Other modules include: crash prediction, driver/vehicle, intersection diagnostic review, policy review, and traffic analysis.

The research documented in this report provided a speed profile model that can be incorporated into the design consistency module of IHSDM. The model can be used to evaluate the design consistency of the roadway or can be used to develop a speed profile for an alinement. The model considers both horizontal and vertical curvature and the acceleration or deceleration behavior as a vehicle moves from one feature to another. The research also demonstrated that predicted speed reduction on a horizontal curve relative to the preceding curve or tangent has a strong relationship to accident frequency.

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NOTICE

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16. Abstract

Design consistency refers to the conformance of a highway’s geometry to driver expectancy. Drivers make fewer errors in the vicinity of geometric features that conform to their expectations. A technique to evaluate the consistency of a design is to evaluate changes in operating speeds along an alignment. To use operating speed as a consistency tool requires the ability to accurately predict speeds as a function of the roadway geometry. In this research project, several efforts were undertaken to predict operating speed for different conditions such as on horizontal, vertical, and combined curves; on tangent sections using alignment indices; on grades using the TWOPAS model; and prior to or after a horizontal curve. The findings from the different efforts were incorporated into a speed-profile model. The model can be used to evaluate the design consistency of the roadway or can be used to develop a speed profile for an alignment. The model considers both horizontal and vertical curvature and the acceleration or deceleration behavior as a vehicle moves from one feature to another.

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1. INTRODUCTION

BACKGROUND

The goal of transportation is generally stated as the safe and efficient movement of people and goods. To achieve this goal, designers use many tools and techniques. One technique used to improve safety on roadways is to examine the consistency of the design. Design consistency refers to a highway geometry's conformance to driver expectancy. Generally, drivers make fewer errors in the vicinity of geometric features that conform to their expectations than at features that violate their expectations.[1]

In the United States, design consistency on two-lane rural highways has been assumed to be provided through the selection and application of a uniform design speed among the individual alignment elements. The design speed is defined by the American Association of State Highway and Transportation Officials (AASHTO) 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."[2] If a road is consistent in design, then the road should not violate the expectations of motorists or inhibit the ability of motorists to control their vehicle safely.[3] Consistent roadway design should ensure that "most drivers would be able to operate safely at their desired speed along the entire alignment."[4]

One weakness of the design-speed concept is that it uses the design speed of the most restrictive geometric element within the section, which is usually a horizontal or vertical curve, as the design speed of the road. Consequently, the design-speed concept currently used in the United States does not explicitly consider the speeds that motorists travel on tangents. Other weaknesses in the design-speed concept have generated discussions and additional research into other methods for evaluating design consistency along two-lane rural highways. Both speed-based and non-speed-based highway geometric design consistency evaluation methods have been considered. These methods have taken several forms and can generally be placed in the following categories: vehicle operations-based consistency (including speed), roadway geometrics- based consistency, driver workload, and consistency checklists.

Some of these methods may be incorporated into the Interactive Highway Safety Design Model (IHSDM). The IHSDM is being developed by the Federal Highway Administration (FHWA) as a framework for "an integrated design process that systematically considers both the roadway and the roadside in developing cost-effective highway design alternatives."[5] The focus of the IHSDM is on the safety effects of design alternatives. Design consistency is one of several modules built around a commercial computer-aided design package in the current vision of the IHSDM.[6] The other modules include: crash analysis, driver/vehicle, intersection diagnostic review, policy review, and traffic analyses.

OBJECTIVES

An earlier FHWA study developed a design-consistency evaluation procedure that used a speed-profile model based on horizontal alignment.[7] This study, "Design Consistency Evaluation Module for the Interactive Highway Safety Design Model (IHSDM)," expanded the research conducted under the previous FHWA study in two directions. These directions were: (1) to expand the speed-profile model, and (2) to investigate three promising design-consistency rating methods. While operating speed is the more common method for evaluating the consistency of a roadway, other methods have been discussed and explored. The three methods selected for additional investigation in this study and documented in another report included: alignment indices, speed-distribution measures, and driver workload.[8]

This report documents the efforts to expand the speed-profile model under the previous FHWA study.[7] The previous study's model estimates speeds along a roadway using horizontal alignment data. Recommendations from that study included conducting further research to validate the developed speed-profile model (including the 85th percentile speeds on curves and long tangents) and to validate the assumed rates and locations relative to curves at which acceleration and deceleration actually occur. The study's objectives were to:

- Develop speed-prediction equations for horizontal and vertical alignments and for other vehicle types.
- Determine the effects of spiral transitions on speeds.
- Determine the deceleration and acceleration rates for vehicles approaching and departing horizontal curves.
- Validate the speed-prediction equations.
- Develop a speed-profile model for inclusion in the IHSDM.
- Identify the relationship of the design consistency module to other modules and components of the IHSDM.

ORGANIZATION OF THE REPORT

The report is organized into 10 chapters:

Chapter 1. Introduction describes the background of the research project and the research objective.

Chapter 2. Previous Work on Predicting Speeds on Two-Lane Rural Highways discusses previous research into the subject area.

Chapter 3. Predicting Speeds on Two-Lane Rural Highway Curves documents the efforts in this research project to collect speed data at 176 two-lane rural highway sites. It also presents the findings from the task that developed speed-prediction equations for horizontal and vertical alignments, the analysis of spiral curves, and the evaluation of speed behavior for different vehicle types.

Chapter 4. Predicting Speeds on Tangents Using Alignment Indices presents information on using alignment indices to predict speeds on tangents.

Chapter 5. Vehicle Performance Using TWOPAS Equations discusses the equations used in the TWOPAS model to estimate speeds for different vehicle types as affected by grades.

Chapter 6. Acceleration/Deceleration Modeling describes the data collection and analysis efforts used to determine appropriate acceleration and deceleration values prior to and after a horizontal curve.

Chapter 7. Validation of Speed-Prediction Equations documents the tasks that evaluated the speed-prediction equations (see chapter 3) and deceleration rates (see chapter 6) in this project.

Chapter 8. Design-Consistency Evaluations and Speed-Profile Model gathers the findings from the different tasks into one procedure that can be used to estimate the speed of a vehicle and to evaluate design consistency along an alignment.

Chapter 9. Relationships of Geometric Design-Consistency Measures to Safety demonstrates how the proposed design-consistency measures are related to safety.

Chapter 10. Summary, Findings, Conclusions, and Recommendations summarizes the study effort and findings and provides conclusions and recommendations.

2. PREVIOUS WORK ON PREDICTING SPEEDS ON TWO-LANE RURAL HIGHWAYS

Since the 1930s, design consistency on two-lane rural highways in the United States has been based on the design-speed concept. The concept was adopted by many countries, but has been modified in recent years after concerns were identified. Research in the United States and other countries has focused on the idea of an operating-speed-based method to ensure consistency. These new methods have generally been based on the prediction of operating speeds using information from the geometric features along the highway alignment as input.

EVALUATION OF DESIGN CONSISTENCY

Two speed-based approaches to evaluate design consistency on two-lane rural highways include design speed and operating speed. The most significant design-speed-based method in the United States is the one used by AASHTO. The operating-speed-based methods are used mostly in Europe and Australia, although methods have been developed and proposed for use in the United States.

Design-Speed-Based Method—AASHTO

The most common approach in the United States to ensure consistency in the design of highways has been the design-speed concept. The concept was developed in the 1930s by Barnett, who thought of the design speed as "the maximum reasonably uniform speed which would be adopted by the faster driving group of vehicle operators, once clear of urban areas."[9] His idea was incorporated into AASHTO policy in the 1940s and is currently used in the United States.

The design-speed concept involves the selection of a design speed based on "the topography, the adjacent land use, and the functional classification of highway."[2] This design speed should be "consistent with the speed a driver is likely to expect."[2] Furthermore, the design speed should be a high-percentile value in a cumulative distribution of vehicle speeds for that type of highway.[2] The design speed selected is then used to establish minimum values for some of the geometric features on the highway.

The premise of the design-speed concept is that a design speed is selected for the entire alignment of a roadway. The individual curves in that alignment must have design speeds equal to or higher than the selected design speed for the roadway. The idea of selecting one speed to which all elements of a highway alignment must comply is sound as long as that speed reflects drivers' expectations and desires. The main problem is that the design speed, as applied in the United States, is the minimum to which any element should be designed. The design speed is used to set minimum curve radius and sight distances; however, AASHTO recommends using higher values whenever "such improvements can be provided as a part of economic design."[2] For example, a road could have a design speed of 90 km/h and only one curve in the entire roadway may have that design speed. Drivers, on the other hand, may presume a safe operating speed higher than the design speed for a curve based on their ad hoc expectancy. Their selection of speeds may result in undesirable speed variations between features.

A limitation of the design-speed concept identified by Krammes and Glascock is that "the design speed applies only to horizontal and vertical curves, not to the tangents that connect those curves."[10] If the tangents are long enough, drivers can attain speeds that are higher than the design speed of the curve at the end of the tangent.

Superelevation is the banking of the pavement on curves to counteract the effect of the centrifugal force. The basic formula that describes the dynamic behavior of a vehicle on horizontal curves is:

$$e + f = \frac{V^2}{127\,R} \tag{1}$$

where:
- e = superelevation rate (m/m)
- f = side-friction factor
- V = vehicle speed (km/h)
- R = radius of curvature (m)

Using the previous equation, AASHTO developed tables relating the design speed, radius of curvature, and superelevation rate for different maximum superelevation rates (e_{max}). Given a maximum superelevation rate and a design speed, the designer selects a radius for the horizontal curve and its respective superelevation rate. A concern with the AASHTO approach is that for different maximum superelevation rates, curves with similar radii and superelevation rates can have different design speeds. An example of this problem is presented in table 1, with data extracted from AASHTO's 1994 *A Policy on Geometric Design of Highways and Streets*.[2]

Hayward discussed the lack of a consistent standard for superelevation throughout the United States and the problems with the AASHTO policy.[11] Even within a particular state, several e_{max} can be allowed, thus increasing the possibility of inconsistent design. Kanellaidis identified the possible differences that could arise between design speeds and actual operating speeds when the AASHTO guidelines are used.[12] He concluded that "the use of design speed to determine individual geometric elements like superelevation rates should be re-evaluated and possibly replaced by operating-speed parameters."[12]

AASHTO's design-speed approach to design consistency has been the standard in the United States for rural highway design. The approach allows for inconsistencies to occur, especially with respect to the application of superelevation. Furthermore, horizontal and vertical alignments are treated separately, and their combination is only addressed in a very limited fashion.

Table 1. Design Speeds for Identical Radii and Different e_{max}.[2]

R	e	e_{max}	V_D
600	0.06	0.06	110
600	0.06	0.08	90
600	0.06	0.10	85
600	0.06	0.12	82

where:
R = radius of curvature (m)
e = superelevation rate (m/m)
e_{max} = maximum superelevation rate (m/m)
V_D = design speed (km/h)

Operating-Speed-Based Methods

The use of operating speeds in lieu of or in addition to design speed has been suggested and implemented in many countries when dealing with design consistency. AASHTO defines operating speed as "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."[2] Krammes et al. found this definition difficult to interpret and use.[13] They preferred to define it as "the speed at which drivers are observed operating their vehicles."[13] In the United States, the 85th percentile of a sample of speeds is accepted as a standard estimate for the operating speed at a specific location.

One of the ways in which operating speeds are used in ensuring design consistency is through the use of speed profiles. Speed-profile models are used to detect speed inconsistencies along road alignments. A speed profile is essentially a plot of operating speeds on the vertical axis versus distance along the roadway on the horizontal axis. Design inconsistencies are identified on the speed profile when there are large differentials in operating speeds between successive alignment features. Switzerland was one of the first countries to use speed-profile models in the geometric design of roadways. Germany and Australia also use operating-speed-based methods to ensure consistency. Three principal methods for checking design consistency have been developed in the United States. The methods were developed by Leisch and Leisch, Lamm et al., and Krammes et al.[7,14-15]

Switzerland

The Swiss design-consistency procedure is one of the oldest in Europe and it is applied only to rural highways.[16] It identifies speed differentials between successive geometric features as a way of detecting inconsistencies in the design.[7,16] In order to identify these speed differentials, a speed profile is prepared using the estimated operating speed on the horizontal curves, the maximum speed on the tangents, and the acceleration rates entering or exiting horizontal curves.[16]

The speeds used in the speed profile represent the 85th percentile speeds on horizontal curves. The speed profile is based only on horizontal alignment because Swiss researchers concluded that grades up to 7 percent had no effect on passenger car operating speeds.[17] Recent Swiss research determined that the 85th percentile speed on curves with radii less than 400 m had increased; however, the decision was made to retain the old 85th percentile values as a standard safe speed.[17-18]

There are three conditions that any speed profile should meet for the horizontal alignment to be considered consistent:[17-18]

1. The maximum speed differential between a curve and the preceding tangent or large radius curve is 5 km/h.
2. The maximum speed differential in successive curves is 10 km/h and any speed differential above 20 km/h should be avoided.
3. The available sight distance should not be less than the length required to change speeds at a rate of 0.8 m/s^2 between successive curves.

Whenever any of the established conditions are violated, the location is marked on the profile and studied for accident experience.[7] If the studies show significant accident experience, the location is corrected by redesigning and realigning the curve.

Germany

In Germany, the design standards for rural highways specify a design speed and an estimated 85th percentile speed for designing roadways. The design speed is used to determine the minimum values for horizontal and crest vertical curve radius and maximum grades.[17] The 85th percentile speed, if greater than the design speed, is used to decide superelevation rates and sight distances.[7] The use of the 85th percentile speed to select superelevation rates and sight distances is justified because it incorporates an additional safety factor into the design.[19]

The Germans do not review individual features in trying to ensure design consistency. They use the curvature change rate (CCR) as a measure of the highway's homogeneity.[19] CCR is defined as the absolute angular change in horizontal direction per unit of distance.[7,17,19] A regression equation based on CCR is used to estimate the 85^{th} percentile speed along the alignment.

The current German guidelines specify that to attain consistency, the design speed and the 85^{th} percentile speed must be even. Therefore, it is required that the 85^{th} percentile speed not exceed the design speed on any given section by more than 20 km/h.[17,19] Furthermore, the maximum difference in 85^{th} percentile speed between successive sections should not exceed 10 km/h.[17,19] Any violation of these conditions will require an adjustment of the horizontal alignment.

Australia

The Australian design-consistency concept is based on research conducted by McLean.[20] He studied 120 horizontal curves on two-lane rural highways in Australia and concluded that when the design speed is lower than 90 km/h, the 85^{th} percentile speed will tend to be higher than the design speed of the geometric features.[20-21] This finding contrasts with the basic assumption that operating speeds will not exceed the design speed. McLean found that when design speeds are higher than 100 km/h, the 85^{th} percentile speeds are generally lower than the design speed.[20-21]

In response to McLean's findings, Australia changed its design procedures for horizontal alignment on lower design-speed highways (i.e., a design speed less than or equal to 100 km/h) to emphasize the importance of the 85^{th} percentile speeds along an alignment.[20] In this case, an estimated 85^{th} percentile speed is used as the design speed. The estimation of the 85^{th} percentile speed is based on the curve radius and the desired speed on the roadway. This desired speed is defined as "the speed at which drivers choose to travel under free-flow conditions when they are not constrained by alignment features."[20] The Australian design standards define guidelines to select the desired speed for the alignment according to the range of curve radii and terrain type.

Leisch and Leisch

Leisch and Leisch concluded in the 1970s that the design-speed concept did not guarantee consistency in highway alignment.[14] They identified two major problems related to the design-speed concept. The basic problem was the variation in operating speeds when the design speed was below 90 km/h. The other problem was the speed differential between passenger vehicles and trucks.

In order to resolve these issues, Leisch and Leisch modified the definition of design speed to make it a "representative potential operating speed that is determined by the design and correlation of the physical features of a highway."[14] They suggested modifying the design-speed concept to include the "15-km/h rule." The "15-km/h rule" follows three principles:[14]

1. Design speed reductions should be avoided, but if they are necessary, they should not exceed 15 km/h.
2. Potential passenger car speeds should not vary more than 15 km/h along an alignment.
3. Potential truck speeds should not be more than 15 km/h lower than average passenger car speeds.

The modified design-speed approach requires a tool to visualize the alignment and its consistency. The device used is a speed profile that is created by plotting speed measurements against distance. The speed profile is done while taking into consideration the horizontal and vertical alignments of the highway. The speed profiles for passenger cars and trucks are prepared separately and then superimposed. If the considerations of the "15-km/h rule" are violated, that portion of the alignment is considered to be inconsistent and should be modified.

Lamm et al.

Lamm et al. suggested a model similar to the German model after studying 260 curves in New York State.(15) The model was developed for use with horizontal alignments and uses degree of curvature (D) as the main variable to determine operating speeds. Lamm et al. define operating speed as the 85th percentile speed of those vehicles traveling on the roadway studied.(15) The German approach uses the curvature change rate as the independent variable in the regression equation to estimate the operating speed. Lamm et al. agree that there are no major differences between using degree of curvature or CCR, but they recommend the degree of curvature for use on most U.S. two-lane rural roads because of its common usage in design.(22) The speed profile is constructed with the estimated operating speed in a manner similar to the Swiss method.

The model by Lamm et al. quantifies design consistency and breaks down highway designs into three categories:(15)

1. Good Design: Change in degree of curvature less than or equal to 5 degrees, or a change in operating speed less than or equal to 10 km/h.
2. Fair Design: Change in degree of curvature greater than 5 degrees and less than or equal to 10 degrees, or a change in operating speed greater than 10 km/h and less than or equal to 20 km/h.
3. Poor Design: Change in degree of curvature greater than 10 degrees, or change in operating speed greater than 20 km/h.

A good design is considered to be consistent. Fair designs have some minor inconsistencies that may affect the driver's behavior. Poor designs have inconsistencies that cause speed differentials in excess of 20 km/h in the speed profile of the roadway.(15)

Krammes et al.

Krammes et al. conducted extensive research in horizontal alignment data to develop a speed-profile model to check design consistency.(7) The speed-profile model suggested incorporates features from the models developed by Lamm et al. and in Switzerland.(7) The study covered 138 curves in 3 regions of the United States.(7) Krammes et al. confirmed what McLean found in his studies in Australia—the operating speed exceeds the design speed of horizontal curves when the design speed is 90 km/h or less.(7)

The speed-profile model uses the change in 85th percentile speed between the tangent and the curve as the main measure of design consistency. The recommended multiple-regression equation to predict the 85th percentile speed is one in which all the variables are related to the geometry of the curve. Another suggested equation includes the speed on the previous tangent as a measure of the desired speed. However, the desired speed was difficult to predict and this version of the model was not recommended.(7) Voigt analyzed the same data used by Krammes et al. and added the effect of superelevation to the 85th percentile speed equation.(23) He found that superelevation enhanced the prediction capability of the equation, but he did not suggest any changes to the speed-profile model.

Design Consistency on Combined Horizontal and Vertical Alignments

Current design-speed-based or operating-speed-based methods to ensure design consistency are oriented toward horizontal alignment. There is no model to measure design consistency on combined horizontal and vertical alignments. There are also no statistical models to estimate operating speeds on combined alignments. Furthermore, in the United States, the operational effects of combined horizontal and vertical alignment have not been studied. Generally, horizontal and vertical designs are done separately to meet quantitative criteria and are then brought together assuming that design consistency will be maintained. This consistency may not always be achieved.

The way the driver assesses combined roadway alignments may be different than the way a designer assesses them. Kanellaidis states that design consistency is indirectly associated with how drivers maneuver geometric features, while a driver's workload is directly related to it.(24) Messer defines driver workload as "the time rate at which drivers must perform a given amount of work or driving tasks."(25) He indicates that driver workload increases with reductions in sight distance and increasing complexity of geometric features.(25) Glascock interpreted from Messer's work that "combinations of [geometric] features increase workload and may be more hazardous to drivers than successive features with adequate separation."(26) For example, a horizontal curve combined with crest vertical curve could increase the driver's workload in two ways: by having a reduced sight distance and by having to control the vehicle in a three-dimensional space. If the combination of horizontal and vertical features includes an unexpected or extreme feature, the workload is increased even more.

Consequently, as the complexity of the geometric feature increases, the higher the workload and the greater the probability of a significant speed change.

AASHTO realizes the limitations of current design procedures and its focus on the designer's point of view. AASHTO attempts to include the driver's perspective by providing a section on how to check combined alignment design in its guidelines. AASHTO tries to define the proper combinations of horizontal and vertical alignment from the driver's perspective. For example, a sharp horizontal curve should not follow a long tangent; a sharp horizontal curve should not be located at or near the top of crest vertical curves; and a sharp horizontal curve should not be located at or near the bottom of sag vertical curves.[2] These examples increase the driver's workload and may cause speed changes and accidents. AASHTO recommends the use of graphical or computer-aided design tools to review combined alignments. Their desire is to achieve the best coordination from the viewpoint of appearance and the driver's perspective.

The use of general guidelines in designing combined horizontal and vertical alignment does not guarantee consistency relative to uniformity of operating speeds. The driver's assessment of the roadway ahead is critical in any speed variations. Combined horizontal and vertical alignments increase the complexity of the driving task and the driver's workload.

ESTIMATION OF OPERATING SPEED

Speed-profile models can estimate the 85th percentile speeds along an alignment. The 85th percentile of a sample of observed speeds is the general statistic used to describe operating speeds on a geometric feature.[13] It is the speed at or below which 85 percent of the drivers are operating.[13,27] Early research by Moyer and Berry found that using percentiles between 85 and 90 percent of the observed speeds provided a satisfactory value for setting speed limits on curves.[28] The 85th percentile is the most common factor used to set speed limits on existing roads in the United States and is internationally accepted as a measure of operating speed.[29] Therefore, the 85th percentile speed is used throughout this research as a measure of the operating speeds on two-lane rural highways.

Horizontal Alignment

Speed prediction on two-lane rural roads has been researched extensively for horizontal curves on relatively flat terrain. Previous research indicates that curve radius is the most important element in determining speeds on horizontal curves. Superelevation and deflection angle are other variables that have been used in some regression equations to predict operating speeds on horizontal curves. For tangents, it is believed that the length of the tangent is the primary alignment parameter that determines the speed. Driveway density and cross-section are two variables that are also believed to affect speeds on tangents on level terrain.

Radius

The use of horizontal radius of curvature as a variable to predict 85th percentile speeds on curves spans more than four decades. During this period, it became customary to predict the 85th percentile speed using geometric factors. Most of the equations were originally developed using degree of curvature instead of radius because that was the standard descriptor of horizontal curvature in the English system of units. The relationship between degree of curvature and radius is given by the following equation, assuming the arc definition of 30.5 m:

$$R = \frac{1746.38}{D} \tag{2}$$

where: R = radius of curvature (m)
D = degree of curvature (deg)

Generally, speed is reduced as the degree of curvature increases. When radius is used, the speed decreases as the radius is reduced. Table 2 lists some of the equations developed to predict operating speeds on horizontal curves as a function of geometric variables. Only the equation by Islam and Seneviratne includes the radius squared.[30]

The equation by Krammes et al. is based on one of the most comprehensive studies on horizontal alignment.[7] Their research covered five States in four geographical areas. It found the degree of curvature, length of curve, and deflection angle to be the most significant independent variables for predicting speeds on curves. Ottesen and Voigt participated in this study and they presented alternative forms of the regression equation.[23,33]

Other models have been developed to predict operating speeds using other independent variables not related to the geometry of the curve. McLean studied the effects of horizontal alignment on speeds in Australia.[20] He concluded that curve radius and the desired speeds of drivers were the most significant variables in determining operating speed on curves.[20-21,34] Desired speed is defined as the speed at which drivers choose to travel under free-flow conditions when they are not constrained by alignment features.[20] Kanellaidis et al. also indicated a strong relationship between operating speeds on curves and curvature.[35]

The models presented in table 2 show the definitive effect that radius has on speeds on curves. Furthermore, they suggest 1/R as the best way to include radius in a regression model to predict operating speeds.

Other Variables Affecting Operating Speeds

Regression equations for estimating operating speeds have not been limited to the use of radius as an independent variable. The study by Krammes et al. found that the length of the horizontal curve and the deflection angle also had a significant effect on operating speeds.[7] It is important to note that the deflection angle is related to the other two independent variables in the model by Krammes et al. Deflection angle, radius, and length are related by the following equation:[23]

$$I = 57.29 \frac{L_H}{R} \tag{3}$$

where:
- I = deflection angle (deg)
- L_H = length of the horizontal curve (m)
- R = radius of horizontal curve (m)

Table 2. Regression Equations for Operating Speeds on Horizontal Curves in the United States.

Author	Equation	R^2	Sample Size	Location	Year
Taragin[31]	$V_{90} = 88.87 - \frac{2554.76}{R}$	0.86	35 curves	Illinois, Maryland, Minnesota, New York, South Carolina	1953
Glennon et al.[32]	$V_{85} = 103.96 - \frac{4524.94}{R}$	0.84	56 curves	Florida, Illinois, Ohio, Texas	1985
Lamm and Choveini[22]	$V_{85} = 94.39 - \frac{3189.94}{R}$	0.79	261 curves	New York	1986
Islam and Seneyirctry[30]	$V_{85} = 103.03 - \frac{4208.76}{R} - \frac{36597.92}{R^2}$	0.98	8 curves	Utah	1994
Ottesen[33]	$V_{85} = 103.64 - \frac{3400.73}{R}$	0.80	138 curves	New York, Oregon, Pennsylvania, Texas, Washington	1993
Krammes et al.[7]	$V_{85} = 102.44 - \frac{2741.81}{R} + 0.012\,L - 0.10\,I$	0.82			

Voigt[23]	$V_{85} = 99.61 - \frac{2951.37}{R} + 0.014\,L - 0.13\,I + 71.82\,e$	0.84			1996

where:
V_{90} = 90th percentile speed on a curve (km/h)
L = length of curve (m)
V_{85} = 85^{th} percentile speed on a curve (km/h)
I = deflection angle (deg)
R = radius of curvature (m)
e = superelevation (m/m)

Voigt expanded the equation by Krammes et al. to include superelevation.[23,7] The length of the curve, the deflection angle, and the superelevation have shown some effect in estimating operating speeds. Care must be employed when interpreting these results because of the collinearity among the independent variables. For example, radius and superelevation are highly correlated.

One final variable that should be mentioned is cross-section. It has been suggested that pavement width, lane width, and shoulder width may have an effect on operating speeds. Lamm and Choueiri reported an effect from cross-section in an equation to predict the 85^{th} percentile speed.[22] Their equation included degree of curvature, lane width, shoulder width, and annual average daily traffic (AADT) as independent variables. The equation was a prelude to the one presented in table 2. They decided to drop the lane width, shoulder width, and AADT from the equation because they only explained about 5.5 percent of the variation in the estimated 85^{th} percentile speed.[22]

Vertical Alignment

Recent vertical alignment research includes the effects of stopping sight distance on accident rates or on crest vertical curves and the effects of grades on speeds. These studies have shown that grades and stopping sight distance have an effect on operating speeds. Passenger cars are affected by short sight distances on two-lane roadways with narrow shoulders and by steep grades. The effects of short sight distances are not known for trucks; however, trucks are affected by the length and steepness of vertical grades.

Vertical Curves

Vertical curves in the United States are parabolic and are described by the rate of vertical curvature. K is the horizontal distance in meters required to effect a 1-percent change in gradient.[2] The mathematical equation has the following form:

$$K = \frac{L_V}{A} \tag{4}$$

where: K = rate of vertical curvature (m/%)

L_V = length of vertical curve (m)
A = $|G_2 - G_1|$ = algebraic difference in grades (%)
G_1 = approach-tangent grade (%)
G_2 = departure-tangent grade (%)

Messer reports that sight distance is the primary factor affecting driver expectancy when looking at the vertical alignment of a road.(25) For crest vertical curves, stopping sight distance is of the utmost importance. Stopping sight distance is defined as the distance that a driver must be able to see ahead along the roadway in order to identify hazards in the roadway and, if necessary, bring the vehicle safely to a stop.

As early as 1953, Lefeve studied crest vertical curves on two-lane rural roads in New York with sight distances between 45.7 m and 152.5 m.(27) He found that drivers do reduce their speeds as they approach curves with short sight distances; however, that reduction in speed was not as much as he thought was required for safe operation.(27) For the range of sight distances studied, Lefeve found no relation between operating speeds at crest vertical curves and the sight distance.(27)

Fambro et al. studied the relationship between operating speed and design speed at crest vertical curves, and concluded that operating speeds are well above the design speeds of crest vertical curves.(36) Their study of 42 curves in 3 States also concluded that "the inferred design speed of a crest curve (without shoulders) is a moderately good predictor of 85th percentile speeds for these types of roadways."(36) The design speed was inferred using the rate of vertical curvature. They classified a two-lane rural road as being without shoulders if the shoulder width was less than 1.8 m. The relationship found for two-lane rural roadways without shoulders was based on 17 curves and it had the following form:

$$V_{85} = 72.74 + 0.47\,V_D \tag{5}$$

where: V_{85} = 85^{th} percentile speed (km/h)
V_D = inferred design speed (km/h)

Fambro et al. established that speed could vary with the design speed or a surrogate of it, such as K.(36) However, it was not possible to establish a statistically significant relationship between operating speed and inferred design speed for roadways with shoulders.(36) The value of K as a possible variable to predict speeds was enhanced because of the results from that study.

For sag curves, AASHTO identifies four criteria for their design. The headlight sight distance is the primary criterion used to obtain the length of the sag curve. The headlight sight distance is similar to the stopping sight distance, and they are used sometimes interchangeably. The comfort criterion is based on the change in vertical direction. AASHTO states that "the effect of change in vertical direction is greater on sag than on crest vertical curves because gravitational and centrifugal forces are combining rather than opposing forces."[2] For both criteria, K is used to specify minimum values according to the design speed. The headlight sight-distance criterion usually controls the design of sag curves; however, for low values of K, the comfort criterion may be the factor that controls the speed selection on sag curves.

Grades

Grades have a physical effect on speeds and should be considered in any model used to estimate operating speeds. AASHTO states that trucks display an increase in speed of less than 5 percent on downgrades and a decrease of more than 7 percent on upgrades when compared to level terrain.[2] In the case of upgrades, the maximum speed that trucks can sustain depends on the weight-to-horsepower ratio of the vehicle as well as the length and steepness of the grade.

Most of the information about grades contained in *A Policy on Geometric Design of Highways and Streets* comes from a 1986 study by Gillespie.[2,37] His formulation for speed reduction on grades and the subsequent tables for different types of trucks and weight-to-horsepower ratios form the basis for design of climbing lanes and passing zones on grades. Although Gillespie's speed-prediction model seems appropriate, the average weight-to-horsepower ratios on trucks have declined, and the capabilities of these vehicles may be underestimated.

Length of a grade or the approach to a vertical curve is of importance because its combination with steep grades can affect speeds. Trucks and recreational vehicles are mostly affected by long grades. AASHTO recognized the importance of the length of a tangent on a grade and conceived the term "critical length of grade" to indicate the maximum length of an upgrade that a truck or recreational vehicle could operate without a significant reduction in speed.[2] AASHTO developed a series of graphs to estimate a reduction in speed based on the critical length of the grade.

Combination of Horizontal and Vertical Alignment

The combination of horizontal and vertical alignment has never been systematically studied. Most studies have been directed toward either horizontal or vertical alignment individually. The most common way of dealing with the combined alignments has been to look at drivers' perspective views.

In 1967, Park and Rowan used the driver's perspective view to alignment coordination by using plotter-drawn perspectives.[38] Smith et al. also used a similar plotter-drawn approach a few years later.[39] Plotter-drawn perspectives are an attempt to represent the three-dimensional view of the driver on a two-dimensional plane. The main purpose of these drawings is to identify visual discontinuities on the alignment and ensure that the alignment has a smooth appearance.

More recently, Smith and Lamm proposed the use of perspective methods for the three-dimensional evaluation of the roadway instead of the common two-dimensional approach.(40) They advocate the use of the drivers' perspective view to ensure that the design meets drivers' expectations and to make a series of recommendations to ensure consistency. Some of these recommendations are: the ratio of horizontal to vertical curve radii should be between 1/5 and 1/10; and when designing in hilly topography, the radius of crest curves should be greater than the radius of sag curves and vice versa when on flat topography.(40) These recommendations are in addition to what AASHTO recommends. In the opinion of Smith and Lamm, failure to consider three-dimensional alignment is the weakest link in the overall design of highways.(40)

None of the proposed perspective-based methods uses quantitative criteria to evaluate design consistency. Drivers' perspective views are useful, but cannot provide a quantitative estimate of the change in speeds caused by the alignment combination. The driver's perspective may be explained to some extent by the geometric variables used in analyzing the physical aspects. For example, how much of a horizontal curve is visible when approaching that curve can affect the perspective view of the driver and impact the desired speed of the vehicle.

The physical aspect of the different geometric features is what most researchers have studied for individual alignments. The radius and the rate of vertical curvature appear to be the most important variables in explaining the speeds. For combined alignments, the same variables used in studying individual geometric features should be used.

Effects of Weather and Light Conditions on Two-Lane Rural Highway Speeds

The speed-profile model developed by Krammes et al. was calibrated using speed data in daylight and dry-weather conditions.(7) A question asked was whether it was necessary to calibrate the model using data for nighttime and wet-weather conditions in order to expand the range of ambient conditions to which the model is applicable. This section assesses the need to account for the effects of weather and light conditions in order to expand the applicability of speed-profile modeling for design-consistency evaluation. First, research on the effects of weather on speed is reviewed. Then, previous studies that compared day and night speeds are summarized.

Speeds During Dry- Versus Wet-Weather Conditions

Several studies have compared speeds during dry- versus wet-weather/pavement conditions. In his 1966 synthesis of *Variables Influencing Spot-Speed Characteristics*, Oppenlander referenced five studies conducted between 1939 and 1956 in observing, "The exact influence of wet pavements on spot-speed characteristics is not definitely defined in the literature, with indications of both a reduction and no significant difference in vehicular speeds on wet as compared to dry pavements."(41)

Among three more recent studies, Blackburn et al. conducted the most extensive comparison of speeds during dry and wet conditions.[(42)] They analyzed passenger-vehicle speed data collected during 1975 by 9 States at 48 sites on urban and two-lane rural, multi-lane uncontrolled-access, and multi-lane controlled-access highways. A statistically significant difference (at a = 0.05) between wet and dry 85th percentile speeds was observed at 18 of the 48 sites; dry-weather speeds were higher at 16 of those sites, and wet-weather speeds were higher at 2 sites. When sites were grouped by roadway type, the 95-percent confidence interval for the difference between dry- and wet-weather 85th percentile speeds on two-lane rural highways was 3.2 km/h ±2.4 km/h, which suggests that the magnitude of the effect of weather is small, even at those sites where the effect is statistically significant.

Between 1984 and 1987, Lamm et al. measured speeds during dry and wet conditions in both directions at 1 tangent and 11 curves (ranging in radius from 213 m to 1,747 m) on two-lane rural highways in New York.[(43)] They concluded, "operating speeds on dry pavements are not statistically different from operating speeds on wet pavements."

Ibrahim and Hall used traffic surveillance system data from a freeway in Mississauga, Ontario, Canada for the time period October 1990 to February 1991 to evaluate the effect of rain and snow on speed-flow-occupancy relationships.[(44)] They concluded, based upon their regression analysis of speed versus occupancy, that light rain had a minimal effect on the relationship (i.e., a difference between dry and light-rain free-flow speeds of only 2 km/h), whereas heavy rain had a greater effect (i.e., a difference between dry and heavy-rain free-flow speed of 5 to 10 km/h).

These empirical results are consistent with AASHTO's conclusion that "Studies show that many operators drive just as fast on wet pavements as they do on dry."[(2)] The results are sufficiently consistent that it was deemed unnecessary to conduct additional data collection and analysis to verify AASHTO's observation. It was concluded, based upon this review of previous research, that accounting for the effect of dry- versus wet-weather conditions would not add substantially to the speed-profile model for design-consistency evaluation.

Speeds During Day Versus Night

Only limited research results are available on the differences between day and night speeds. In his 1966 synthesis, Oppenlander observed, "As many articles on highway traffic characteristics indicate, average spot speeds in the daytime are about 1 mph higher in urban areas and 2 to 8 mph higher in rural areas, depending on the particular roadway facility, than the corresponding speed values during the nighttime. However, several speed-characteristic studies did not show significant differences between average daytime and nighttime speeds."[(41)] Oppenlander references 21 studies, dating from 1937 to 1961. Due to the age of these references, primary reviews were not conducted.

A search of more recent literature uncovered only two U.S. studies in which day and night speeds were collected on two-lane rural highways. Both studies evaluated the effectiveness of alternative roadway delineation treatments. In 1972, Taylor and McGee reported day and night speed data for the inside and outside lanes of one horizontal curve (88-m radius) and its approach tangents.(45) Speed data were collected for each of four delineation treatments (combinations of old or new centerlines and edgelines, with or without retroreflective raised pavement markers). Day and night speeds were not significantly different (at a = 0.05) on either the inside or outside lanes of this curve for any of the delineation treatments.

In 1977, Stimpson et al. reported day and night speed data that were collected at nine sites for their field evaluation of alternative delineation treatments for two-lane rural highways.(46) The sites included five tangent locations, two locations on windy alignments, and two isolated horizontal curves (one 250-m radius and one 349-m radius). Among the nine sites, differences between day and night speeds were statistically significant at only one of the tangents.

In summary, previous research on the effect of light conditions is much more limited than research on the effect of weather conditions on speeds. For this reason, Guzman undertook a small-scale empirical study to collect additional data on day and night speeds as a basis for deciding whether a larger scale effort was required under this contract.(47)

Small-Scale Study on Day Versus Night Speeds

Guzman collected speed data at eight horizontal curve sites on two-lane rural highways in Texas.(47) The sites satisfied the site-selection controls and criteria that were specified for the data collected to calibrate the speed-profile model.(7) The roadside environment at the study sites was typical of rural, central Texas—a mixture of pasture and forest, with widely scattered residences. There was no roadside lighting at any of the sites except for a single residence near some sites. The traffic control devices at the curve sites are also typical of practices in Texas. All of the sites had centerlines, but only four of the eight sites had edgelines. At all of the sites, retroreflective raised pavement markers supplemented the painted centerline. All but the 582-m radius curves had curve warning signs with advisory speed panels. The sharpest curve had a turn sign, large arrow sign, and chevrons.

At each site, speed data were collected during the day and night on both lanes. Speed-measurement locations included the middle of the approach tangent and the midpoint of the horizontal curve in each lane. Only data collected during dry-weather conditions were analyzed. Only weekday data (between midnight Monday and sunrise Friday) were used in the analysis. Speeds were retained in the database only for passenger vehicles that could be tracked between the tangent and curve measurement points and that were unaffected by other vehicles. Vehicles maintaining less than a 10-s headway to the leading vehicle in the same direction were excluded from the database. Vehicles crossing the midpoint of the curve within 10 s of each other were also excluded from the database. Speeds were measured using traffic counters/classifiers with on-pavement sensors. Three dependent variables were analyzed: (1) speed at the midpoint of the approach tangent, (2) speed at the midpoint of the horizontal curve, and (3) speed change from the tangent to the curve. Using only data for vehicles tracked through a site ensured that the speeds on the curve and the speeds on the tangent were for the same sample of drivers/vehicles for a given light condition and lane of travel. The speed change for each tracked vehicle was computed as the difference between the vehicle's speed on the approach tangent and its speed on the horizontal curve.

As a first step in the analysis, a three-factor analysis of variance with full interactions was performed on each dependent variable to test for differences among their means. The three main effects were site, lane of travel, and light condition. Light condition was significant for speeds at the midpoint of the curve and for the speed change from the approach tangent to the curve. The significance of the three-way interaction term (site x lane x light) suggests that the effect of light condition varied among the sites and lanes of travel. As a result, it may be an overgeneralization to draw conclusions about the effect of light condition based solely on the analysis of variance results. Therefore, a site-by-site analysis was undertaken in order to determine for which combinations of site and lane the effect of light condition was significant. For each measurement point in each lane of each site, a two-sample t-test was conducted to identify statistically significant differences between mean speeds during the day and night. F-tests were conducted to identify statistically significant differences between the variances of speeds during the day and night. All analyses were performed using the Statistical Analysis System (SAS)®. All tests were performed at a 0.05 significance level (a).

The results of statistical comparisons of day versus night speeds at eight horizontal curves and their approach tangents on two-lane rural highways in Texas can be summarized as follows:

- Mean speeds during the day and night differed significantly at some sites (56 percent of the curve lanes and 38 percent of the tangent lanes), but not at others. At most sites (both curve and tangent) where differences were significant, mean speeds were higher during the day. Speed variances were significantly different at 31 percent of the curve sites and 25 percent of the tangent sites. The magnitude of the differences between day and night mean speeds (ranging from 2.1 to 6.2 km/h on the curves and from 3.0 to 6.1 km/h on the approach tangents) was small compared to the previously observed variability in speeds among curve sites and the precision of regression models used to predict speeds on curves.[7]

- Mean speed changes from the approach tangent to the curve were significantly different (higher during the night) at 31 percent of the site lanes. Differences in speed-change variances were almost evenly divided between those where night variances were higher and those where day variances were higher. The magnitude of the differences in both the means (1.1 to 2.8 km/h) and variances (1.2 to 5.5 km/h) of the speed changes was small.

- Differences in geometry and traffic control devices at sites cannot be used to distinguish those sites at which day and night speeds differed significantly and those sites at which day and night speeds did not differ significantly.

The limited (eight sites) empirical study reported herein indicates that the speeds of free-flowing passenger vehicles differ significantly between day and night at some locations, but not at others, and that the magnitude of the differences is small. These results were similar to the findings of previous research. Therefore, it was also concluded that sufficient information was available on light conditions to recommend that no additional research on this topic was necessary as part of efforts to develop speed-profile models for design-consistency evaluation, although such research may be justified in the broader interest of better understanding all facets of driver speed behavior.

SUMMARY OF LITERATURE REVIEW

In the United States, AASHTO's design-speed approach for design consistency is the standard; however, it has some problems that do not guarantee the desired consistency in all situations. International practices and U.S. research confirm that methods based on operating speeds should also be used to ensure design consistency.

The 85th percentile of a sample of speeds measured at a specific location is generally accepted as a measure of the operating speeds at that location. Therefore, the ability to predict the 85th percentile speed using geometric variables is critical to the operating-speed-based methods. Research and foreign practices have identified horizontal radius as the main variable when estimating speeds on horizontal curves. Passenger car speeds on vertical curves can be affected by the rate of vertical curvature and steep grades. Trucks and recreational vehicles are affected by the length and steepness of a grade.

There are two main issues with existing approaches to design consistency. One is their inability to study combinations of horizontal and vertical alignments. Horizontal alignments combined with vertical alignments increase the driver workload and the potential for speed changes. Presently, the only alternative to determine consistency on combined alignments is to look at a three-dimensional perspective of the alignment. This method is not quantitative and cannot measure the effect of the alignment on the driver's behavior. The other issue is that they were developed only for passenger cars. Trucks and recreational vehicles may be affected differently than passenger cars by combinations of horizontal and vertical alignment.

The previous research cited in this section consistently supports AASHTO's assumption that light rain and wet pavement have little effect on passenger vehicle speeds. This previous research was deemed to provide a sufficient basis for concluding that no further research was necessary to make recommendations about whether to account for weather conditions in design-consistency evaluations.

It is concluded, based upon the literature review on the effects of weather and the literature review and small-scale empirical study on the effects of light that neither weather (dry versus wet) nor light (day versus night) have a sufficiently large effect on two-lane rural highway speeds to impact design-consistency evaluations or, therefore, to necessitate their inclusion in the speed-profile modeling for that purpose.

3. PREDICTING SPEEDS ON TWO-LANE RURAL HIGHWAY CURVES

One objective of this research was to develop speed-prediction equations that consider combinations of horizontal and vertical alignment. Regression models were developed to predict the 85^{th} percentile speed of passenger vehicles on horizontal curves, vertical curves, and combined horizontal and vertical curves based only on the geometry of the curves. Three possible ways in which combined horizontal and vertical alignments affect operating speeds are:

1. Horizontal and vertical alignment act independently of each other and their effect is linear and additive.
2. The effect of horizontal or vertical alignment depends on the value of the other and there is an interaction between them.
3. The most severe alignment condition influences the operating speed and the other has no marginal effect and can be ignored.

The first alternative may apply to horizontal curves combined with vertical crest or sag curves. In this alternative, both alignments influence speeds, but there is no interaction between the alignments. The effect of vertical alignment on operating speeds is in addition to the effect of horizontal alignment. The regression equation for such a condition would have no interaction terms between the horizontal and vertical alignment variables.

The second alternative would occur if an interaction between the horizontal and vertical alignment exists. The effect of the horizontal alignment is conditional upon the vertical alignment with which it is combined. The interaction may be identified by the presence of an interaction term in a regression equation that has variables related to both alignments. It may also be identified as an equation with horizontal variables, but with different equations for various types of vertical curves or grades.

The final alternative would occur when the most extreme alignment condition controls the selection of operating speeds, i.e., either the horizontal or vertical alignment controls the selection of speeds. This condition may be encountered in any combination of horizontal and vertical alignment. For this condition, there will be a regression equation for when the horizontal alignment influences operating speed and another one for when the vertical alignment influences operating speed.

Regression analysis was used to determine which of the above hypotheses best describes how geometry influences the speeds of passenger cars. The analysis began with tests for normality and the development of correlation matrices to identify relationships. Regression equations were then developed using models based on the above three hypotheses. All models were tested for statistical significance and the model that best predicted speed for the set of conditions was selected.

In addition, regression analyses were also performed to determine if the presence of spiral transitions influenced the speed of passenger car drivers. In addition to evaluating passenger car speeds, the speeds of trucks and recreational vehicles (RVs) were also examined. Even though sites were selected to maximize the amount of truck and RV data, the quantity of data collected was insufficient for a statistical evaluation. Therefore, a graphical review was conducted.

DATA COLLECTION METHODOLOGY

This section describes the site selection and data collection methodology used. The data collection had two main components: the geometric data of the site and the speed data. These components were merged during the data-reduction efforts. Approximately half of the data were used in model development and half were used to validate equations developed previously by Krammes et al. and the equations developed in this study.[7]

Site Selection

Data were collected in six States: Minnesota, New York, Pennsylvania, Oregon, Washington, and Texas. These States were selected because they cover much of the range of two-lane rural highway conditions in the United States. In addition, these States had readily available computerized geometric information or design plans.

The general criteria used to select sites are summarized in table 3. These criteria were selected to result in a database that represents the most common conditions found in the United States. However, it must be noted that the site sample was not random and selection bias may exist because databases of all possible roads in the United States are not available. The criteria are similar to those used in the previous research conducted by the Texas Transportation Institute (TTI) to allow comparisons and the expansion of the speed-profile model developed as part of that research.[7]

Several additional criteria were also used in selecting the sites. When vertical curves were combined with horizontal curves, the points of inflection of both curves could not be more than 30 m apart, and a significant overlap had to exist between the lengths of both curves. Sites with suspected high truck and/or RV activity were given high priority during the selection process. Lower volume facilities (less than 2,000 vehicles per day) were preferred to reduce the potential for restricted vehicle flow. Also, the curves selected could not be close to towns or developed areas that may significantly affect the speed patterns on the curves.

Candidate sites were selected by reviewing plans and/or computerized alignment records. Potential sites were marked on maps, and the proximity of a site in relation to other sites was a deciding factor in site selection. Final determination of appropriate sites was made after visitingthe locations.

Table 3. Site Selection Criteria.

Control	Criteria
Area Type	Rural
Density of Access Points	# 3 per km
Functional Classification	Collector or minor arterial
Design Speed	# 120 km/h
Posted Speed Limit	75 km/h to 115 km/h
Terrain	No restrictions
Radii	110 m to 3,500 m
Grade	-10% to +10%
Traffic Volumes	500 to 4,000 vehicles per day
Lane Widths	2.74 m to 3.66 m
Horizontal Curve Length	No restrictions
Vertical Curve Length	$ 60 m
Tangent Length	No restrictions
Vertical Curve	Type I or II (as defined by AASHTO)

In order to organize the site selection procedure, the horizontal and vertical alignments were divided into groups or cells that included the desired combinations. Table 4 shows the different combinations of horizontal and vertical features that were studied. The table also shows the desired and actual number of sites at which data were collected for model development and model validation. At each site, data were collected on the preceding tangent as a control point and consequently horizontal tangents on grade were not included explicitly in the study site matrix.

Horizontal alignment was divided into five categories of radii, including tangents that have an infinite radius. The radii range from less than 144 m to infinity. The vertical alignment was divided into seven categories: four are ranges of grades and three are types of vertical curves. Crest vertical curves are divided into limited sight-distance (LSD) and non-limited sight-distance (NLSD) curves. A K-value of 43 was used to divide the categories because curves with values less than 43 are those that have a limited sight distance for a design speed of 90 km/h.(2)

Two additional factors were used to select sites—cross-section and tangent length. Cross-sections were divided into two categories—typical cross-section and narrow cross-section. Typical cross-sections had lanes 3.6 m or wider and shoulders 1.8 m or wider. Narrow cross-sections had lanes less than 3.6 m and shoulder widths less than 1.8 m. Slightly more than half of the sites had the narrow cross-section. In the previous speed-profile model, there were three categories of horizontal tangent lengths, and an effort was made to obtain a sample of each category. The three categories are: long tangents, moderate tangents, and short tangents. Long tangents are those where the driver can reach and maintain the desired speed. Moderate-length tangents allow the driver to accelerate and reach the desired speed, but not sustain it. Finally, short tangents do not allow the driver to accelerate to the desired speed before reaching the next geometric feature. Half of the sites had long tangent lengths and 27 percent had short tangent lengths.

The study site matrix in table 4 was designed to ensure that the effects of horizontal and vertical alignment could be separated during the statistical analysis. The data collection goal was to collect data at a minimum of 175 sites. Data were collected at 176 sites in the 6 States. Of the 176 sites, 32 were used to validate the regression model developed by TTI in the previous research (see chapter 7); 103 were used in the development of the regression models for this research (as discussed in this chapter); and 41 were allocated to the validation of the regression models developed in this research (see chapter 7). After the validation was completed, all sites were used to develop regression equations as discussed in chapter 8.

Table 4. Study Site Matrix.

Vertical Alignment (% Grade or Type of Curve)		**Horizontal Alignment Range of Radii (m)**					
		Tangent	**400 - 3,500**	**200 - 399**	**145 - 199**	**#144**	**Total**
0 to +5%		-	6/4/4[c]	6/4/4	6/2/4	6/2/3	24/12/15
>+5%		-	5/4/2	5/5/2	5/3/2	5/2/1	20/14/7
<-5%		-	5/2/2	5/3/3	5/3/2	5/3/1	20/11/8
-5 to 0%		-	6/2/4	6/4/4	6/2/4	6/3/5	24/11/17
Crest	NLSD[a]	5/2/0	6/5/2[d]	6/3/2	6/1/0	6/0/0	29/11/4
	LSD[b]	5/6/4	6/4/2	6/6/2	6/4/2	6/2/0	29/22/10
Sag		5/6/3	6/4/3	6/4/2	6/4/2	6/4/2	29/22/12
Total		15/14/7	40/25/19	40/29/19	40/19/16	40/16/12	175/103/73

[a]NLSD = non-limited sight distance (i.e., K$43 m/%)
[b]LSD = limited sight distance (i.e., K< 43 m/%)
[c]Desired Number of Sites / Actual Number of Sites for Development of Models / Actual Number of Sites for Validation of Models
[d]Combined horizontal and vertical curves with a maximum separation of 30 m between the points of inflection were studied.

Geometric Data

Several sources of geometric data were used during the study. Most of the data came from original construction plans obtained from the respective State departments of transportation. In Minnesota and Washington, the main source was computerized databases created by the States for the Highway Safety Information System (HSIS). Oregon also provided a similar computerized database. For these States, an effort was made to obtain the original construction plans to corroborate the databases. In New York, Pennsylvania, and Texas, geometric data were extracted from original construction plans.

The geometric data required for this study included information about horizontal curves, vertical curves, and the tangent preceding these curves. For a horizontal curve, the data obtained prior to the field visit included the degree of curvature (D), deflection angle (I), radius (R), length (L_H), grade (G), and milepoint or station of the beginning of the curve (PC); the point of intersection (PI); and the end of the curve (PT).

For vertical curves, the geometric data included the approach grade (G_1) and the departure grade (G_2), the length (L_V), and the milepoint or station of the beginning of the curve (VPC); the point of intersection (VPI); and the end of the curve (VPT). A note was also made to identify the vertical curve as crest or sag. The length (L_T) and grade (G_T) of the approach tangent to all curves were also recorded from the plans or the database. The beginning and ending milepost or station of the tangent were recorded.

On-site, the pavement, lane, and unpaved shoulder widths were measured. The cross-section elevations at the center of the horizontal curves (i.e., left, center, and right side of the pavement) were measured to calculate the superelevation rate. At all sites, the construction plans and/or the database were verified in the field to ensure that they were current and matched the existing roadway.

The clear zone may affect driver speed behavior; however, questions on whether the clear-zone dimension should change because an isolated tree or other object is present must be addressed when identifying the clear-zone distance. Therefore, Zegeer's seven-point roadside rating scale of roadside hazards was used to describe the roadway surroundings.[48] The rating scale classifies roadways that have no roadside hazards as a 1 and roadways with severe roadside hazards as a 7. Roadside hazard rating incorporates environment and clear zone into one descriptive parameter to represent driver perception of the roadway.

Researchers used photo logs and video recordings of the study sites to independently evaluate roadside features to determine a rating for each study location. The evaluation considered the proximity of roadside features to the travel lane and compared these roadside features to the picture scales developed in Zegeer's FHWA report.[48] Qualitative ratings were independently given by two researchers for each study site. The researchers then compared the individual ratings and, when a difference in value occurred, a consensus was reached after reviewing the photo and video recordings of the site.

Speed Data

The speed data were collected between July 1996 and January 1997 during daylight, off-peak periods, and under dry-weather conditions. At least 100 observations were taken at each site. Vehicle type was identified on site by observation or from the traffic counter by using the number and spacing of the axles. The speed data were collected using radar meters and on-pavement piezoelectric sensors connected to traffic counters/classifiers. These two types of speed-measuring devices were selected after performing a series of tests comparing different devices. The two selected devices provided speed measurements that were within 4 km/h of a fifth-wheel reading.

For horizontal curves, speeds were recorded at the center of the horizontal curve and at the midpoint of the preceding tangent. When the piezoelectric sensors were used, they were located as shown in figure 1. The sensors themselves were placed 3.05 m apart, and the traffic counter/classifier unit was concealed. When the radar meters were used, the data collectors were located as shown in figure 2.

When radar meters were used to record speeds, the operators were located where they could see the speed-measurement point. Furthermore, they were concealed as much as possible when taking speed readings to prevent drivers from being influenced. The four radar meters used in the study were modified by the manufacturer to collect speeds only when the trigger was pulled. This modification prevented their early detection by drivers using radar detectors.

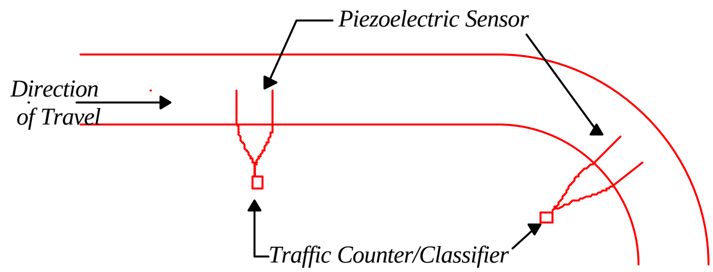


Figure 1. Location of Piezoelectric Sensors on Horizontal Curves.

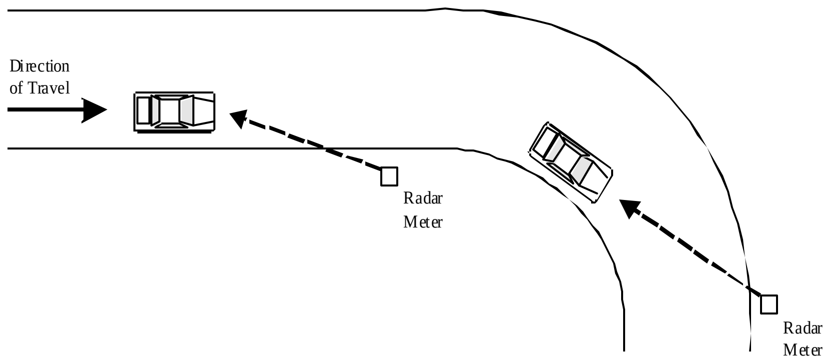


Figure 2. Location of Radar Meters on Horizontal Curves.

Speeds were measured at two points on sag and non-limited sight-distance crest vertical curves and at three points on limited sight-distance curves. The three points were the midpoint of the vertical curve, the minimum sight-distance point, and the midpoint of the preceding tangent. The minimum sight-distance point was defined as the point that had an elevation 1.07 m lower than the highest point on the vertical curve just prior to the crest of the curve (see figure 3). The height of 1.07 m corresponds to AASHTO's driver eye height. Radar meters were not used to collect speeds on limited sight-distance curves because of the complexity of placing the data collectors and identifying the minimum sight-distance point for speed measurements.

For combined horizontal and vertical curves, speeds were recorded at the midpoint of the horizontal curve and at the minimum sight-distance point of the vertical curve, if it was a limited sight-distance curve. Otherwise, the speeds were recorded midway between the horizontal curve point of intersection (PI) and the vertical curve point of intersection (VPI). In all cases, speeds were recorded at the midpoint of the preceding tangent.

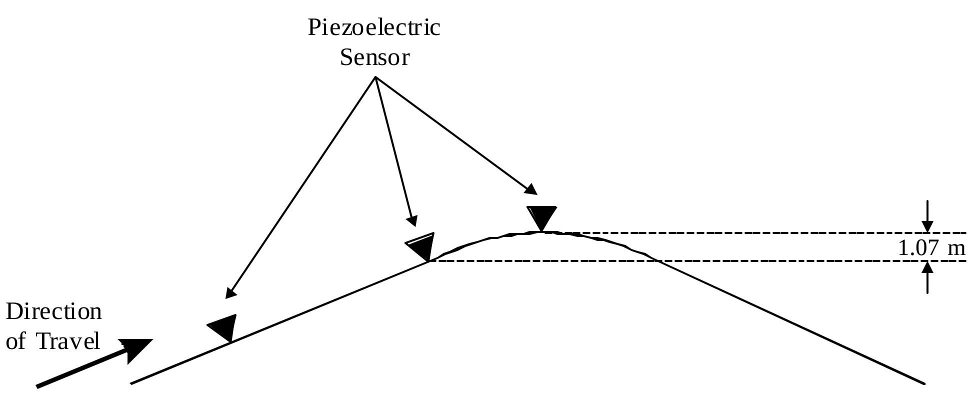


Figure 3. Location of Piezoelectric Sensors on Vertical Crest Curves With Limited Sight Distance.

Data Reduction

Initial preparation of the raw data files required the use of a spreadsheet to eliminate those vehicles that were not at free-flow speeds. Free-flow vehicles were determined using a minimum headway of 5 s between vehicles.[49] After removal of non-free-flow vehicles, speeds for passenger vehicles, trucks, and recreational vehicles were separated into three different categories based on the vehicle axle spacings. Limitations in the amount of data available for trucks and recreational vehicles made it more reasonable to do most of the analyses with the passenger vehicle data and later verify or modify the results for trucks and recreational vehicles.

The geometric data from the data collection sheets were entered into a spreadsheet. Due to the large number of sites, the data collection sheets were rechecked against the geometric data- base for errors in data entry. The errors were corrected when found. Additional preparation of the raw data files required an observational check of the data to ensure the reasonableness of the measured speeds and to convert the speeds from English units to metric units. Upon completion of the conversion, descriptive statistics were calculated using Statistical Analysis Software (SAS®). These statistics included mean speed, 85th percentile speed, standard deviation, skewness, kurtosis, and coefficient of variation. The descriptive statistics were merged with the geometric data to create a master file. The master data file included all the geometric variables considered as possible independent variables in this research.

SPEED-PREDICTION EQUATIONS FOR PASSENGER CARS ON HORIZONTAL AND VERTICAL CURVES

Table 5 presents illustrative examples of regression models that follow logically from the research hypotheses. The hypothesized regression models include only the radius of horizontal curvature (R) and the rate of vertical curvature (K). Radius is included as 1/R because that form has been used in previous regression models. Since K can be used to approximate the radius of a vertical curve, it is reasonable to use it in the same form as R. Using 1/R and 1/K in the regression equation allows the estimated speed values to reach a maximum as R and K increase, thus representing speeds on tangents. Other models that include additional independent variables (i.e., L_H, I, G, and e) were also considered.

All the models were tested for statistical significance. The purpose of the tests was to check if any of the regression parameters ($ß_i$) were not significantly different from zero and if their contribution was not significant to the estimation of speeds. The p-value reported for all tests is the probability of observing an extreme or more extreme value of the statistic when the null hypothesis (H_o) is true. The null hypothesis is the claim that is believed to be true or that is favored initially. The opposite claim that contradicts H_o is the alternative hypothesis termed H_a. The significance level (a) for all tests was set at 0.05. If the p-value was less than 0.05, then H_o was rejected.

Table 5. Hypothesized Regression Models.

Model to Estimate V_{85}	**Condition**	**H_o**	**H_a**
$V_{85} = \beta_0 + \beta_1 \frac{1}{R} + \beta_2 \frac{1}{K}$	Horizontal and vertical alignment act independently of each other, and their effect is linear and additive.	$\beta_i = 0$	$\beta_i \neq 0$
$V_{85} = \beta_0 + \beta_1 \frac{1}{R} + \beta_2 \frac{1}{K} + \beta_3 \frac{1}{R\,K}$	The effect of one depends on the value of the other and there is an interaction.	$\beta_i = 0$	$\beta_i \neq 0$
$V_{85} = \beta_0 + \beta_1 \frac{1}{R}$	Horizontal alignment influences speed and the vertical has no marginal effect and can be ignored.	$\beta_i = 0$	$\beta_i \neq 0$
$V_{85} = \beta_0 + \beta_1 \frac{1}{K}$	Vertical alignment influences speed and the horizontal has no marginal effect and can be ignored.	$\beta_i = 0$	$\beta_i \neq 0$

PRELIMINARY MODEL DEVELOPMENT

The speed data were tested for normality using the Shapiro-Wilk test. The Shapiro-Wilk test is suitable for analyzing speeds since speeds have a continuous distribution.[50] Also, stem-and-leaf plots and normal probability plots were created to check the normality of the speed data. The normality test results indicated that at many of the sites, speeds did follow a normal distribution. Those sites that failed the test were checked by observing the normal probability plots. In most cases, the plots indicated a normal distribution. As a result, the data were assumed to be normal.

In the preliminary analyses, several methods were used to identify possible relationships between the independent variables and the 85th percentile speed. Using scatter plots, correlation matrices, and other criteria, a list of tentative variables to be included in the regression models were identified. The next step in developing the regression models was to identify the variables with a higher explanatory value by performing linear regression analysis.

In order to identify the most promising combinations of independent variables, an all-possible-regressions selection procedure was used. The all-possible-regressions selection procedure analyzes all possible combinations of variables presented. The purpose of this approach is to identify the best regression models according to specified criteria. After the best models were selected, more detailed tests were performed on these models. A combination of two criteria was used to aid in selecting the best-fitting models: the adjusted coefficient of multiple determination (R_a^2) and the C_p statistic.[50-51]

The adjusted coefficient of multiple determination (R_a^2) adjusts the commonly used coefficient of multiple determination (R^2) by accounting for the number of parameters in the regression model. The R^2 measures the proportionate reduction of total variation in the dependent variable associated with using a particular set of independent variables.[51] High values of R^2 are usually related to good regression models; but R^2 does not account for the number of parameters in the regression equation. Actually, as the number of parameters increases, so does the value of the coefficient of multiple determination. R_a^2 is a better criterion to observe because it includes consideration of the increase in R^2 caused by additional variables. R_a^2 can have values between 0 and 1, with higher values of R_a^2 being desirable.

The second criterion was the C_p statistic. For this criterion, the goal is to obtain small values that are close to the number of parameters (p) in the regression equation. The mean squared error (MSE) has two components: a bias error and a random error.[51] It is always desired that the bias component be close to zero. Small values of C_p translate into a small MSE and small bias in the regression model.[51]

A model that shows a high R_a^2- and a C_p-value near the number of parameters in the equation would favorably explain the variability of the dependent variable and also show little bias. Models with low R_a^2- and high C_p-values may be biased and would not explain the sources of variability in the dependent variable. Using the R_a^2 and C_p criteria, the best combination of variables was identified for each alignment combination as shown in table 6. At this stage, no conclusions were drawn about the statistical significance of the variables or the models. The objective was to identify the combinations of variables that resulted in the higher values of R_a^2 and values of C_p close to the number of parameters.

The variables identified in the previous step were used in regression models for further analysis. In each model, a t-test was used to test the statistical significance of each variable. The null hypothesis for the t-test was that a specific parameter estimate was equal to zero. Diagnostic tests were conducted to identify collinearity among independent variables and influential observations. The variance inflation factor (VIF) was used to test for collinearity. The VIF measures how much the variances of the estimated regression coefficients are inflated as compared to when the independent variables are not linearly related.[51-52] The largest or maximum VIF value is an indicator of collinearity if it exceeds 10.[51]

No single measure can be used to identify an observation as being influential. Therefore, the decision to delete an observation was based on studying several statistical measures and deciding if the observation is really a result of an error and does not belong in the database. The main statistical measures used to detect influential observations in this research were Cook's distance (Cook's D), studentized residuals (RSTUDENT), and the hat matrix (Hat Diag H).

Table 6. Independent Variable Combinations to Estimate V_{85}.

Alignment Combination	Available Variables	Variables Selected	R_a^2	C_p	n
Horizontal Curves on Grades	1/R, L_H, e, e/R, G	1/R, e, G	0.66	1.25	48
Vertical Crest Curves on Horizontal Tangents	1/K, L_V	1/K	0.03	1.12	8
Vertical Sag Curves on Horizontal Tangents	1/K, L_V	1/K, L_V	0.54	3.00	6
Horizontal Curves Combined With Crest Vertical Curves	1/R, L_H, 1/K, e, e/R	1/R, L_H, 1/K, e	0.50	4.33	25
Horizontal Curves Combined With Sag Vertical Curves	1/R, 1/K, e, e/R	1/R	0.95	0.05	16

Cook's D measures the change to the predicted value that results from deleting each observation.[52] If the Cook's D is 0.50 or more, then the observation may be too influential.[53] Studentized residuals are the residuals obtained from the regression analysis divided by an unbiased estimator of the variance.[51] RSTUDENT values greater than the absolute value of 2 indicate that an observation may be too influential.[52-53] Finally, the hat matrix refers to the diagonal elements of the least-squares projection matrix used to determine the predicted values.[53] An observation may be too influential if the Hat Diag H value exceeds 2p/n, where p is the number of parameters in the model and n is the number of observations.[52-53]

Horizontal Curves on Grades

For horizontal geometry on level and mild grades, the principal independent variable has generally been radius. Therefore, the first plots were of the 85th percentile speed at the center of the curve against the radius and two transformations of the radius. The transformations were the inverse of the radius (1/R) and the square root of the radius ($R^{1/2}$). The inverse of the radius has been significant in regression equations previously developed in the United States.[7,33] These three scatter plots are shown in figures 4 through 6.

From the scatter plots, it can be seen that as R increases, the 85th percentile speed increases and seems to level with radii larger than 300 m to 400 m. The plot of 1/R shows a linear relationship with speed. The scatter plot of $R^{1/2}$ is similar to the plot of R. Horizontal curve length, grade, deflection angle, and superelevation were also plotted against the 85th percentile speed, but failed to show obvious relationships.

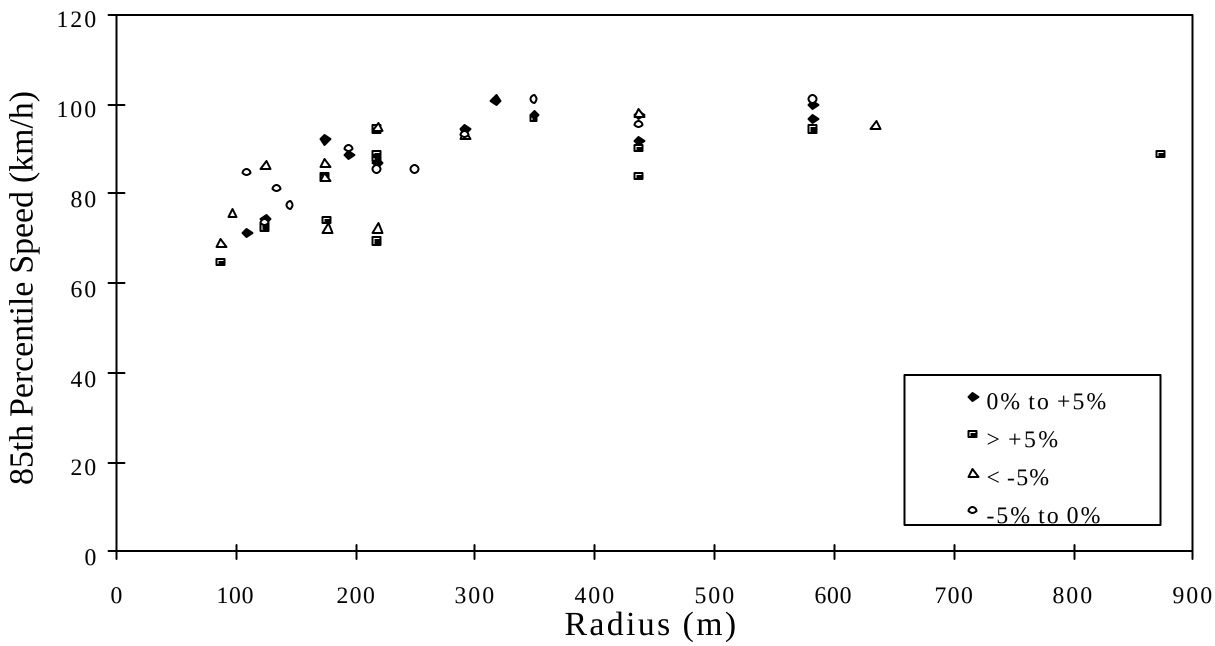


Figure 4. Horizontal Curves on Grades: V_{85} Versus R.

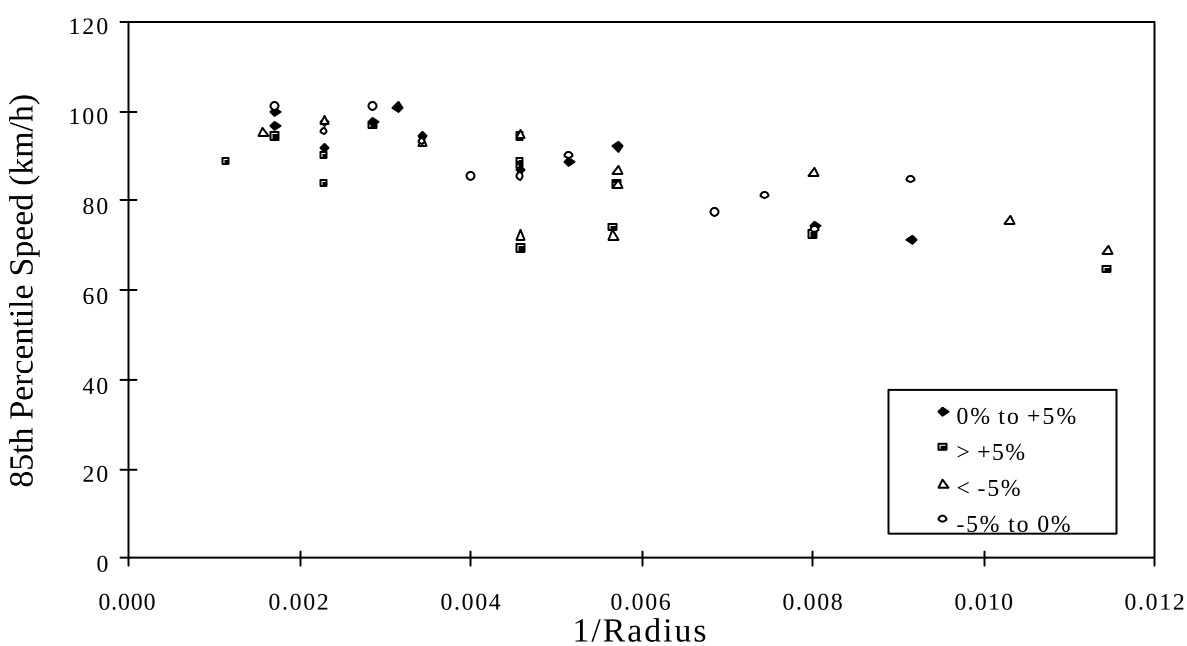


Figure 5. Horizontal Curves on Grades: V_{85} Versus 1/R.

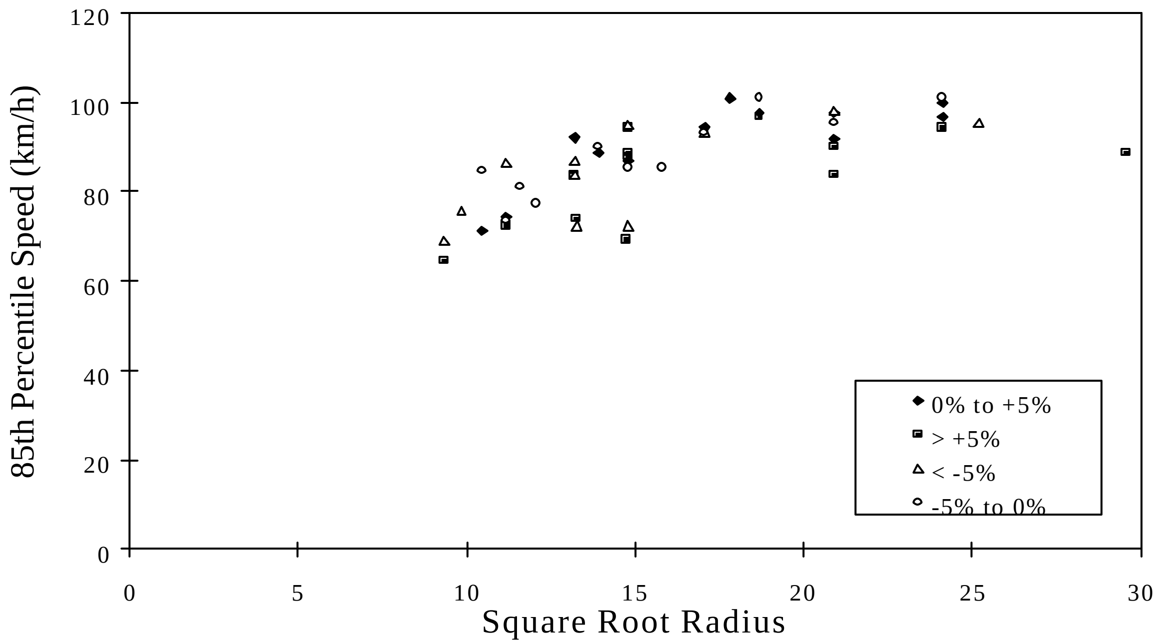


Figure 6. Horizontal Curves on Grades: V_{85} Versus $R^{1/2}$.

The correlation analysis for horizontal curves on grades used 48 study sites. Among the three forms of radius, 1/R had the strongest correlation with V_{85}. The length (L_H), deflection angle (I), and the interaction of superelevation and radius (e/R) had statistically significant degrees of association with V_{85}. The deflection angle was not included for further analysis because it is highly correlated to the radius. Meanwhile, the grade (G) and superelevation did not show a statistically significant correlation with V_{85}. All three transformations of R were significantly correlated to e.

Based on the results from the scatter plots and the correlation matrix, the following independent variables were tested in the regression analysis: 1/R, e, e/R, L_H, and G. Although e was not highly correlated at this stage of the analysis, it was necessary to include it in order for e/R to be used. Grade was selected because it was thought to be too important to delete without further consideration. From this list, variables were deleted as the statistical analyses progressed. The inverse of the horizontal radius was included as the main independent variable, which agrees with previous models suggested in the United States.[(7,33)] The length was included because it was part of the equation developed by Krammes et al., and it showed a statistically significant correlation to V_{85}.[(7)]

The regression model fitted for horizontal curves on grades included the independent variables 1/R, e, and G. This model and some of the analysis results are summarized in table 7. The model had an R^2-value of 0.68.

Table 7. Preliminary Parameter Estimates of the Regression Equation for Horizontal Curves on Grades.

Variable	Parameter Estimate ($ß_i$)	p-Value
Intercept	99.28	0.0001
1/R	-3390.65	0.0001
e	56.10	0.0149
G	-0.34	0.0691

The t-test indicated that grade was not significant; however, its p-value was very close to 0.05. Therefore, an F-test was conducted to check if regression equations without the variable G_1 from horizontal curves on upgrades and horizontal curves on downgrades had different parameter estimates. SAS® was used to conduct this test using indicator variables and the conclusion was that a significant difference exists between the two groups.

After considering the results from all the tests, separate models were fitted using 1/R and e for the horizontal curves on upgrades and horizontal curves on downgrades. The models failed to show e as a significant variable, and it was eliminated from both equations.

Horizontal Curves on Upgrades

The equations for horizontal curves on upgrades were tested to check if there was a difference between the effect of upgrades greater and less than 4 percent. The continuous variable G was not statistically significant; however, it was tested as a blocking factor because it was considered important. The F-test shows a significant difference between the mild and steeper grades. The test results indicate a significant difference between the intercepts, but not between the slopes of the equations. In the case of grades greater than 4 percent, the regression line is shifted down (i.e., speeds are lower). This effect was expected.

The final check on the regression equations was a test for the effect of cross-section using pavement width as the independent variable. Pavement width was defined as the distance from one edge of the pavement to the other. It includes lane widths plus paved shoulders widths. For each grade condition, pavement widths were divided into those less than or equal to 8.5 m and those greater than 8.5 m. An F-test was used to conduct this comparison using indicator variables to identify each category of pavement width. The width of 8.5 m was selected because it was approximately the median value for the pavement widths. For both horizontal curves on grades less than 4 percent and horizontal curves on grades greater than or equal to 4 percent, there was no significant difference between the regression lines that could be attributed to pavement width.

Horizontal Curves on Downgrades

Horizontal curves on downgrades were tested in a similar way to the upgrades. The downgrades were tested to see if there was a statistical difference between downgrades greater than or equal to -4 percent and downgrades less than -4 percent. The test on 22 observations indicated that there was no significant difference between the two categories of downgrades. Therefore, only one regression equation was developed.

Vertical Crest and Sag Curves on Horizontal Tangents

Crest vertical curves on horizontal tangents were divided into those with limited sight distances (LSD) and those with non-limited sight distances (NLSD). Since this study only collected data at eight sites, data collected during the National Cooperative Highway Research Program (NCHRP) Stopping Sight Distance (SSD) study that were collected in the States used in this research were added.[36] The additional sites were from two-lane rural roads with shoulders or with narrow shoulders in Texas and Washington. The additional data were collected under similar conditions and reflected speeds of passenger vehicles only. The data were added to help identify relationships between 85th percentile speeds and the independent variables. Two sites were added to the NLSD data and 16 to the LSD data.

The rate of curvature (K) was identified in the literature review as the primary independent variable for crest vertical curves. The correlation matrix for crest vertical curves on horizontal tangents did not reveal any statistically significant relationships between the independent variables and V_{85}. The strongest correlations with V_{85} were from 1/K and L_V. Because these variables are of engineering value, they were included in additional analyses. The rate of curvature was plotted against the 85th percentile speed as 1/K in figure 7. The scatter plot does not show a clear relationship between the variables. The same lack of relationships was found for the other variables.

Sag curves on horizontal tangents were treated in a similar fashion to the crest curves. Only six sites were available for the analysis of these curves. From the literature review, the rate of curvature (K) and length (L_V) were identified as important independent variables. The rate of curvature was plotted against the 85th percentile speed as 1/K in figure 8. As with the crest curves, the scatter plot does not show a clear relationship between the variables. The lack of relationship is repeated with all the other variables. Six observations were insufficient to show a relationship on a scatter plot.

None of the tested variables had a statistically significant correlation with V_{85}; however, G_1 had a statistically significant correlation with 1/K. Therefore, the decision was made to include only 1/K and not G_1. Two variables were selected for further analysis: 1/K and L_V. They were selected because of their importance from an engineering point of view.

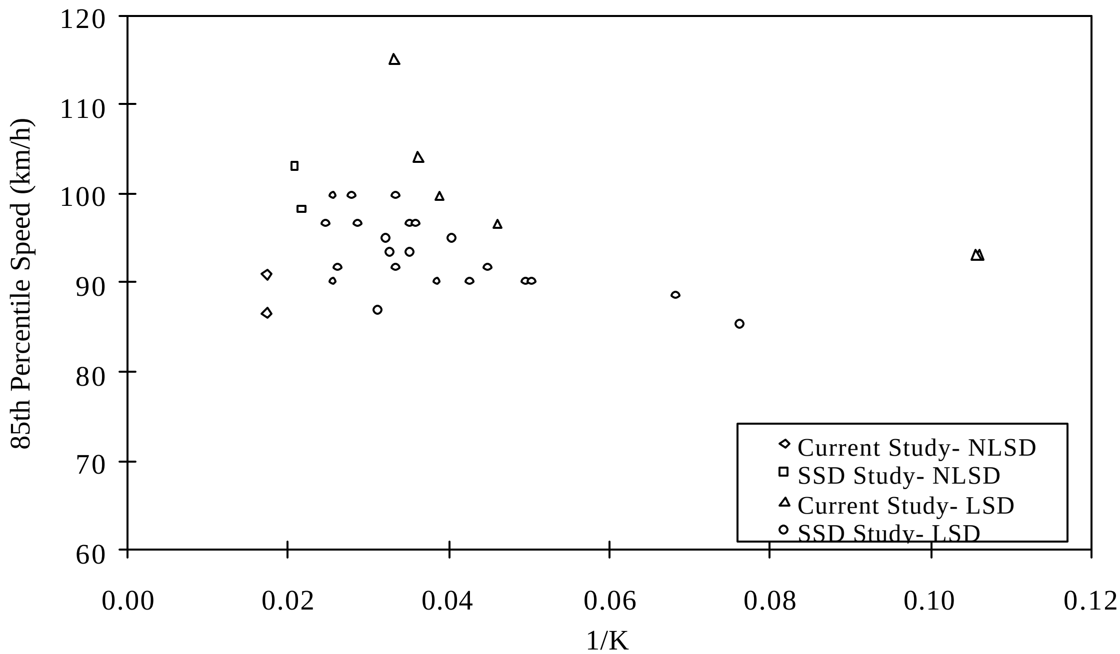


Figure 7. Vertical Crest Curves on Horizontal Tangents: V_{85} Versus 1/K.

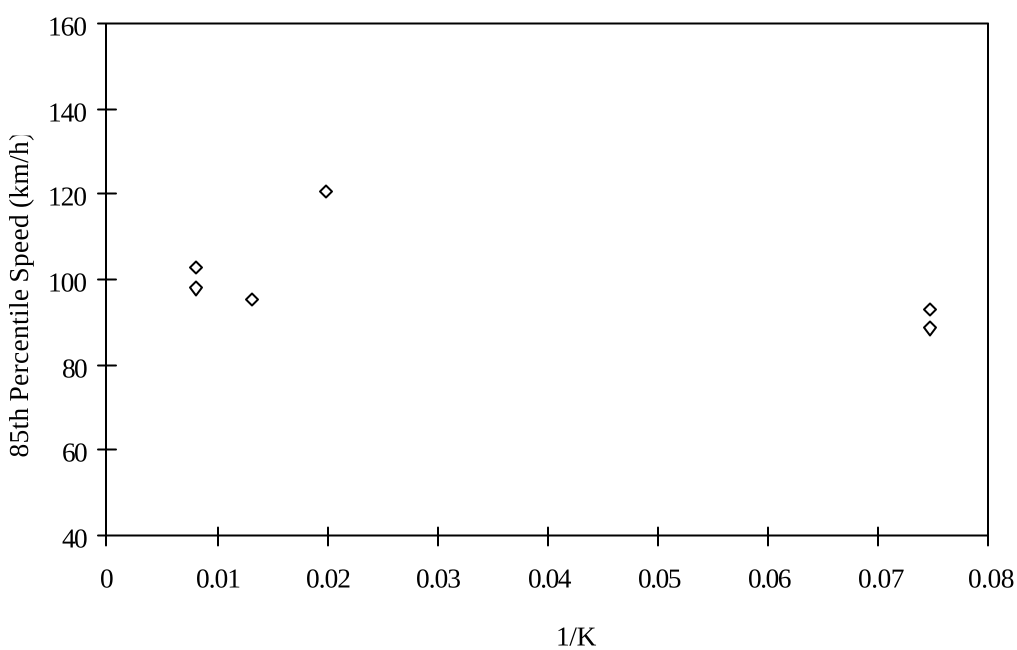


Figure 8. Vertical Sag Curves on Horizontal Tangents: V_{85} Versus 1/K.

The vertical curves on horizontal tangents failed to show statistically significant relationships between the independent variables and the 85th percentile speed. Other factors, such as the environment, may affect the selection of speeds, but since they were not the primary controls, their effect may be hidden. For both types of vertical curves, 1/K was used as an independent variable. Sag curves also had L_V as an independent variable.

In the case of crest curves, an attempt was made to develop an equation that included non-limited (i.e., K > 43 m/%) as well as limited sight-distance curves (i.e., K # 43 m/%). The results were not statistically significant, and it was decided to separate the non-limited from the limited sight-distance curves. With the data available (six observations), it was not possible to identify any relationships for the non-limited sight-distance curves. A useful equation was obtained for the limited sight-distance curves.

The limited sight-distance data produced the results listed in table 8. Since 1/K was not statistically significant for a = 0.05, the same model was fitted to the data collected in the SSD study. The model developed using that data was statistically significant as indicated in table 8. An F-test that compared both regression equations indicated that the equations have significantly different intercepts, but the slopes are not statistically different. The difference in intercept may be because the SSD data were collected prior to the elimination of the national maximum speed limit. Since the equations were similar, it was decided to accept the equation developed with the data from this research as a good equation, even though the data used to develop it were limited. The R^2 for the regression equation was 0.54.

For sag curves, the elimination of L_V from the equation did not make 1/K significant. In the case of sag curves, there was one influential observation that was the only one of the six sites with a speed limit of 112.7 km/h. If this observation is removed, the number of observations is reduced to five and the equation changes to include only 1/K with a smaller R^2, but the p-value for 1/K is closer to being significant. This model is not statistically significant, but its form was included in the validation to check its significance.

Table 8. Parameter Estimates of the Regression Equation for Crest Curves With Limited Sight Distance (i.e., K# 43 m/%) on Horizontal Tangents.

	Variable	Parameter Estimate ($ß_i$)	p-Value
New Data	Intercept	111.07	0.0001
	1/K	-175.98	0.0941
Data Collected in the SSD Study	Intercept	101.83	0.0001
	1/K	-219.82	0.0008

Horizontal Curves Combined With Vertical Curves

Separate correlation matrices were prepared for horizontal curves combined with crest vertical curves and horizontal curves with sag vertical curves. The correlation matrix for these combined curves included the variables that showed a statistically significant degree of association with the 85th percentile speed in the horizontal and vertical correlation matrices.

The association between the independent variables and the 85th percentile speed for the combined horizontal and vertical crest curves showed similarities with the individual horizontal and vertical curves. Once again, 1/R showed a strong correlation to V_{85} and 1/K was on the borderline of being statistically significant. Therefore, it was decided to use 1/R, 1/K, L_H, e, and e/R in the regression analysis of horizontal curves combined with crest curves. For horizontal curves combined with sag curves, 1/R, 1/K, e, and e/R were selected for the regression analysis. The variable 1/K was included because it was considered important even though it was not correlated significantly with V_{85}.

The estimation of operating speeds on combined horizontal and vertical curves was one of the primary objectives of this research. The horizontal curves combined with crest curves were fitted with a regression model using 1/R, 1/K, e, L_H, and e/R. For these conditions, the regression analysis results are summarized in table 9.

The p-values indicated that 1/R was the only significant variable. Therefore, a model using only 1/R was fitted. The model, listed in table 10, has an R^2 of 0.43. One observation was found to be influential from the combination of statistical measures used to check for outliers. The particular observation was removed from the analysis, but it was later returned to the database because there was no indication that it was the result of an error in the data collection, and the elimination of the variable from the analysis did not change the regression parameters significantly.

Table 9. Preliminary Parameter Estimates of the Regression Equation for Horizontal Curves Combined With Crest Vertical Curves.

Variable	Parameter Estimate ($ß_i$)	p-Value
Intercept	99.30	0.0001
1/R	-3647.51	0.0029
1/K	-81.44	0.1068
e	79.68	0.2710
L_H	0.02	0.1957

Table 10. Parameter Estimates of the Regression Equation for Horizontal Curves Combined With Crest Vertical Curves.

Variable	Parameter Estimate ($ß_i$)	p-Value
Intercept	107.70	0.0001
1/R	-3780.59	0.0004

The low value of R^2 raised the concern that some other factor was affecting speeds. Therefore, a test was conducted to examine the influence of limited sight distance on the vertical component of the curve. The test indicated that there was a statistically significant difference between horizontal curves with limited sight-distance crest curves and horizontal curves with non-limited sight-distance crest curves. The results prompted the development of separate regression equations for the two sight-distance conditions. For the horizontal curves with non-limited sight-distance curves, a statistically significant regression equation could not be obtained with any of the variables tried. Therefore, it was decided to exclude non-limited sight-distance curves from further analysis. Note that data were only available for the larger horizontal radii; a relationship may be present, but it cannot be determined with available data. The combination of large horizontal radii and non-limited sight-distance crest curves may be interpreted by drivers as a geometry that is not severe enough to require reduced speeds.

For the horizontal curves with limited sight-distance crest curves, the regression analysis resulted in a statistically significant equation that is listed in table 11. The R^2 for this equation was 0.78 and the MSE was 3.95, and it was developed from the remaining 16 observations. Even though 1/K was not statistically significant as an independent variable in any of the suggested equations, it was considered beneficial to check its significance again during the validation phase.

Table 11. Parameter Estimates of the Regression Equation for Horizontal Curves Combined With Limited Sight-Distance (i.e., K# 43 m/%) Crest Vertical Curves.

Variable	Parameter Estimate ($ß_i$)	p-Value
Intercept	101.90	0.0001
1/R	-3283.01	0.0001

The final test for these curves was to check for cross-section effects. The test was conducted only on the horizontal curves with limited sight-distance crest curves. In this situation, there was a statistically significant difference between pavement widths greater and less than the median width of 8.5 m. It was the only occasion throughout the research that such a difference was found. This finding resulted in the estimation of two new regression equations (See table 12 for the parameter estimates.) The R^2 for pavement widths less than or equal to 8.5 m was 0.87 using a total of nine observations. The MSE was 3.57. For pavement widths greater than 8.5 m, seven observations were used and the R^2 was 0.86 and the MSE was 2.89. These equations are presented here for completeness of the analysis, but are not included in the following sections because they were not considered to be of practical significance from the engineering point of view.

The horizontal curves combined with sag curves were fitted with a regression model using 1/R. This model tends to indicate that this alignment combination may be controlled by the horizontal features. To finalize the analysis of horizontal curves combined with sag curves, the effect of the cross-section was tested. The test found that there was no significant difference that could be attributed to pavement width.

Table 12. Parameter Estimates of the Regression Equation for Horizontal Curves Combined With Limited Sight-Distance (i.e., K# 43 m/%) Crest Vertical Curves and Different Pavement Widths.

Pavement Width (m)	Variable	Parameter Estimate ($ß_i$)	p-Value
# 8.5 m	Intercept	106.18	0.0001
	1/R	-4580.31	0.0002
> 8.5 m	Intercept	99.15	0.0001
	1/R	-2474.64	0.0026

Horizontal Curves on Grades

The horizontal curves on grades have three equations describing the three possible conditions that affect the speeds of passenger vehicles. Figure 9 shows plots of the three equations. Speeds on level or mild upgrades (0% # G < 4%) are higher than on steeper upgrades (4% # G < 9%) or downgrades (-9% # G < 0%). Speeds on steep upgrades are lower than on mild upgrades because as the grade increases, so do the grade resistance forces. On the other hand, speeds on downgrades could be higher than on level grades since the force is acting in the direction of movement. Drivers, though, probably use their brakes on downgrades approaching a horizontal curve to prevent losing control of the vehicle.

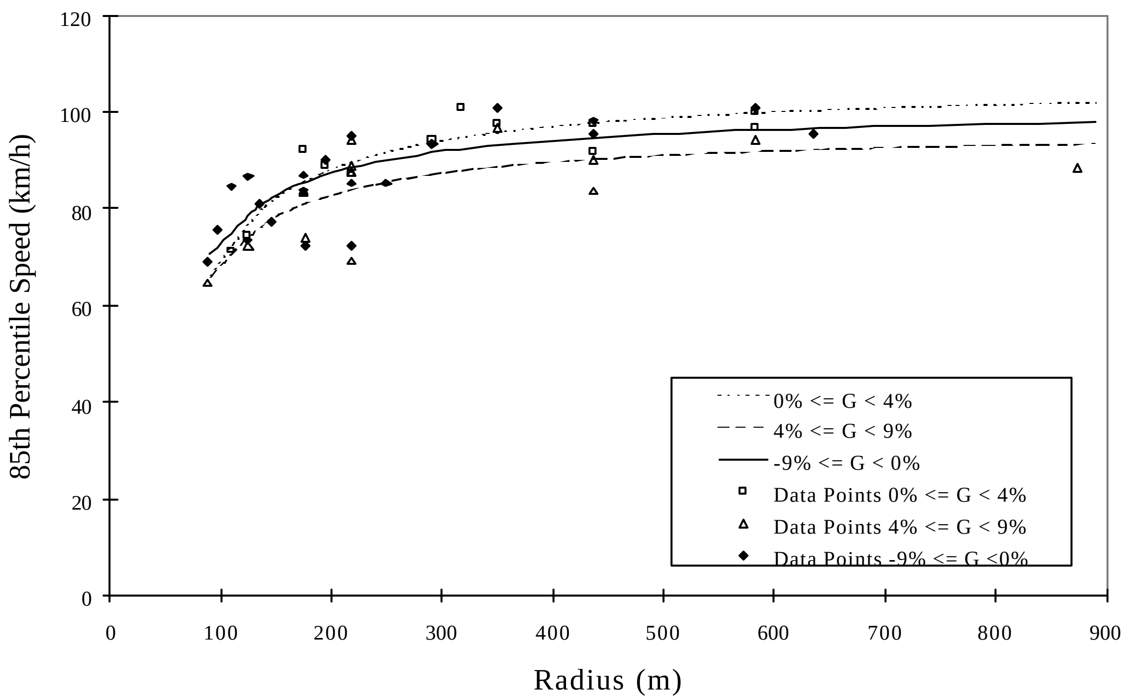


Figure 9. Estimated V_{85} Versus Radius: Horizontal Curves on Grades.

Two interesting points are noticeable in figure 9. The first point is that after the radius exceeds approximately 400 m, the 85th percentile speed does not appear to be affected by the radius. Furthermore, when the radius is approximately 800 m, the 85th percentile speed is close to the assumed operating speed for long tangents that was estimated as 97.9 km/h. A radius of 400 m is greater than most of the minimum radii for a design speed of 120 km/h in the American Association of State Highway and Transportation Officials' (AASHTO's) *A Policy on Geometric Design of Highways and Streets.*[2] Therefore, it could be argued that when the radius is greater than approximately 800 m, the grade, and not the radius, is the controlling factor in the selection of operating speeds for horizontal curves on grades.

The other aspect that should be noticed is that for radii less than approximately 250 m, the 85th percentile speeds drop sharply regardless of the grade. The reduction is approximately 2 km/h for every 10 m of reduction in radius. The radius of 250 m coincides with the minimum radius for a design speed of 90 km/h found in AASHTO's *A Policy on Geometric Design of Highways and Streets.*[2]

Horizontal curves on mild upgrades have been the principal subject of research in geometric design in the United States. In the literature review, some of the models developed previously were presented, and it was reasonable to compare those models to the model developed in this research. Figure 10 shows the simple linear regression models developed by Lamm et al., Ottesen, Glennon et al., and the new model suggested.[19,32-33] It is interesting to note that all the linear models with 1/R as the independent variable show the same sharp drop in speeds for radii less than approximately 250 m. They also show a leveling on speeds when the radius exceeds approximately 800 m. All the models have similar characteristics, but it was not possible to compare them statistically due to the lack of statistical information from the previous studies. All of the models have reported R^2-values around 0.82. Ottesen's model is approximately 2 km/h below the suggested model. These similarities are encouraging since different researchers have found similar equations for similar conditions. The differences between the equations may be related to the year that the studies were conducted. The present study is the only one conducted after the elimination of the 88.5-km/h national speed limit, although most of the study sites were on roadways with 88.5-km/h speed limits.

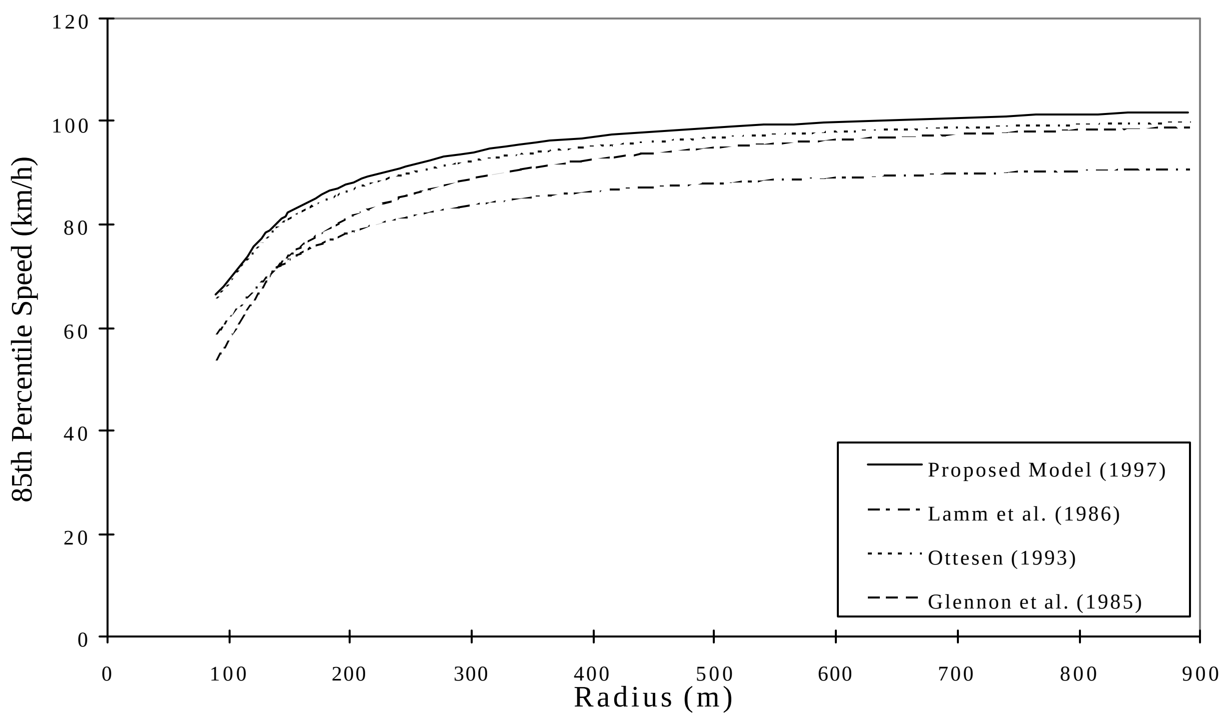


Figure 10. Estimated V_{85} Versus Radius: Horizontal Curves on 0- to 4-Percent Grades.

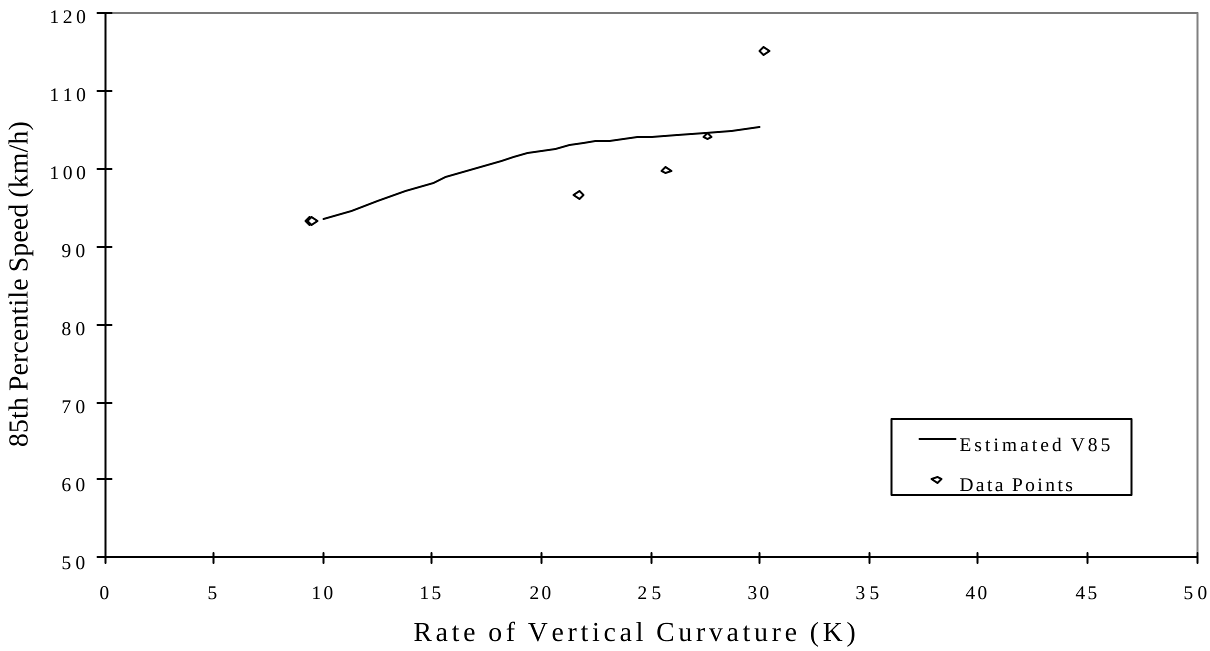


Figure 11. Estimated V_{85} Versus K: Limited Sight Distance Crest Vertical Curves.

Given the suggested equations, it was expected that the curve speeds would become asymptotic to the 85th percentile speed on long tangents as R increased. Krammes et al. reported that they were unable to develop a statistically significant equation for predicting 85th percentile speeds on long tangents.[7] They calculated the average 85th percentile speed for long tangents to be 97.9 km/h. A separate task of the present research confirmed that value. Therefore, it was decided to substitute that value in the equations developed in this research for horizontal curves on grades, and solve them for R. On mild upgrades, a radius of approximately 450 m or more would result in speeds of 97.9 km/h or more. Krammes et al. also found that the speeds on horizontal curves with radii greater than 450 m were not significantly different from speeds on long tangents.[7] For steeper upgrades, the speed of 97.9 km/h would never be reached. Finally, on downgrades, the radius would have to be greater than 915 m to attain 97.9 km/h.

For passenger vehicles, the average 85th percentile speed for long tangents may be used as an estimate for the 85th percentile speed on horizontal curves on mild upgrades when the radius is equal to or exceeds 450 m. Horizontal curves on steep upgrades may never approach the average 85th percentile speed on long tangents; however, if the radius is larger than 800 m, the change in speed would be approximately 3 km/h or less. For horizontal curves on downgrades, radii larger than 915 m may not influence the speed on downgrades.

Vertical Curves on Horizontal Tangents

Operating speeds on vertical curves on horizontal tangents were related to 1/K. For crest curves, only curves with limited sight distances showed a statistically significant relationship between 1/K and the 85th percentile speed. Figure 11 shows the estimated values of the 85th percentile speed using the developed regression equation. Because of the limited number of data points available, the shape of the regression curve was selected using the findings from the NCHRP SSD study and the findings from the sag curves in this project.[36]

According to AASHTO's *A Policy on Geometric Design of Highways and Streets*, a vertical crest curve with a design speed of 90 km/h requires a K-value of at least 43.[2] From figure 11, it can be seen that the estimated 85th percentile speed for a K-value of 43 could be greater than the design speed. The fact that the estimated operating speeds could be greater than the design speed would confirm the findings by McLean and Krammes et al. for horizontal curves, and Fambro et al. for crest vertical curves.[20,7,36] All have reported that operating speeds are higher than design speeds at least for design speed values less than or equal to 90 km/h.

The average 85th percentile speed on long tangents was used to calculate K with the equation suggested for crest curves on horizontal tangents. It was thought that since the equation had 1/K as the independent variable, the estimated speed would approach the V_{85} for long tangents as K increased. The calculated K-value for a speed of 97.9 km/h was approximately 14. A K-value of 14 is recommended by AASHTO for a design speed of 60 km/h based on stopping sight distance.[2] Using the guidance from Fambro et al. for a K-value of 14, the design speed would be approximately 65 km/h.[36] The recommended design speed is lower in both cases than the estimated speed. It is interesting that Fambro et al. also report K-values to meet the comfort criteria on vertical curves assuming a vertical acceleration of 0.03 g's.[36] When the comfort criteria is used, for a K-value of 14, the design speed is approximately 40 km/h.[36] It could be argued that drivers may feel comfortable driving at the average V_{85} of tangents while on crest vertical curves with K-values greater than 14.

The operating speeds on sag curves on horizontal tangents had a statistically significant relationship with 1/K also. Figure 12 shows a graph of the estimated 85th percentile speed versus K. Again, the estimated 85th percentile speed is greater than the inferred design speed from AASHTO's *A Policy on Geometric Design of Highways and Streets.* For example, a design speed of 90 km/h requires a K-value between 30 and 40.[2] Both values of K result in estimated 85th percentile speeds that are higher than the design speed. If a speed of 97.9 km/h is substituted for the estimated V_{85} in the equation, the resulting K-value is approximately 55. According to AASHTO, for a K-value of 55, the design speed would be approximately 110 km/h.[2] For a K-value of 55, Fambro et al. suggest a design speed of approximately 70 km/h for headlight considerations, or in excess of 120 km/h for comfort considerations.[36] It would seem logical to assume that for sag curves on horizontal tangents, K-values higher than 55 do not affect the operating speed of drivers at least during daylight conditions. Indeed, drivers could attain speeds on these curves similar to those on long tangents.

Combination of Horizontal and Vertical Curves

Two equations were developed for horizontal curves combined with vertical curves. One of the equations was for horizontal curves combined with limited sight-distance crest curves and one for horizontal curves combined with sag curves. For the crest curves, a statistically significant equation was possible only for the limited sight-distance curves. It is presumed that any effect of radius on horizontal curves combined with non-limited sight-distance crest curves was undetectable because the data available had radii larger than 250 m. Fambro et al. had reported that on crest curves with limited sight distances and shoulders less than 1.8 m, the 85th percentile speed tends to increase linearly as the K or inferred design speed increases.[36] Those results were confirmed in this research.

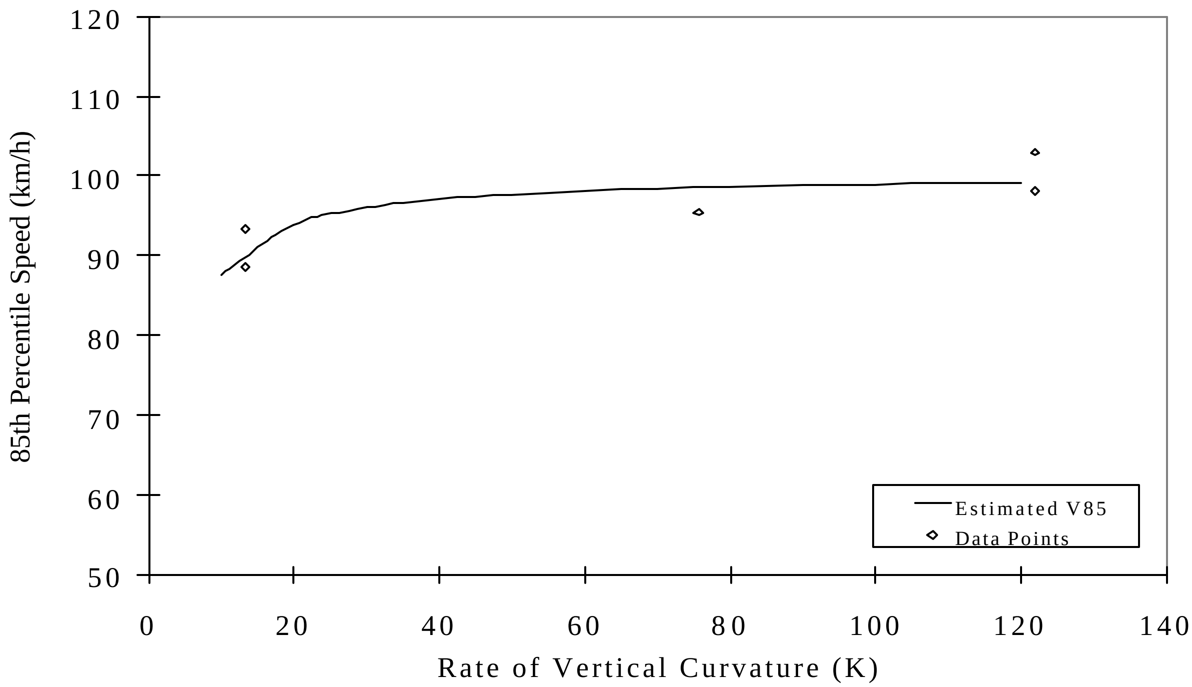


Figure 12. Estimated V_{85} Versus K: Vertical Sag Curves.

If the average 85th percentile speed on long tangents of 97.9 km/h is used in the suggested equation for horizontal curves combined with limited sight-distance crest curves, the resulting radius is approximately 820 m. This result is significant because this value is similar to what AASHTO recommends for horizontal curves with design speeds of 120 km/h. Radii larger than 800 m were identified as not affecting the speeds on horizontal curves on grades, and vertical curves with K-values larger than 14 had speeds similar to the average operating speed for long tangents. Therefore, it could be presumed that when there is a combination of a horizontal curve with a radius equal to or larger than

800 m and a vertical curve with a K-value of 14 or greater, drivers are able to reach the average operating speed observed on long tangents.

Horizontal curves combined with sag curves had a regression equation with radius as the only significant independent variable. The operating speeds on these curves were very similar to the operating speeds on horizontal curves on level or mild upgrades. Figure 13 shows the speeds estimated with both equations. An F-test was performed to compare both equations. The equations were not statistically significantly different, and a single equation was developed. The equation was developed with 28 observations and had an R^2 of 0.92 and an MSE of 2.84. The summarized results are listed in table 13.

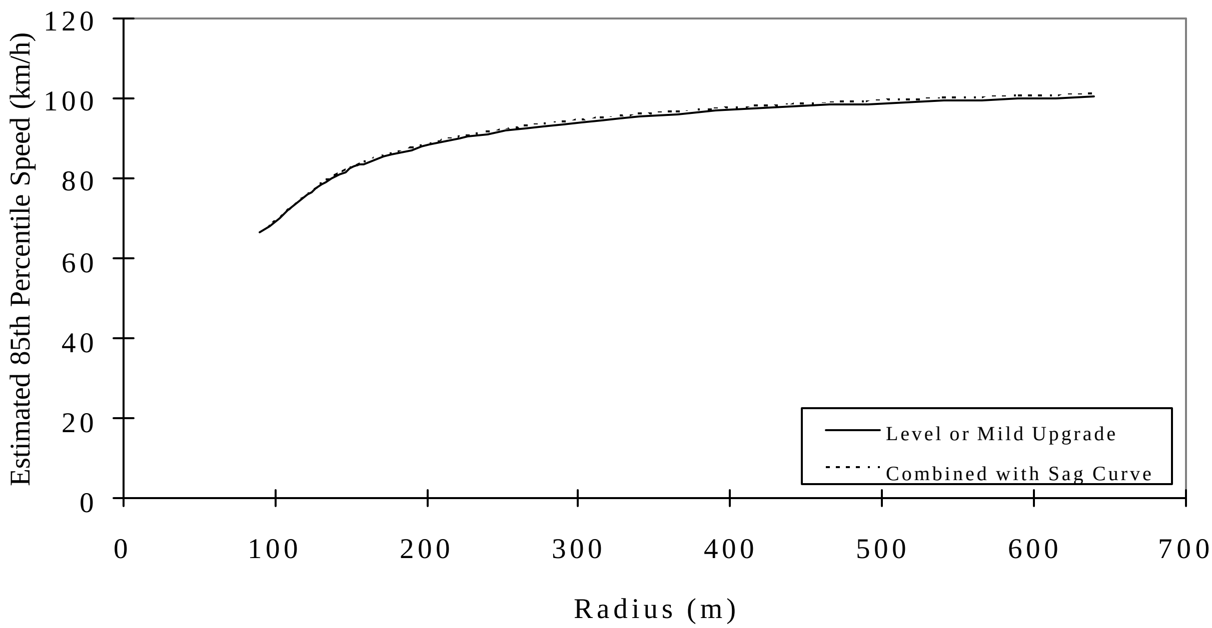


Figure 13. Estimated V_{85} Versus R: Horizontal Curves With Different Vertical Alignment Conditions.

Table 13. Parameter Estimates of the Regression Equation for Horizontal Curves on Level or Mild Upgrades (i.e., 0% # G < 4%) or Horizontal Curves Combined With Sag Vertical Curves for Passenger Vehicles.

I	Variable	Parameter Estimate ($ß_i$)	p-Value
0	Intercept	106.30	0.0001
1	1/R	-3595.29	0.0001

In horizontal curves combined with sag curves, the driver has generally an unlimited sight distance, and the main concern may be comfort. The driver's perception of the sharpness of the curve may not be affected by the presence of the sag curve. Furthermore, the driver may feel that there is no large difference in the forces acting on the vehicle. It could be concluded that drivers maneuver horizontal curves combined with sag curves the same as they would horizontal curves on level or mild upgrades.

The average 85th percentile speed on long tangents was used to calculate the radius of the suggested equation. The average operating speed on long tangents was determined as the mean of the observed 85th percentile speeds on long tangents.[(7)] For a speed of 97.9 km/h, the radius is approximately 430 m. The result is very similar to what was obtained for horizontal curves on mild upgrades and, therefore, confirms that horizontal curves combined with sag curves display similar operating speeds as horizontal curves on mild upgrades and could be treated in the same.

Summary

The data analysis started with preparation for analysis and the checks for normality. It was followed by the identification of the independent variables that were correlated with 85th percentile speed. Then a series of statistical procedures were applied to the data to identify the best combination of variables to predict V_{85}.

The variables selected in the initial analyses were used to refine and select the regression models for passenger vehicle speeds on the different alignment combinations. The alignment combinations that included a non-limited sight-distance crest curve did not show any statistical significance when analyzed. It was concluded that the non-limited sight-distance crest curves approach the speed characteristics of tangents because the curvature is not so restrictive as to influence speeds.

All other alignment combinations had statistically significant regression equations that estimated 85th percentile speed. These models were evaluated for differences in the regression equations due to the cross-section of the curve. Only horizontal curves with limited sight-distance crest curves showed a statistically significant difference related to the pavement width.

Regression models were developed for each possible alignment combination except for those that included non-limited sight-distance crest curves. The regression models and the conditions in which they apply are summarized in table 14. These equations should be most accurate when used under conditions similar to those where the data were collected. Due to the small number of observations used in developing these regression models, extreme care should be taken before using the equations. It is believed that the forms of the regression models are appropriate, but the values of the parameters may vary.

Separate regression models were developed for passenger vehicles in most of the combinations of horizontal and vertical alignment (see table 14). Statistically significant regression equations were not developed for alignments that included non-limited sight-distance crest curves.

Table 14. Regression Equations Recommended for Validation.

Alignment Condition	Passenger Vehicles	N	R^2	MSE
Horizontal Curve on Grade: 0% ≤ G < 4% Horizontal Curve Combined With Sag Vertical Curve	$V_{85} = 106.30 - \frac{3595.29}{R}$	28	0.92	2.84
Horizontal Curve on Grade: 4% ≤ G < 9%	$V_{85} = 96.46 - \frac{2744.49}{R}$	14	0.56	6.86
Horizontal Curve on Grade: -9% ≤ G < 0%	$V_{85} = 100.87 - \frac{2720.78}{R}$	22	0.59	6.38
Horizontal Curve Combined With Limited Sight-Distance Crest Vertical Curve (i.e., K ≤ 43)	$V_{85} = 101.90 - \frac{3283.01}{R}$	16	0.78	3.95
Vertical Crest Curve With Limited Sight Distance (i.e., K ≤ 43) on Horizontal Tangent	$V_{85} = 111.07 - \frac{175.98}{K}$	6	0.54	6.30
Sag Vertical Curve on Horizontal Tangent	$V_{85} = 100.19 - \frac{126.07}{K}$	5	0.68	3.51

where: N = number of observations
V_{85} = 85th percentile speed of passenger cars (km/h)
R = radius of curvature (m)
G = grade (%)
K = rate of vertical curvature

EVALUATION OF SPIRAL TRANSITIONS

The objective of the study of spiral transition curves was to determine if the presence of spiral transition curves influenced the speed of drivers at the midpoint of a horizontal curve. If spirals influence the speeds of drivers, the relationship can be included in speed-prediction equations. Most conclusions on spiral curves are based on computer simulations or assumptions from vehicle operation and performance on circular curves. Little information is available on studies that use field-collected data.

The initial goal of this study was to use matched pairs of spiral transition and simple circular curve sites. Because appropriately matched pairs could not be identified, the analysis was performed by comparing a group of spiral curves to a group of simple circular curves whose geometry parameters fell within the same range of values. Data were collected at 12 curves with spiral transitions. This group was compared to 42 circular curves collected for the speed-prediction efforts that had similar geometry to the spiral transitioned curves. A summary of the data used in the analysis is shown in table 15. Both regression and cumulative analysis methods were employed in the evaluation. The regression analysis used an indicator variable to identify whether the presence of a spiral transition was significant. The cumulative analysis presented plots of the speed data for the spiral curves and the circular curves for specific radii to illustrate any differences between the two types of curves.

Table 15. Summary of the Data Collected.

Degree of Curve (deg)		1-4	5-8	9-12	Total
Range of Radii (m)		**400 - 3,500**	**200 - 399**	**145 - 199**	
Spiral Curve	Number of Sites	5	4	3	12
	Posted Speed Limit (km/h)	88.5	88.5	88.5	88.5
	Superelevation Rate (m/m)	0.03 to 0.12	0.10 to 0.12	0.07 to 0.08	0.03 to 0.12
	Horizontal Curve Length (m)	336.7 to 340.9	293.5 to 430.9	132.2 to 135.3	132.2 to 430.9
	Deflection Angle (deg)	11.0 to 44.4	17.1 to 18.7	50.6 to 52.1	11.0 to 52.1
	Grade	-1.5 to +1.5	-0.4 to +0.4	-0.2 to +0.2	-1.5 to +1.5
	Pavement Width (m)	8.1 to 8.3	8.5 to 10.3	7.9 to 8.1	7.9 to 10.3
Circular Curve	Number of Sites	14	16	12	42
	Posted Speed Limit (km/h)	88.5 to 96.6[a]	88.5	80.5[b] to 88.5	80.5 to 96.6
	Superelevation Rate (m/m)	0.02 to 0.07	0.02 to 0.12	0.05 to 0.12	0.02 to 0.12
	Horizontal Curve Length (m)	85.4 to 272.2	65.2 to 271.1	75.2 to 130.5	65.2 to 272.2
	Deflection Angle (deg)	5.5 to 33.0	12.8 to 56.0	24.9 to 51.4	5.5 to 56.0
	Grade	-2.6 to +2.6	-3.9 to +3.9	-4.3 to +4.3	-4.3 to +4.3
	Pavement Width (m)	6.2 to 11.9	6.7 to 13.4	7.1 to 9.4	6.2 to 13.4

[a] One site located in Texas.
[b] Two sites located in Texas.

Regression Analysis

Plots of the combined data were constructed to see if a clear difference existed between the spiral and circular curves. Figure 14 is a plot of 85th percentile speed versus the horizontal curve radius, and figure 15 is a plot of 85th percentile speed versus the horizontal curve length. No clear trends signaling that the presence of spiral curves affects the 85th percentile speed were visible from the analysis of these plots.

Regression analysis was performed using 85th percentile speed as the dependent variable and the inverse of the radius, curve length, deflection angle, superelevation, and an indicator variable (TYPE) as the independent variables. The indicator variable had a value of 0 if the curve was a circular curve, and a value of 1 if the curve has a spiral transition. The indicator variable was included to determine if the presence of spirals is statistically significant in determining 85th percentile speeds at the center of horizontal curves.

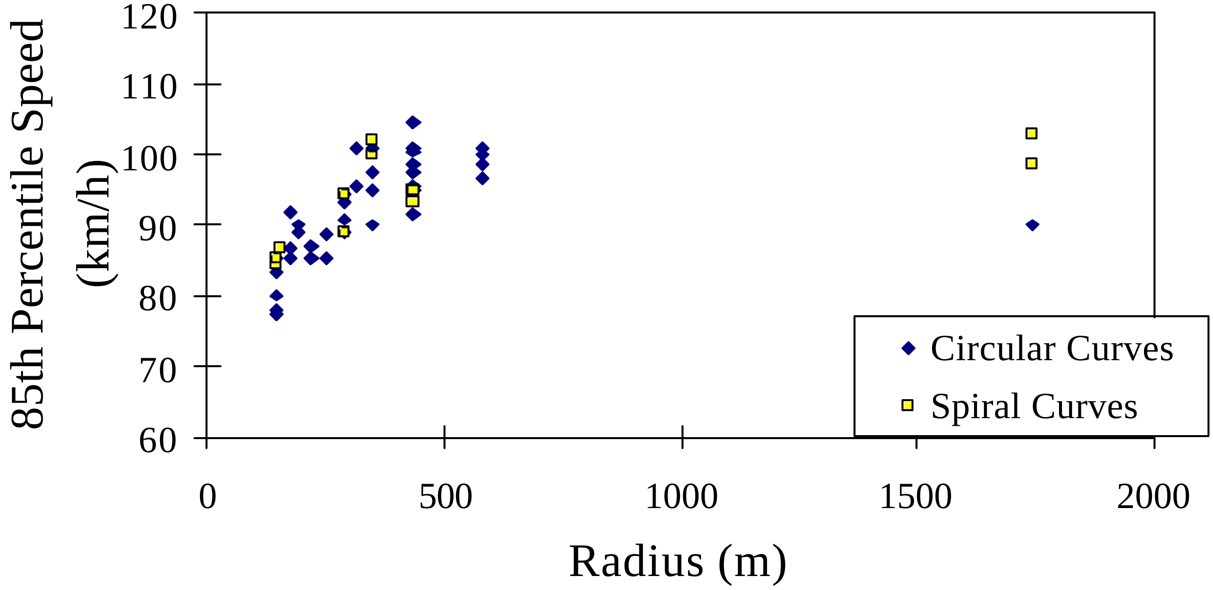


Figure 14. V_{85} Horizontal Curve Versus Radius of Curve.

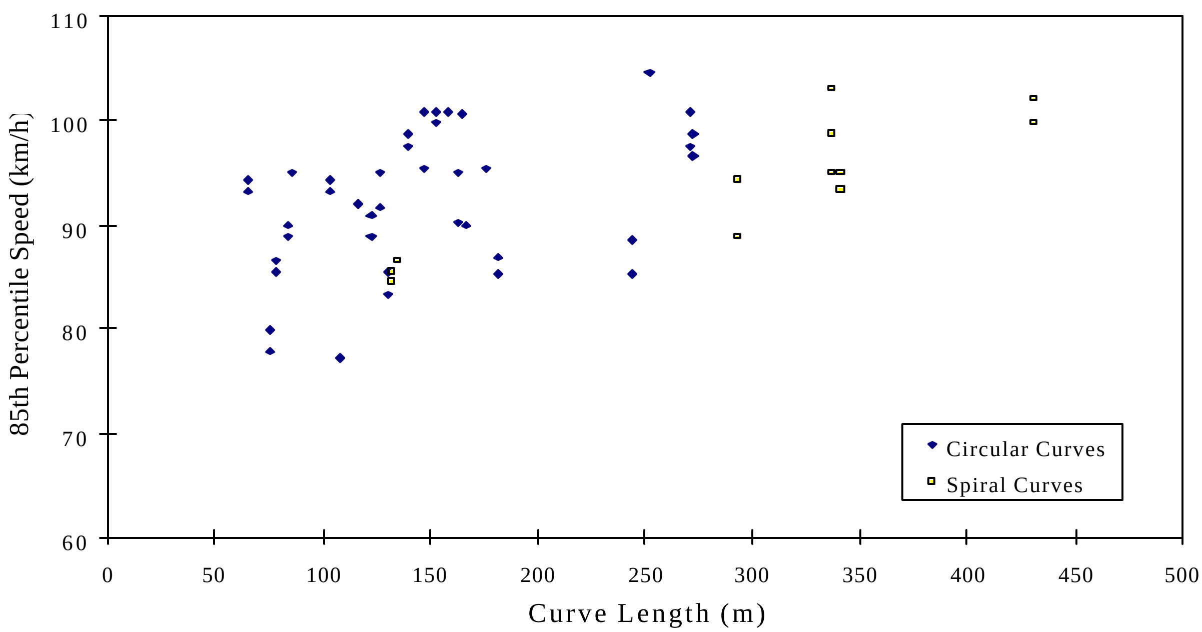


Figure 15. V_{85} Horizontal Curve Versus Curve Length.

A stepwise regression procedure was used to identify models that best predicted 85th percentile operating speeds for spiral and circular curves. The specific procedure ranked the different model combinations in order of decreasing adjusted coefficient of multiple determination (R_a^2) values. The R_a^2-value was chosen because it considers the number of parameters in the model and balances the gain in the proportion of variation explained by the model against the addition of more parameters.[54]

Models with high R_a^2-values were further analyzed to ensure that the independent variables were not correlated, and that each variable in the model was significant. This process involved analyzing each potential model and checking that all independent variables had a p-value of less than 0.05. Independent variables that did not have p-values less than 0.05 were removed from the model.

The result of the stepwise regression procedure was that the indicator variable (TYPE) was not included in the models with the highest R_a^2-values. By not being included in the models with the highest R_a^2-values, the indicator variable did not have a significant effect on predicting 85th percentile speeds on curves. This means that the presence of spiral transitions does not have a significant effect on 85th percentile speeds at the center of horizontal curves.

Further analysis of the output reaffirms that the model containing only the inverse of the radius is the best predictor for both spiral and circular curves. Although the model with the inverse of the radius and the deflection angle has a higher adjusted R^2-value, the parameter estimate for the deflection angle is not significant. This is also the case for the model containing the inverse of the radius and the curve length. Although the adjusted R^2-value is slightly higher than for the model containing only the inverse of the radius, the parameter estimate for the horizontal curve length is insignificant.

Cumulative Analysis

The results from the regression analysis suggested that spiral transitions did not significantly affect the 85th percentile speed of vehicles traversing horizontal curves. To determine if any trends or differences existed between spiral and circular curves with the same radius value, an analysis of the individual free-flow vehicle speed data was performed on the spiral and circular curve data for sites with a curve radius of 146 m, 291 m, 349 m, and 437 m. Those radii were chosen for analysis because data were collected for at least two spiral curve sites and two circular curve sites.

It is hypothesized that any differences between curves with spiral transitions and circular curves will be most evident for lower values of curve radius. By being subjected to a curve radius corresponding to minimum or below-minimum design standards, drivers will probably drive faster when a spiral transition is present. Thus, it is also hypothesized that for curves with a large radius, no difference in driver speed on curves with spiral transitions and circular curves will be observed. Table 16 contains a summary of the individual speed-data analysis.

Table 16. Summary of Cumulative Analysis.

Radius (m)	Type	Number of Sites	Mean Speed (km/h)	Standard Deviation (km/h)	Percentile Speed (km/h)	
					15th	85th
145.5	Circular	3	73.7	10.4	64.4	83.4
	Spiral	2	76.7	8.0	69.0	84.5
291.1	Circular	6	84.2	8.6	75.6	92.1
	Spiral	2	80.5	12.0	67.9	92.1
349.3	Circular	4	90.9	8.2	82.2	98.6
	Spiral	2	90.6	10.7	80.0	100.9
436.6	Circular	9	89.8	9.0	81.1	98.7
	Spiral	3	87.2	7.4	78.9	95.0

In an attempt to find a formal statistical procedure to determine if the cumulative distributions constructed during the analysis were statistically different, many alternatives were examined. Common procedures such as the Chi-square test and the Kolmogorov-Smirnov (K-S) test, which are traditionally used to test sample distributions versus hypothesized distributions, would not be appropriate to compare two sample distributions. Although tests such as Smirnov's maximum deviation test for identical populations and a modified version of the K-S test exist to compare two sample distributions, these tests are generally not exact.[55-56] Errors are often encountered from a few large differences in dispersion between the two distributions that may lead to incorrect conclusions. Therefore, none of these procedures were used in examining the relationship between spiral and circular curves.

Figure 16 shows the cumulative percentage versus vehicle speed for spiral and circular curves with a radius of 146 m. As hypothesized, it appears that vehicles travel at a slightly higher speeds on curves with spiral transitions due to the small curve radii. This observation is especially evident at lower percentiles where the two lines are furthest apart; however, the actual difference is no greater than approximately 5 km/h.

Figure 17 shows the cumulative percentage versus vehicle speed for spiral and circular curves with a radius of 291 m. Figure 17 shows the opposite of what was observed for a radius of 146 m in figure 16. For a radius of 291 m, it appears that vehicles will travel at a slightly higher speed on circular curves. As was the case with a radius of 146 m, the plots are very similar for the 65th percentile and higher, but seem to differ for lower percentile values. The largest speed difference is at the 15th percentile and has a value of approximately 8 km/h.

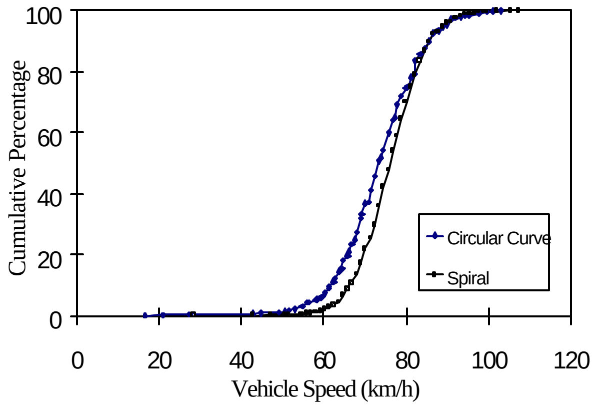


Figure 16. Cumulative Plot for Radius = 146 m.

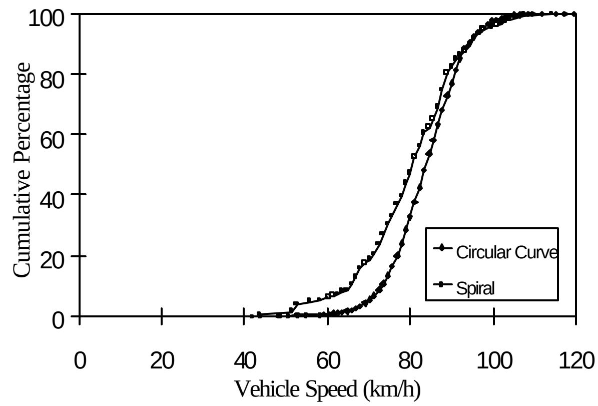


Figure 17. Cumulative Plot for Radius = 291 m.

Figure 18 shows the cumulative percentage versus vehicle speed for spiral and circular curves with a radius of 349 m. Little difference between vehicle speeds on spiral transition and circular curves can be observed from figure 18. The largest difference is at the 95th percentile where speeds are approximately 6 km/h greater for spiral transition curves. At the 85th percentile and 15th percentiles, speeds differ only by approximately 2 km/h.

Figure 19 shows the cumulative percentage versus vehicle speed for spiral and circular curves with a radius of 437 m. As was the case with a radius of 291 m, it appears that vehicles will travel at a slightly higher speed on circular curves. A difference of 3 km/h exists from the 15th percentile to the 85th percentile, with a difference of 5 km/h at the 95th percentile.

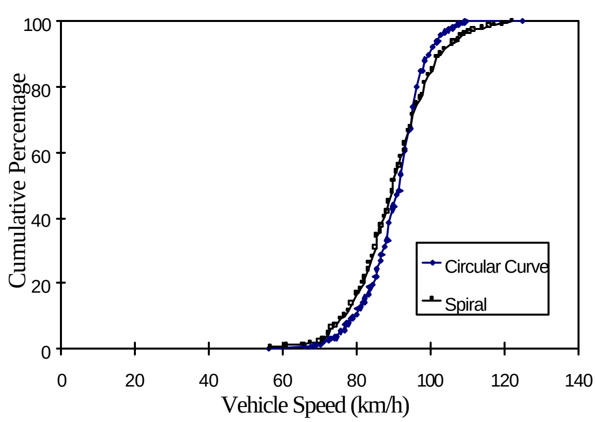


Figure 18. Cumulative Plot for Radius = 349 m.

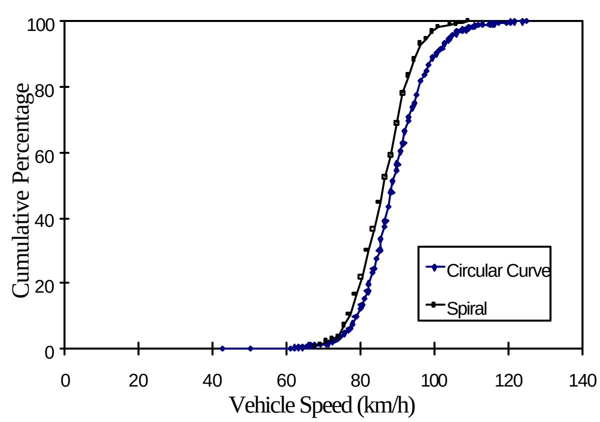


Figure 19. Cumulative Plot for Radius = 437 m.

The results of the speed data analysis agreed with the hypothesis that any differences between curves with spiral transitions and circular curves would be most evident for lower values of curve radius. By observing that vehicle speeds were slightly higher on spiral transition curves with a radius of 146 m, spiral transitions may be a benefit for low values of curve radii.

OTHER VEHICLE TYPES

Classification of other vehicle types were based on the axle-spacing data recorded in the speed-data files by the traffic classifiers. Preliminary classification of vehicles was based on AASHTO defined dimensions.[(2)] Eight vehicle types other than passenger vehicles were used for the preliminary analysis. An attempt was made to perform the same analyses that were done on passenger vehicles on the seven truck types and on recreational vehicles.

The amount of data collected for trucks and recreational vehicles was not as extensive as for passenger vehicles even though extra efforts were used to identify sites with high truck and RV activity. For the regression analysis, the minimum number of observations per site was 100 or more, but only passenger vehicles had that many observations at all the sites. It was decided to include those sites for which the 85th percentile speed could be calculated with 10 or more observations. The desired procedure to analyze these data would have been to follow all the steps performed for the analysis of the passenger vehicle data.

Due to the non-random selection of the sites and the extremely small sample size of trucks and recreational vehicles at each site, the results were combined into two classifications: recreational vehicles and all truck types. Reclassification increased the number of sites with more than 10 trucks, although the data set was still limited. Table 17 lists the number of sites available for analysis by the alignment condition used in the passenger car analysis.

It was not reasonable to obtain regression equations for trucks and recreational vehicles on the different alignment combinations due to the limited amount of data available and the small number of spot-speed measurements used to determine the 85th percentile value. Therefore, the models developed for passenger vehicles were compared to the truck and recreational vehicle data. Figure 20 shows the equations for grades between 0 percent and 4 percent; figure 21 shows the equations for grades greater than or equal to 4 percent; and figure 22 shows the equations for grades less than 0 percent. The data for trucks and recreational vehicles on horizontal curves display a speed behavior that is similar to passenger vehicles. As with passenger vehicles, there is a sharp drop in speed when the radius is less than approximately 250 m.

Table 17. Number of Truck and Recreational Vehicle Sites by Alignment Conditions.

Alignment Condition	Number of Sites With 10 or More Spot-Speed Observations	
	Trucks	Recreational Vehicles
Horizontal Curve on Grade: 0% # G < 4%	14	4
Horizontal Curve Combined With Sag Vertical Curve	11	4
Horizontal Curve on Grade: 4% # G < 9%	12	5
Horizontal Curve on Grade: -9% # G < 0%	25	12
Horizontal Curve Combined With Limited Sight-Distance Crest Vertical Curve (i.e., K # 43)	11	7
Vertical Crest Curve With Limited Sight Distance (i.e., K # 43) on Horizontal Tangent	3	0
Sag Vertical Curve on Horizontal Tangent	5	0

The 85^{th} percentile speed data for trucks and recreational vehicles shown in figure 20 are slightly lower than for the passenger vehicle model, but the behavior of the data parallels the model. Truck and recreational vehicle data are very similar to the predicted passenger car speeds on 4- to 9-percent grades (see figure 21). Figure 22 shows that only a few of the truck and recreational vehicle 85^{th} percentile speeds are higher than the predicted passenger car speeds. The data also indicate that for sharp horizontal curves (radius less than 200 m) on a downgrade, truck and recreational vehicles may have lower speeds than passenger cars.

Given the similarity in speed trends and the limitations of the truck and RV data available from this study, a design-consistency evaluation method should be based on passenger cars rather than on trucks or recreational vehicles. Additional data or another method should be explored if the objective is to predict truck or recreational vehicle speeds for a given set of alignment conditions.

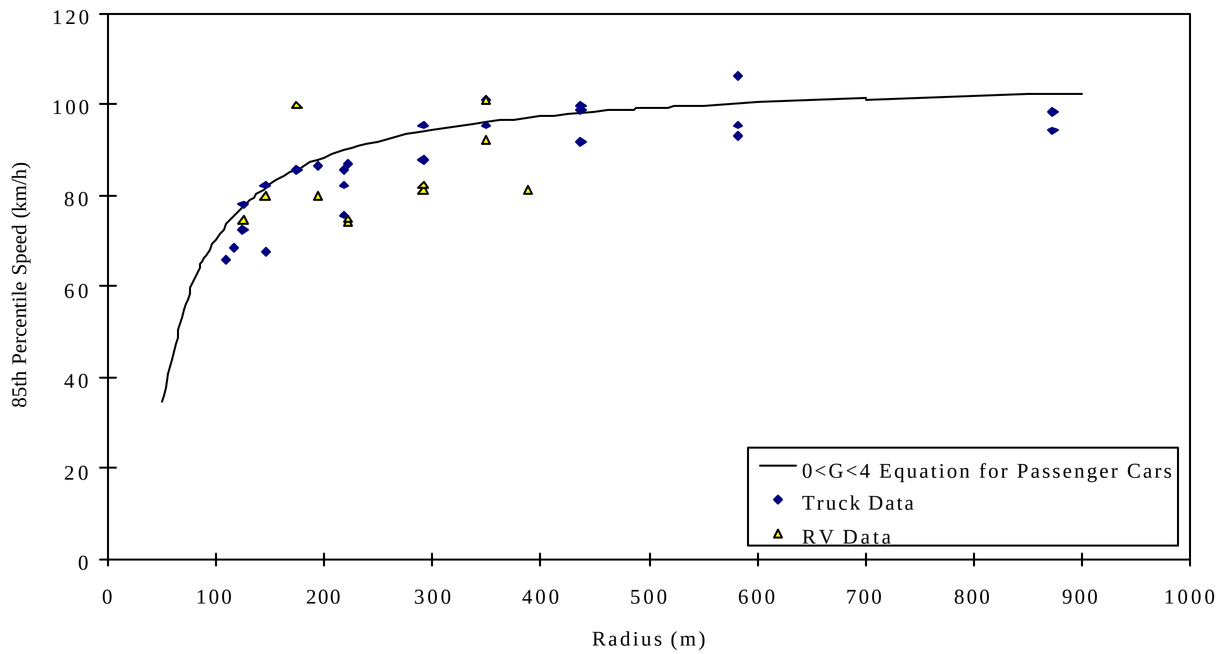


Figure 20. Truck and Recreational Vehicle Data Versus Radius (Passenger Vehicle Model, Grade: 0 percent to 4 percent).

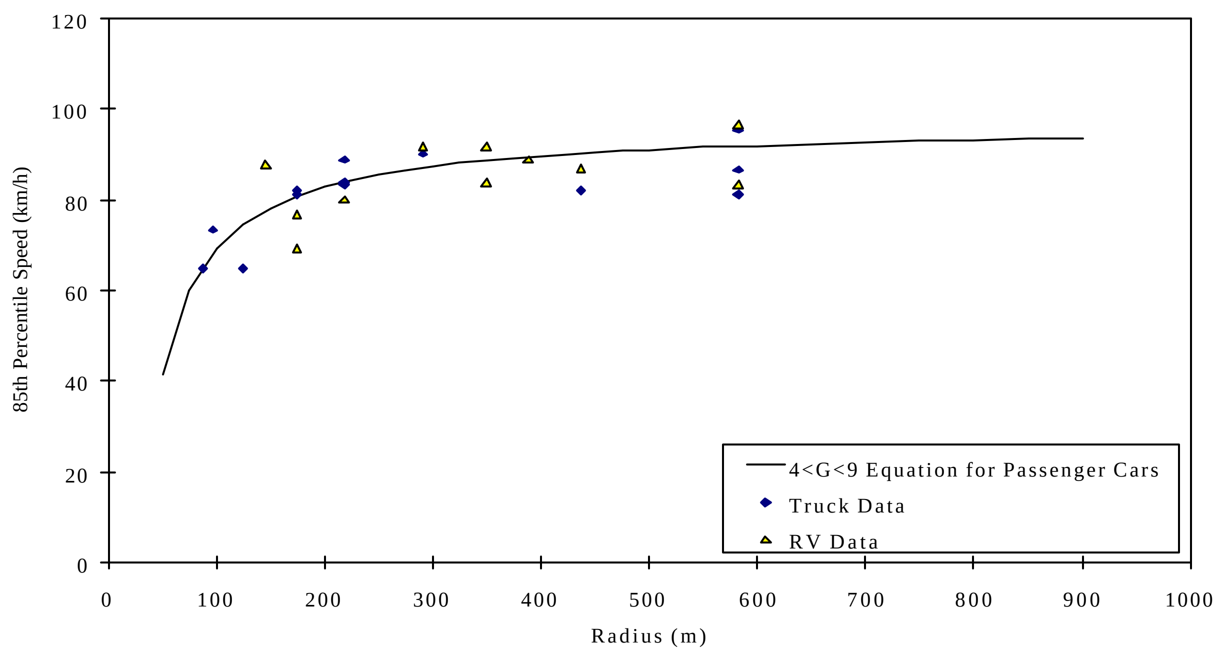


Figure 21. Truck and Recreational Vehicle Data Versus Radius (Passenger Vehicle Model, Grade: 4 percent to 9 percent).

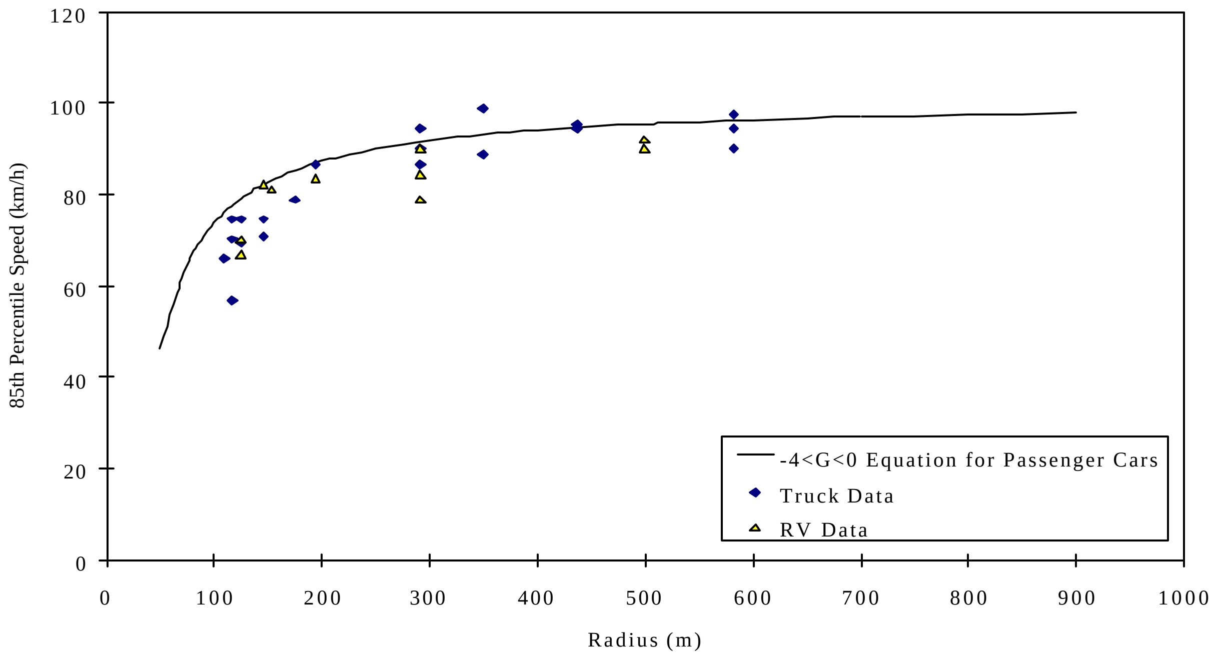


Figure 22. Truck and Recreational Vehicle Data Versus Radius (Passenger Vehicle Model, Grade: -9 Percent to 0 Percent).

SUMMARY

A primary objective of this research was to develop speed-prediction equations that consider combinations of horizontal and vertical alignment. Regression models were developed for horizontal curves for different ranges of vertical grade and in combination with sag and limited sight-distance crest vertical curves. They were also developed for limited sight-distance crest vertical curves and sag curves on horizontal tangents. Statistically significant regression equations were not developed for alignments that included non-limited sight-distance crest curves. These findings were based on speed and geometry data from 103 sites distributed across 6 states (Minnesota, New York, Pennsylvania, Oregon, Washington, and Texas).

Additional objectives of the speed-prediction research were to investigate spiral transitions and other vehicle types. The analyses indicated that spiral transitions did not significantly affect the 85^{th} percentile speed of vehicles traversing horizontal curves and that any differences between curves with spiral transitions and circular curves were most evident for lower values of curve radius. Because of the limited amount of data available for truck and recreational vehicles, it is not reasonable to obtain regression equations for these vehicle types for the different alignment conditions. Plots of the truck and RV data compared with plots of the regression equations for passenger cars demonstrated that similar trends exist. Therefore, a design-consistency evaluation method should be based on passenger cars.

4. PREDICTING SPEEDS ON TANGENTS USING ALIGNMENT INDICES

The objective of this part of this study was to evaluate the applicability of alignment indices as estimators of 85th percentile speeds on long tangents of two-lane rural highways. Alignment indices use the geometric characteristics of the roadway to provide quantitative measures of the general character of a roadway's alignment. It is hypothesized that the previous geometry of the roadway influences the subsequent speed expectations and desires of motorists. Therefore, these indices, which are based on the upstream alignment, can possibly reflect the expectancy of motorists when estimating their desired speeds on long tangents.

Several approaches have been used to estimate desired speeds on long tangents for speed-based design-consistency evaluations. The most relevant to the current contract is the approach used in the Australian rural design guide and the approach presented in Report No. FHWA-RD-94-034.(7,57)

McLean performed much of the research that led to the method adopted in the Australian design guide for rural roads.(20) McLean was also the first to define desired speed in the way it is used in this report—"the speed at which drivers choose to travel under free-flow conditions when they are not constrained by alignment features."(20) McLean suggested that desired speed was influenced by such factors as the purpose of the trip, proximity to urban areas, and the amount of time traffic was on the road; he also suggested that desired speed was influenced by the geometric characteristics, or the overall standard of alignment, of the roadway.(20,34) The Australian design guide provides a table of standard values for the speed environment of a roadway (i.e., desired speed), based upon McLean's work, for different terrain types (flat, undulating, hilly, mountainous) and ranges of horizontal curve radii.(57)

Report No. FHWA-RD-94-034 covers attempts to model 85th percentile speeds on long tangents; however, these attempts did not yield a statistically significant model. (7) Independent variables that were considered included the travel-way width, total pavement width, characteristics of the preceding and succeeding curves, tangent length, annual average daily traffic, terrain, and region of the country. The only significant difference was between the speeds on long tangents for level terrain in Texas and on long tangents for rolling terrain in the East (New York, Pennsylvania) and West (Oregon and Washington). As a result, the speed-profile model in that publication used a value of 97.9 km/h as the desired speed on long tangents, the mean of the 85th percentile speeds observed on the 78 long tangents at which data were collected. The use of this mean value does not explain the variability among observed 85th percentile speeds at those tangents, which ranged from 85.3 to 112.7 km/h.

PREVIOUS USES OF ALIGNMENT INDICES

While limited research on alignment indices has been performed in the United States, research efforts on alignment indices have been the focus of other countries, specifically Germany and England. In addition, Polus has examined the issue. These two countries have established methods of using alignment indices as both a measure of the design consistency and in estimating 85th percentile operating speeds.

Germany

In determining the design consistency, Germany uses a parameter for the horizontal alignment called the Curvature Change Rate (CCR). The CCR is the sum of angular changes in the horizontal alignment divided by the length of the highway section. This parameter is used in an attempt to prevent unsafe changes in operating speeds and to describe the overall operating characteristics of a road.(19) The formula below is used to calculate the CCR:(58)

$$CCR = \frac{\sum ?_c + ?_1 + ?_2}{L} \tag{6}$$

where:

	CCR	=	curvature change rate (deg/km)
	$?_C$	=	angular change in circular curve (deg)
	$?_1$	=	angular change in first transition curve (deg)
	$?_2$	=	angular change in second transition curve (deg)
	L	=	total length of section (km)

In calculating the CCR, the road being examined must be divided into sections that have similar or homogeneous alignments. These sections are determined by plotting the sum of the horizontal curvature (1/radius) against the length of the entire road. Sections of the alignment that exhibit similar characteristics (sections with a similar slope) would be determined subjectively, and the CCR would then be calculated for those sections.(5)

Germany has developed a nomograph that uses the CCR and pavement width to estimate the 85th percentile speed over a section of roadway.(22) Extensive investigations have shown that speed characteristics on horizontal alignments can be adequately described in terms of the CCR, and that there is a strong correlation between the 85th percentile speed of the roadway section and the CCR for all lane widths.(22)

England

England estimates the probable 85th percentile speed on wet pavement conditions based on the

calculated alignment and layout constraints. An estimated 85th percentile speed, which represents a calculated design speed, is inferred from the actual or proposed alignment and cross-section. For new roads, this comparison is used "to identify locations where elements of the trial alignment may be relaxed to achieve cost or environmental savings, or conversely, where the design should be upgraded, according to the calculated design speed."[59] For existing roadways,
this method can be used to determine if and where disparities exist between the design speed stated on roadway plans and probable 85th percentile speeds (which in the English method correspond to a calculated design speed), thereby indicating potential speed inconsistencies.

The alignment constraint measures the degree of constraint imparted by the alignment of the road and is a function of the bendiness and the harmonic mean visibility. For a two-lane road, the following equation is used to calculate the alignment constraint:[59]

$$A_C = 12 - VISI / 60 + 2B / 45 \tag{7}$$

where:

A_C = alignment constraint
VISI = harmonic mean visibility (m)
B = bendiness (deg/km)

Bendiness, identical to the CCR used in Germany, represents the sum of the horizontal angular changes in the roadway per kilometer. The harmonic mean visibility is the harmonic mean of individual sight-distance observations along the roadway and it should be measured over a minimum section length of 2 km.[59]

The layout constraint measures the degree of constraint imparted by various charac- teristics of the roadway and is determined based on roadway type, cross-section width, and access density.[9]

Polus

Polus suggested several alignment indices to quantify the general character of the alignment of a roadway.[60] Indices based on horizontal alignment characteristics included the average curvature in degrees per kilometer, the average radius of curvature, the ratio of the maximum to minimum radius, and two horizontal alignment indices using the radii of curves. Polus also suggested one index based on vertical alignment, the average gradient along a section of roadway. Polus evaluated the correlation between these indices and accident rates on 20 segments of roadway in Israel. Among the indices evaluated, only the ratio of the maximum to the minimum radius was significantly related to the accident rate. Polus observed that as this value approaches 1, or as the radii of the roads become more consistent, a reduced accident rate may be expected.

DEVELOPMENT OF ALIGNMENT INDICES

The hypothesis that alignment indices (representing the character of the preceding alignment) influence drivers' desired speeds (measured as the 85^{th} percentile speeds on long tangents) was motivated by experiences in Australia, Germany, and England. In Australia, desired speeds are estimated based on the terrain and range of the radii of horizontal curvature. In Germany, curvature change rate and lane width are used to estimate 85^{th} percentile speeds along a segment of roadway. In England, the average speeds along a roadway are estimated based on an alignment constraint (essentially, an alignment index) and a layout constraint (represents the cross-section of the roadway).

Identification of Alignment Indices

The initial step in determining the alignment indices consisted of identifying all possible indices that may be useful for this research. Therefore, some of the alignment indices that had been used in other countries or proposed for use were included. In addition, other indices thought to have some worth in estimating the desired speed of motorists on long tangents were developed.

All of these alignment indices could possibly provide an indication of the geometry motorists experience upstream on the roadway. Since it is hypothesized that the previous geometry of the roadway influences motorists' speeds, these indices may be able to quantify the effects of the preceding alignment on the speeds of motorists. A list of the alignment indices that were initially developed for this study is provided in table 18.

Selection of Alignment Indices

As part of this evaluation, some of the proposed alignment indices in table 18 were computed for a small sample of roadways. After examining these results and the proposed alignment indices more closely, it was determined that some of the indices may be more worthwhile for use in estimating the desired speeds of motorists on long tangents than others. Two other criteria used in determining whether the indices should be further evaluated in this study were:

- Alignment indices had to be strictly a function of the geometry of a roadway.
- There had to be a reasonable hypothesized relationship between the alignment index and the 85^{th} percentile speeds.

Therefore, those indices that were thought to be better measures were evaluated in this research while the others were eliminated from consideration. The rationale used in making these decisions is described in the following sections.

Table 18. Initial Alignment Indices Identified.

Horizontal Alignment Indices	Vertical Alignment Indices
1. Angular Change in Direction • Average Deflection Angle per Length (Bendiness or Curvature Change Rate) • Average Deflection Angle per Curve • Average Degree of Curvature per Length • Average Degree of Curvature per Curve • Ratio of Curve Length to Total Roadway Length 2. Radii Measures • Average Radius • Average Radius / Minimum Radius • Maximum Radius / Minimum Radius 3. Tangent Length • Average Tangent Length 4. Sight Distance • Harmonic Mean Visibility	1. Angular Change in Direction • Algebraic Difference in Grades / Length (Percent per Kilometer) (Degrees per Kilometer) • Average Rate of Vertical Curvature 2. Elevation Measures • Average Gradient 3. Sight Distance • Sight Distance **Composite Alignment Indices** 1. Angular Change in Direction 2. Average Highway Speed

Horizontal

The length-based alignment indices for the horizontal angular change in direction were selected for further evaluation. Since the length-based indices consider the length of the roadway, it was assumed that these alignment indices were more representative of the general character of the alignment of the roadway. The average deflection angle and average degree of curvature per curve were not evaluated further for the horizontal alignment indices using the angular change in direction because the Germans and English both calculate their indices on a per length basis rather than based on the number of features. Furthermore, using the average radius as an alignment index would provide similar information to these feature-based indices. For example, stating that the previous 3 km of roadway had a degree of curvature alignment index value of 12 deg/km describes the angular change motorists experience better than simply stating that the average degree of curve for the same distance was 5 degrees. A large value for these indices indicates that the road either contains a large number of curves or there are long or sharp curves in that section. It was expected that an increase in the value of these indices would decrease the desired speeds of motorists.

Correspondingly, the ratio of curve length to total roadway length was selected because it could also provide a good indication of the character of the alignment. This alignment index provides information on the proportion of the roadway that is on curved sections. As the road includes more curves or longer curves, this proportion increases, and the speeds motorists drive would be expected to decrease.

The horizontal indices that compare the ratio of the average and maximum radii to the minimum radius were not chosen for further evaluation. While these indices appear to provide indications of design consistency rather well (the closer the value is to 1, the more consistent the design), it was thought that these indices did not quantitatively represent the character of the roadway adequately to predict speeds on tangents. For example, a value of 2 may be the ratio between either a 1,746-m (1-deg) and a 873-m (2-deg) curve or a 350-m (5-deg) and a 175-m (10-deg) curve. It is hypothesized that motorists would react differently to each situation, thereby making speed estimation difficult.

The average radius and average tangent length were selected for further study because they could represent what a driver would typically experience along sections of the previous alignment. The average radius expresses what motorists typically encounter on curved sections of the road. A large average radius would indicate curves that are typically not very sharp. Therefore, it is expected that higher speeds would exist for these values compared to smaller average radius values. The average tangent length indicates the length of tangent that is typically available to motorists between curved sections of the roadway. A large value for this index would indicate that the road has tangent sections that are typically long, therefore, motorists' speeds would be expected to be higher than for roads with a smaller value.

The use of a sight-distance-based alignment index in the horizontal plane was not considered further because an automated method of calculating the actual sight-distance available to the motorists from roadway design plans was not available. In addition, the amount of sight distance available to motorists is based on subjective parameters, such as driver eye height and object height. The intent of this research was to use alignment indices that were geometry-based.

Table 19 provides a list of the horizontal alignment indices selected for use in this study. This table also shows the equations necessary to compute the indices and the indices'units.

Vertical

For the vertical alignment indices, all the indices initially developed were studied further except for vertical sight distance. Sight distance as a vertical alignment index was not selected because an automated method of calculating the actual sight distance available to the motorists from roadway design plans does not exist. In addition, the sight distance available to motorists is not based on the geometric parameters of the road, but rather it uses subjective parameters such as driver eye height and object height.

Table 19. Alignment Indices Selected for Evaluation.

Horizontal Alignment Indices	Vertical Alignment Indices
• Curvature Change Rate - CCR (deg/km) $\frac{j\ ?_i}{j\ L_i}$ where: ? = deflection angle (deg) L = length of section (km)	• Vertical CCR - V CCR (deg/km) $\frac{j\ A_i}{j\ L_i}$ where: A = absolute difference in grades (deg) L = length of section (km)
• Degree of Curvature - DC (deg/km) $\frac{j\ (DC)_i}{j\ L_i}$ where: DC = degree of curvature (deg) L = length of section (km)	• Average Rate of Vertical Curvature - V AVG K (km/percent) $\frac{j\ \frac{L}{*A*}}{n}$ where: L = length of section (km) A = algebraic difference in grades (%) n = number of vertical curves
• Curve Length:Roadway Length - CL:RL $\frac{j\ (CL)_i}{j\ L_i}$ where: CL = curve length (m) L = length of section (m)	• Average Gradient - V AVG G (m/km) $\frac{j\ *?E_i*}{j\ L_i}$ where: ?E = change in elevation between VPI_{i-1} and VPI_i (m) L = length of section (km)
• Average Radius - AVG R (m) $\frac{j\ R_i}{n}$ where: R = radius of curve (m) n = number of curves within section	**Composite Alignment Indices**
• Average Tangent - AVG T (m) $\frac{j\ (TL)_i}{n}$ where: TL = tangent length (m) n = number of tangents within section	• Combination CCR - COMBO (deg/km) $\frac{j\ ?_i}{j\ L_i} \% \frac{j\ A_i}{j\ L_i}$ where: ? = deflection angle (deg) A = absolute difference in grades (deg) L = length of section (km)

In the angular change in vertical direction category, the algebraic difference in grades per length and the average rate of vertical curvature were included for use in this study. The algebraic difference in grade per length measures the angular change in the vertical direction of a road in the same way that CCR measures the angular change in the horizontal direction. Therefore, this index was renamed the vertical CCR, and the units for this alignment index were converted to match the units of the horizontal CCR. A large vertical CCR value would indicate the existence of many changes in the vertical direction of a roadway or the presence of steep grades. For either condition, a large value is hypothesized to result in lower operating speeds by motorists.

The average rate of vertical curvature was used as an alignment index because it provides a measure of the sharpness of the vertical curves of a road, which resembles the average radius alignment index in the horizontal plane. It was thought that this index provides a good indication of the vertical character of the alignment. A large value for this index would indicate that motorists are provided with more sight distance when traveling on the vertical curves of the roadway. Therefore, the effects of a steep grade, if present, may be reduced and the speeds of the motorists may increase as a result.

The average gradient was included for evaluation because this index represented the absolute change in vertical direction along a roadway. As the average gradient increases in value, this indicates that either there is typically a large change in elevation between the vertical point of intersections or that the vertical alignment is hilly. It is hypothesized that a large value for this index would result in low operating speeds.

Table 19 shows the vertical alignment indices evaluated in this study. The equations used to calculate the indices and their units are also included. Although Germany and England do not consider vertical alignment indices in their design practices, this research attempts to use vertical alignment indices in estimating the desired speeds of motorists on long tangents. One weakness associated with these vertical alignment indices is that they treat crest and sag vertical curves equally. This equal treatment is a concern because each type of vertical curve has a different effect on the speeds of motorists. Since crest curves typically limit the sight distance of motorists more so than sag curves, crest vertical curves may have a greater effect on the speeds of motorists. In the evaluation of the vertical alignment indices, these effects must be considered.

Composite

Of the two composite alignment indices identified, the average highway speed was eliminated from consideration because it is based on the design speed of the individual alignment elements of the roadway. While this index uses the geometric characteristics of the road, it also utilizes the design-speed concept. As discussed previously, there are several weaknesses of the design-speed concept that hinder its ability to ensure operating-speed consistency, and thus, its ability to estimate speeds of motorists on long tangents.

The composite angular change in direction combines the horizontal and vertical angular change in direction of the roadway. This index was termed the combination CCR because it was computed by simply adding the horizontal CCR value and the vertical CCR value. As the value of the combination CCR increased, indicating more angular changes in either the horizontal and/or vertical direction of the roadway, it was hypothesized that the speeds of motorists would tend to decrease.

In evaluating this index, it is important to note that the total angular change in the horizontal direction is much greater than the total angular change in the vertical direction. Therefore, by simply adding these two effects, greater emphasis is placed on the effect of the horizontal alignment than the vertical alignment. While it is assumed that the horizontal alignment plays a greater role in the determination of speeds, the magnitude of this effect is not known. The composite alignment index being evaluated is shown in table 19. The units for this index and the equation are also shown.

ANALYSIS METHODOLOGY

Identification of Geometric Variables

In addition to the alignment indices, other geometric characteristics thought to influence the desired speeds of motorists on long tangents were identified. Five independent variables identified for evaluation were vertical grade at the tangent site, total pavement width at the tangent site, roadside hazard rating, driveway density, and the posted speed limit of the roadway.

The vertical grade on the tangent was examined because the vertical grade of the roadway can have an effect on the speeds that motorists drive. It was hypothesized that the 85th percentile speeds would be lower for approach tangents to horizontal curves on upgrades compared to tangents on downgrades. More specifically, the steeper the upgrade, the more reduction in speed is expected.

It was also hypothesized that as the total pavement width at the tangent site increased, the speeds of motorists would also tend to increase. Wider pavements on the roadway would allow motorists more room to maneuver their vehicles along the road. Motorists may feel more comfortable driving faster at locations where extra space exists between them and other vehicles on one side and the pavement edge on the other.

The consequences of running off the road increase in severity as the roadside hazard rating increases. Therefore, it was hypothesized that desired speeds would decrease as the roadside hazard rating increased. A seven-point scale was used to describe the roadway surroundings with 1 representing a clear roadside and 7 representing severe roadside hazards.

Driveway density is a measure of the roadside friction associated with adjacent land development. It was hypothesized that desired speeds would decrease as driveway density increases. Since rural areas are characterized by low driveway densities, it was further hypothesized that desired speeds would not be sensitive to driveway densities within the range of values at the sites in this study.

A logical relationship exists between the posted speed limit of a roadway and the speeds motorists want to drive. As the legal speed limits increase, the 85th percentile speeds of motorists are expected to increase. Motorists tend to drive at or above the speed limit of a roadway.

Because of the relationships described above, these five independent variables were analyzed together with the alignment indices to determine their ability to predict desired speeds of motorists. It was hypothesized that either one or more of these variables could be used in addition to the alignment indices in predicting the desired speeds of motorists on long tangents of two-lane rural highways.

The factors mentioned above are only a few of the possible influences on motorists' desired speeds on long tangents of two-lane rural highways. Other characteristics of the roadway may play a role in the speeds motorists drive on tangents. These characteristics include items such as:

- Familiarity of roadway to motorist.
- Available sight distance to the motorist.
- Available roadside clear distance.

However, these factors are difficult to quantify and were not investigated in this research.

Data Collection and Reduction

For this effort, the spot-speed and roadway alignment data collected for other tasks were used; these data were described in chapter 3. Alignment data were extracted from highway design plans obtained from the corresponding State's department of transportation and geometric characteristics were collected in the field. Table 20 shows the data that were required from the roadway plans in order to compute the alignment indices. The roadway design plans were converted into a computerized spreadsheet format and were used to efficiently calculate the alignment indices.

Table 20. Alignment Data Required From Highway Design Plans.

Horizontal	Vertical
Station of PC	Station of Vertical PC
Station of PT	Station of Vertical PI
Tangent Length	Station of Vertical PT
Curve Length	Grades
Radius	Curve Length
Degree of Curve	Algebraic Difference in Grades
Deflection Angle	Rate of Vertical Curvature
	Elevation of Vertical PI

Alignment indices were computed for only 68 of the 176 sites available. The number of sites was reduced because the sites had to meet the criteria listed below:

- Available speed data for the horizontal curve site.
- Available speed data on the preceding tangent of a horizontal curve site.
- Identified tangent had to be considered an independent tangent.
- Selected preceding tangent length of curve site must be greater than 200 m.

An independent tangent is classified in the speed-profile model as a tangent that is long enough for motorists to accelerate to and maintain their desired speed for some distance. The equation below, which considers the degree of curvature preceding and following the tangent, was developed for use in the speed-profile model to determine whether a tangent could be considered an independent tangent:[7]

$$TL_C = \frac{2V_f^2 - V85_1^2 - V85_2^2}{25.92\ a} \tag{8}$$

where: TL_c = critical tangent length (m)
V_f = 85th percentile desired speed on long tangents = 97.9 km/h
$V85_n$ = 85th percentile speed on curve n (km/h)
a = acceleration/deceleration = 0.85 m/s^2

In this equation, the critical tangent length represents the length of tangent necessary for motorists to accelerate to and maintain their desired speed. If the actual tangent lengths were greater than the critical tangent length value, then a desired speed could be reached on that particular tangent. The tangents used in this study satisfied this criterion.

In addition to being considered an independent tangent, the length of the tangent had to be greater than 200 m. Two factors influenced the selection of this limiting value. First, a previous study on tangent speeds of two-lane rural highways considered a tangent length of 244 m as an independent tangent.[7] Second, a visual inspection of a plot of 85th percentile tangent speeds against tangent length, shown in figure 23, appeared to reveal that speeds on a tangent became constant after a tangent length of 200 m. The slope of a linear regression line using tangent lengths greater than 200 m was not significantly different from zero.

Statistical Analysis

Several techniques were utilized in analyzing the data collected for this study. A graphical analysis was performed to provide an initial visual indication of any relationships that may exist between the 85th percentile tangent speeds and the alignment indices. After examining the graphical analysis results, a simple linear regression analysis was performed to determine if one of the alignment indices was a significant predictor of desired speeds. This analysis was used to confirm or deny the existence of relationships that may have been determined visually through the graphical analysis. Geometric variables were then introduced to determine if they could be used in combination with alignment indices to estimate the desired speeds of motorists. A correlation analysis was then used in determining if any relationships existed among the alignment indices and the geometric variables. The results of this analysis were used in the multiple linear regression analysis that followed. If any of the alignment indices or geometric variables were correlated, then they should not be included as predictors in the same multiple regression model. The multiple linear regression analysis determined which indices and geometric variables may be useful in combination to predict the desired speeds of motorists. After all the possible relationships between the 85th percentile speeds and alignment indices were examined, an analysis of variance (ANOVA) was performed to determine if there were significant differences in the observed 85th percentile speeds due to the geometric variables of vertical grade, total pavement width, posted speed, driveway density, and roadside rating. In addition, this analysis was also used to determine if there were regional differences in the observed 85th percentile tangent speeds. All of the statistical analyses were performed using SAS® and were tested at a 90-percent confidence interval (i.e., significance level of $a = 0.10$).

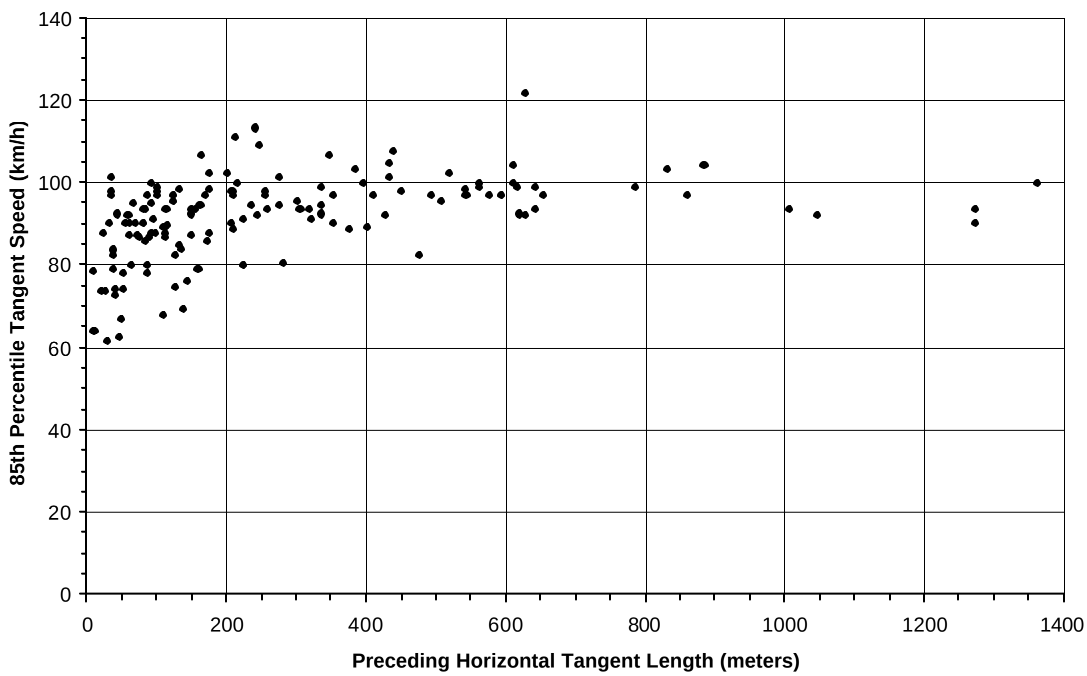


Figure 23. Observed 85th Percentile Tangent Speeds Versus Preceding Horizontal Tangent Length.

RESULTS

Summary of Site Characteristics and Values

There were 68 sites in this study that satisfied the criteria mentioned previously. These sites were located in six States (see table 21). The geometric and speed characteristics at each tangent site were either collected in the field or obtained from roadway design plans. A summary of these characteristics for this study is presented in table 22. Ranges of values for the alignment indices using 5 km of the preceding roadway alignment are shown in table 23.

Table 21. Number of Sites by State.

State	No. of Sites	No. of Road Segments
Minnesota	2	2
New York	9	3
Oregon	21	4
Pennsylvania	8	3
Texas	13	7
Washington	15	7
Total	68	26

Table 22. Summary of Site Characteristics.

	Minimum	Maximum	Average
Tangent Length (m)	201.4	1368.8	485.0
Preceding Roadway Alignment Available (km)	0.1	23.6	8.2
Grade on Tangent (%)	-9.0	8.0	0.6
Total Pavement Width (m)	6.2	12.7	8.0
Posted Speed (km/h)	80.5	112.7	89.3
Observed 85^{th} Percentile Tangent Speed (km/h)	80.0	121.7	96.8

Previous Alignment Characteristics

One of the issues in the development of the alignment indices was the determination of the amount of previous alignment characteristics that is a factor in the speeds that motorists drive. The length of the preceding alignment characteristics that affect motorists was hypothesized to be between 3 and 7 km. The lower value of 3 km was selected because a distance of 1 and 2 km was thought to inadequately include the characteristics of the previous roadway. The number of geometric features encountered by motorists within 2 km was considered to be too small for motorists to create any expectations of the upcoming roadway. In addition, assuming motorists typically travel 1.6 km/min on rural highways, there may not be enough time for motorists to build up any expectations of the road ahead. The higher value of 7 km was selected for different reasons. As the preceding alignment characteristics for distances greater than 7 km were used, it was hypothesized that the indices would not be affected much by changes in the upstream alignment characteristics. In addition, using greater than 7 km of the previous alignment characteristics may exceed the short-term memory expectation limits of motorists. It was assumed that the roadway encountered greater than 7 km upstream has little effect on the current expectation of the roadway by motorists.

To test these theories, values for all of the alignment indices were initially calculated using 1 to 10 km of previous alignment data. Graphs of the 85th percentile tangent speed against each of the alignment indices were created for each 1-km increment. While it was hoped that these graphs could provide some indication as to the length of the preceding alignment data to be used in this study, the appropriate distance to be used could not be determined based on these plots.

Table 23. Range of Alignment Index Values.

		Minimum	**Maximum**	**Average**
HORIZONTAL	CCR (deg/km)	12.5	293.5	62.5
	DC (deg/km)	2.1	146.3	15.4
	Ratio of Curve Length to Road Length	0.13	0.55	0.33
	Ave. Radius (m)	168.0	2043.3	588.6
	Ave. Tangent (m)	81.4	1104.9	371.0
VERTICAL	Vertical CCR (deg/km)	0.9	17.0	6.1
	Ave. Gradient (m/km)	5.7	58.9	21.2
	Ave. Rate of Vertical Curvature (m/%)	2.8	187.0	58.1
COMPOSITE	Combination CCR (deg/km)	18.8	303.7	72.1

To adequately determine the effect that the previous alignment data had on the alignment indices, figure 24 was developed. This figure represents the average change in the horizontal, vertical, and composite alignment indices from the previous 1-km increment. This figure shows that after a distance of 5 km, the values of the alignment indices remain constant; therefore, using alignment characteristics greater than 5 km upstream of the tangent site does not significantly change the value of the indices. In addition, analyses of the results using upstream distances of 1 to 10 km were similar to the results obtained for 5 km of upstream alignment. Therefore, alignment characteristics a distance of 5 km upstream of and including the tangent site were used and reported for the remaining analyses in this study. Using this distance, the sample size of tangent sites was reduced from 68 to 40 for the horizontal alignment indices and to 35 for the vertical alignment indices.

The sample size was reduced from the original 68 sites because the length of the preceding alignment data available varied by individual roads. As the amount of preceding alignment distance being used increased, the number of available tangent sites decreased. In addition, there were fewer tangent sites for the vertical alignment indices because several of the roadway design plans did not contain any vertical alignment information.

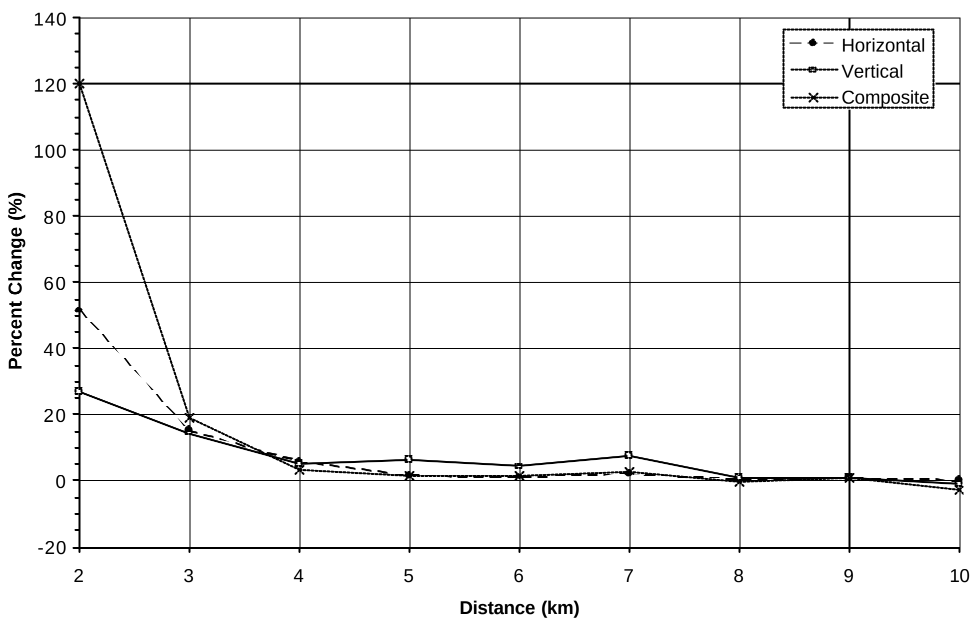


Figure 24. Average Change in Alignment Indices by Distance.

Examination of Alignment Indices as Individual Estimators of Desired Speed

In determining whether any of the alignment indices were able to individually predict 85th percentile tangent speeds on long tangents of two-lane rural highways, graphical and simple linear regression analyses were used. The graphical analysis was performed initially to assist the regression analysis.

Graphical Analysis

The ability of individual alignment indices to predict the 85th percentile tangent speeds was first examined graphically. For all nine of the alignment indices, graphs of the observed 85th percentile tangent speeds against the alignment indices were developed. These graphs, which are shown in figures 25 through 33, were created in order to visually inspect those indices that may have promise in estimating the desired speeds of motorists on long tangents.

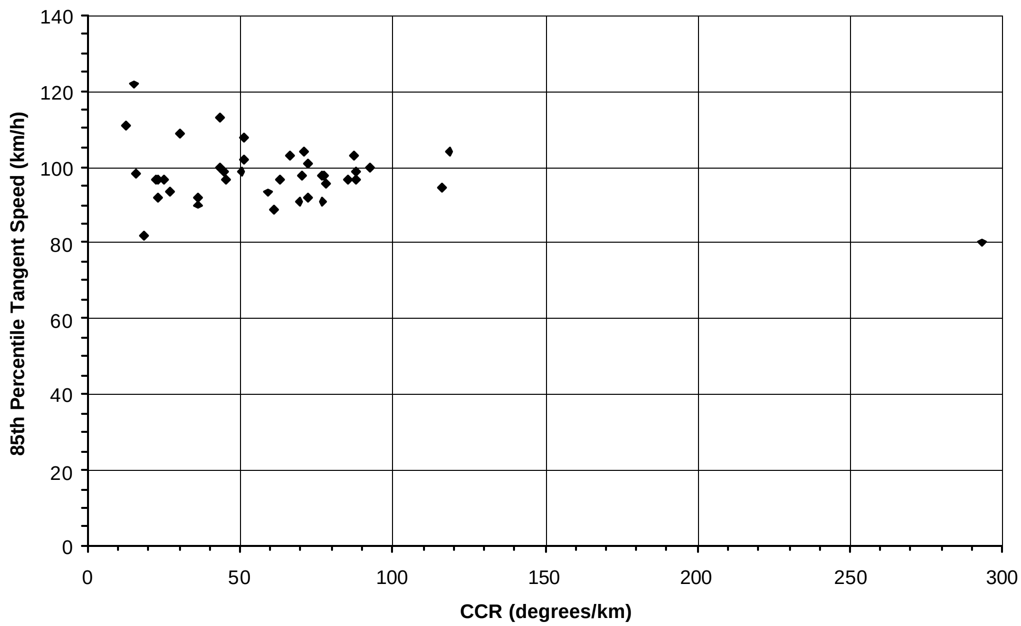


Figure 25. 85th Percentile Tangent Speed Versus CCR.

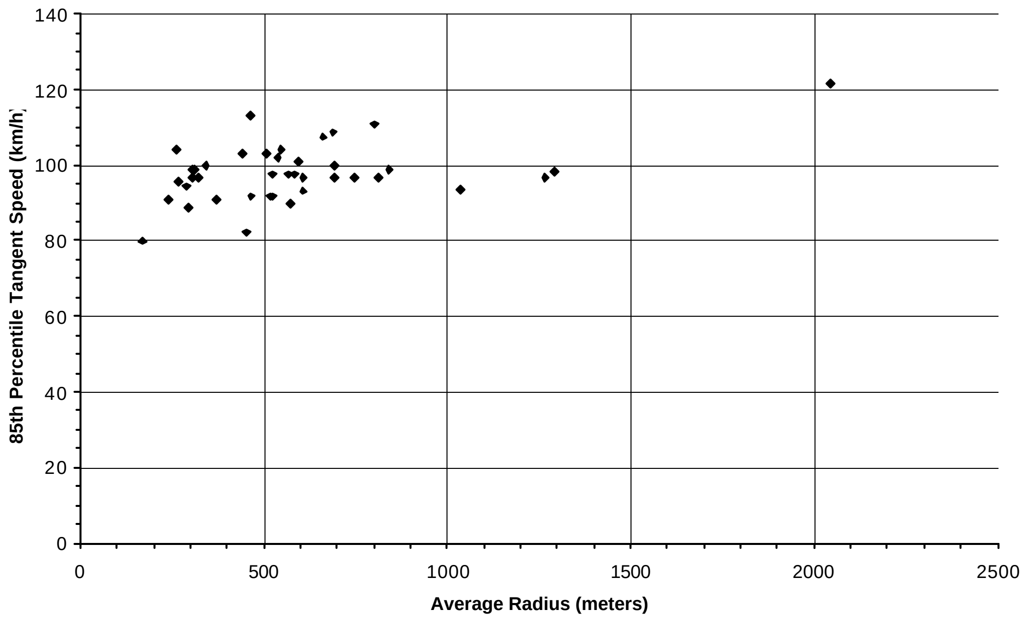


Figure 28. 85th Percentile Tangent Speed Versus Average Radius.

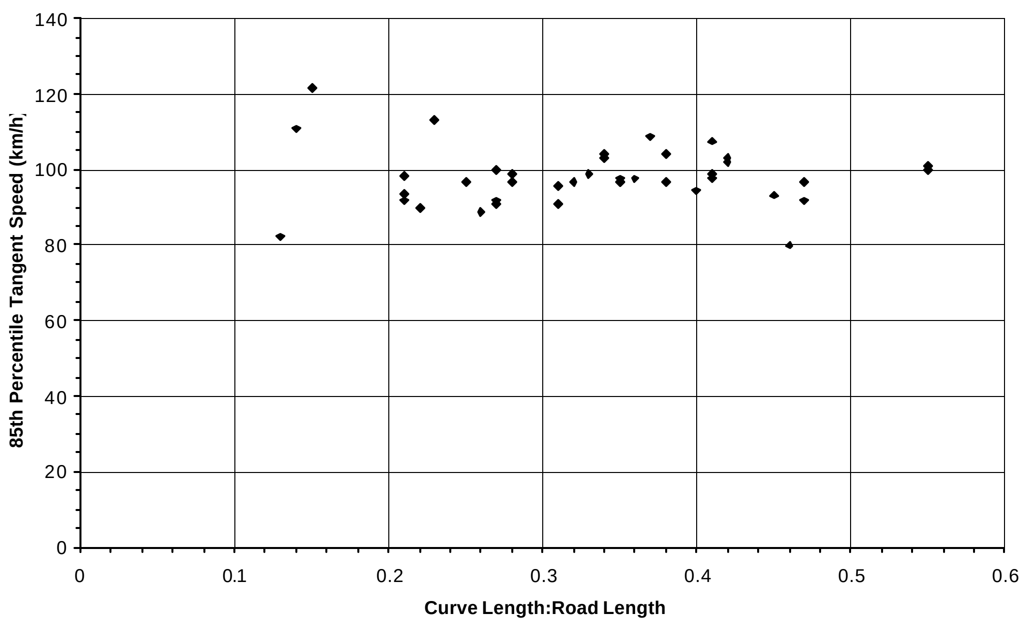


Figure 27. 85th Percentile Tangent Speed Versus Ratio of Curve Length to Road Length.

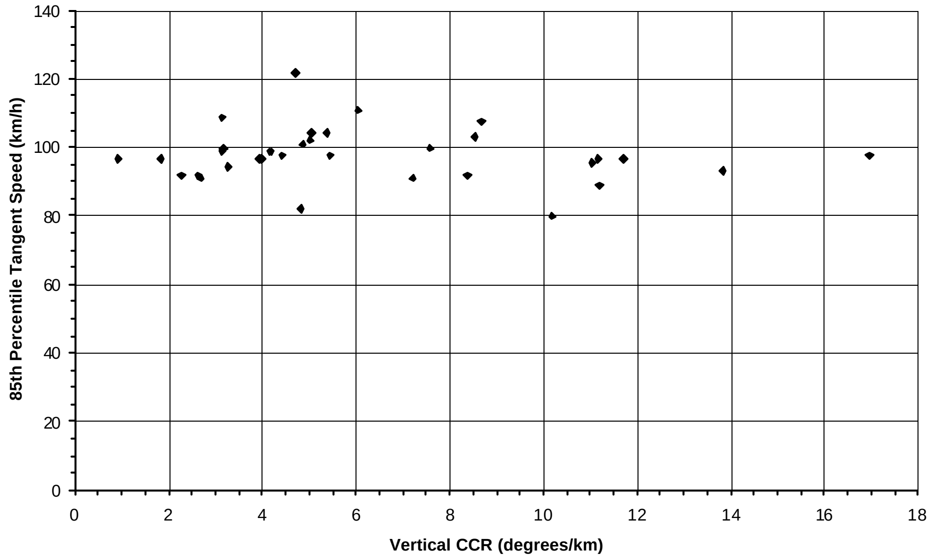


Figure 30. 85th Percentile Tangent Speed Versus Vertical CCR.

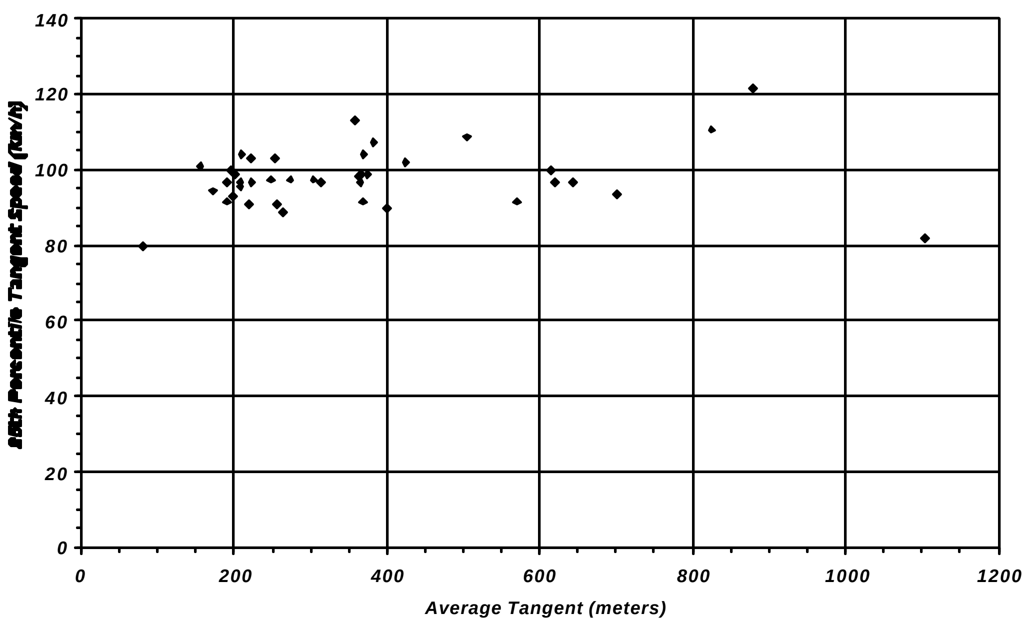


Figure 29. 85th Percentile Tangent Speed Versus Average Tangent.

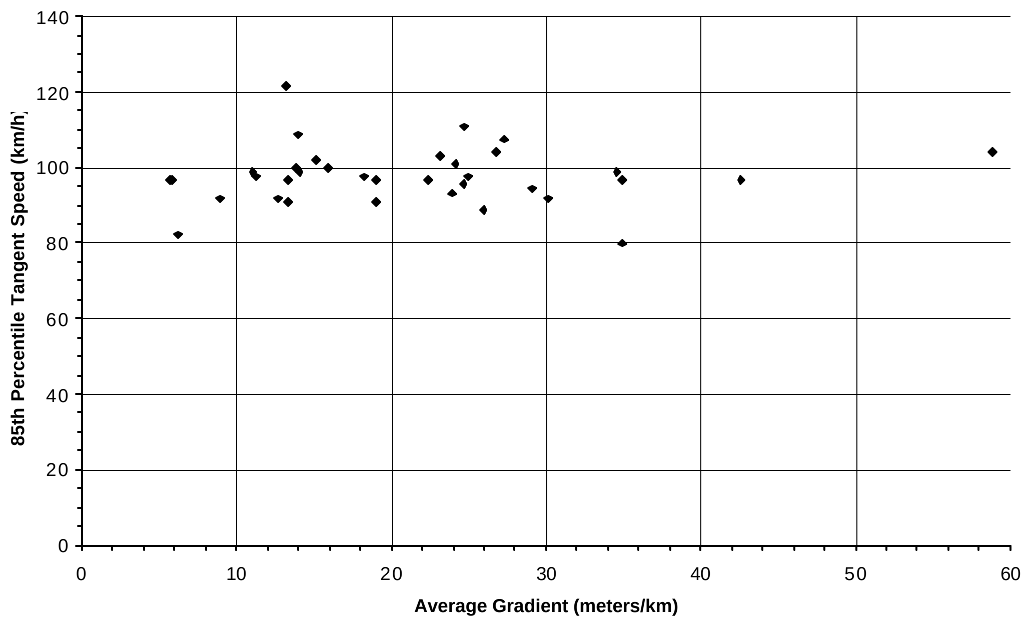


Figure 32. 85th Percentile Tangent Speed Versus Average Gradient.

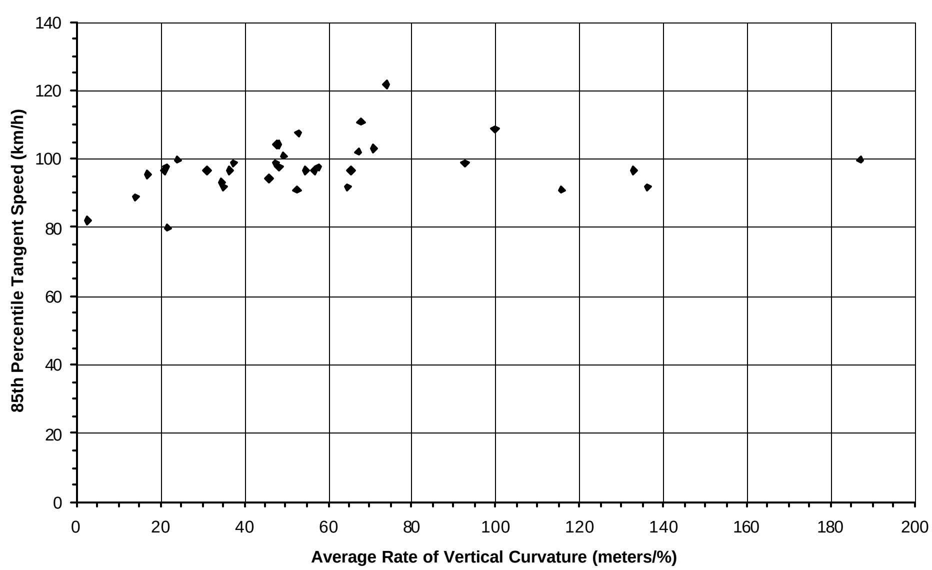


Figure 31. 85th Percentile Tangent Speed Versus Average Rate of Vertical Curvature.

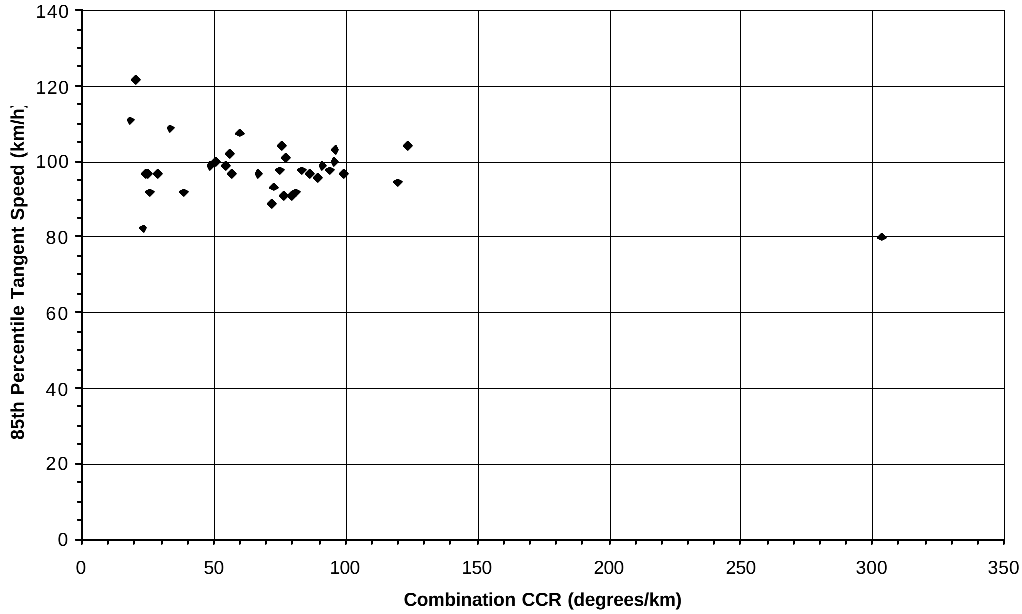


Figure 33. 85th Percentile Tangent Speed Versus Combination CCR.

From these graphs, there appeared to be a relationship between the 85th percentile speeds and several of the alignment indices. The apparent relationships are indicated below:

- As the curvature change rate increased, the 85th percentile tangent speeds decreased.
- As the degree of curvature index increased, the 85th percentile tangent speeds decreased.
- As the average radius increased, the 85th percentile tangent speeds increased.
- As the average tangent increased, the 85th percentile tangent speeds increased.
- As the vertical CCR increased, the 85th percentile tangent speeds decreased.
- As the combination CCR increased, the 85th percentile tangent speeds decreased.

The relationships that were identified from the graphs were consistent with the hypothesized relationships discussed in the development of the alignment indices. However, most of the trends observed in these graphs appeared to be slight, indicating that the alignment indices did not have much of an effect, if any, on the desired speeds of motorists on long tangents. Finally, several of the relationships suggested by these graphs appeared to be influenced by one or more extreme values.

Simple Linear Regression Analysis

To determine whether there was a statistically significant relationship (at a = 0.10) between the 85th percentile tangent speeds and these alignment indices as shown in the graphs, simple linear regression analyses were performed. The results of the regression analyses are listed in table 24.

Table 24. Regression Results.

		$ß_0$	$ß_1$	p-Value	R^2	MSE[a]
HORIZONTAL	CCR	101.53	-0.06	0.03	0.12	53.79
	DC	100.06	-0.14	0.01	0.17	50.73
	CL:RL	99.52	-4.93	0.69	0.00	61.11
	AVG R	91.57	0.01	0.01	0.23	47.20
	AVG T	95.33	0.01	0.22	0.04	59.00
VERTICAL	V CCR	99.41	-0.29	0.41	0.02	59.15
	V AVG K	94.78	0.05	0.15	0.06	56.64
	V AVG G	97.04	0.03	0.81	0.00	60.27
COMPOSITE	COMBO	102.10	-0.06	0.02	0.16	50.78
[a]MSE = Mean Square Error						

From the p-values in table 24, all of the alignment indices that visually appeared to have an effect on the 85th percentile tangent speed did have a significant effect (p-value < a=0.10), except average tangent and vertical CCR. However, there were concerns about these results. The first issue was the low coefficient of determination (R^2) for models containing the individual alignment indices. These low R^2-values indicate that the alignment indices do not explain much of the variability in speeds on long tangents of two-lane rural highways.

The second issue concerns the graphs of 85th percentile tangent speeds versus the alignment indices. In these graphs, there appeared to be several data points that controlled the general trend of the data and, therefore, had a great influence on the resulting relationships. Using an INFLUENCE option in the regression procedure for SAS, several indicators were used to determine the amount of influence each observation had on the estimates of the regression analysis. This investigation identified four sites that had a significant amount of influence on the predictive power of the alignment indices. These sites were usually located at or near the extreme values of the alignment indices. In addition, looking at the characteristics of these influential sites, one of these sites had the lowest observed 85th percentile tangent speed—80 km/h. Of the other three data points, one observation was on a roadway with a posted speed of 80.5 km/h. The other two data points represented locations where the posted speed limit was the highest—112.7 km/h. Excluding these four sites, the remaining locations all had posted speeds of 88.5 km/h. Therefore, it can be concluded that these observations were influential due to their alignment index values and their posted speeds.

In order to determine the effects these influential points had on the relationships identified using these observations, simple linear regression analyses were performed again without including these four data points. These analyses were also performed to see whether the results obtained without the influential observations were the same as the results obtained including the influential observations.

The results of these simple regression analyses are in table 25. The results show that none of the alignment indices were statistically significant predictors of 85th percentile tangent speeds. In addition, the coefficient of determination was near zero for all of the alignment indices. These results indicate that at a significance level of a = 0.10, none of the alignment indices were individually able to estimate the 85th percentile tangent speeds of motorists.

After further examining the results of the simple linear regression analyses, it was decided that the influential observations were to be excluded from the data sets. With such limited data available at the extreme values, it was difficult to accept with confidence the results that were obtained by including these extreme values. Based on the regression results without the influential observations, it was determined that none of the alignment indices were significant predictors of the 85th percentile speeds of motorists.

Table 25. Regression Results Without Influential Points.

		$ß_0$	$ß_1$	p-Value	R^2	MSE
HORIZONTAL	CCR	97.18	0.01	0.77	0.00	30.26
	DC	99.12	-0.10	0.37	0.02	29.61
	CL:RL	93.11	13.59	0.19	0.05	28.79
	AVG R	97.03	0.00	0.71	0.00	30.21
	AVG T	97.15	0.00	0.76	0.00	30.25
VERTICAL	V CCR	98.22	-0.12	0.61	0.01	23.57
	V AVG K	96.87	0.01	0.64	0.01	23.60
	V AVG G	95.94	0.07	0.35	0.03	23.08
COMPOSITE	COMBO	97.12	0.01	0.86	0.00	23.76

Examination of a Multiple Linear Regression Model in Estimating Desired Speed

Since none of the alignment indices were determined to individually predict the 85th percentile speeds for motorists on tangents in the simple linear regression analyses, the use of a combination of alignment indices and geometric variables as predictors in a multiple linear regression model was examined. However, the use of posted speed as an independent variable was not considered here, because all the sites had the same posted speed (88.5 km/h). Stepwise multiple linear regression analysis was performed to determine if any combination of variables could estimate 85th percentile speeds. These results are in table 26.

The results provided a multiple linear regression model that appears to be adequate in predicting the 85th percentile tangent speed of motorists. All of the p-values were significant and the model R^2 of 0.51 was acceptable; however, this suggested model contains independent variables that were identified as being highly correlated. The variables in this model that were highly correlated are listed below and should not be used in the same model:

- Ratio of Curve Length to Road Length and Average Tangent.
- Average Tangent and Average Gradient.

Using a combination of the four variables suggested by the stepwise multiple linear regression results, and not including the highly correlated variables together in the same model, multiple regression analyses were performed again. The results of these analyses are in table 27.

Table 26. Stepwise Multiple Regression Analysis Results.

Model	Variable Entered	R^2		p-Value	MSE
		Partial	Model		
1	CL:RL	0.12	0.12	0.05	20.83
2	AVG T	0.19	0.32	0.01	16.79
3	WIDTH	0.13	0.44	0.02	14.18
4	V AVG G	0.06	0.51	0.07	13.04

Only one of these models that used combinations of the four variables was determined to be a significant estimator of 85th percentile speeds. According to the p-values of the variables, the model containing the ratio of curve length to road length alignment index and the total pavement width was significant; however, there were two concerns with using this model to predict speeds. The first concern was the low R^2 value (0.14) associated with this model. The second concern was that the signs of the parameter estimates were counter-intuitive. This model indicates that as motorists drive a higher proportion of curves along the road, their speeds on long tangents will increase; however, a curvilinear alignment is believed to reduce operating speeds. Although once a driver reaches a long tangent after a series of curves, a higher speed could result because the driver welcomes the opportunity to be able to drive faster. In addition, however, this model indicates that as the total pavement width of the roadway increases, the desired speed of motorists on long tangents decreases. In requirements for minimum lane and shoulder width values for two-lane rural highways, previous research suggests that wider pavements usually result in higher operating speeds by motorists.[60] Due to these concerns, it was determined that a model including only these two variables to estimate desired speeds on long tangents was not acceptable.

Possible Influences on Desired Speed

The results from the previous sections indicated that none of the alignment indices examined were able to predict the desired speeds of motorists on long tangents, even in combination with geometric variables. Therefore, this section considers other factors and variables that may have an influence on the speeds motorists drive on long tangents. In examining these influences, all 68 tangent sites were included unless otherwise stated.

Regional Speed Differences

The mean 85th percentile speeds of the different regions in this study were examined to determine if there was a difference in the speed characteristics of motorists by region. Regional differences in speed may be attributed to the different alignment and motorist characteristics that are believed to exist in different locations across the United States. Table 28 lists the mean 85th percentile tangent speeds from each State.

Table 27. Multiple Regression Analysis Results.

Model		CL:RL	AVG T	V AVG G	WIDTH	R^2	MSE	$ß_0$	No. of Obs.
1	$ß_1$	13.59				0.05	28.79	93.11	36
	p-value	0.19							
2	$ß_1$	17.87			-1.25	0.14	26.89	101.53	36
	p-value	0.08			0.07				
3	$ß_1$	18.96		0.05		0.14	21.27	89.65	31
	p-value	0.07		0.54					
4	$ß_1$	30.99		0.04	-1.70	0.28	18.36	98.91	31
	p-value	0.01		0.62	0.03				
5	$ß_1$		0.002			0.00	30.25	97.15	36
	p-value		0.76						
6	$ß_1$		0.002		-0.96	0.06	29.45	104.87	36
	p-value		0.79		0.17				

Table 28. 85th Percentile Speeds on Long Tangents by State.

State	No. of Sites	85th Percentile Tangent Speed (km/h)
Minnesota	2	99.2
New York	9	93.2
Oregon	21	96.3
Pennsylvania	8	93.1
Texas	13 (9)*	104.3 (103.2)
Washington	15	94.8
* Nine of the Texas sites had posted speeds of 88.5 km/h or less and four had posted speeds greater than 88.5 km/h.		

Performing an analysis of variance on these results, it was determined that the 85th percentile speeds for long tangents in Texas significantly differed from all of the other States, except Minnesota. These speed differences may be due to the presence of four sites in Texas that had posted speeds greater than 88.5 km/h. In addition, factors such as the roadside clear zone and the familiarity of motorists with the roads are two other possibilities for this speed difference between Texas and the other States.

In a previous study, differences in 85th percentile speeds by region were also examined.[(7)] The previous research used the same States as in this study, with the exception of Minnesota, and investigated tangents greater than 244 m in length. Table 29 provides a comparison between the previous results and this research. In order to provide a fair comparison to the previous research, the results from this study were grouped into the same regions used in the previous research, and only tangent lengths greater than 244 m from this current research were used. In addition, only roads with posted speeds of 80.5 km/h and 88.5 km/h from the current research were included, since these were the posted speeds examined in the previous research.

Table 29. Comparison of Regional Differences in 85th Percentile Tangent Speeds.

Region	Previous Study[(7)]	Current Research[a]			
	Grade Between -5 and +5 percent	All Grades		Grades Between -5 and +5 percent	
	Speed (km/h)	Speed (km/h)	No. of Sites	Speed (km/h)	No. of Sites

South	99.8	105.3	7	105.3	7
East and West	96.0	95.5	49	96.4	43
All Regions	97.9	96.7	56	97.7	50
[a] Includes sites with tangent lengths greater than 244 m and posted speeds of 80.5 or 88.5 km/h.					

The East and West region combined the results of New York, Oregon, Pennsylvania, and Washington speed data. The South region consisted of speed data in Texas. Based on the results of these two studies, it appears that there are State variations in the desired speeds of motorists on long tangents. Analysis of variance of these results confirmed that 85^{th} percentile speeds on long tangents in Texas were significantly greater than the 85^{th} percentile speeds for the other States.

Further comparison of the results of these two studies indicates that the means of the 85^{th} percentile speeds on long tangents of two-lane rural highways were similar (97.9 and 96.7 km/h). A t-test was performed to compare these two means, and the results indicated no significant difference in the 85^{th} percentile tangent speeds for these two studies. In addition, including tangents greater than 200 m in this study, the mean of the 85^{th} percentile tangent speeds (96.2 km/h) was also not significantly different from the previous research. Therefore, the estimate of the desired speeds of motorists on long tangents used in the speed-profile model (97.9 km/h) should be retained under the following conditions:

- Tangent lengths > 200 m.
- Posted speed = 88.5 km/h.

Total Pavement Width

The total pavement width at the tangent sites for this study ranged from 6.2 to 12.7 m. Figure 34 is a plot of the 85^{th} percentile tangent speeds against the total pavement width for the 68 tangent sites. For analyzing the pavement width data, the widths were classified into three levels. These levels divided the width of the roadways into three categories: narrow, average, and wide. It was hypothesized that if there were any differences in speeds, they would exist between the narrowest and widest pavement widths. These classes of pavement width and their respective means are shown in table 30.

Analysis of variance results indicates that the mean 85^{th} percentile tangent speeds for these classes of widths were not significantly different from each other (p-value = 0.2614). Although travel-way width was found to be a significant predictor of 85^{th} percentile speeds on tangents in other studies, this research indicated that total pavement width was not a significant predictor of desired speeds of motorists on long tangents of two-lane rural highways.[15,62] A previous study also investigated the effects of pavement width on speeds on long tangents and concluded that pavement width did not affect motorists speeds.[7] Therefore, there are mixed results in determining whether total pavement width has

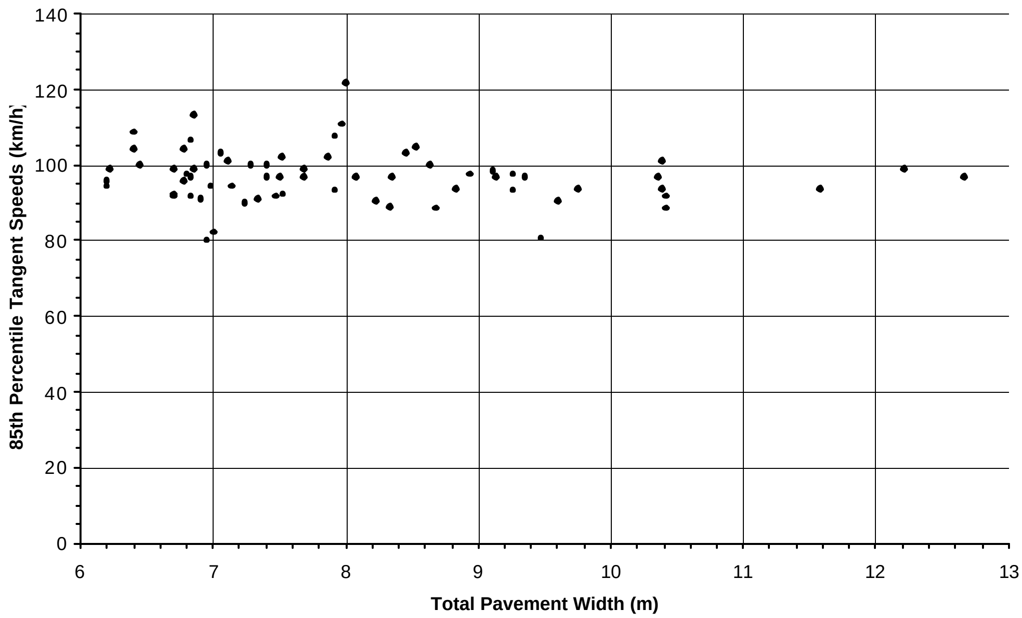


Figure 34. 85th Percentile Tangent Speed Versus Total Pavement Width.

any effect on the desired speeds of motorists.

Table 30. 85th Percentile Speeds by Total Pavement Width.

Total Pavement Width (m)	No. of Obs.	85th Percentile Tangent Speed (km/h)
Width # 7.2	26	97.0
7.2 < Width # 8.4	20	98.4
Width > 8.4	22	95.1

Vertical Grade on Tangent

An additional independent variable examined for its effect on 85th percentile tangent speeds was the vertical grade present at the tangent site. A graph of 85th percentile tangent speeds against the vertical grade on tangents can be seen in figure 35. Analysis of variance indicated that a difference in the mean 85th percentile speeds existed (p-value = 0.0249) for different vertical grades on tangents. Tukey's studentized range test identified that the mean 85th percentile speeds for tangents on vertical grades greater than 4 percent were significantly lower than for tangents on vertical grades between and including -4 and +4 percent. While these results are for passenger vehicles only, it is expected that a greater difference in the 85th percentile speeds would exist for heavy trucks and recreational vehicles. Table 31 shows the results of this analysis.

Driveway Density

Driveway density is a measure of the number of driveways per length of roadway. High driveway density is believed to cause lower speeds. In this study, it would be preferable to conduct an analysis of the effects of driveway density on the speeds present at the 68 tangent sites used in the alignment indices study. Data could only be obtained, however, for those States for which video logs were available (Minnesota, Oregon, and Washington). Also, another major limitation was that only sites with low driveway density were selected. Therefore, most of the sites had no driveways, and only one of the sites had as many as three driveways.

In total, 41 sites (18 from Oregon, 17 from Washington, and 6 from Minnesota) were available for the analysis. Therefore, while not a direct comparison, the findings from the 41 sites could provide an appreciation for the effects of driveway density on speed at this project's study sites. The number of driveways was determined from the video logs, and the driveway density was computed by dividing the number of driveways by the length of the tangent section. As expected, the number of driveways within the study sites was small. At 44 percent of the tangents, there were no driveways. An additional 34 percent of the tangents had only one driveway. The maximum number of driveways was three. Because the length of many of the tangents and curves was short, however, some of the driveway densities were quite high. For example, one short tangent that had only one driveway had a driveway density of 63 driveways per kilometer. Therefore, the analysis looked at the number of driveways rather than driveway density.

Table 31. 85th Percentile Speeds by Vertical Grade on Tangent.

Grades (%)	**No. of Obs.**	**85th Percentile Tangent Speed (km/h)**
Grade < -4	**6**	**95.3**
-4 # Grade < 0	**24**	**97.4**
0 < Grade # 4	**26**	**98.9**
4 < Grade	**12**	**91.9**

Figure 36 is a plot of the 85th percentile speed versus the number of driveways on long tangents. As expected, given the limited range of number of driveways that were observed, no relationship between 85th percentile speed and number of driveways was evident. Regression analysis results (p-value) corroborate the lack of a significant relationship between these variables.

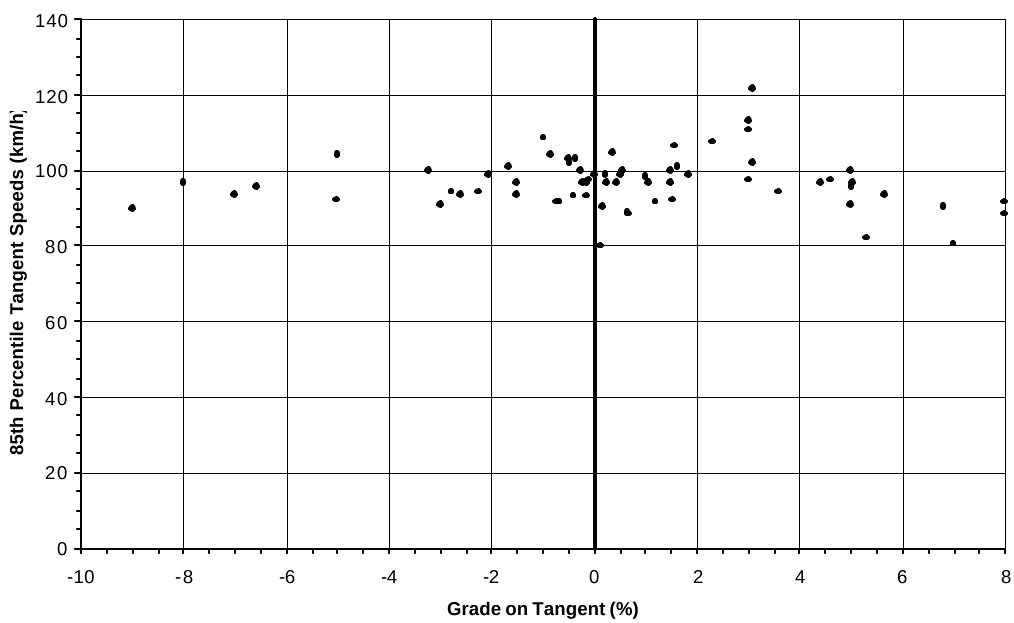


Figure 35. 85th Percentile Tangent Speed Versus Vertical Grade on Tangent.

Roadside Rating

The roadside environment was classified using Zegeer's seven-point scale.[48] The scale considers the type and proximity of roadside features to the travel lane. A value of 1 represents an open environment, while a value of 7 represents a roadway with roadside features close to the travel way. Photo logs and video recordings were used by two researchers working independently when determining the roadside rating. The ratings for the 68 long tangents are shown in figure 37. A notable variation in speed exists for each of the roadside ratings. The variable was not significant when used within a regression analysis to determine whether it could be used to predict speeds on the 68 long tangents (p-value = 0.1797).

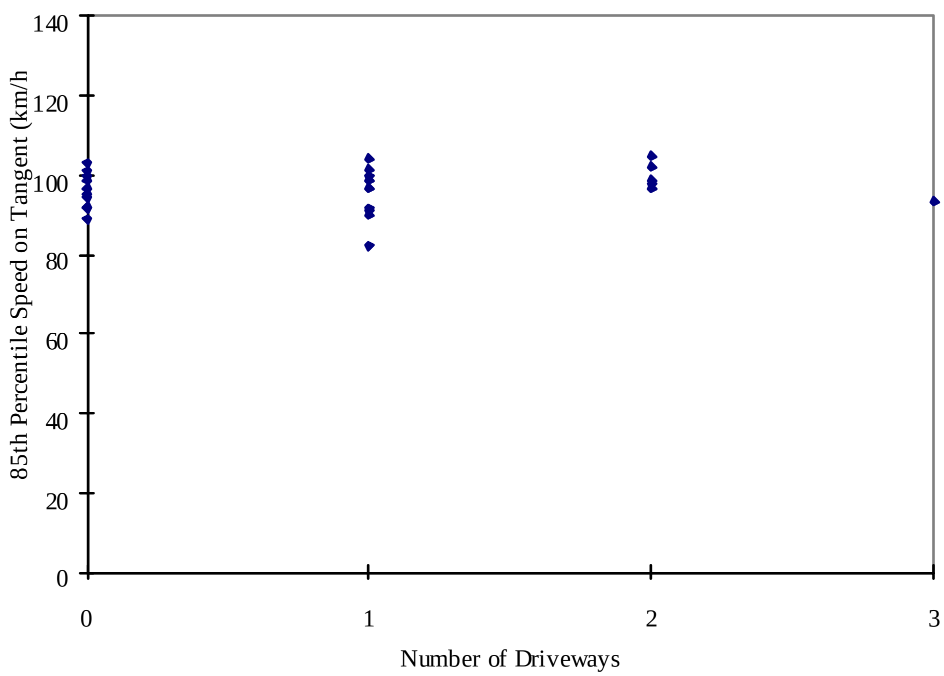


Figure 36. 85th Percentile Tangent Speed Versus Driveway Density for 41 Two-Lane Rural Highway Study Sites.

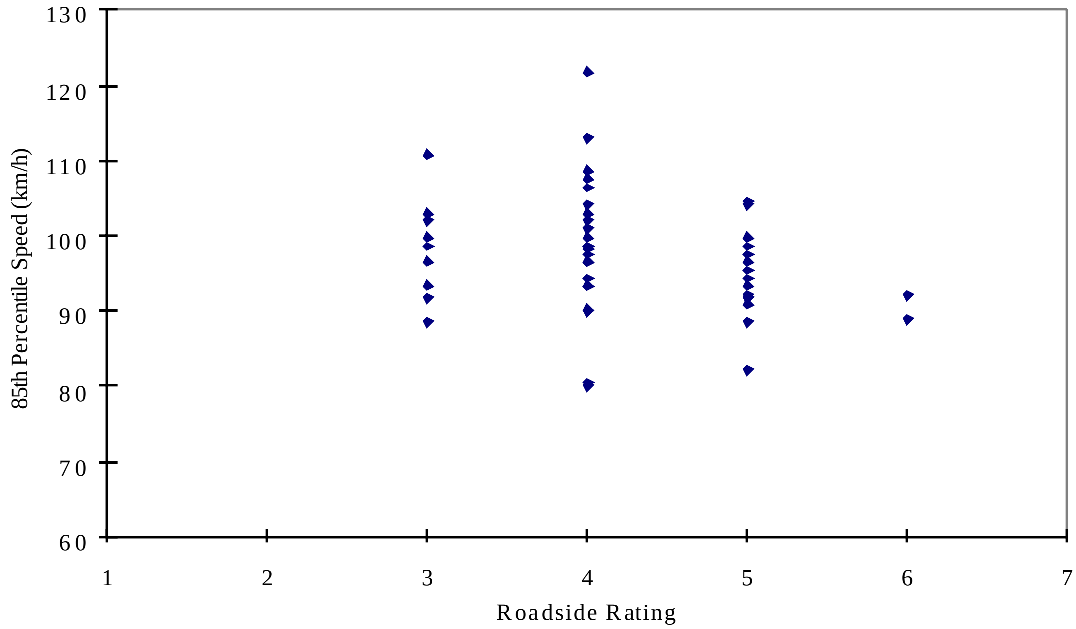


Figure 37. 85th Percentile Tangent Speed Versus Roadside Rating.

SUMMARY FOR ALIGNMENT INDICES

It was hypothesized that motorists select the speed they drive based on previous alignment characteristics they have encountered. Since alignment indices quantify the general character of the roadway alignment, they were used in this research to represent the geometry motorists encounter upstream of a long tangent. Using this principle, the objective of this effort was to evaluate the applicability of using alignment indices in estimating the desired speeds of motorists on long tangents of two-lane rural highways.

Alignment indices were identified and developed using previous research efforts. In order to compute and evaluate the indices, spot-speed and alignment data were collected from sites located in six States across the United States. After the alignment data were entered into a computerized format, the alignment indices were calculated.

These alignment indices were then compared to the observed 85th percentile speeds on long tangents. Graphical and statistical analyses were performed to determine if the alignment indices were significant predictors of the desired speeds of motorists on long tangents of two-lane rural highways. In addition, other possible influences of desired speeds of motorists were examined.

The findings indicated that combinations of alignment indices and other geometric variables were not able to significantly predict the 85th percentile speeds of motorists on long tangents of two-lane rural highways. The 85th percentile tangent speeds on two-lane rural highways were determined to be affected by the region of the country and the vertical grade of the tangent. These results were similar to those obtained in a previous study on speeds of long tangents of two-lane rural highways.[(7)]

5. VEHICLE PERFORMANCE USING TWOPAS EQUATIONS

Chapter 4 dealt with passenger car speeds primarily for level tangents. However, it has been demonstrated in table 31 that steep upgrades reduce passenger car speeds. This effect is more pronounced for heavy vehicles, such as trucks and recreational vehicles. The following discussion presents equations that can be used to represent the effect of grades on the speeds of specific vehicle types, including passenger cars, trucks, and recreational vehicles.

Upgrades have the effect of limiting the accelerations that vehicles can achieve, thus making it difficult for drivers to maintain their desired speed. If the grade a vehicle is ascending is steep enough, the vehicle will be forced to decelerate. If the grade is long enough, the vehicle will eventually decelerate to a crawl speed. A vehicle at its crawl speed can continue up the grade at a constant speed without decelerating further, but cannot accelerate. A vehicle's crawl speed on a specific grade is a function of the steepness of the grade and the performance characteristics of the vehicle. The length of grade traversed until the vehicle reaches its crawl speed is a function of the steepness of the grade and the performance characteristics of the vehicle, as well as the vehicle's initial speed as it enters the grade.

This section of the report presents equations that can be used to estimate the speed profiles of particular vehicles as they proceed up particular grades. Thus, the equations can be used to predict the speed of the vehicle at any point on the grade if its initial speed at the entry to the grade is known. More generally, the model can be used to determine the maximum acceleration that a vehicle can achieve at any point on the road. This model can be used, for example, to determine whether vehicle performance capabilities will limit the rate at which a vehicle can accelerate when moving from a horizontal curve to a tangent, not just on an upgrade, but even on the level or on a downgrade. In other words, if a speed-profile model is being applied to any geometry, the equations presented below can be used to check whether the speed change projected at any point on the road exceeds the vehicle capability and, thus, whether the acceleration rate (i.e., the magnitude of the speed change per unit time) could be limited accordingly.

TWOPAS MODEL

The vehicle performance equations presented below are those used in the TWOPAS model.[63] TWOPAS is a microscopic simulation model of traffic on two-lane rural highways first developed in the mid-1970s and revised most recently in the mid-1980s. The operational analysis procedures for two-lane highways in the current chapter 8 of the *Highway Capacity Manual* (HCM) are based on an early version of the TWOPAS model.[49] TWOPAS is currently being improved in NCHRP Project 3-55(3) to serve as the basis for updating the HCM analysis procedures for the next HCM revision anticipated in 2000. The vehicle performance equations have proven to be very stable over time and are not expected to change as a result of this update. However, the performance-related characteristics of vehicles used as input data for those equations may well need to be updated. Note that the

TWOPAS equations are in English units. After the equations are used to determine vehicle speeds and distances, the values will need to be converted into metric equivalent values for use in the speed-profile model.

Research has also shown that drivers exercise restraint and do not generally use the full performance capabilities of their vehicle on grades.[(64)] Even on level sections or downgrades, drivers use limited acceleration and deceleration rates when accelerating or decelerating toward their desired speed. The performance equations presented below include mathematical models to account for such driver restraints and preferences, which should also be considered in estimating realistic vehicle-speed profiles.

VEHICLE-PERFORMANCE EQUATIONS FOR PASSENGER CARS AND RECREATIONAL VEHICLES

Extensive vehicle performance tests by St. John and Kobett found that passenger car and recreational vehicle acceleration rates on level terrain vary linearly, but inversely, with speed.[(64)] The following equation represents passenger car and recreational vehicle performance:

$$a = a_0 \left(1 - \frac{V}{V_m} \right) \tag{9}$$

where:
- a = acceleration capability of vehicle (ft/s^2 [9.81 m/s^2]) at speed V
- a_0 = maximum acceleration (ft/s^2) at zero speed, i.e., the highest acceleration that can be achieved by the vehicle at any speed
- V = current vehicle speed (ft/s)
- V_m = maximum speed (ft/s) attainable by the vehicle on level terrain

Thus, vehicle performance for passenger cars and recreational vehicles is modeled in equation (9) as a function of two fundamental vehicle characteristics:

- a_0, the vehicle's maximum acceleration capability (which occurs at zero speed, using maximum horsepower when starting from a stop).

- V_m, the vehicle's maximum speed when using maximum horsepower on level terrain.

These vehicle performance characteristics are often available in, or can be estimated from, tabulated results of vehicle performance tests. Figure 38 illustrates the fundamental vehicle performance relationship. The figure illustrates that the zero-speed acceleration and the maximum speed represent the y-intercept and the x-intercept, respectively, of the vehicle performance curve.

Another term must be added to the performance equation to account for grade effects:

$$a = a_0 \left(1 - \frac{V}{V_m} \right) - gG \tag{10}$$

where:
- g = acceleration due to gravity (32.17 ft/s^2)
- G = gradient expressed as a decimal fraction [9.81m/s](i.e., percent grade divided by 100; + for upgrades, - for downgrades)

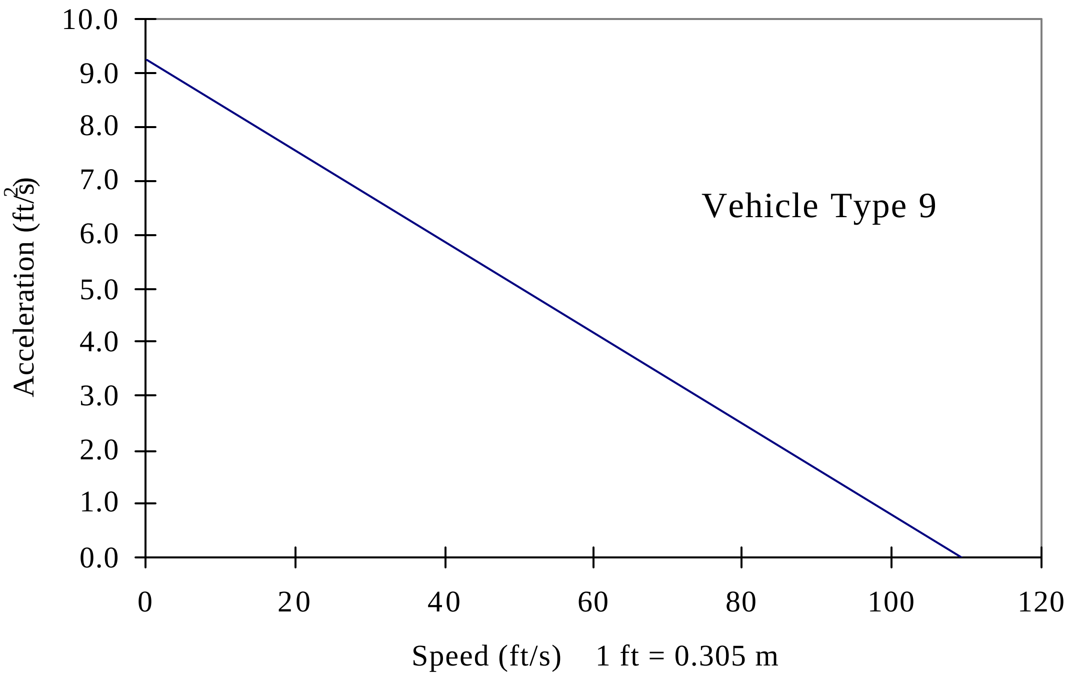


Figure 38. Example Passenger Car Performance Curve.

Finally, field studies have shown that drivers do not use the full acceleration capabilities of their vehicles for sustained periods of time. Drivers generally use, at most, 73 percent of their vehicle's acceleration capability and will not generally exceed 90 percent of their vehicle's maximum speed. Vehicle performance under these driver restraints can be modeled as:

$$a = 0.73\, a_0 \left(1 - \frac{V}{0.90\ V_m} \right) - gG \tag{11}$$

Although passenger car drivers generally employ the restraint on their speeds and accelerations indicated by equation (11), they occasionally do use the larger accelerations for short periods of time. Realistic driver acceleration and speed behavior have been modeled in TWOPAS with an algorithm based on an interplay between equations (10) and (11) that is described as follows: if a in equation (11) is less than or equal to zero, then the vehicle is constrained to that acceleration; however, when a in equation (11) becomes positive, then the driver will use the larger positive acceleration represented by the value of a in equation (10).

The algorithm described above is applied sequentially in TWOPAS over a 1-s interval. The value of acceleration (a) resulting from this algorithm represents the maximum acceleration rate that will be used during each 1-s interval. Where a has a negative value, it represents the largest deceleration rate that the vehicle can use over a 1-s interval. Where a has a positive value, it represents the largest acceleration rate that the vehicle can use over a 1-s interval.

Where vehicle performance limitations govern, the new speed at the end of the 1-s interval would be determined as:

$$V_n = V + a\,t \tag{12}$$

where: V_n = new speed (ft/s) at end of time interval t
t = duration of time interval (s) (generally, t=1)

Research has found that even when drivers are not limited by vehicle performance, the driver's preferred acceleration rate will be limited as a function of the magnitude of the difference between the driver's current speed and the desired speed.[64] Such driver preferences generally come into play only when the acceleration rate that would need to be used by the driver in returning to the desired speed exceeds 1.2 ft/s^2 (0.4 m/s^2). The following three equations represent limitations on the new speed based on the maximum preferred accelerations (or decelerations) for drivers for three specific cases:

If $|V_d - V| \leq 1.2$ then,

$$V_n = V_d \tag{13}$$

If $|V_d - V| > 1.2$ and $V_d - V > 0$ then,

$$V_n = V + (1.2 + 0.108\ |V_d - V|\ )\ t \tag{14}$$

If $|V_d - V| > 1.2$ and $V_d - V < 0$ then,

$$V_n = V - 1.2\ t \tag{15}$$

where: V_d = driver desired speed (ft/s)

If the new speed based on equation (13), (14), or (15), as appropriate, is lower than the new speed based on equations (10) through (12), then the lower speed based on equation (13), (14), or (15) will govern. Maximum acceleration preferences generally govern speed choices on the level, on downgrades, and on minor upgrades, but not on steep upgrades.

Once the speed at the end of the 1-s interval is known, the acceleration during the interval and the position at the end of the interval can be computed based on simple kinematic relationships as:

$$a = \frac{V_n - V}{t} \tag{16}$$

$$X_n = X_o + Vt + 0.5\ a\ t^2 \tag{17}$$

where: X_n = position at the end of the time interval of length t
X_o = position at the beginning of the time interval of length t

As explained above, vehicle performance limitations for passenger cars are a function of two vehicle characteristics, a_0 and V_m. Table 32 presents values of a_0 and V_m for five passenger cars with a range of performance capabilities. These five vehicles are not intended to represent any particular passenger car makes or models. Rather, it has been shown that their collective performance, when present in the proportions shown in the table, reproduces the collective performance of the passenger car population on two-lane highways in the mid-1980s. NCHRP Project 3-55(3) has found that the performance of passenger cars has increased since the mid-1980s and a revised version of table 32 is under development.

The algorithm presented above, represented by equations (10) through (17) can be applied to determine the maximum speed of a passenger car traveling through any specified sequence of grades, given an initial speed of that car at the beginning of the section. These equations are applied interactively by 1-s intervals so that the predicted speed at the end of any 1-s interval becomes the initial speed at the beginning of the next 1-s interval.

Table 32. Passenger Car Performance Characteristics.

Vehicle Type[a]	Maximum Acceleration (ft/s^2)[b]	Maximum Speed (ft/s)[c]	Percentage of Passenger Car Population (%)	Passenger Car Performance
9	9.28	109.1	10	lowest
10	9.77	114.9	15	
11	10.09	118.7	20	
12	10.43	122.7	25	
13	11.20	131.8	30	highest

[a]Vehicle type identifier used in TWOPAS.
[b]Maximum acceleration at zero speed on level grade using maximum available horsepower.
[c]Maximum speed on level grade using maximum available horsepower.

To illustrate the application of the algorithm presented above, figure 39 presents a speed profile for a medium-performance passenger car (Vehicle Type 11 in table 32) that enters a constant 5-percent upgrade at a speed of 88 ft/s (60 mph) [96.6 km/h]. The passenger car will decelerate until it reaches a speed of 76.8 ft/s (52.5 mph) [84.5 km/h] and will then continue up the grade at that speed. The passenger car reaches this crawl speed after 48 s and at a distance of 4,200 ft (1,280 m) up the grade.

Figure 40 shows that a passenger car slowed by a 5-percent upgrade can recover its speed very quickly when the grade ends. When a level grade follows the 5-percent upgrade of 4,500 ft (1,372 m) in length, the vehicle will return to its desired speed of 88 ft/s (60 mph) [96.6 km/h] within 3 s. Table 33 lists a sample of the calculations performed that result in the speed-distance relationship shown in figure 40. The speeds and accelerations in figures 39 and 40 and in table 33 are computed as shown in table 34.

The algorithm from TWOPAS can be used to supply the effect of vertical geometry on speed profiles where the effect of horizontal geometry is determined from the speed-profile models. Any vertical geometric alignment, including straight grades or vertical curves or a mix, can be introduced by modifying the local grades shown in Column (5). The local grade in Column (5) should be the grade at the position shown in Column (4). The effects of horizontal curvature can be accounted for by reducing the desired speed in Column (2) of the tables at appropriate locations along the roadway. Consideration should be given to whether the driver acceleration preferences indicated in equations (13), (14), and (15) agree with driver deceleration/acceleration behavior entering and leaving horizontal curves observed in the field.

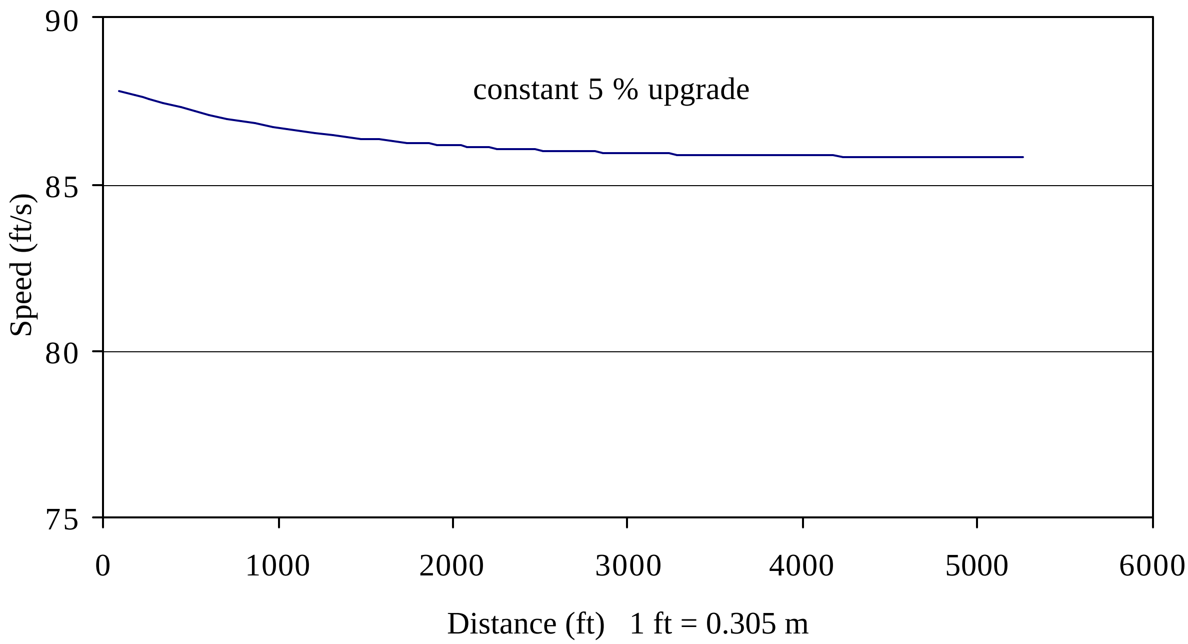


Figure 39. Passenger Car (Vehicle Type 11) on a Constant 5-Percent Upgrade.

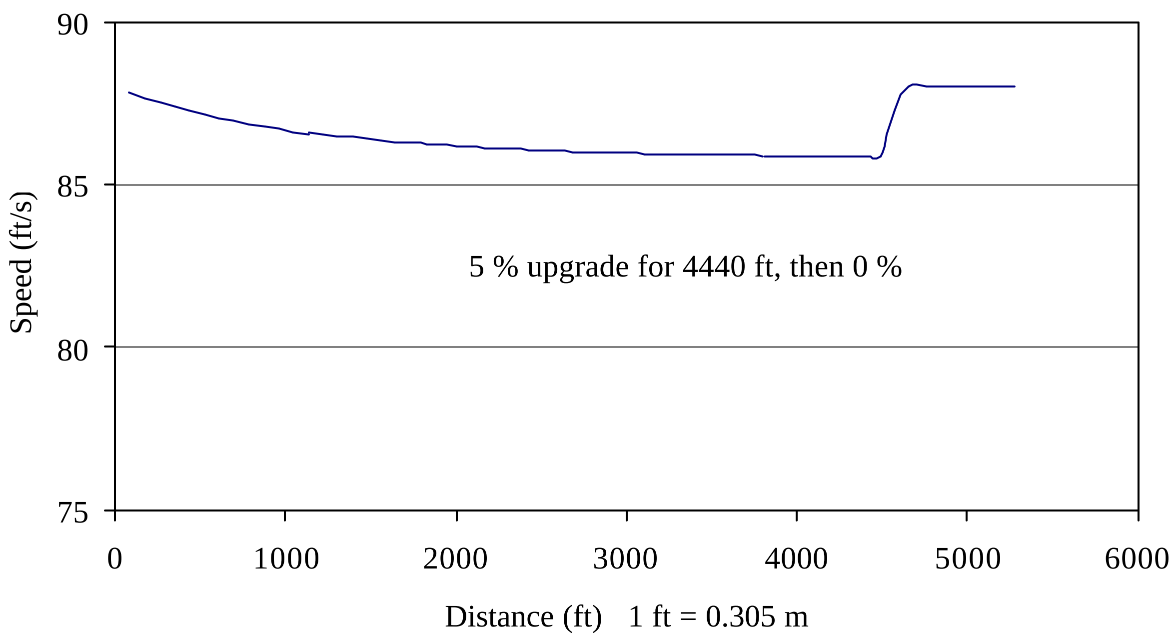


Figure 40. Passenger Car on a 5-Percent Upgrade Followed by a Level Grade.

Table 33. Limiting Accelerations and Speeds for a Medium Performance Passenger Car (Vehicle Type 11) on a Sustained 5-Percent Upgrade Followed by a Level Roadway (Sample of Data).

(1) Elapsed Time (s)	(2) Desired Speed (ft/s)	Start of 1-s Interval		(5) Local Grade (%)	Limiting Acceleration (ft/s^2)			(9) New Speed Based on Vehicle Performance (ft/s)	Limiting Acceleration and Speed Based on Driver Preferences		(12) Actual Acceleration (ft/s^2)	End of 1-s Interval	
		(3) Speed (ft/s)	(4) Position (ft)		(6) Without Driver Restraint	(7) With Driver Restraint	(8) Net		(10) Speed (ft/s)	(11) Acceleration (ft/s^2)		(13) New Speed (ft/s)	(14) New Position (ft)
0	88.0	88.0	0.0	5.0	1.00	-0.17	-0.17	88.8	88.0	0.00	-0.17	87.8	87.9
1	88.0	87.7	87.8	5.0	1.02	-0.16	-0.16	87.7	88.0	0.17	-0.16	87.7	175.7
2	88.0	87.4	175.4	5.0	1.03	-0.14	-0.14	87.5	88.0	0.32	-0.14	87.5	263.3
3	88.0	87.1	262.7	5.0	1.04	-0.13	-0.13	87.4	88.0	0.47	-0.13	87.4	356.7
4	88.0	86.9	349.7	5.0	1.05	-0.12	-0.12	87.3	88.0	0.60	-0.12	87.3	438.1
5	88.0	86.6	436.4	5.0	1.06	-0.11	-0.11	87.2	88.0	0.72	-0.11	87.2	525.1
6	88.0	86.4	523.0	5.0	1.07	-0.10	-0.10	87.1	88.0	0.83	-0.10	87.1	612.4
7	88.0	86.2	609.3	5.0	1.08	-0.10	-0.10	87.0	88.0	0.94	-0.10	87.0	699.4
8	88.0	86.0	695.4	5.0	1.09	-0.09	-0.09	86.9	88.0	1.03	-0.09	86.9	786.4
9	88.0	85.9	781.4	5.0	1.10	-0.08	-0.08	86.8	88.0	1.12	-0.08	86.8	873.2
49	88.0	85.8	4231.5	5.0	1.18	-0.01	-0.01	85.8	87.3	1.43	-0.01	85.8	4317.3
50	88.0	85.8	4317.3	5.0	1.18	-0.01	-0.01	85.8	87.3	1.43	-0.01	85.8	4403.2
51	88.0	85.8	4403.2	5.0	1.18	-0.01	-0.01	85.8	87.3	1.43	-0.01	85.8	4489.0
52	88.0	85.8	4489.0	0.0	2.79	1.61	2.79	88.6	87.3	1.43	1.43	87.3	4575.6
53	88.0	87.3	4575.6	0.0	2.67	1.50	2.67	89.9	88.0	0.73	0.73	88.8	4663.2
54	88.0	88.0	4663.2	0.0	2.61	1.44	2.61	90.6	88.0	0.00	0.00	88.0	4751.2
55	88.0	88.0	4751.2	0.0	2.61	1.44	2.61	90.6	88.0	0.00	0.00	88.0	4834.2
56	88.0	88.0	4837.2	0.0	2.61	1.44	2.61	90.6	88.0	0.00	0.00	88.0	4927.2

1 ft = 0.305 m

Table 34. Speed and Acceleration Computation Procedure.

Column	Value
(1)	Elapsed time (s).
(2)	Driver desired speed (ft/s), V_d.
(3)	Vehicle speed (ft/s) at start of 1-s interval, V.
(4)	Vehicle position at start of 1-s interval.
(5)	Local grade (%).
(6)	Acceleration (ft/s^2) as limited by vehicle performance without driver restraint, a in equation (10).
(7)	Acceleration (ft/s^2) as limited by vehicle performance with driver restraint, a in equation (11).
(8)	Net acceleration based on vehicle performance limits. If Column (7) is less than or equal to zero, use Column (7), otherwise, use Column (6).
(9)	Speed (ft/s) at end of 1-s interval as limited by vehicle performance, based on equation (12).
(10)	Speed (ft/s) at end of 1-s interval based on equation (13), (14), or (15), as appropriate.
(11)	Acceleration rate over 1-s interval based on the difference between the speed shown in Column (3) to the speed shown in Column (10).
(12)	Actual acceleration (ft/s^2) over the 1-s interval based on the difference between the speed shown in Column (3) to the speed shown in Column (13). This represents the more critical of the acceleration rates shown in Columns (8) and (11).
(13)	Speed (ft/s) at the end of the 1-s interval. Based on the lower of the speeds in Columns (9) and (10).
(14)	New position (ft) at the end of the 1-s interval, based on equation (17).

The same logic shown above can be used to model the speed profiles of recreational vehicles, which are more severely affected than passenger cars by grades. Table 35 presents vehicle performance characteristics for four recreational vehicles, comparable to those shown for passenger cars in table 32, that collectively represent the population of recreational vehicles on two-lane highways. As in table 32, these vehicles do not represent any particular make or model of recreational vehicle, but collectively, in the proportions shown in the table, they represent the overall performance of the recreational vehicle population. The vehicle characteristics shown in table 35 are based on the work of Werner during the 1970s.[(65)] This is considered the definitive study of recreational vehicle performance. NCHRP Project 3-55(3) is updating table 35 based, in part, on the Werner data and in part on the increased performance of light trucks found during the updating of table 32. The latter approach is appropriate because many recreational vehicles use the same engine and chassis as comparable light trucks, although recreational vehicles typically weigh more and have greater aerodynamic drag.

The speed of a recreational vehicle on a grade can be determined with the same model as the speed of a passenger car using the vehicle performance characteristics for recreational vehicles shown in table 35 (or its successor).

Table 35. Recreational Vehicle Performance Characteristics.

Vehicle Type[a]	Maximum Acceleration (ft/s^2)[b]	Maximum Speed (ft/s)[c]	Percentage of Passenger Car Population (%)	Recreational Vehicle Performance
5	8.22	78.7	1.2	lowest
6	8.64	89.7	48.8	
7	8.75	96.0	48.8	
8	8.76	97.5	1.2	highest

[a]Vehicle type identifier used in TWOPAS.
[b]Maximum acceleration at zero speed on level grade using maximum available horsepower.
[c]Maximum speed on level grade using maximum available horsepower.

VEHICLE-PERFORMANCE EQUATIONS FOR TRUCKS

Modeling of vehicle performance for trucks requires different performance equations because additional vehicle characteristics come into play. In addition to engine performance, gearshift delays and aerodynamic drag are important considerations. Specifically, the truck characteristics that influence performance on grades are:

- Weight-to-net-horsepower ratio (lb/hp).
- Weight-to-frontal-area ratio (lb/ft^2).
- Elevation correction factor for net horsepower.
- Elevation correction factor for aerodynamic drag.

The performance equations to determine the limiting value of truck acceleration on a grade are as follows:

$$a_c = -0.2445 - 0.0004\,V' - \frac{0.021\ C_{de}\ (V')^2}{(W/A)} - \frac{222.6\ C_{pe}}{(W/NHP)\ V'} - gG \qquad (18)$$

$$a_p = \frac{a_c + \dfrac{15368\ C_{pe}}{(W/NHP)V'}}{1 + \dfrac{14080}{(W/NHP)(V')^2}} \tag{19}$$

$$a_e = \frac{0.4\ V'\ a_p}{0.4\ V' + \dfrac{1.5\ a_p}{|a_p|}\ (a_p - a_c)},\ V \geq 10\ \text{ft/s} \tag{20}$$

$$a_e = \frac{10\ a_p}{10 + \dfrac{1.5\ a_p}{|a_p|}\ (a_p - a_c)},\ V < 10\ \text{ft/s} \tag{21}$$

where:

- a_c = coasting acceleration (ft/s^2) during gear shifts
- a_p = horsepower-limited acceleration (ft/s^2)
- a_e = effective acceleration (ft/s^2), including an allowance of 1.5 s for gearshift delays
- V' = larger of speed at beginning of interval (V) and 10 ft/s
- C_{de} = correction factor for converting sea-level aerodynamic drag to local elevation = $(1 - 0.000006887E)^{4.255}$
- C_{pe} = altitude correction factor for converting sea-level net horsepower to local elevation, equals 1 - 0.00004E for gasoline engines and equals 1 for diesel engines
- E = local elevation (ft)
- W/A = weight to projected frontal area ratio (lb/ft^2)
- W/NHP = weight to net horsepower ratio (lb/hp)

Equation (18) represents the coasting acceleration of the truck. Equation (19) represents the acceleration as limited by engine horsepower. Equations (20) and (21) combine the coasting and horsepower-limited accelerations into an effective acceleration that allows the truck to use maximum horsepower except during gearshift delays of 1.5 s, during which the truck is coasting (with no power supplied by the engine). This model of truck performance is based on Society of Automotive Engineers (SAE) truck-performance equations that were adapted by St. John and Kobett to incorporate gearshift delays.[64,66] There are no driver restraints on using maximum acceleration or maximum speed on upgrades because, unlike passenger car engines, truck engines are designed to operate at full power for sustained periods.

Truck performance is limited on grades where passenger car performance would not be limited. However, where the truck is not performance-limited, the driver acceleration preferences shown in equations (13), (14), and (15) also apply to trucks. The performance-limited speed of the truck on grade can be determined from the speed at the beginning of the 1-s interval, the limiting speed based on the performance-limited acceleration rate, and driver acceleration preferences in the same manner as used for passenger cars.

Table 36 presents values of these truck characteristics for four trucks that collectively represent the performance of the truck population on two-lane highways in the mid-1980s. NCHRP Project 3-55(3) has found that truck performance has increased since the mid-1980s. Specifically, the performance of the lowest performance trucks (such as Vehicle Types 1 and 2 in table 36) has increased substantially, while the performance of the highest performance trucks (such as Vehicle Types 3 and 4 in table 36) has changed very little. An updated version of table 36 is currently being developed in NCHRP Project 3-55(3).

Table 36. Truck Performance Characteristics.

Vehicle Type[a]	Weight to Net Horsepower Ratio (lb/hp)	Weight to Projected Frontal Area Ratio (lb/ft)	Altitude Correction Factor for:		Percentage of Truck Population (%)
			Net Horsepower	Aerodynamic Drag	
1	266	620	1.00	0.957	12.0
2	196	420	1.00	0.957	25.6
3	128	284	1.00	0.957	34.0
4	72	158	1.00	0.957	28.4

[a]Vehicle types identified are used in TWOPAS.

Equations (18) through (21) can be applied interactively with equations (13) through (15) to determine the speed of any specified truck at the end of any specified 1-s interval given the speed of the truck at the beginning of the 1-s interval and the local grade on which the truck is traveling.

Using TWOPAS for a 128-lb/hp (58.1-kg/hp) truck (Vehicle Type 3 in table 36) ascending a constant 5-percent grade, the truck decelerates on the 5-percent upgrade from its initial speed of 60 mph (96.6 km/h) to a crawl speed of 38.5 mph (62.0 km/h). This deceleration requires approximately 120 s and the truck reaches its crawl speed approximately 7,500 ft (2,286 m) up the grade when it ceases to decelerate (i.e., when its acceleration rate becomes zero). A 196-lb/hp (89.0-kg/hp) truck (Vehicle Type 2 in table 36) is less powerful than a 128-lb/hp (58.1-kg/hp) truck and reaches a lower crawl speed (27.1 mph [43.6 km/h] on a similar 5-percent grade). Deceleration to the truck's crawl speed still requires approximately 120 s, but the truck reaches only about 6,000 ft (1,829 m) up the grade before reaching its crawl speed.

Figure 41 presents the speeds for a 128-lb/hp (58.1-kg/hp) truck moving through rolling terrain alternating every 2,500 ft (762 m) between 2-percent upgrades and 2-percent downgrades. The figure shows that the truck decelerates from 88.0 to 85.5 ft/s (26.8 to 26.1 m/s) on each 2,500-ft (762-m) upgrade, but recovers its speed within 300 ft (91.4 m) on the subsequent downgrade.

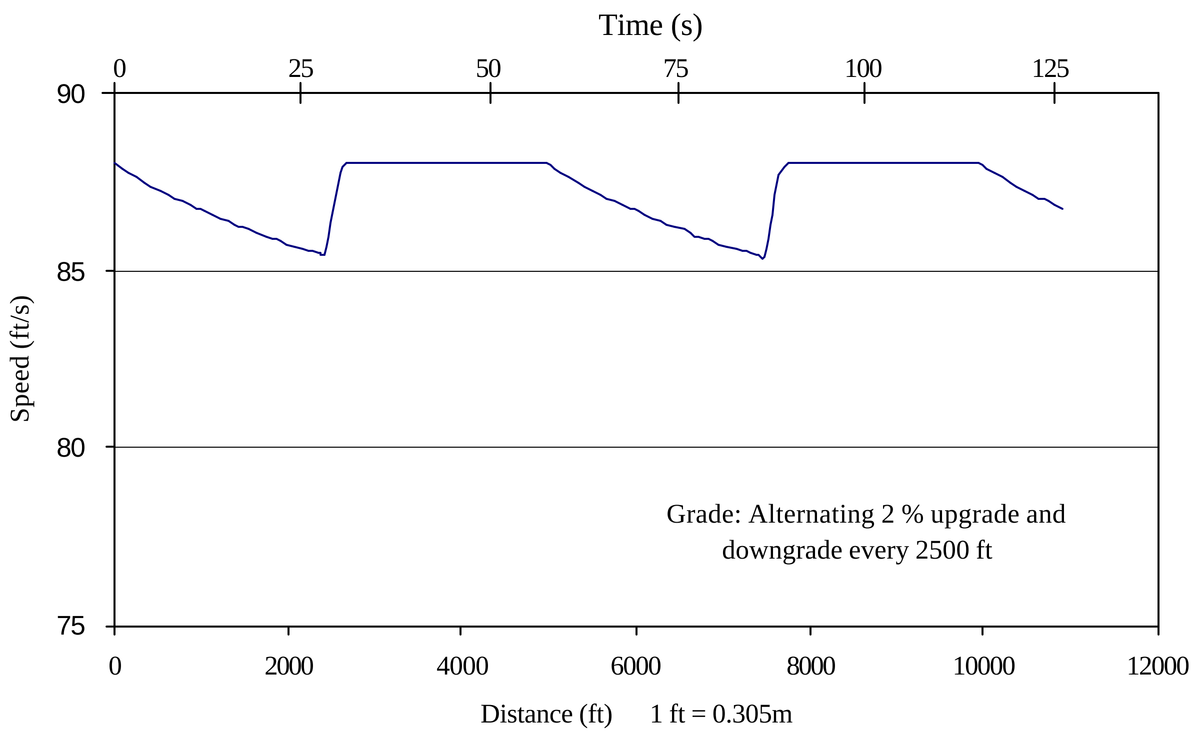


Figure 41. Truck Speed on Alternating 2-Percent Upgrades and Downgrades.

The truck findings are derived in a manner similar to the passenger car findings except that:

- Column (6): Coasting acceleration (ft/s^2), a_c, based on equation (18).
- Column (7): Horsepower-limited acceleration (ft/s^2), a_p, based on equation (19).
- Column (8): Effective acceleration (ft/s^2), a_e, based on equations (20) and (21).

The logic shown in the procedure presented above can be used to model truck performance as affected by vertical and horizontal geometry. Grades and vertical curves can be represented by using the appropriate local gradients in Column (5). The effects of horizontal curvature can be accounted for by modifying the desired speeds in Column (2) based on the developed speed-profile models.

SUMMARY FOR GRADE EFFECTS

The preceding discussion has shown how the effect of a grade on the speed of a passenger car, recreational vehicle, or truck can be determined. Speeds determined in this manner can be used as limiting values for tangent speeds determined in chapter 4. For example, if it is established that the 85th percentile speed for trucks on level tangents in the southern United States is 105 km/h, but the truck in question is ascending a grade for which the limiting speed is 82 km/h, then the tangent speed for that truck should be assumed to be 82 km/h. This same concept could be used to establish limits for speeds for horizontal curves that are also located on grades.

6. ACCELERATION/DECELERATION MODELING

This chapter discusses the acceleration and deceleration issues involved in the formulation of the speed-profile model. In the model developed initially under Report No. FHWA-RD-94-034, there were two basic assumptions employed:[7]

- All acceleration and deceleration occur outside the limits of the horizontal curve.
- Acceleration and deceleration rates are constant and equal to 0.85 m/s^2.

These two assumptions are examined here using new field data from 21 sites in Texas and Pennsylvania. The following sections of the report describe the data collection, statistical analysis procedures, and results used in the validation of the acceleration/deceleration model assumptions.

ACCELERATION/DECELERATION DATA COLLECTION

The vehicle acceleration and deceleration speed data were collected at 21 sites in 2 States (12 in Pennsylvania and 9 in Texas). These sites represent two geographic regions: south and northeast. The sites were selected based on the general criteria presented previously in table 3. In addition to these criteria, some additional constraints were imposed to ensure that the effects of the individual design element (i.e., the horizontal curve) would be isolated, and thus provide a true measure of acceleration and deceleration rates. Therefore, the approach tangent was a minimum of 244 m and the vertical alignment was limited to ± 5-percent grades. Vertical curves were beyond the scope of the acceleration/deceleration portion of the study. The study site matrix shown in table 37 summarizes the number of study sites by site characteristics.

After each site was identified and inspected, a variety of data were collected, including alignment geometry, width of pavement, width of lanes, cross-section information, weather conditions, traffic control devices, lighting conditions, and terrain/environment description. Table 38 lists the basic geometric characteristics for the 21 sites.

Table 37. Study Site Matrix.

Horizontal Alignment					
Range of Radii		**> 400 m**	**300-399 m**	**200-299 m**	**< 200 m**
Vertical Alignment (% Grade)	0 to +5%	3 sites	5 sites	1 site	1 site
	0 to -5%	3 sites	6 sites	1 site	1 site

Table 38. Study Site Characteristics.

Site	Deflection Angle (E)	Length of Curve (m)	Radius (m)	Length of Approach Tangent (m)
PA1	18.2	139	437	306
PA2	18.2	139	437	180
PA3	7.0	107	873	283
PA4	7.0	107	873	273
PA5	55.0	243	250	153
PA6	32.9	167	291	153
PA7	16.0	127	437	559
PA8	21.6	164	437	294
PA9	11.1	85	437	461
PA10	32.9	167	291	367
PA11	43.5	531	699	448
PA12	43.5	531	699	378
TX13	40.0	203	291	724
TX14	33.0	260	437	165
TX15	93.4	407	250	675
TX16	60.9	310	291	481
TX17	29.5	257	500	355
TX18	29.5	257	500	504
TX19	60.9	310	291	481
TX20	89.4	273	175	269
TX21	89.4	273	175	537

Vehicle speeds were collected at each site using Light Detection and Ranging (Lidar) guns. Lidar is a multi-purpose tool that has become increasingly popular as an alternative to conventional radar speed guns. The use of Lidar in speed data collection has two major advantages over radar. First, Lidar can measure distances to a vehicle as well as the speed of the vehicle, while the radar guns can only measure speed. The second advantage is that the signal transmitted by Lidar travels in a straight line as opposed to the conical patterns of radar transmissions, making it easier to pinpoint the vehicle whose speed is being measured with the Lidar device, and the narrower laser beam is harder to detect by radar/laser detectors. The information that the Lidar devices provide (i.e., both the distance to, and the speed of the vehicle) enables the speeds of individual vehicles to be measured at known points along the roadway. This information can be translated into acceleration/deceleration rates.

The Kustom-Pro-Laser was used in this data-collection effort. This is the off-the-shelf device used by police in speed enforcement. It has the capability of “tracking” a vehicle’s speed over a given distance and downloading the speed and distance information onto a laptop computer. A computer program (Laser-Speed 1.0) developed by the Midwest Research Institute was used to store data in an ASCII file.[67] In addition to the speed and distance data collected by Lidar, the user has the capability of manually recording additional information, such as the type and color of each vehicle. This information was recorded and used in the data reduction.

Figure 42 is a sketch of a typical data-collection setup using three Lidars. Each of the Lidars collected speed and distance data for the areas shown in figure 42. The data collection procedures using the Lidar guns began with the determination of the point of curvature (PC), point of tangent (PT), and midpoint for each horizontal curve. After these points were determined, three data collectors set up the Lidar equipment in a direct line with the entering or exiting tangents and the midpoint of the curve as shown in figure 42.

Prior to the speed data collection, the clocks on the three laptop computers were synchronized to ensure that each vehicle could be easily identified and tracked through all three zones. Observers then positioned themselves so that they remained hidden from oncoming traffic in order to minimize the effects of their presence on the speed data. For the speed data collection, the Lidars were aimed at the vehicles’ most reflective point, preferably the license plate, headlights, or bumper. The vehicle speed measurements were collected starting at a minimum of 200 m prior to the PC of the curve. Speeds were then recorded as a vehicle proceeded through the study site. Similarly, on the departure tangent, the vehicle speeds were recorded at a maximum of 200 m beyond the PT.

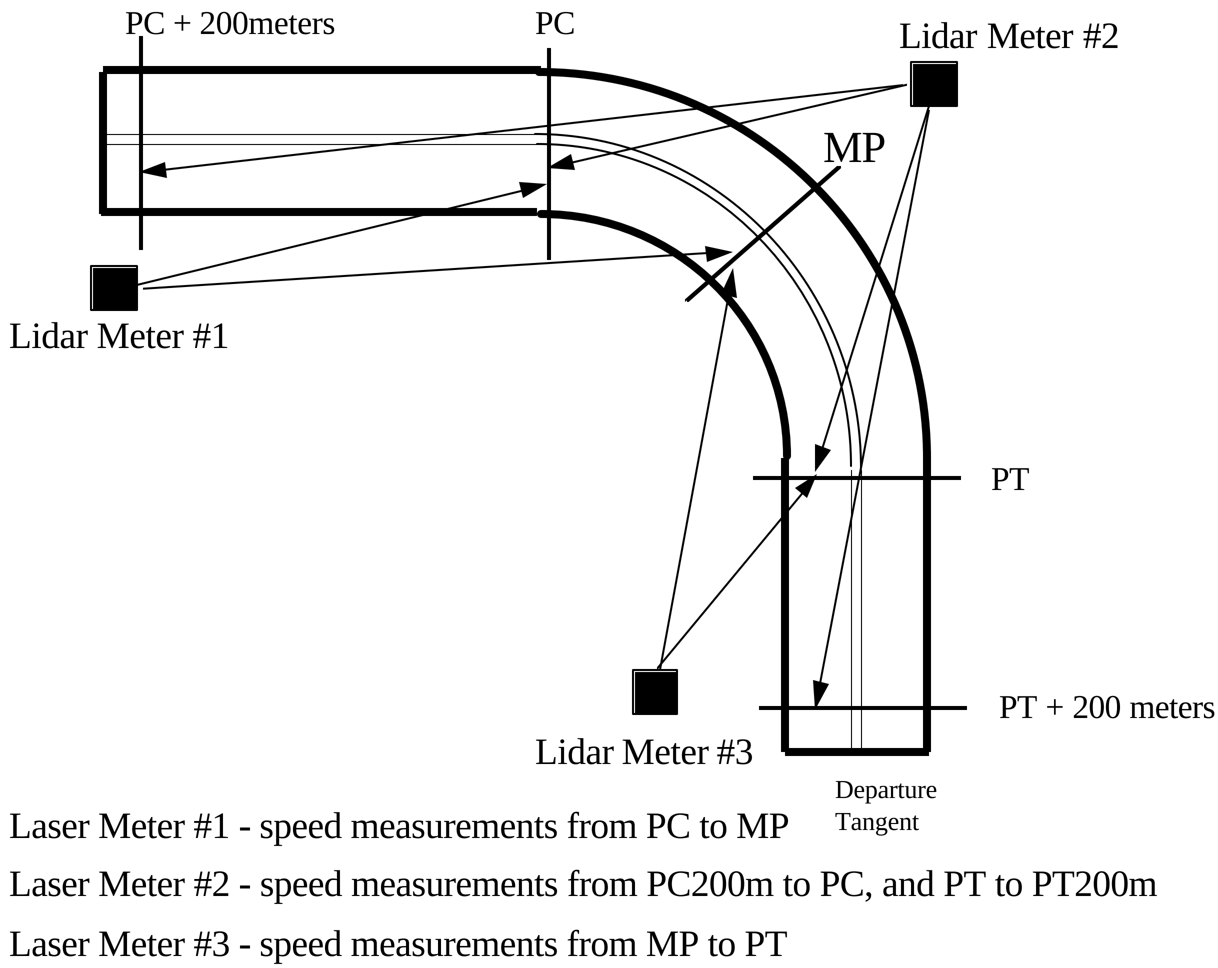


Figure 42. Typical Three-Laser-Meter Acceleration/Deceleration Data Collection Setup.

Vehicles selected for data collection had a minimum of 5 s of headway to ensure that they were not impeded. Vehicles were classified as passenger cars, pickup trucks, single-unit trucks, tractor-trailers, or recreational vehicles. At each acceleration/deceleration site, speed data were collected for a minimum of 100 passenger cars. For each vehicle, there were a minimum of 30 speed measurements from the approach tangent to the departure tangent. Sketches and pictures of each site were attached to the data collection forms to assist in data reduction.

DATA REDUCTION

The speed-data reduction process consisted mainly of identifying each speed-measurement location relative to the horizontal curve, and calibrating it to account for the cosine error. The cosine error occurs when the speeds are not measured in the direct path of the vehicle. In this case, the ratio of the Lidar speed reading to the true forward speed of the vehicle is equal to the cosine of the angle between the Lidar and the vehicle's direction of travel. As long as the location of the Lidar device relative to the horizontal curve can be identified, the angle can be determined and a "cosine correction" can be applied to estimate the actual vehicle speed. On tangents, where Lidar can measure speeds in or near the vehicle path, the cosine error is zero, or negligible. For vehicles on the horizontal curve, the angle between the laser beam and the vehicle's forward direction increases and is constantly changing through the curve. The equation for estimating the correct speed of the target vehicle is as follows:

$$\mathit{Correct\ Speed} = \mathit{Measured\ Speed} \times [1 + (1 - \cos(\frac{\mathit{arcsin\ cord\ length}}{\mathit{curve\ radius}}))] \qquad (22)$$

where the arcsin cord length is measured for each speed/distance reading from the Lidar gun to the vehicle.

After the correction of each speed measurement, point speeds were calculated at each of the locations shown in figure 43. These locations included the PC and PT points; one-quarter (QP), one-half (midpoint), and three-quarter (3QP) distances within the curve, as well as 50-m increments up to 200 m beyond either end of the curve. The calculation at the different locations will allow for data analysis at corresponding locations along the alignment for the different curves. Speeds were determined at each location by interpolating between the nearest corrected speed measurements on either side of the location. Acceleration/deceleration rates were also calculated for each of the zones shown in figure 43.

In calculating acceleration and deceleration rates, two potential approaches were considered.[(68)] The first approach involved estimating the average acceleration and deceleration rate for each vehicle over a selected number of zones, and then calculating the 85th percentile rate of these average acceleration/deceleration rates. In the second approach, the acceleration and deceleration rates are calculated directly from the 85th percentile speed at each location. The second approach was selected, since it would correspond to the 85th percentile speed profile.

ACCELERATION/DECELERATION ASSUMPTIONS—VALIDATION TESTS

This section describes the statistical procedures and analysis results conducted to evaluate the two acceleration and deceleration assumptions previously employed in the speed-profile model.

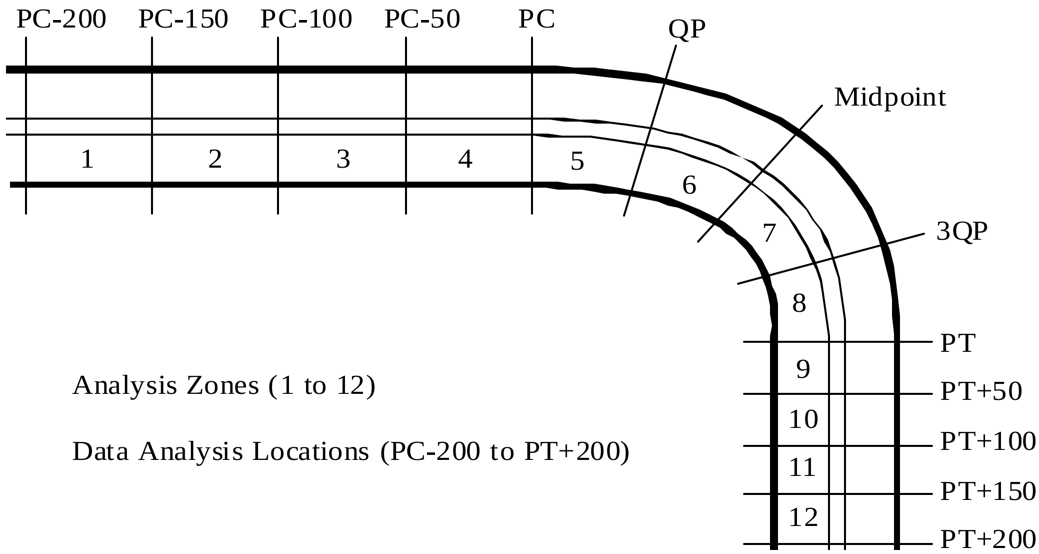


Figure 43. Speed Locations Used for Analysis.

Approach-Tangent Speed Check

Sites were selected to ensure that all drivers could reach their maximum desired speed on the approach tangent. Thus, the data obtained would include the full range of deceleration, since vehicles approaching the horizontal curve would be at their maximum desired speed. The previous speed-profile model assumes that the desired 85th percentile speed on approach tangents is equal to 97.9 km/h.[(7)] The first step in the data analysis was to determine whether the 85th percentile speed at the first location (PC-200) was equal to this desired speed on approach tangents (97.9 km/h). A t-test was conducted for the 21 sites to determine whether the average V_{85} speed of 97.08 km/h was statistically different from the assumed tangent speed of 97.9 km/h. It was concluded that the average V_{85} speed at PC-200 for the sites examined was not significantly different than the assumed speed (p-value=0.73). Thus, the average V_{85} did not start dropping until a point closer than 200 m to the PC.

Acceleration/Deceleration Assumptions Validation

This section of the report investigates the assumptions that acceleration and deceleration occur outside the limits of the curve, and they are both equal to 0.85 m/s^2. In estimating average acceleration or deceleration rates, two possible approaches emerged, related to the distances across which these rates are to be measured. The first approach estimates the average acceleration and deceleration rates occurring outside the limits of the curve and for 200 m on either side of it. This approach is consistent with the assumption that the speed through the horizontal curve is constant; it does not consider, however, the possibility that acceleration and/or deceleration may occur over different distances and locations at each site. For example, drivers may start decelerating before they pass the PC-200 point, or continue decelerating past the PC. The second approach would be to estimate the maximum acceleration and deceleration rates for each site, regardless of the distance over which these occurred. The second approach would result in higher average acceleration/deceleration rates than the first one. Both approaches were examined, and a total of five tests were conducted:

1. Test whether the rate of deceleration from PC-200 to the PC is significantly different from 0.85 m/s^2.

2. Test whether the rate of acceleration/deceleration within the curve limits is significantly different from 0.

3. Test whether the rate of acceleration from the PT to PT+200 is significantly different from 0.85 m/s^2.

4. Test whether the maximum rate of deceleration is significantly different from 0.85 m/s^2.

5. Test whether the maximum rate of acceleration is significantly different from 0.85 m/s^2.

In conducting these tests, the average acceleration or deceleration value was obtained for all 21 sites examined. The assumed rate of 0.85 m/s^2 from the Krammes et al. model applies to all sites, regardless of their geometric characteristics.(7) The t-test was selected for performing these analyses, because the sample size of the 85th percentile acceleration/deceleration rates was small (n=21). The results of the tests are presented and discussed in the following section.

Deceleration Rate From Approach Tangent to Point of Curvature (PC-200 to PC)

A t-test was performed to test whether the deceleration rate from PC-200 to the PC was equal to 0.85 m/s^2. The average deceleration rate within this zone for the 21 sites was 0.1143 m/s^2. The test found that the measured deceleration rate is significantly different from 0.85 m/s^2 (p-value < 0.0001). Thus, the statistical results showed that there is a significant difference between the assumed deceleration rate and the rate observed between PC-200 and the PC. The average rate was estimated for all 21 sites studied, and thus includes a variety of estimated deceleration values (from 0.01 to 0.54 m/s^2), which may be a function of geometric alignment and particularly the horizontal curve radius. Thus, drivers may apply higher deceleration rates for sharper (i.e., smaller radii) curves.

Another statistical test (one-sided t-test) was conducted on whether the observed deceleration rate was <u>less</u> than the assumed 0.85 m/s^2. The results of this t-test showed that the deceleration rate was significantly lower than 0.85 m/s^2. Thus, the assumed deceleration rate of 0.85 m/s^2 was a liberal estimate for the set of sites included in the study. In other words, the previous speed-profile model underestimates the distance used for deceleration between the tangent and the horizontal curve.

Acceleration/Deceleration Within the Horizontal Curve Limits (PC to PT)

A t-test was performed to examine whether there is any acceleration/deceleration occurring within the limits of the curve. This test showed that there is acceleration and deceleration occurring within the limits of the curve (p-value=0.015). The magnitude of this acceleration is, on average, 0.0724 m/s^2, which is small compared to assumed rates outside the limits of the curve (0.85 m/s^2).

Acceleration Rate From the PT Through the Departure Tangent (PT to PT+200)

A t-test was performed next to examine whether the acceleration rate from the PT through the departure tangent was significantly different from 0.85 m/s^2. The test showed that the acceleration rate is significantly different from 0.85 m/s^2 (p-value < 0.0001). The average acceleration rate within this zone for all 21 sites was 0.0448 m/s^2. Similar to estimated deceleration rates, there is a wide range of acceleration rates observed for the 21 sites (0.01 to 0.32 m/s^2), and thus it is possible that the acceleration rate is a function of a variety of geometric elements at each site.

A second t-test (one-sided) was performed to determine if the acceleration rate is significantly lower than 0.85 m/s^2. The t-test showed that the acceleration rate was significantly lower than 0.85 m/s^2. Thus, the assumed acceleration rate of 0.85 m/s^2 was a liberal estimate resulting in a shorter acceleration distance than observed in the field.

Maximum Observed Deceleration Rate

A t-test was performed to test whether the maximum deceleration rate observed was equal to the assumed rate of 0.85 m/s^2. The test showed that the maximum deceleration rate is significantly different from 0.85 m/s^2 (p-value < 0.0001).

Maximum Observed Acceleration Rate

A t-test was performed to test whether the maximum observed acceleration rate was equal to the assumed rate of 0.85 m/s^2. The test showed that the maximum acceleration rate is significantly different from 0.85 m/s^2 (p-value < 0.0001).

Validation Results Summary

The validation results indicate that the acceleration and deceleration assumptions employed in the existing speed-profile model are not valid for the set of selected study sites. The acceleration/deceleration values predicted and the acceleration/deceleration values measured in the field are statistically different at the 95-percent confidence level. As shown, the assumed rates are too liberal to be applied to all sites. Closer examination of the field data revealed that the only sites with acceleration and deceleration rates which approached 0.85 m/s^2 were those with curve radii less than 250 m.

As a result of the validation process for the acceleration/deceleration assumptions, it was concluded that new acceleration/deceleration models would need to be developed to consider the effects of the curve radius and potentially other geometric design elements in the vicinity of the horizontal curve. The following section describes the model formulation process and presents the recommended models developed.

DEVELOPMENT OF ACCELERATION/DECELERATION MODELS

The acceleration/deceleration database was used for developing revised models and rates. First, plots of 85th percentile speed profiles and acceleration/deceleration profiles for each site were constructed to observe general trends in the data. Figures 44 through 47 present speed profiles developed for each of the 21 sites, grouped by curve radius. In general, these plots show that speed drops are smaller for the larger curve radii and become much larger for sites with smaller curve radii. In addition, the locations of minimum and maximum speed vary widely from site to site, especially for the larger curve radii. After observing these plots, three potential analysis scenarios were considered:

Scenario 1—Analysis based on the following two analysis zones:

- PC-200 to midpoint of horizontal curve.
- Midpoint of horizontal curve to PT-200.

Scenario 2—Analysis based on three analysis zones:

- PC-200 to PC.
- PC to PT.
- PT to PT+200.

Scenario 3—Analysis based on the maximum acceleration and deceleration rates at each site.

Table 39 provides the data for the first two scenarios, while tables 40 and 41 provide the acceleration and deceleration data for the third scenario. Table 39 presents the estimated acceleration and deceleration rates by zone for the two scenarios for each of the 21 sites.

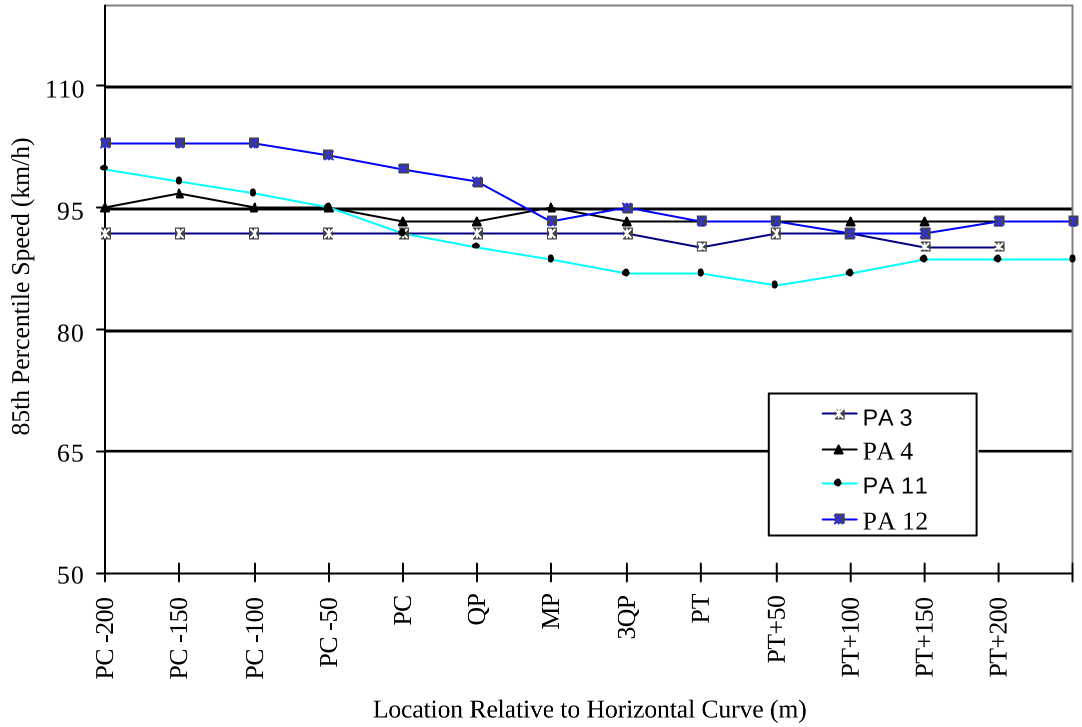


Figure 44. Speed Profile for Curves With Radii Greater Than 600 m.

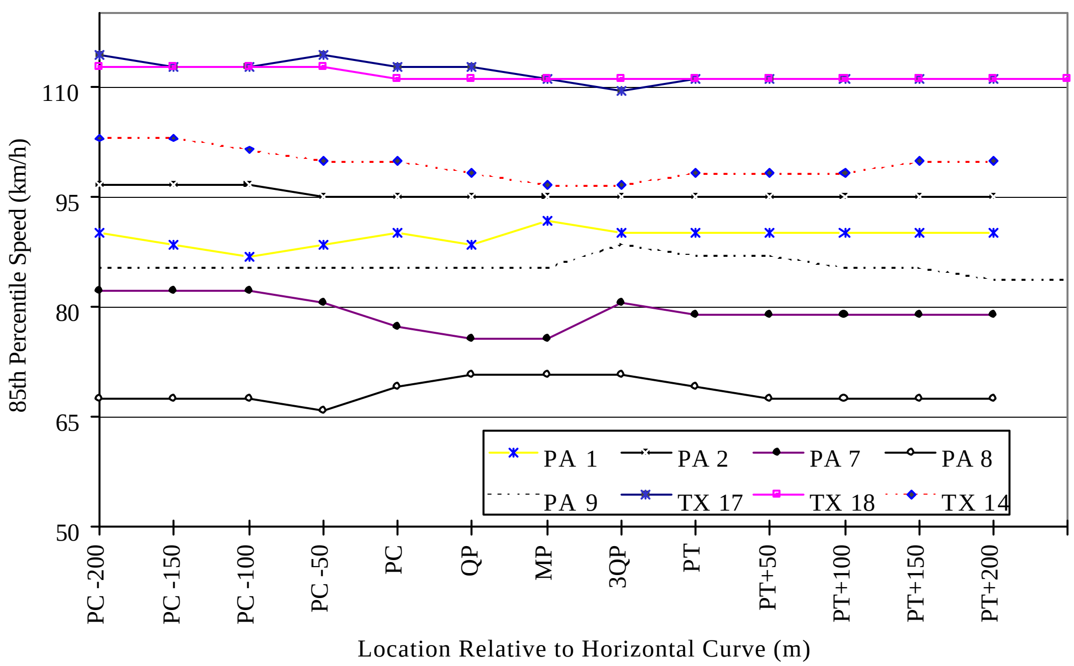


Figure 45. Speed Profile for Curves With Radii Between 437 and 499 m.

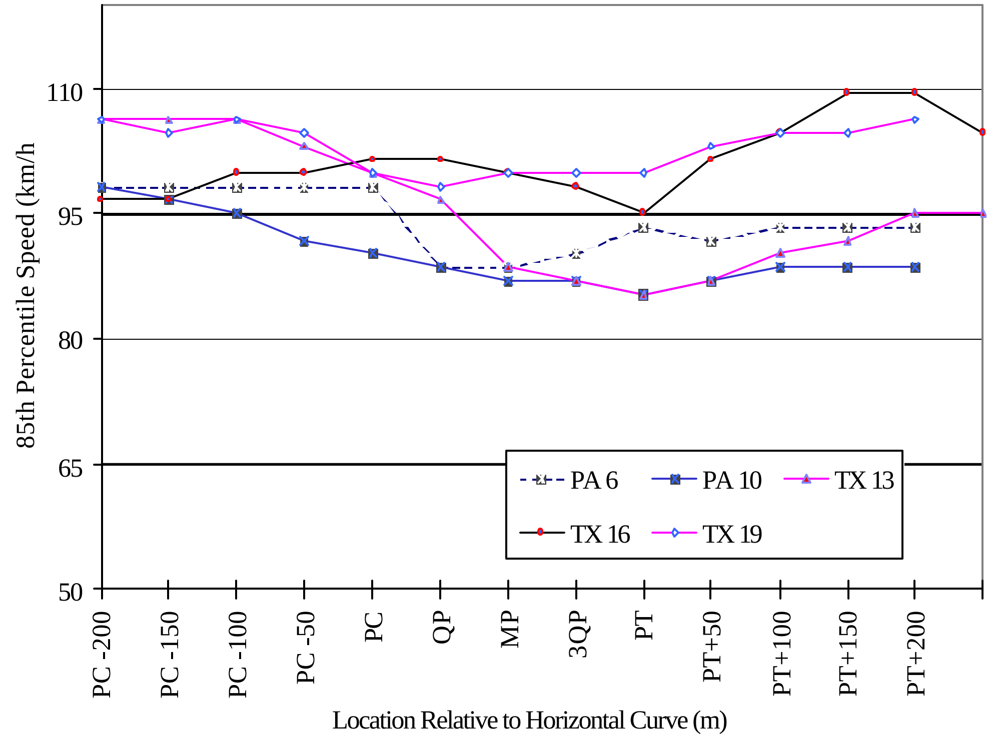


Figure 46. Speed Profile for 291-m Radii Curves.

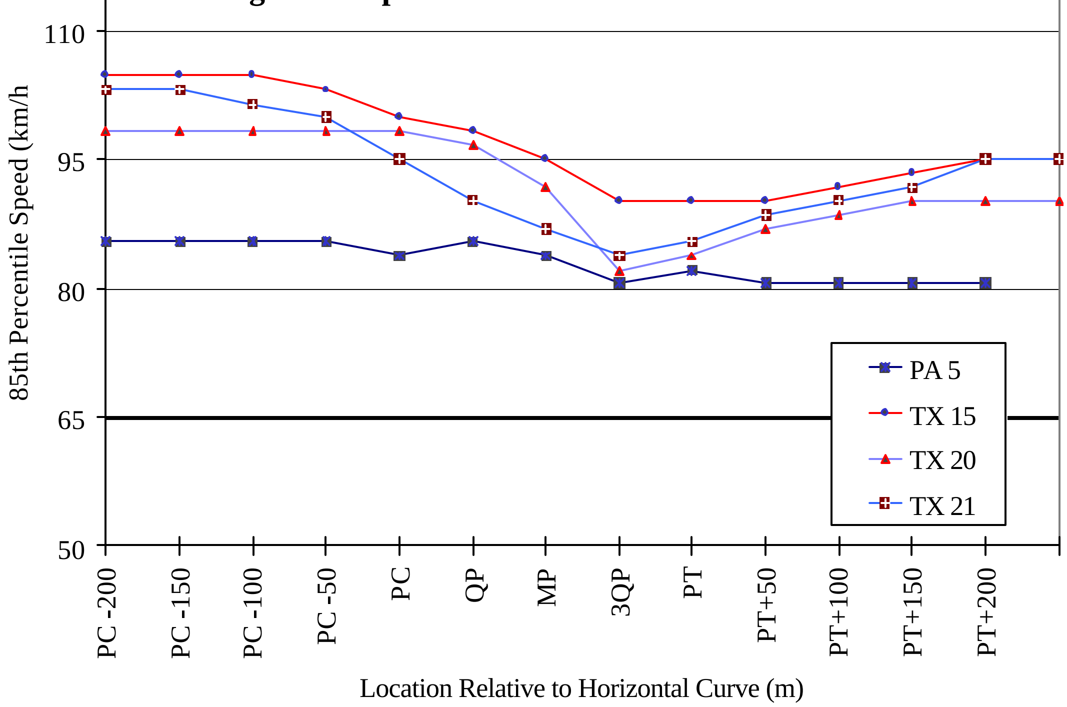


Figure 47. Speed Profile for Curves With Radii Less Than 250 m.

Table 39. Acceleration/Deceleration for Scenarios 1 and 2.

Site	Scenario 1—Acc/Dec Rates (m/s^2)		Scenario 2—Acc/Dec Rates (m/s^2)		
	PC-200 to Midpoint	Midpoint to PT+200	Approach Tangent	Within Curve	Depart Curve
PA1	-0.04	0.00	-0.04	0.00	0.00
PA2	0.00	-0.08	-0.01	0.13	-0.20
PA3	-0.01	-0.01	-0.01	-0.02	0.00
PA4	-0.01	0.00	-0.01	-0.02	-0.01
PA5	-0.03	-0.04	-0.01	-0.12	0.00
PA6	-0.08	0.08	-0.04	0.00	0.05
PA7	-0.09	-0.04	-0.11	0.00	-0.05
PA8	0.07	0.08	0.01	0.02	-0.05
PA9	-0.17	-0.01	-0.20	0.00	-0.01
PA10	-0.45	0.01	-0.32	-0.14	0.05
PA11	-0.09	0.00	-0.21	0.00	0.01
PA12	-0.14	-0.01	-0.12	-0.04	0.00
TX13	-1.19	0.19	-0.44	-0.48	0.32
TX14	-0.07	0.01	-0.05	-0.01	0.01
TX15	-0.52	0.06	-0.19	-0.17	0.11
TX16	0.01	0.11	-0.11	0.00	0.04
TX17	-0.06	-0.01	-0.03	-0.05	0.00
TX18	-0.10	0.00	-0.01	0.11	0.11
TX19	-0.11	0.12	-0.19	0.00	0.21
TX20	-0.76	0.19	-0.01	-0.34	0.05
TX21	-1.10	0.37	-0.54	-0.06	0.21
Note: Negative value implies deceleration, while positive value indicates acceleration.					

Table 40 presents the locations at which the maximum and minimum speeds were observed at each site. The maximum deceleration rate, estimated from the maximum and minimum speeds and their locations, is also shown. From table 40, it is evident that deceleration may start at different locations upstream from the PC. Note that for the second site (PA2), the maximum speed occurs at the PT, because the horizontal curve is not very sharp, and the speed does not fluctuate significantly from the approach tangent through the horizontal curve. The same can be observed for other, relatively flat horizontal curves, at which speed variations seem to be occurring randomly through the tangent and horizontal curve. Minimum speeds were also observed at various locations ranging from PC-50 to PT+50. As shown, the 3QP has the highest number of minimum speed occurrences (eight). Table 41 presents the minimum and maximum speeds observed and used in estimating the maximum acceleration rates. As for the maximum deceleration rates, there is wide fluctuation observed in the minimum and maximum speed locations.

Table 40. Scenario 3— Location of Maximum and Minimum 85th Percentile Speeds and Maximum Deceleration Rates for Sites.

Site No. (Radius [m])	PC-200	PC-150	PC-100	PC-50	PC	QP	MP	3QP	PT	PT+50	Max Dec Rate (m/s^2)
PA1 (147)		**max**							**min**		-0.04
PA2 (437)									**max**	**min**	-0.26
PA3 (873)			**max**						**min**		-0.05
PA4 (873)				**max**			**min**				-0.01
PA5 (250)			**max**					**min**			-0.15
PA6 (291)				**max**		**min**					-0.45
PA7 (437)	**max**							**min**			-0.14
PA8 (437)							**max**			**min**	-0.18
PA9 (437)	**max**				**min**						-0.20
PA10 (291)	**max**								**min**		-0.45
PA11 (699)	**max**							**min**			-0.16
PA12 (699)		**max**						**min**			-0.17
TX13 (291)		**max**						**min**			-1.44
TX14 (437)		**max**						**min**			-0.10
TX15 (250)		**max**					**min**				-0.59
TX16 (291)					**max**			**min**			-0.17
TX17 (499)				**max**				**min**			-0.14
TX18 (499)			**max**	**min**							-0.05
TX19 (291)			**max**			**min**					-0.33
TX20 (175)				**max**			**min**				-1.38
TX21 (175)	**max**						**min**				-1.10
Totals—Max	5	5	4	4	1		1		1		20

Totals—Min				1	1	2	4	8	3	2	20

Table 41. Scenario 3—Location of Maximum and Minimum 85th Percentile Speeds and Maximum Acceleration Rates for Sites.

Site No. (Radius [m])	**PC-50**	**PC**	**QP**	**MP**	**3QP**	**PT**	**PT+50**	**PT+100**	**PT+150**	**PT+200**	**Max Acc. Rate (m/s^2)**
PA1 (437)	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	0.00
PA2 (437)		**min**			**max**						0.17
PA3 (873)		**min**		**max**							0.00
PA4 (873)					**min**	**max**					0.00
PA5 (250)			**min**	**max**							0.01
PA6 (291)			**min**				**max**				0.12
PA7 (437)					**min**	**max**					0.08
PA8 (437)		**min**		**max**							0.97
PA9 (437)						**min**	**max**				0.00
PA10 (291)						**min**		**max**			0.10
PA11 (699)					**min**			**max**			0.21
PA12 (699)								**min**	**max**		0.05
TX13 (291)					**min**				**max**		0.57
TX14 (437)					**min**				**max**		0.04
TX15 (250)						**min**			**max**		0.15
TX16 (291)					**min**			**max**			1.77
TX17 (499)						**min**	**max**				0.05
TX18 (499)					**min**	**max**					0.98
TX19 (291)			**min**					**max**			0.27
TX20 (175)				**min**				**max**			0.44
TX21 (175)				**min**					**max**		0.64
Totals—Max				3	1	3	3	5	5		20
Totals—Min		3	3	2	7	4		1			20

After examining these tables, it was concluded that the most appropriate analysis method is Scenario 3—maximum acceleration and deceleration rates. As shown in tables 40 and 41, the location at which drivers start decelerating or accelerating varies by site. The maximum and minimum speeds do not occur at the same location for each site, and analyzing the data based on only one pre-selected zone would not capture the actual acceleration and deceleration rates, nor the full effect of the horizontal curve on speeds.

Acceleration/Deceleration Rates—Model Development

Using the maximum acceleration and deceleration rates (Scenario 3), the following alternatives were considered for developing models to predict acceleration/deceleration rates:

1. Regression models were developed using all curve radii sites.
2. Regression models were developed using sites with radii < 436 m and using constant (average) rates for sites with radii $\geq$ 436 m.
3. Regression models were developed using sites with radii < 291 m and using constant (average) rates for sites with radii $\geq$ 291 m.
4. Regression models were developed using sites with radii between 175 and 436 m and using constant (average) rates for sites with radii $\geq$ 436 m and radii < 175 m.
5. Acceleration Rates—Constant average rates for all sites.
Deceleration Rates—Adjusted regression models developed using sites with radii < 436 m and constant acceleration for sites with radii $\geq$ 436 m.

Acceleration/Deceleration Rate Recommendations

Based on the results from the alternatives outlined above, it is recommended that for deceleration rates, the linear regression model should be used for curves with radii > 175 m, and assume -1.0 m/s^2 deceleration for curves with radii < 175 m and 0 m/s^2 for radii $\geq$ 436 m. The recommended deceleration model is shown in figure 48. This model was recommended because of the relatively seamless transition between the two site groups (curve radii < 175 m and curve radii $\geq$ 175 and < 436 m).

The recommended acceleration model is shown in figure 49. As shown, a step-function is recommended instead of a regression-type model. Regression analyses resulted in very low R^2 for all analysis alternatives, and thus it is recommended that constant (average) rates for sites based on their curve radii be used.

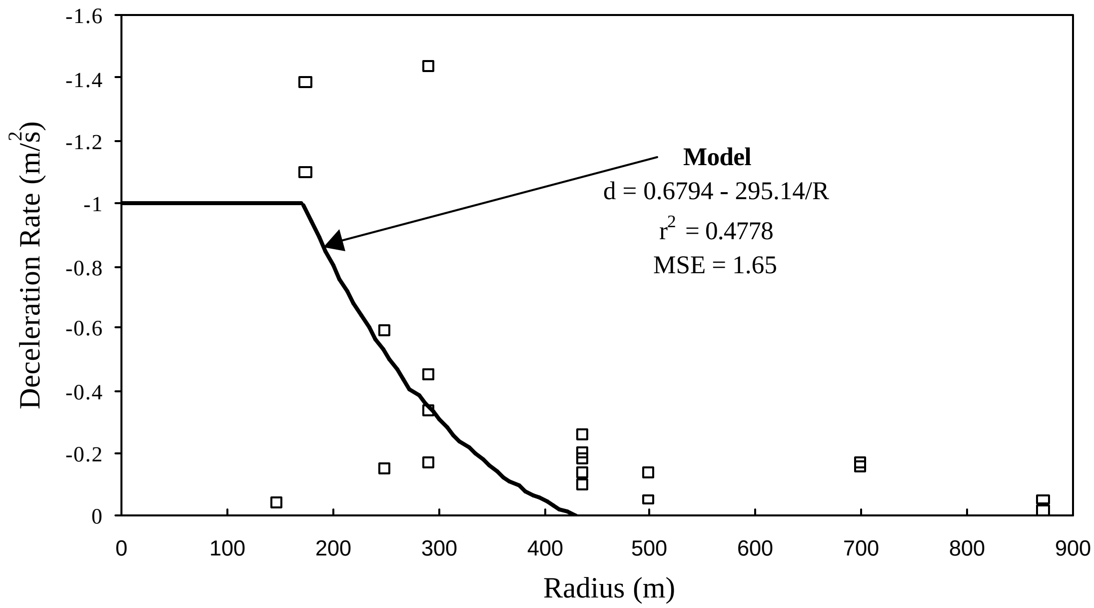


Figure 48. Deceleration Rates.

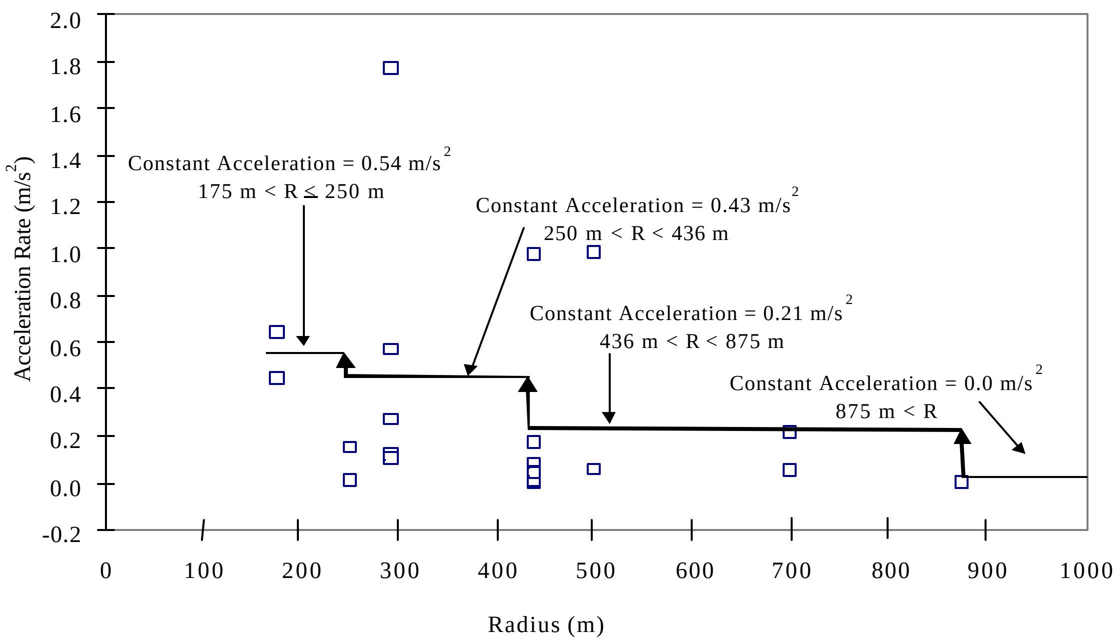


Figure 49. Constant Acceleration Rates.

7. VALIDATION OF SPEED-PREDICTION EQUATIONS

This chapter presents the validation results for the speed prediction equations developed in chapter 3 and presented in table 14. The objective of this validation effort is to evaluate the accuracy with which the speed prediction equations developed estimated passenger car speeds along various combinations of horizontal and vertical alignment segments.

In the first part of the chapter, we present the validation results for the equations estimating the V_{85} at the midpoint of the curve. The second part of the chapter includes the validation of the speed changes predicted between the midpoint of the curve and the midpoint of the preceding tangent. Note that the validation of the speed changes was conducted prior to the full development of the speed-profile model presented in chapter 8, and thus the results reported in the second part of the chapter are not a reflection of the accuracy of the final speed-profile model.

Equation Development Database

The V_{85} speed-prediction equations were developed using data collected in six States representing four regions of the country: East (New York and Pennsylvania), South (Texas), Midwest (Minnesota), and West (Oregon and Washington). The database consisted of free-flow passenger car speeds and various geometric data from 92 horizontal curves/approach tangent pairs. The descriptive statistics for the variables used in the equation development are shown in table 42. Chapter 3 provides additional information on data-collection and data-reduction efforts and equation development.

Equation Validation Database

As noted in chapter 3 of this report, data were collected at sites for both equation development and validation. Ultimately, 68 sites were used for validation of the 6 speed-prediction equations. The validation data were collected in the four regions as mentioned above. The descriptive statistics for the validation database are included in table 43.

VALIDATION OF SPEED-PREDICTION EQUATIONS FOR HORIZONTAL AND VERTICAL CURVES

This section of the chapter presents the validation results for the following six speed-prediction equations (SPE) listed in table 14, chapter 3:

$$\text{SPE-1: } V_{85} = 106.3-3595.29/R \tag{23}$$
$$\text{SPE-2: } V_{85} = 96.46-2744.49/R \tag{24}$$
$$\text{SPE-3: } V_{85} = 100.87-2720.78/R \tag{25}$$
$$\text{SPE-4: } V_{85} = 101.90-3283.01/R \tag{26}$$
$$\text{SPE-5: } V_{85} = 111.07-175.98/K \tag{27}$$
$$\text{SPE-6: } V_{85} = 100.19-126.07/K \tag{28}$$

where: V_{85} = 85th percentile speed at the midpoint of the horizontal/vertical curve (km/h)
R = radius of curvature (m)
K = rate of vertical curvature (m/%)

These equations estimate the 85^{th} percentile speed at the midpoint of the curve for various horizontal and vertical alignment combinations and for passenger vehicles during daylight and dry pavement conditions.

Table 42. Descriptive Statistics for Variables Used in the Development of Speed-Prediction Equations.

Parameter	SPE-1	SPE-2	SPE-3	SPE-4	SPE-5	SPE-6
Sites	28	14	22	14	6	5
Radius (m), Range	87-582	87-873	87-635	125-1164	N/A	N/A
Radius (m), Mean	298	306	249	339	N/A	N/A
Radius (m), Std. Dev.	171	214	153	266	N/A	N/A
Sag VC (m), Range	45-243	N/A	N/A	N/A	N/A	91-434
Sag VC (m), Mean	117	N/A	N/A	N/A	N/A	152
Sag VC (m), Std. Dev.	53	N/A	N/A	N/A	N/A	83
CLSD VC (m), Range	N/A	N/A	N/A	61-426	61-290	N/A
CLSD VC (m), Mean	N/A	N/A	N/A	180	130	N/A
CLSD VC (m), Std. Dev.	N/A	N/A	N/A	115	90	N/A
V_{85th} (km/h), Range	63-100	65-97	69-100	78-100	93-115	89-121
V_{85th} (km/h), Mean	89	84	86	88	100	101
V_{85th} (km/h), Std. Dev.	11	10	10	8	8	12

Table 43. Descriptive Statistics for Sites Used in the Validation of Horizontal/Vertical Alignment Equations

Parameter	SPE-1	SPE-2	SPE-3	SPE-4	SPE-5	SPE-6
Sites	22	10	25	4	4	3
Radius (m), Range	116-1747	97-582	109-873	175-499	N/A	N/A
Radius (m), Mean	347	273	279	353	N/A	N/A
Radius (m), Std. Dev.	340	188	181	135	N/A	N/A
Sag VC (m), Range	61-244	N/A	N/A	N/A	N/A	183-435
Sag VC (m), Mean	98	N/A	N/A	N/A	N/A	267
Sag VC (m), Std. Dev.	61	N/A	N/A	N/A	N/A	145
CLSD VC (m), Range	N/A	N/A	N/A	61-91	61-290	N/A
CLSD VC (m), Mean	N/A	N/A	N/A	76	126	N/A
CLSD VC (m), Std. Dev.	N/A	N/A	N/A	18	110	N/A
V_{85} (km/h), Range	62-103	57-96	59-105	81-103	82-100	97-120
V_{85} (km/h), Mean	89	82	87	92	94	106
V_{85} (km/h), Std. Dev.	10	12	12	9	8	13

The following analyses were performed for each of the six speed-prediction equations:

1. Calculate the predicted V_{85} at the midpoint of the curve for each study-site location in table 43 using equations 23 through 28 as appropriate. Plot the predicted V_{85} versus the observed V_{85} at the midpoint of the horizontal curve for all the sites presented in table 43.

2. Calculate the mean squared error, the mean absolute difference, and the mean absolute percent difference across all validation sites and separately for each of the six equations, defined as follows:

   $$\text{Mean squared error} = (\text{variance} + \text{bias squared}) \quad (29)$$

   $$\text{Mean absolute difference} = \text{Mean of } |\text{observed}_i - \text{predicted}_i| \quad (30)$$

   $$\text{Mean absolute percent difference} = \text{Mean of } \left| \frac{\text{observed}_i - \text{predicted}_i}{\text{predicted}_i} \right| x\,100\% \quad (31)$$

   These statistics are used to describe the discrepancy between the equation-predicted and the observed measurements of speed.

3. Develop box plots of the MSE, the mean absolute difference, and the mean absolute percent difference across all validation sites. In addition, develop box plots of the mean absolute percent difference for the six individual equations. These box plots are used to illustrate the distribution of these statistics overall and for the individual equations.

Predicted Versus Observed V_{85} at the Midpoint of the Curve

A plot of the equation-predicted V_{85} at the midpoint of the curve versus the field-observed V_{85} for all 68 sites was developed and is shown in figure 50. Different symbols are used to depict each of the six equations and their corresponding alignment conditions. For alignment condition 1 (horizontal curve on grade between 0 and +4 percent or horizontal curve combined with sag vertical curve), most points are close to the 45-degree line. For alignment condition 2 (horizontal curve on grade +4 percent to +9 percent), the predicted V_{85} is slightly underestimated in most cases. Finally, alignment condition 6 (sag vertical curve on horizontal tangent), includes a site with a relatively high observed speed (120 km/h), for which the predicted speed is significantly lower.

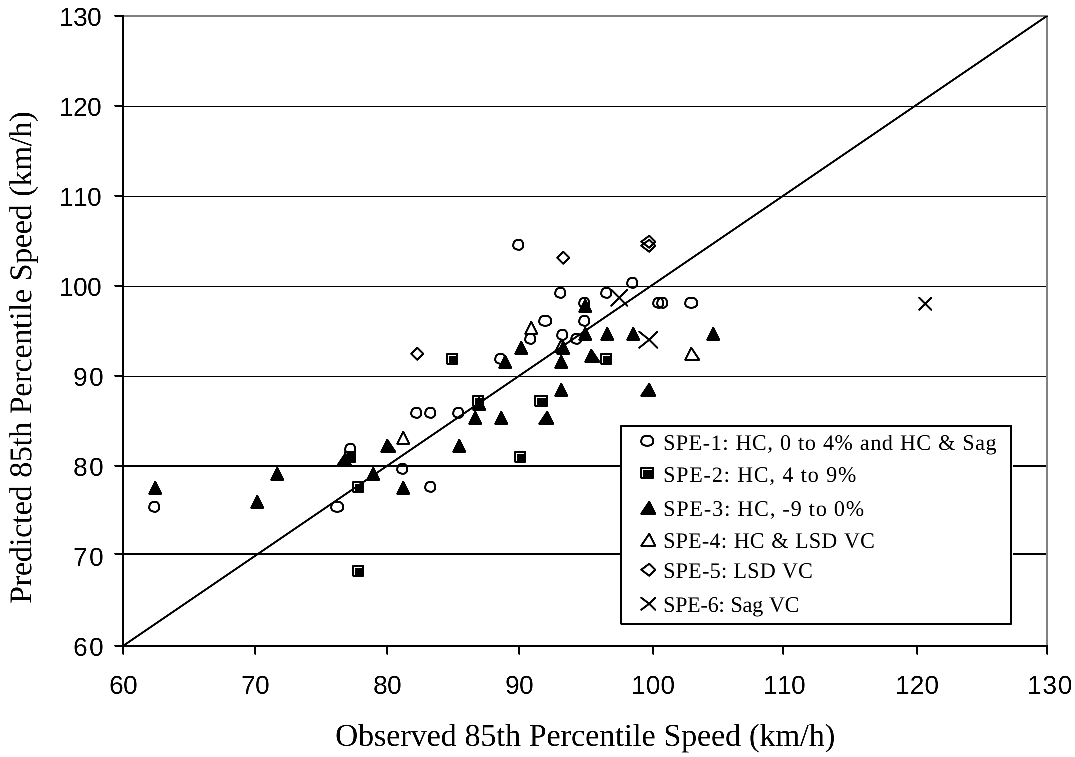


Figure 50. Predicted Versus Observed V_{85} at Midpoint of Curve.

Statistics for Speed-Prediction Equations

To determine the accuracy of the six equations, the mean squared error (MSE), the mean absolute difference (MAE), and the mean absolute percent difference (MAPE) for each of the equations were calculated separately and overall. The results are presented in table 44.

Table 44. Statistics for Speed-Prediction Equations.

	Overall	**SPE-1**	**SPE-2**	**SPE-3**	**SPE-4**	**SPE-5**	**SPE-6**
No. of Sites	68	22	10	25	4	4	3
MSE	47.08	27.31	67.39	44.11	44.19	62.67	236.05
MAE	4.93	3.735	6.13	4.67	4.28	7.30	9.91
MAPE	5.7	4.1	7.8	5.6	4.6	7.3	10.0
?^{2} Calc.	36.14	6.46	7.10	12.97	1.44	2.45	5.71
?$^2_{5\%}$ Critical	88.24	33.92	18.31	37.65	9.49	9.49	7.81

As shown in table 44, SPE-6 is the equation with the largest errors and with the least number of sites. The overall performance of the six equations is greatly influenced by the performance of SPE-6. However, it should be taken into account that SPE-4 through SPE-6 had very few sites for validation and SPE-5 and SPE-6 had very few sites for equation development (six and five sites, respectively). In general, the individual equations range in MAPE from 4.1 to 10.0 percent, while overall, the performance of the equations results in a MAPE of 5.7 percent.

Chi-Square Test

A Chi-square test was performed to assess the fit of the equations developed to predict V_{85} at the midpoint of horizontal/vertical curves. The Chi-square statistic is determined by:

$$c^2 = \sum_{i=1}^{n} \frac{(observed_i - predicted_i)^2}{predicted_i} \qquad (32)$$

If the Chi-square statistic is greater than the critical Chi-square value taken from a standardized table, then it can be concluded that there is a significant difference between the observed and predicted V_{85} at a given confidence interval. From table 44, it can be seen that there is no statistically significant difference between the observed and predicted V_{85} at the midpoint of the curve.

Box Plots for the Six Speed-Prediction Equations

Box plots were developed to illustrate the distribution of the absolute percent difference for the six individual equations as well as the difference, absolute difference, and absolute percent difference for the entire validation database. Figure 51 includes box plots for the entire data set. A box plot consists of a box, whiskers, and outliers. The solid line inside the box represents the median. The top and bottom of the box reflects the third and first quartile, respectively. The whiskers are the lines that extend from the top and bottom of the box to the lowest and highest observations that are still inside a calculated region. The region above the box, for example, is defined as being 1.5 times the difference between the values for the first and third quartile (interquartile range) added to the value of the third quartile. Outliers are points outside the lower and upper limits and are plotted with asterisks (*).

Figure 51 displays the range of error associated with the equation's prediction of V_{85} on the curve versus the field-observed V_{85} at the midpoint of the curve. As is shown in figure 51, there are quite a few data points that can be considered outliers (as indicated by the asterisk), with absolute differences much greater than 10 km/h. To examine how each of the six equations were performing, box plots were developed for each equation representing the absolute percent difference between field observations and equation-predicted V_{85} at the midpoint of the curve (see box plots in figure 52).

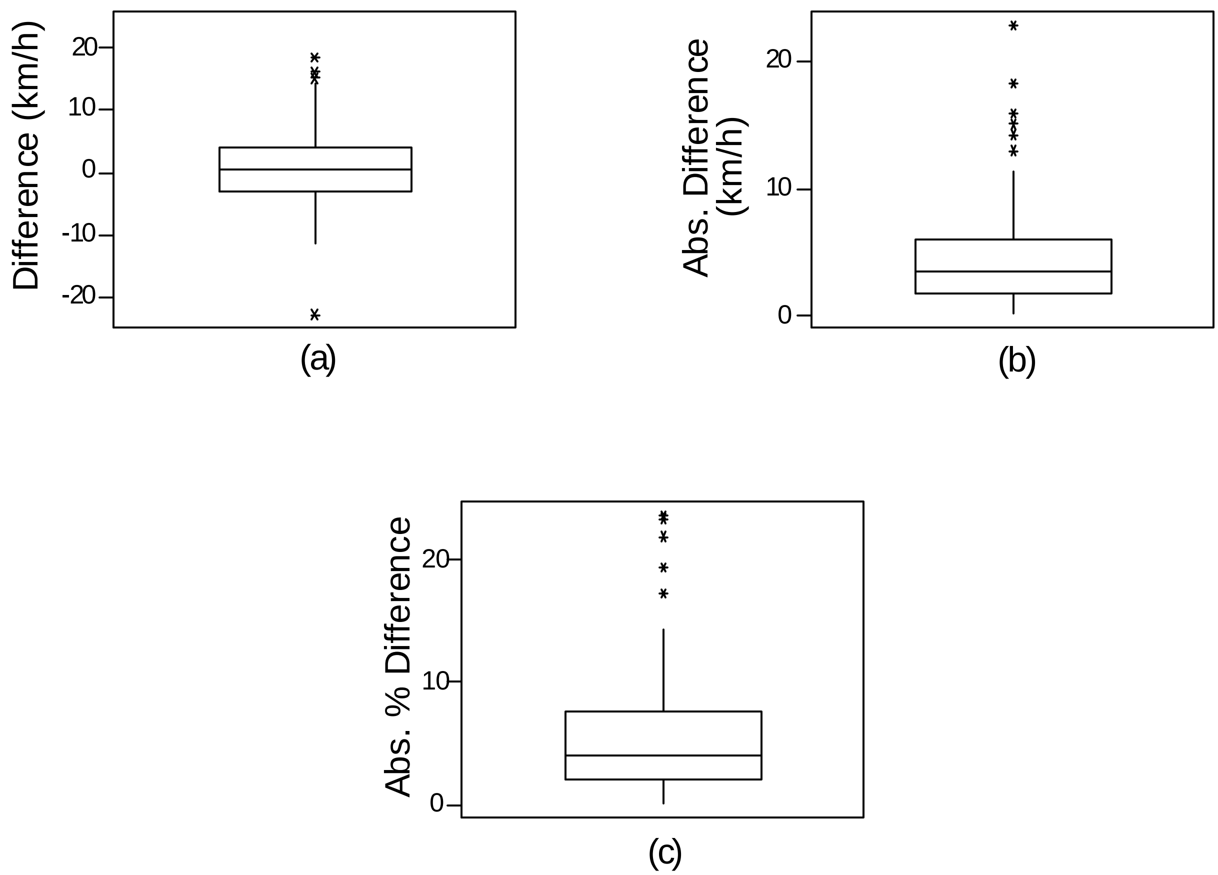


Figure 51. Equations Versus Field Observations (All Data).

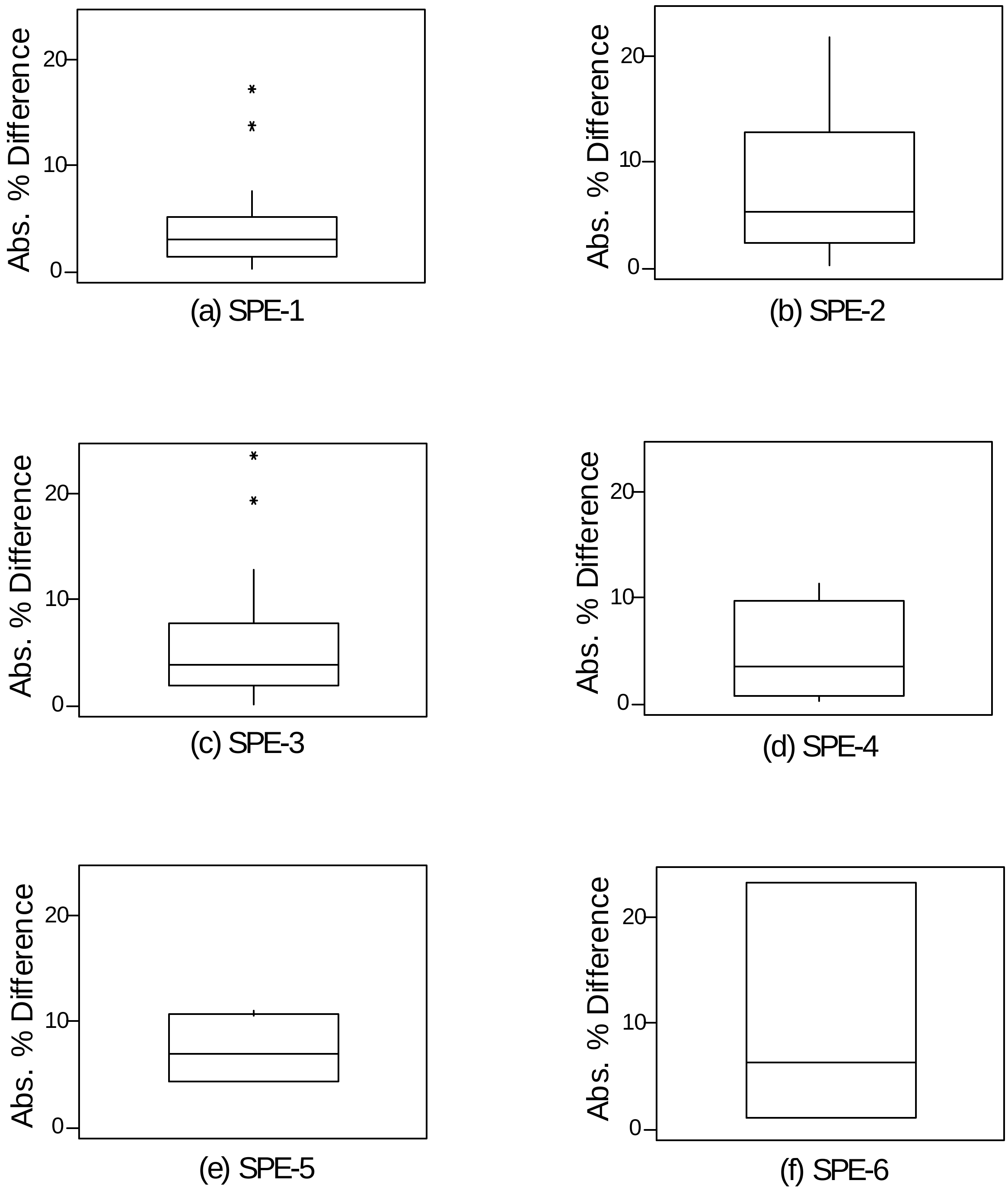


Figure 52. Absolute Percent Difference for Each Equation.

Figure 52 shows the range of error for the six individual speed-prediction equations. Given that SPE-1 through -3 had many more validation data points, more information can be gleaned from the box plots for these equations than for SPE-4 through -6. In figure 52(a), for instance, it can be seen that there are two significant outliers and a mean absolute percent error of approximately 3 percent. Also, from this box plot, it can be seen that the absolute percent error is smaller than the error associated with SPE-2 (see figure 52(b)).

VALIDATION OF PREDICTED-SPEED CHANGES

This section of the chapter includes the validation of the speed change predicted between the midpoint of the tangent and the midpoint of the curve for various alignment conditions. As mentioned above, the method used in this chapter for calculating speed changes is a simplification of the method used in the final speed profile model presented in chapter 8. Thus, the results reported here are not a reflection of the accuracy of the final speed-profile model, but constitute an intermediate step in the final model development. For example, the predicted speed changes used in this chapter do not check to determine if the tangent length available for speed changes should affect the selected deceleration rate. To model the change in speed between the midpoint of the tangent and midpoint of the horizontal curve, the models developed to estimate V_{85} at the midpoint of a horizontal curve were utilized. (Note: SPE-1 through -4 were for horizontal curves, while SPE-5 and -6 were developed for vertical curves on tangents and thus they were excluded from this analysis.) Next, the appropriate deceleration rate was used (see chapter 6), along with the tangent length, to calculate the estimated V_{85} at the midpoint of the tangent. The difference between these two measurements was then compared to the field-measured difference between V_{85} at the midpoint of the tangent and V_{85} at the midpoint of the horizontal curve. The steps taken to estimate speed at the midpoint of the approach tangent are outlined below with a numerical example.

Example for Calculating Speed Change:

Horizontal curve with the following characteristics:

Grade = 5 percent
Radius = 300 m
Preceding tangent length = 500 m

1. Calculate speed at the midpoint of the horizontal curve using the equations in table 14.

   Example: Use SPE-2 (equation (24) or second equation in table 14).

$$\begin{aligned} V_{85} &= 96.46 - 2744.49/R \\ &= 96.46 - 2744.49/300 \\ &= 87.31 \text{ km/h} \end{aligned}$$

2. Determine the appropriate deceleration rate from chapter 6.

   Example: Apply the following equation: +0.6794 – 295.14/R

   Deceleration rate = 0.6794 – 295.14/300 = -0.3044 m/s^2

3. Apply the following equation to estimate the speed at the midpoint of the tangent:

$$V_f^{\,2} = V_0^{\,2} - 2ad \tag{33}$$

   where:

   V_f = speed at the midpoint of the horizontal curve (m/s)
   V_0 = speed at the midpoint of the approach tangent (m/s)
   a = acceleration (deceleration) (m/s^2)
   d = distance (m)

Example:

$$\begin{aligned} v_0 &= \sqrt{(24.25)^2 - (2 \cdot (-0.3044) \cdot 500 \cdot 0.5)} \\ &= 27.21 \ m/s \\ &= 97.95 \ km/h \end{aligned}$$

4. If the estimated speed at the midpoint of the tangent is greater than 100 km/h, substitute the calculated value with 100 km/h.

The same tasks performed in the first section of this chapter were performed on this data set as well to test the accuracy of the speed-change models.

Predicted Versus Observed Speed Change in V_{85} Between Tangent and Curve

Figure 53 contains a plot of the estimated change in V_{85} speed between the midpoint of the approach tangent and the midpoint of the horizontal curve versus the field-measured change in V_{85} between these same locations. As can be seen in the figure, the predicted change in speed is in most cases greater than the observed change in speed between these two locations.

Statistics for Estimated Speed Change

To determine the accuracy of the individual models and overall in predicting the change in speed between the midpoint of the tangent and the midpoint of horizontal curve, three statistics were calculated: the mean squared error (MSE), the mean absolute difference (MAE), and the mean absolute percent difference (MAPE). These statistics are presented in table 45.

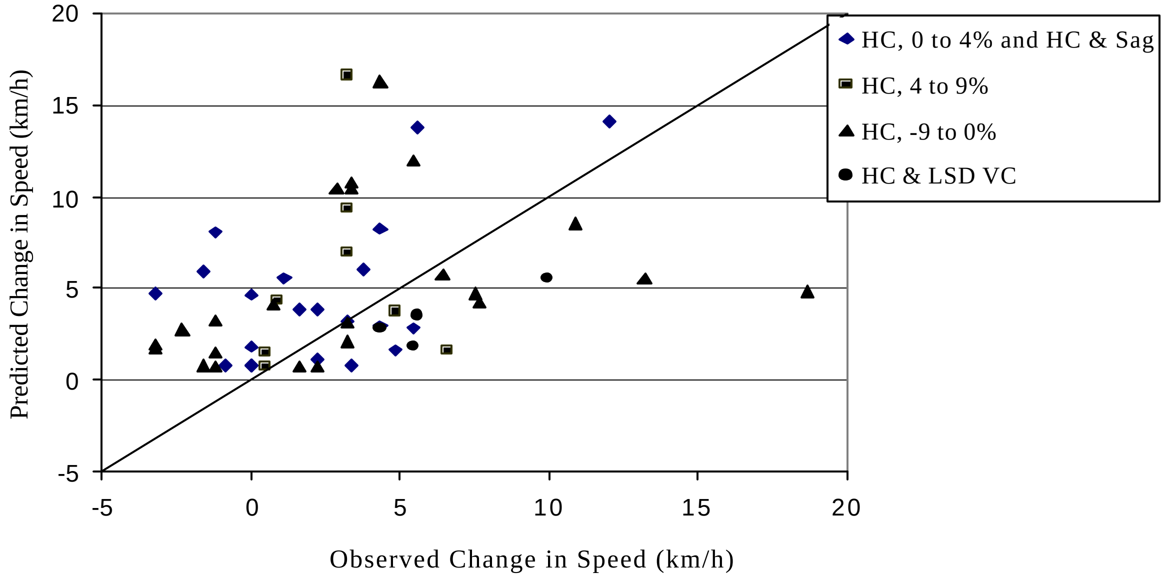


Figure 53. Predicted Versus Observed Change in V_{85} Speed Between Midpoint of Tangent and Midpoint of Curve.

Table 45. Statistics for Estimated Speed Change.

	Overall	Alignment 1	Alignment 2	Alignment 3	Alignment 4
No. of Sites	**59**	**22**	**8**	**25**	**4**
MSE	**26.93**	**17.85**	**37.27**	**36.40**	**10.38**
MAE	**3.97**	**3.22**	**4.24**	**4.74**	**2.93**
MAPE (%)	**97.87**	**86.81**	**98.85**	**107.36**	**97.42**
?^{2} Calc.	**261.92**	**73.00**	**7.91**	**139.87**	**12.72**
?$^{2}_{5\%}$ Critical	**77.92**	**33.92**	**15.51**	**37.65**	**9.49**

As shown in table 45, overall, the models differ from the observed change in speed by nearly 98 percent. The Chi-square statistics show that, overall, the estimated change in speeds differs significantly from the observed change in speed between the midpoint of the approach tangent and the midpoint of the horizontal curve. The same conclusion can be reached for each of the individual models, with the exception of Alignment 2.

Box Plots for Speed-Change Estimates

Box plots were developed to illustrate the distribution of the absolute percent difference for the four speed-prediction models for horizontal curves (Alignments 1 through 4) as well as the difference, absolute difference, and absolute percent difference for the entire validation database. Figure 54 includes box plots for the entire data set. These three box plots display the range of error between the model-predicted change in V_{85} (i.e., the difference between the V_{85} at the midpoint of the approach tangent and the V_{85} at the midpoint of the horizontal curve) and the field-observed change in V_{85} between these two points.

As can be seen in figure 54, the models were mostly found to overestimate the change in speed between the midpoint of the tangent and the midpoint of the horizontal curve. To evaluate each of the four alignment conditions, box plots were developed for each model, representing the absolute percent difference between field observations and model-predicted change in V_{85} between the midpoint of the approach tangent and the midpoint of the horizontal curve. As can be seen in figure 55, the change in speed is greatly overestimated for all alignment conditions.

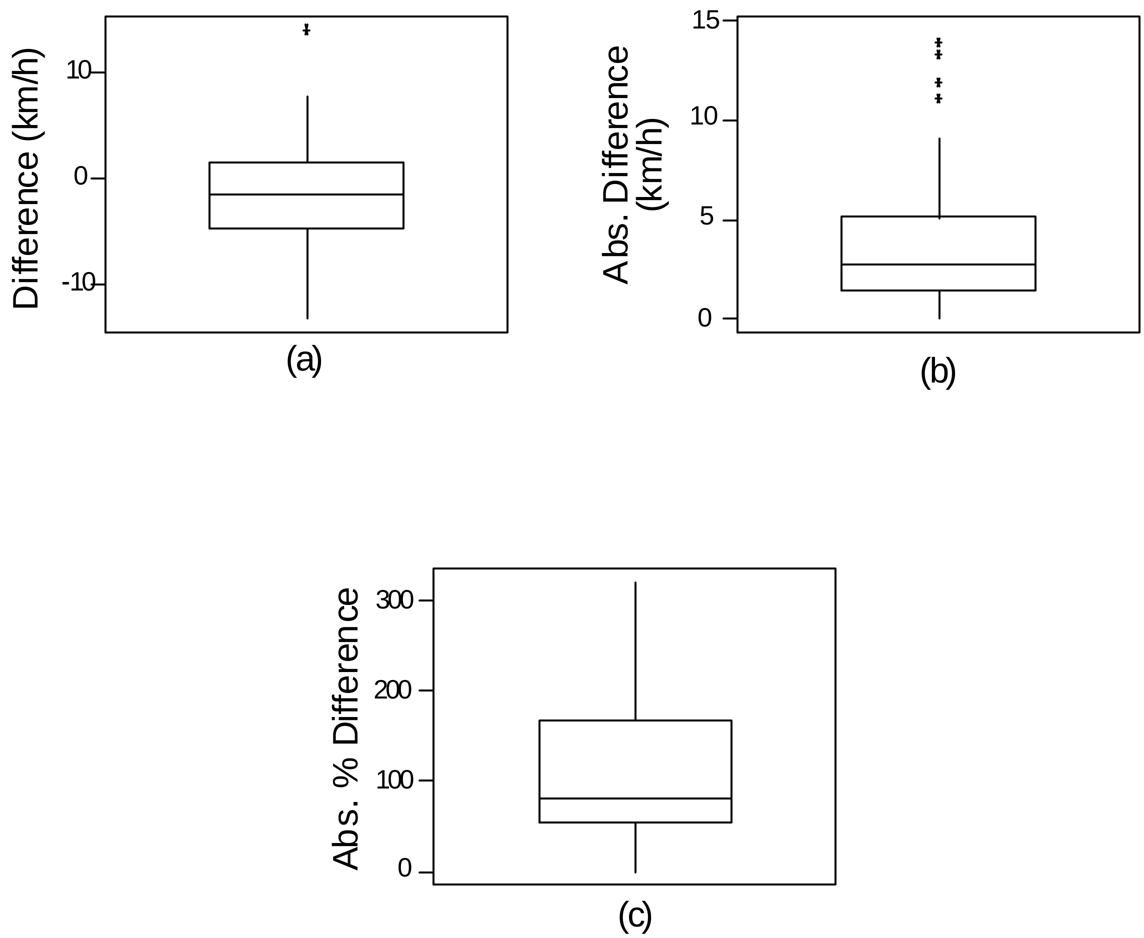


Figure 54. Alignment Model Versus Field Observation (All Data).

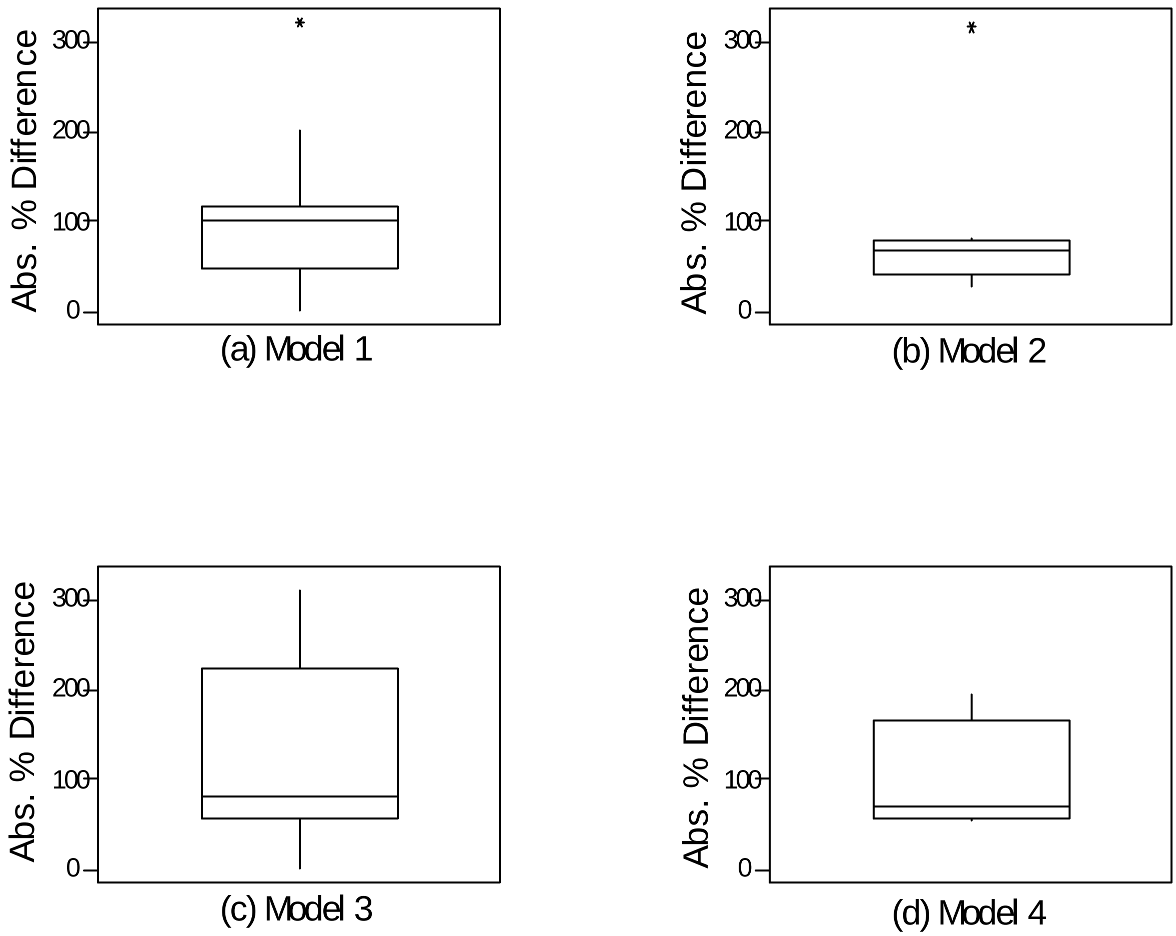


Figure 55. Absolute Percent Difference for Each Alignment Model.

SUMMARY-VALIDATION OF SPEED-PREDICTION EQUATIONS AND ACCELERATION/DECELERATION MODELS

The validation of the six speed-prediction equations for horizontal and vertical curves was performed by comparing the equation-predicted V_{85} at the midpoint of the curve to the field-observed V_{85} at the midpoint of the curve. The six speed-prediction equations perform well in that they range in mean absolute percent error between 4.1 percent and 10 percent and have an overall mean absolute percent error of 5.7 percent. Box plots were developed to demonstrate the range in error between the equations and the field measurements as well as displaying any outliers that may be present in the data sets.

A validation was also conducted for the predicted speed change between the midpoint of the approach tangent and the midpoint of the horizontal curve. In general, the speed changes predicted by the simplified method used in this chapter were found to differ from the observed changes in speed between the tangent and the horizontal curve by an average of 98 percent. As noted previously, this difference is not a reflection of the accuracy of the final speed-profile model presented in the following chapter.

8. DESIGN-CONSISTENCY EVALUATIONS AND SPEED-PROFILE MODEL

SPEED-PREDICTION EQUATIONS USING ALL AVAILABLE DATA

The equations recommended in chapter 3 used 103 of the 176 study sites. The remaining 73 sites were selected for validation of the previous research conducted by Krammes et al. and the equations presented in chapter 3.[7] Based on the validation results of chapter 7 and a desire to use all available data, a decision was made to use all collected data to generate equations that would be based on more study sites than any previous efforts. The following sections discuss the new equations.

Horizontal Curves on Grades

Four different vertical grade conditions were considered in the preliminary evaluation of horizontal curves on grades: upgrades (0 to 4 percent), steep upgrades (greater than 4 percent), downgrades (-4 to 0 percent), and steep downgrades (less than 4 percent). The data were analyzed based on these grade separations. A correlation matrix was developed to identify relationships between independent and dependent variables. Based on the correlation matrix and the statistical findings in chapter 3, the independent variables evaluated in the regression analysis were radius and inverse radius. Figures 56 and 57 show the data for the 94 sites that had a horizontal curve on a grade. Figure 56 shows the speed data versus the radius of the horizontal curve, while figure 57 shows the speed data versus the inverse radius of the horizontal curve. Both of these figures also include plots of the developed equations.

Figure 56 shows that as R increases from 0 to 400 m, the 85th percentile speeds increase notably for all study locations. For radii greater than 400 m, the increase in speed is not as dramatic. The inverse of the radius was the variable most highly correlated to the 85th percentile speed of all the variables included within the correlation matrix. The regression model developed to fit the data for horizontal curves on grades included the single independent variable 1/R. The plot of 1/R versus 85th percentile in figure 57 shows a linear relationship with speed.

Table 46 lists the equations developed using all available data. Not unexpectedly, some of the R^2 values are lower than for the chapter 3 equations. This could be caused by the greater number of observations included for each alignment type. The similarity of the magnitude of the Beta values between the previous equations and the equations using all data for each alignment condition supports the validation process of the chapter 3 equations.

Three of the four revised equations have intercept values greater than the 97.9 km/h recommended by Krammes et al. on long tangents.[7] Therefore, under certain situations, the equations would predict speeds higher than the assumed speed on a long tangent. Observed speeds on long tangents ranged from 93 to 104 km/h (average 85th percentile speed by State).

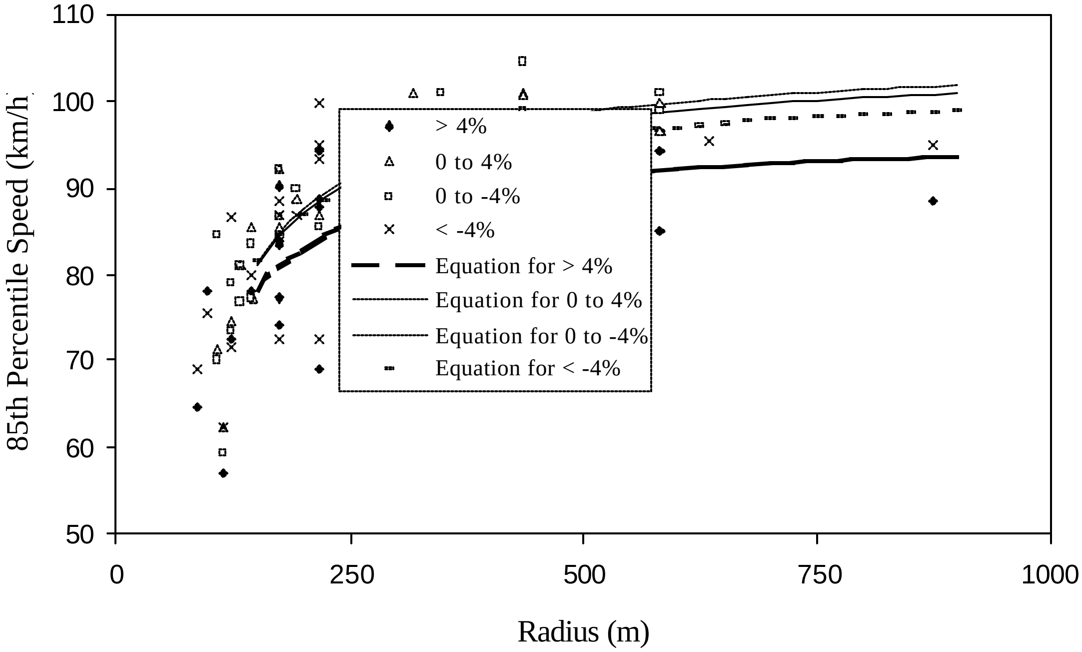


Figure 56. Horizontal Curves on Grades: V_{85} Versus R.

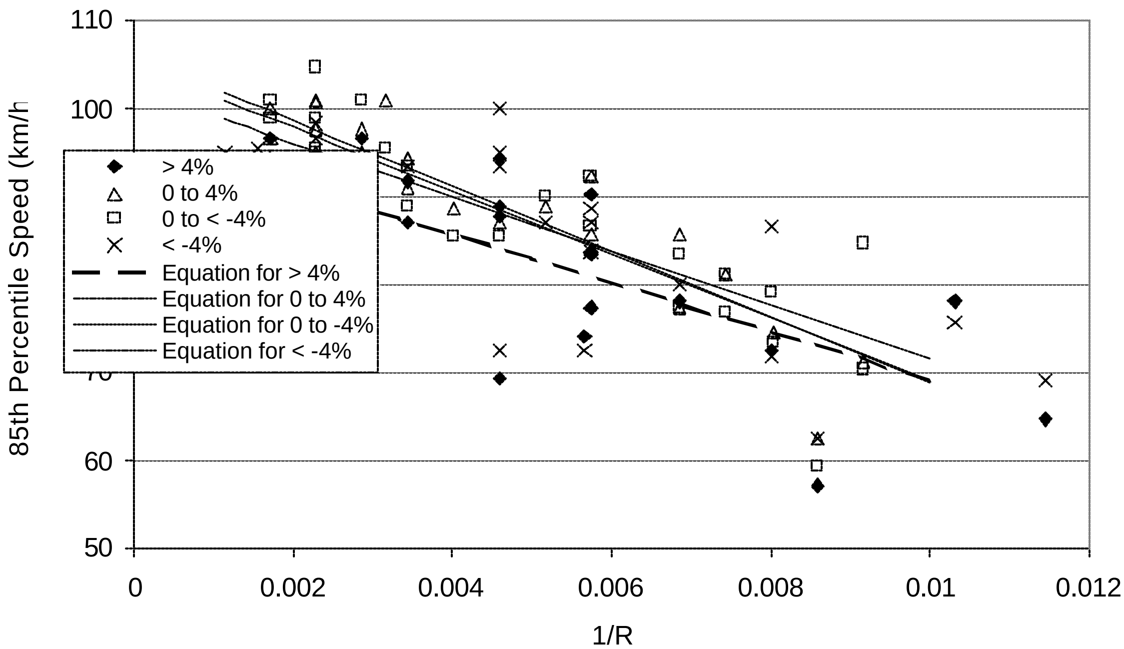


Figure 57. Horizontal Curves on Grades: V_{85} Versus 1/R.

Table 46. Parameter Estimates of Horizontal Curves on Grade.

Alignment Condition	Independent Variable	Beta Parameter Estimate	p-Value	Number	R^2
0%# G < 4%	1/R	-3574.51	0.0001	25	0.76
	Intercept	104.82	0.0001		
4%# G < 9%	1/R	-2752.19	0.0001	23	0.53
	Intercept	96.61	0.0001		
-4%# G < 0%	1/R	-3709.90	0.0001	25	0.76
	Intercept	105.98	0.0001		
-9%# G < -4%	1/R	-3077.13	0.0001	21	0.58
	Intercept	102.10	0.0001		

Based on the data and engineering judgment, the maximum operating speed on horizontal curves and tangents could be rounded to 100 km/h. Thus, operating speeds on large-radius horizontal curves should be truncated to 100 km/h (or to another desired operating speed) when the predicted speed exceeds this value.

Vertical Curves on Horizontal Tangents

Vertical curves on horizontal tangents were divided into three categories: non-limited sight-distance (NLSD) crest curves, limited sight-distance (LSD) crest curves, and sag curves. A combined total of 21 study sites were collected for the vertical curves on tangents: 2 for NLSD crest curves, 10 for LSD crest curves, and 9 for sag curves.

Four different correlation matrices were developed for vertical curves on tangents: individual matrices containing the data for each vertical curve type and one matrix that combined the NLSD and LSD crest curves. The independent variables considered included K and 1/K. Figure 58 shows the 85th percentile speed versus K. Of the independent variables, 1/K was most highly correlated to the 85th percentile speeds, even though the correlation was low for some conditions. The relationship between 85th percentile speed and 1/K is shown in figure 59. Also included on these figures are the data from the NCHRP Stopping Sight Distance (SSD) study and the plot of the selected regression equation for the limited sight-distance condition.[36]

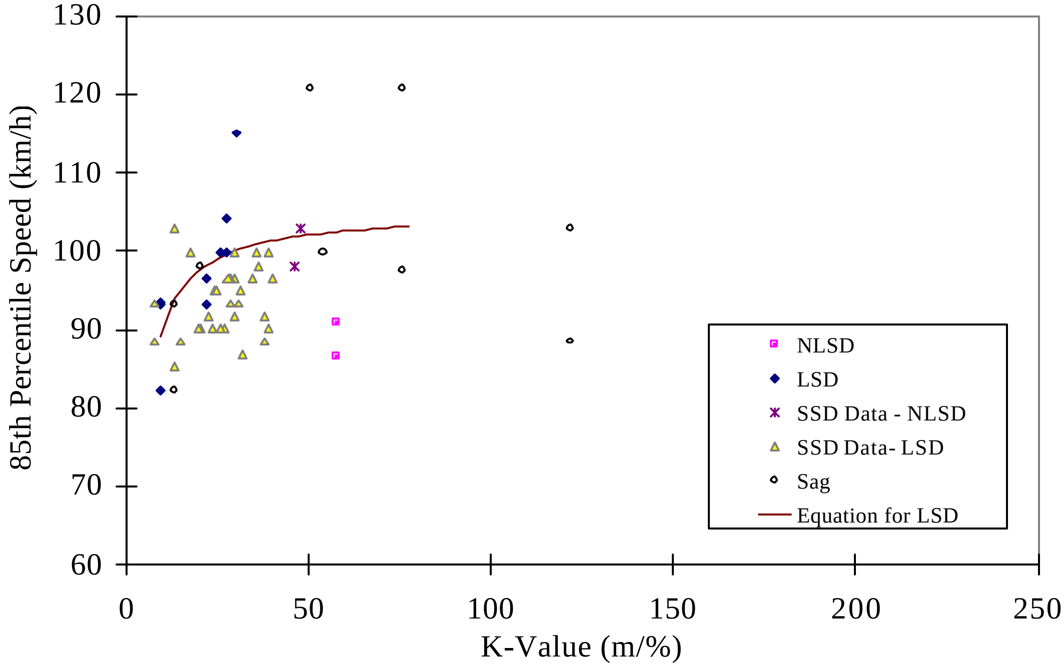


Figure 58. Vertical Curves on Horizontal Tangents: V_{85} Versus K.

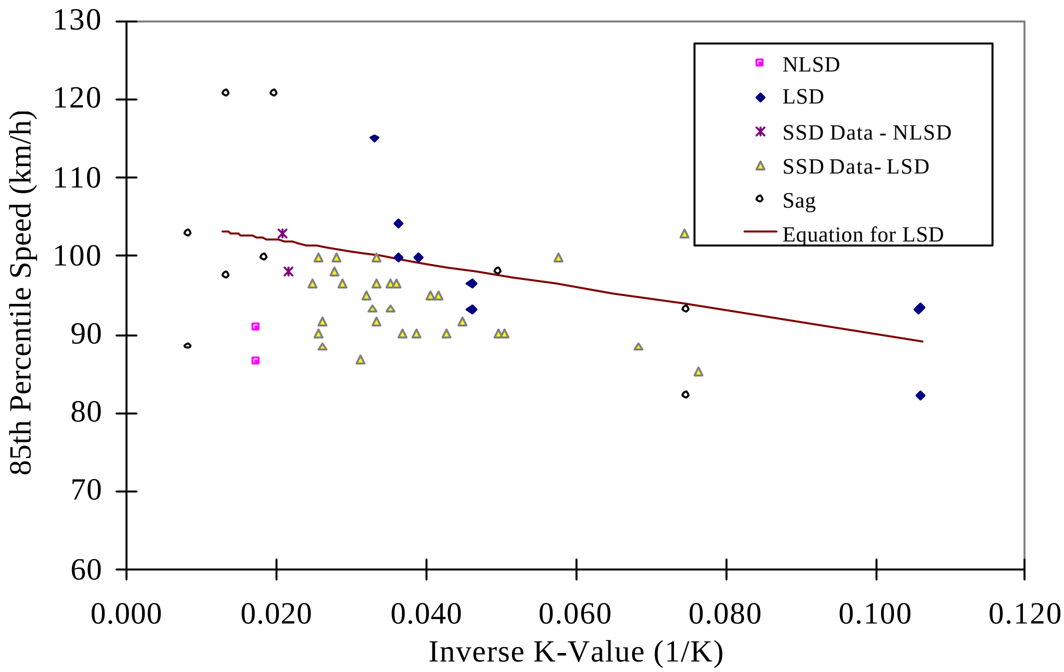


Figure 59. Vertical Curves on Horizontal Tangents: V_{85} Versus 1/K.

Based on the small data set for NLSD vertical curves (only two sites), the data from the NCHRP SSD Study for NLSD curves were added to the correlation matrix.[36] The addition of the SSD study data increased the number of study sites to four data points. Based on the small amount of data and the wide variability in speeds observed, no statistically significant regression equation was found for NLSD curves on horizontal tangents. Therefore, the desired speed for long tangents is assumed for this condition. This recommendation is based on the graphical representation of the four sites and engineering judgment.

Ten study sites were available from the current study and 32 sites from the SSD study for the evaluation of the LSD sites. The review of the correlation matrix found 1/K to be correlated with the 85th percentile speeds. Regression analysis showed that 1/K was significant (0.033); however, the R^2 value was extremely low (0.10). A possible reason for the low R^2 was that there was a study location where the posted speed was 112.7 km/h, resulting in a large variation in 85th percentile speeds. While all the SSD sites had a speed limit of 88.5 km/h, the data were collected prior to the repeal of the nationally mandated speed limit and there was an observable difference in the plot of the data. The SSD speed data were consistently lower then the speed data observed in the current research. Therefore, the SSD data were not included in the final equation development. The equation was developed from the nine observations where the posted speed was less than or equal to 88.5 km/h. The parameter estimates are listed in table 47.

Table 47. Parameter Estimates of LSD Crest Curves on Horizontal Tangents.

Independent Variable	Parameter Estimate	P-Value	Number of Sites	R^2
Intercept	105.08	0.0001	9	0.60
1/K	-149.69	0.0134		

A total of nine sag curves on horizontal tangents sites were available for the analysis. As with the crest curves, the scatter plot does not show a clear relationship between the variables (see figure 58 or 59). One possible reason for the poor relationship between 85th percentile speed and K or 1/K could be due to the different posted speeds where data were collected. Of the nine sites, two of the sites had posted speeds of 112.7 km/h. A statistical test showed these sites to be data outliers and they were eliminated from the analysis. An equation was developed for the remaining seven sites. Table 48 shows the parameter estimates and statistics of the equation. The R^2-value for the equation was 0.49; however, the independent variable, 1/K, was not significant. Therefore, based on the plots and the regression analysis, it is recommended that the desired speed on long tangents be used for this alignment condition. Extreme sag vertical curves where the K-value is less than 15 may result in reduced operating speeds; however, the available data are too sparse to make a definitive conclusion on the issue.

Table 48. Parameter Estimates of Sag Curves on Horizontal Tangents.

Independent Variable	Parameter Estimate	P-Value	Number of Sites	R^2
Intercept	100.21	0.0001	7	0.49
1/K	-106.34	0.0779		
Recommend: Use desired speed on long tangent.				

Combination of Horizontal and Vertical Curves

The analysis of the combination curves (i.e., sites with both a horizontal curve and a vertical curve) began with plotting the speed data versus R, 1/R, K, and 1/K. These plots are shown in figures 60, 61, 62, and 63, respectively. Initial evaluation of the plots indicated that both R and K could influence the speed along the combination of curves.

For non-limited sight-distance crest vertical curves in combination with horizontal curves, the efforts discussed in chapter 3 did not find a statistically significant regression equation. One of the reasons was that the data used in the analyses were for larger radii curves. Drivers on a combination of large horizontal radii and non-limited sight-distance crest curves may not feel the need to reduce their speed in response to the geometry. The inclusion of all available data from this study also did not identify a regression equation with significant variables. All tested models that used variations of R and K had both insignificant variables and very low R^2 values. Therefore, engineering judgment must be used to determine the predicted speed for a horizontal curve combined with a non-limited sight-distance crest vertical curve. Based on a review of the data available for this condition and for similar conditions, the lowest speed predicted using the equation developed for the following conditions is recommended:

- Assumed maximum desired speed on long tangents.
- Predicted speed using the horizontal curve radius equation for the upgrade.
- Predicted speed using the horizontal curve radius equation for the downgrade.

Using the lowest predicted speed will ensure that the speed predicted along the combined vertical and horizontal curve will not be better than if just the horizontal curve was present.

Limited sight-distance crest curves combined with horizontal curves were evaluated using the 22 study sites available. Regression analysis compared the influence of 1/K, 1/R, and an interaction term. The analysis demonstrated that only 1/R was significant in predicting 85th percentile speeds. Table 49 lists the parameter estimates for the developed equation.

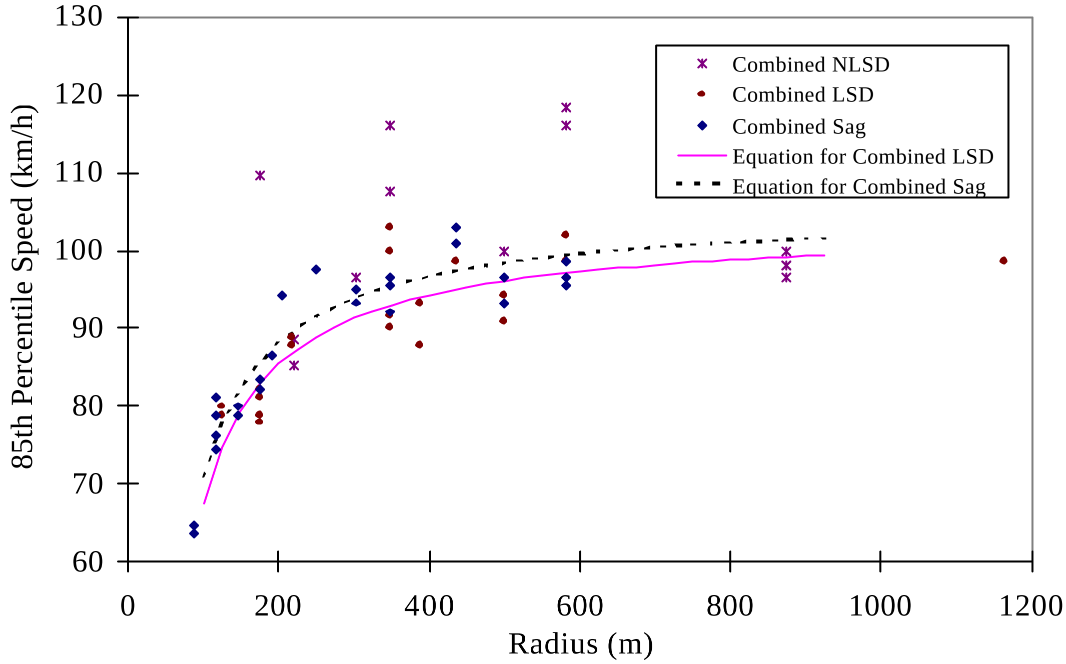


Figure 60. Combination Curves: V_{85} Versus R.

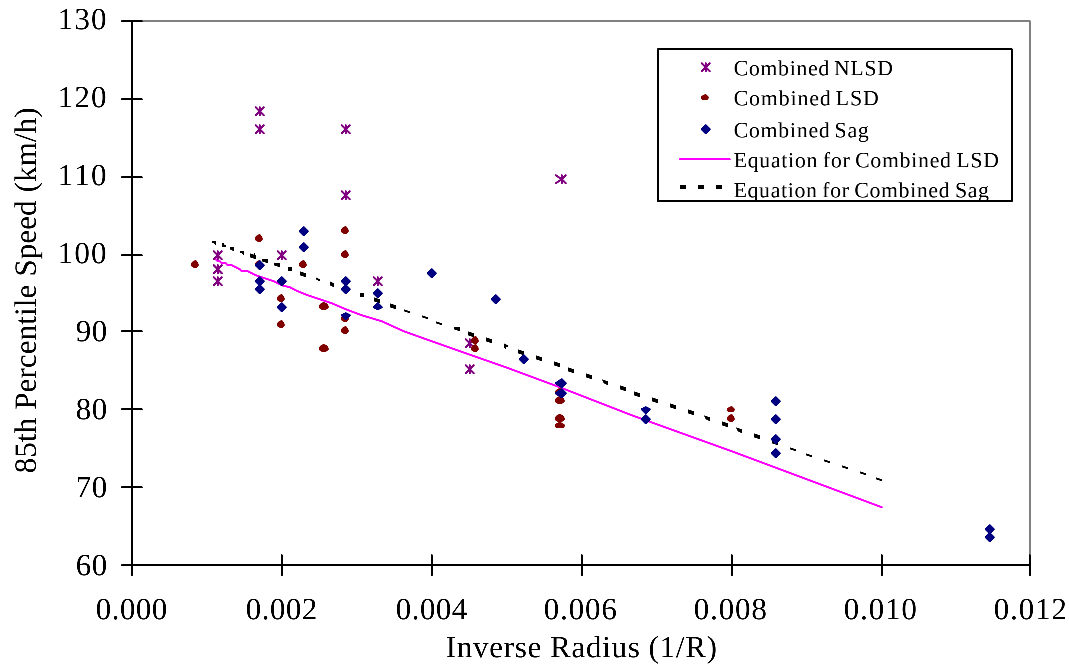


Figure 61. Combination Curves: V_{85} Versus 1/R.

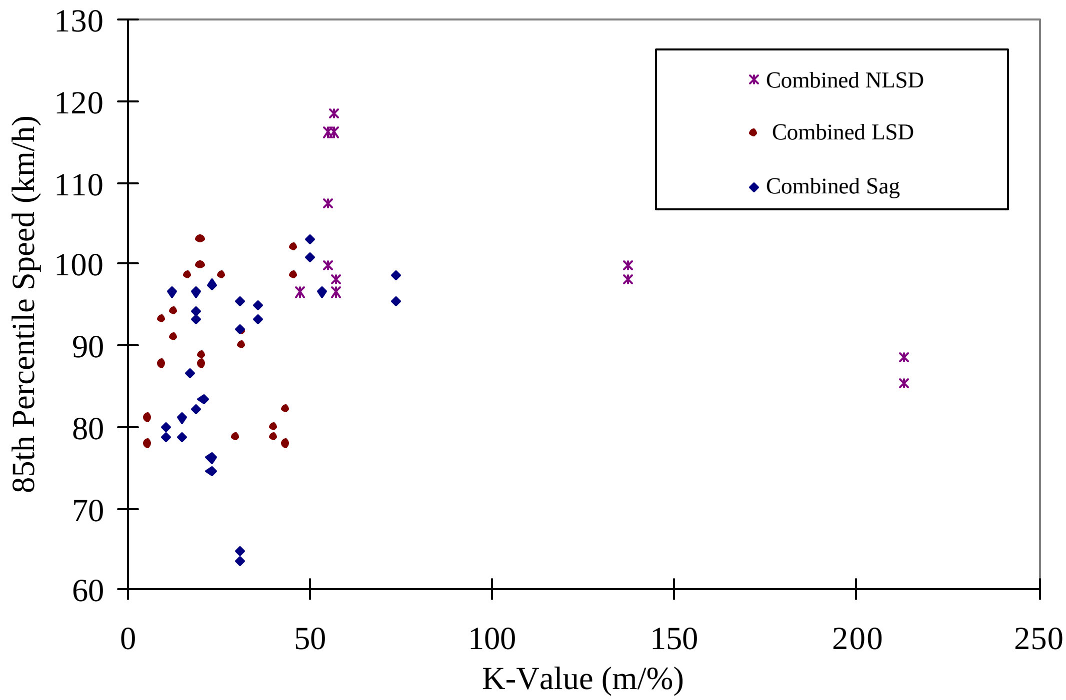


Figure 62. Combination Curves: V_{85} Versus K.

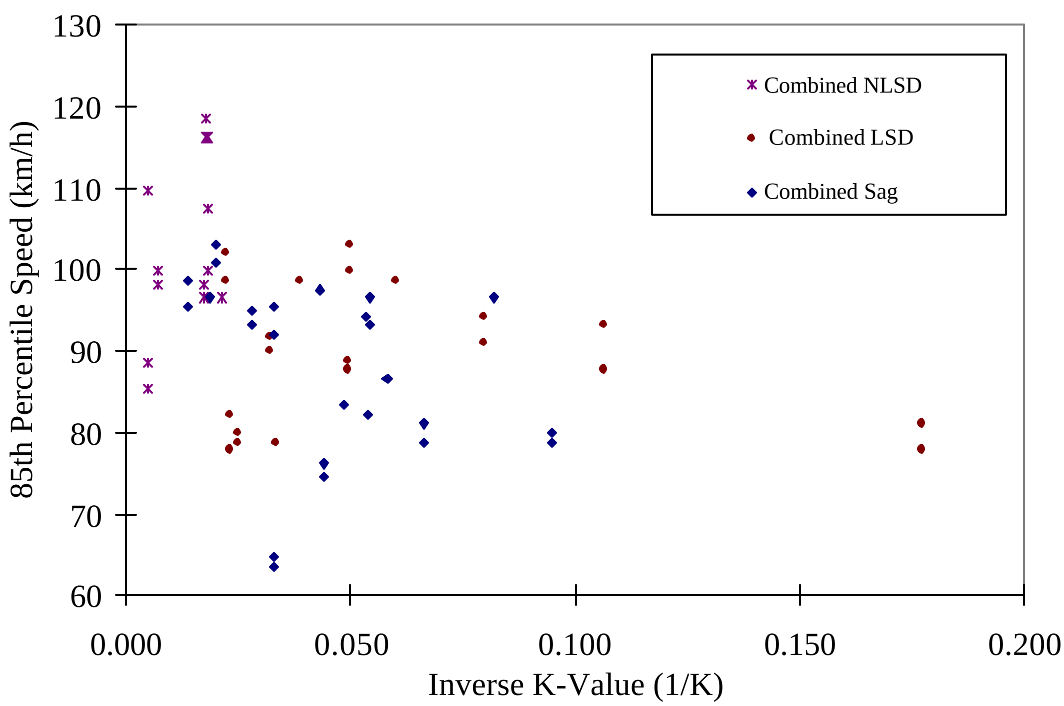


Figure 63. Combination Curves: V_{85} Versus 1/K.

Table 49. Parameter Estimates of Combined Horizontal and LSD Crest Curves.

Independent Variable	Parameter Estimate	P-Value	Number of Sites	R^2
Intercept	103.24	0.0001	22	0.74
1/R	-3576.51	0.0001		

The revised equation developed for the combination of sag vertical curves and horizontal curves had an additional nine observations included in the analysis. The equation developed with the data from the 25 sites revealed that 1/R was the only significant independent variable. Table 50 lists the equation parameter estimates.

Table 50. Parameter Estimates of Combined Horizontal Sag Curves.

Independent Variable	Parameter Estimate	P-Value	Number of Sites	R^2
Intercept	105.32	0.0001	25	0.92
1/R	-3438.19	0.0001		

Comparison of Regression Equations

The equations using radius were tested to determine if there were differences between the slopes of the regression lines. A t-test and indicator variables were used to check pairs of regression equations. The regression equation for horizontal curves on grades greater than 4 percent was significantly different from the equations for horizontal curves on a 0- to 4-percent grade, horizontal curves on a 0- to -4-percent grade, and horizontal curves combined with sag curves. Observed speeds for upgrades greater than 4 percent were lower than the speeds for the other horizontal curves on grade conditions resulting in an equation that was statistically different from the other equations. This difference is apparent in figure 64. Table 51 lists the results of the statistical evaluations.

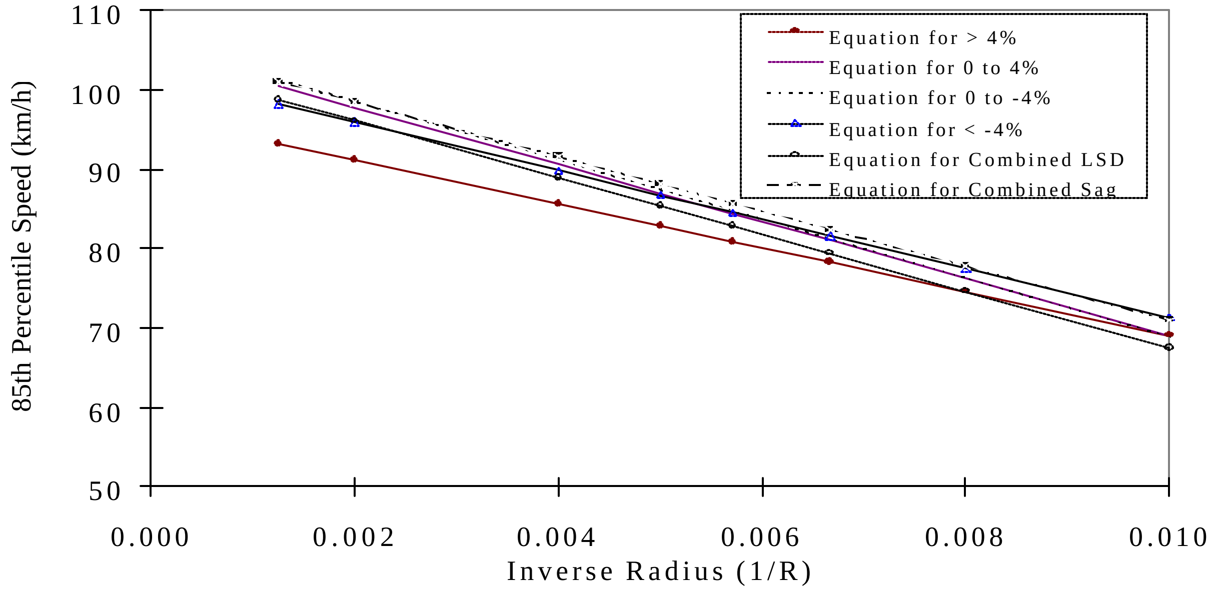


Figure 64. Plot of Speed-Prediction Equations Using Radius as a Variable.

Table 51. Comparison of Regression Equations.

Equations Compared		Number of Sites	p-Value for Indicator Variable
0 to 4 % and	> 4 %	48	0.0169*
	< -4 %	46	0.8585
	0 to -4 %	51	0.7071
	Combined Sag	50	0.3447
	Combined LSD	47	0.2523
> 4 % and	< -4 %	44	0.0861
	0 to -4 %	49	0.0139*
	Combined Sag	48	0.0018*
	Combined LSD	45	0.1002
< -4 % and	0 to -4 %	47	0.7400
	Combined Sag	46	0.4080
	Combined LSD	43	0.5626
0 to - 4 % and	Combined Sag	41	0.5777
	Combined LSD	48	0.3921
Combined Sag and Combined LSD		47	0.3775
* Indicates a significant difference.			

DESIGN-CONSISTENCY EVALUATION AND SPEED-PROFILE MODEL PROCEDURE

The design-consistency evaluation and the speed-profile model consists of several steps as illustrated in figure 65. The following is a discussion of each of these steps:

1. Select desired speed for the alignment.
2. Predict speed for each curve using the developed equations.
3. Predict grade-limited speed using the TWOPAS equations.
4. Select the lowest speed for each element along the alignment.
5. Adjust speeds for acceleration and deceleration.
6. Perform design-consistency evaluation.
7. Complete the speed profile for the alignment.

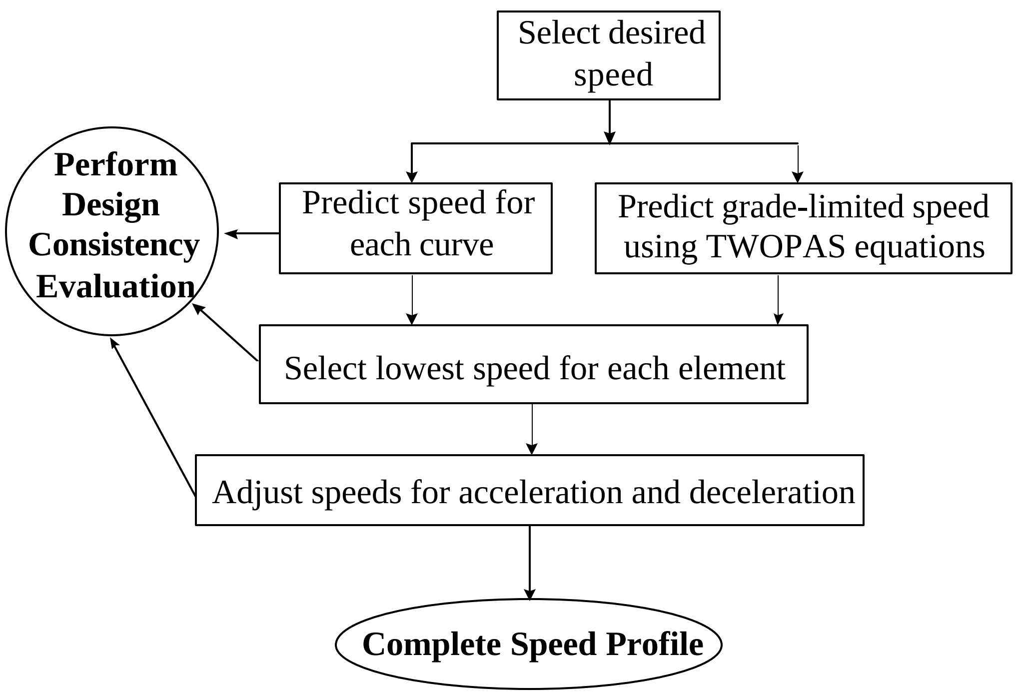


Figure 65. Design-Consistency Evaluation and Speed-Profile Model Flow Chart.

Select Desired Speed

The initial step is to select the desired speed along the roadway. Desired speed is the speed that drivers select when not constrained by the vertical or horizontal alignment. The desired speed for a driver cannot be measured; however, the speeds observed on a tangent beyond the influence of horizontal or vertical alignment can serve as an assumed desired speed. Based on the findings reported in chapter 4, the average 85^{th} percentile speeds on long tangents range between 93 and 104 km/h for the different States in this study. Therefore, a speed of 100 km/h is a good estimate of the desired speed along a two-lane rural roadway when seeking a representative, rounded speed. The long tangent is defined as a tangent that is long enough for a driver to accelerate to and maintain a desired speed for some distance.

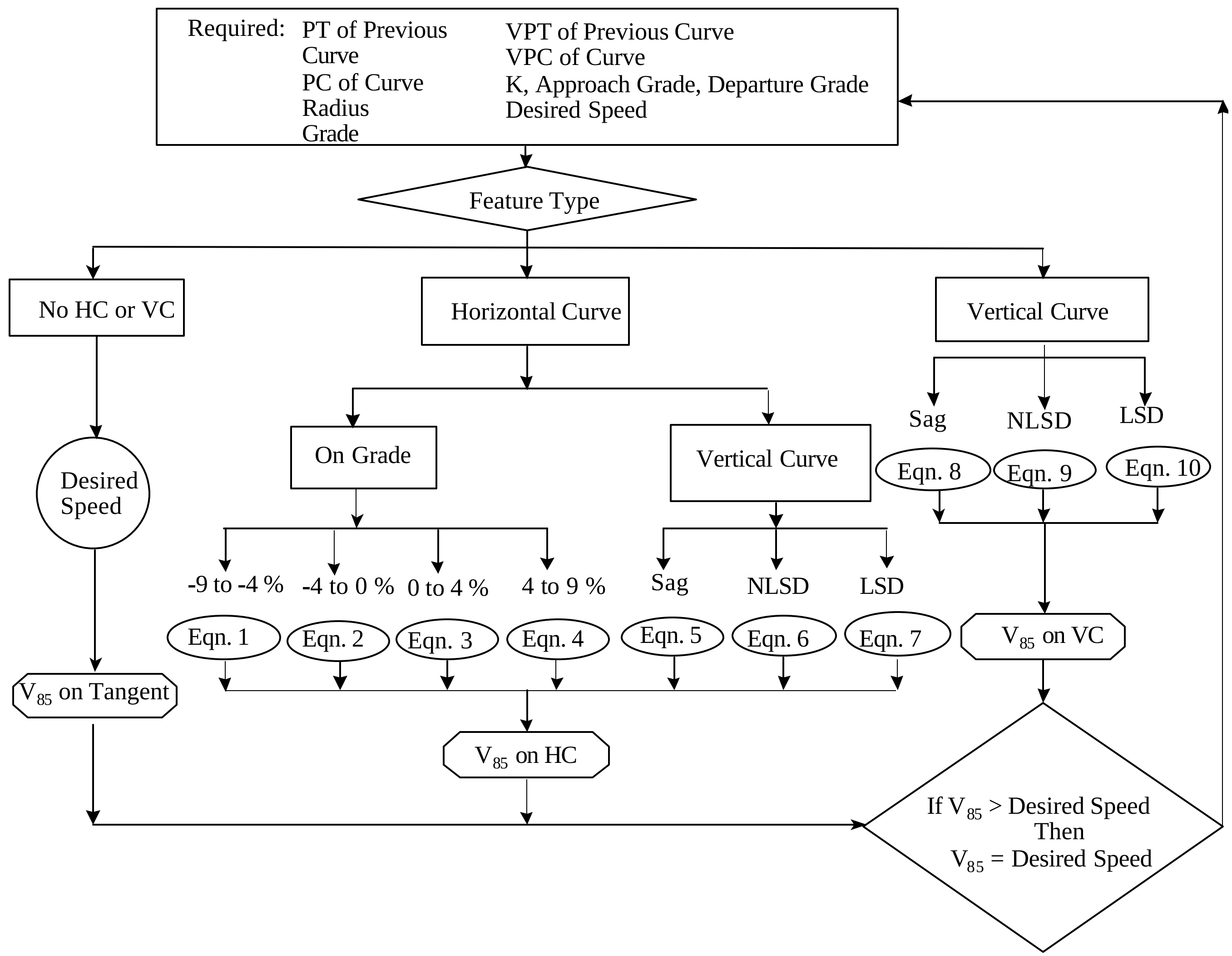
Predict Speed for Each Curve Using the Developed Equations

Several equations were developed to predict speeds for different alignment conditions in this study. Figure 66 illustrates the basic logic for using the speed-prediction equations listed in table 52. The characteristics of the roadway needed for use with the speed-prediction equations are listed at the top of figure 66 and include the beginning and ending points of the curves, the radius of the horizontal curve, the vertical grade, and/or the K-value for the vertical curve. This information determines which of the three major categories to use: horizontal curve, vertical curve, or tangent section (i.e., no horizontal or vertical curve).

The speeds predicted from the equations represent the speeds measured at the midpoint of the curve. The model currently assumes that this speed is constant throughout the horizontal or vertical curve; however, as the data in chapter 6 illustrate, speeds vary throughout the curve. Because a defined pattern of how speed varies through the curve was not present for the 21 horizontal curves (see tables 40 and 41), the previous assumption of constant speed for the entire horizontal curve was retained.

Predict Grade-Limited Speed Using the TWOPAS Vehicle Performance Equations

The TWOPAS equations can be used to check the performance-limited speed at every point on the roadway (upgrade, downgrade, or level). If, at any point, the grade-limited speed is less than the tangent or curve speed predicted using the speed-prediction equations or the assumed desired speed, then the grade-limited speed will govern. Chapter 5 presents the TWOPAS equations and examples of the effects of grade on passenger cars and trucks.



Note: Alignment-Condition Equations are listed in table 52.

Figure 66. Speed-Prediction Equations.

Table 52. Speed-Prediction Equations for Passenger Vehicles.

AC EQ No. [1]	Alignment Condition	Equation [2]	No. of Obser.	R^2	MSE
1.	Horizontal Curve on Grade: -9% # G < -4%	V_{85} ' 102.10 & $\frac{3077.13}{R}$	21	0.58	51.95
2.	Horizontal Curve on Grade: -4% # G < 0%	V_{85} ' 105.98 & $\frac{3709.90}{R}$	25	0.76	28.46
3.	Horizontal Curve on Grade: 0% # G < 4%	V_{85} ' 104.82 & $\frac{3574.51}{R}$	25	0.76	24.34
4.	Horizontal Curve on Grade: 4% # G < 9%	V_{85} ' 96.61 & $\frac{2752.19}{R}$	23	0.53	52.54
5.	Horizontal Curve Combined With Sag Vertical Curve	V_{85} ' 105.32 & $\frac{3438.19}{R}$	25	0.92	10.47
6.	Horizontal Curve Combined With Non-Limited Sight-Distance Crest Vertical Curve	(see note 3)	13	n/a	n/a
7.	Horizontal Curve Combined With Limited Sight-Distance Crest Vertical Curve (i.e., K # 43 m/%)	V_{85} ' 103.24 & $\frac{3576.51}{R}$ (see note 4)	22	0.74	20.06
8.	Sag Vertical Curve on Horizontal Tangent	V_{85} = *assumed desired speed*	7	n/a	n/a
9.	Vertical Crest Curve With Non-Limited Sight Distance (i.e., K > 43 m/%) on Horizontal Tangent	V_{85} = *assumed desired speed*	6	n/a	n/a
10.	Vertical Crest Curve With Limited Sight Distance (i.e., K # 43 m/%) on Horizontal Tangent	V_{85} ' 105.08 & $\frac{149.69}{K}$	9	0.60	31.10

NOTES:
1. AC EQ No. = Alignment Condition Equation Number
2. Where: V_{85} = 85th percentile speed of passenger cars (km/h) K = rate of vertical curvature
   R = radius of curvature (m) G = grade (%)
3. Use lowest speed of the speeds predicted from AC EQ No. 1 or 2 (for the downgrade) and AC EQ No. 3 or 4 (for the upgrade).
4. In addition, check the speeds predicted from AC EQ No. 1 or 2 (for the downgrade) and AC EQ No. 3 or 4 (for the upgrade) and use the lowest speed. This will ensure that the speed predicted along the combined curve will not be better than if just the horizontal curve was present (i.e., that the inclusion of a limited sight-distance crest vertical curve results in a higher speed).

Select the Lowest Speed for Each Element

The speeds predicted from the previous three methods (assumed desired speed, speeds predicted using the speed-prediction equations, and the speeds from the TWOPAS equations) are compared and the lowest speed is selected. The speeds for the different alignment features could then be compared to identify unacceptable changes in speed between alignment features. For example, a design-consistency evaluation could be conducted and a flag raised if the speed change from one curve to another is greater than a pre-set number. If a continuous speed profile for the alignment is desired or if the deceleration or acceleration from one curve to another needs to be checked, these lowest speeds would then be used in the following step.

Adjust Speeds for Acceleration and Deceleration

The next step of the design-consistency evaluation is to compare the calculated deceleration and acceleration rates using the lowest speeds determined in the previous step with the recommended rates present in table 53. In developing a speed-profile model, the next step of the process is to adjusted the speed prior to and departing from a curve. If the speed on a curve is less than the desired speed, the driver decelerates when approaching the curve and then accelerates after departing the curve. Table 53 lists the values determined from either the studies conducted as part of this research (see chapter 6) or the values selected based on engineering judgment and previous research.

The values selected for the speed-profile model are based on the observed speeds at the 21 horizontal curves studied in this project (see chapter 6). Three sets of acceleration and deceleration values were identified for the design-consistency checks. The range for the initial set, called *good* design, was selected as being between those observed at the 21 horizontal curves (see chapter 6) and the value for *normal* acceleration or deceleration for speed changes between 50 and 60 mph (80.5 and 96.6 km/h) as presented in ITE's *Transportation and Traffic Engineering Handbook*.[69] The *fair* design range for acceleration was from the *normal* acceleration rate to the maximum acceleration rate.[69] For deceleration, ITE's *Handbook* has a *reasonable comfortable* rate of 2.46 m/s^2; however, only approximately 90 percent of *wet* pavements provide that level of friction (see figure III-1, part B in AASHTO's *A Policy on Geometric Design of Highways and Streets*).[2] To include consideration for wet pavements, the recommendation is that the *fair* design range for deceleration be between *normal* deceleration and 2.00 m/s^2. *Poor* design for deceleration would exceed the 2.00-m/s^2 value and for acceleration, it would exceed 1.25 m/s^2.

Figure 67 illustrates the different conditions that can occur when evaluating and/or predicting acceleration and deceleration speeds. Six conditions were identified. Supporting equations for these conditions are listed in table 54.

Table 53. Deceleration and Acceleration Rates.

Deceleration Rate, d (m/s^2)	Alignment Condition		Acceleration Rate, a (m/s^2)
Speed Profile			
<u>Radius, R (m)</u> — <u>d</u> $R \geq 436$ — 0.00 $175 \leq R < 436$ — $\left\lvert -0.6794 + \frac{295.14}{R} \right\rvert$ $R < 175$ — 1.00	1-4	Horizontal Curves on Grade: $-9\% \leq G < 9\%$	<u>Radius, R (m)</u> — <u>a</u> $R > 875$ — 0.00 $436 < R \leq 875$ — 0.21 $250 < R \leq 436$ — 0.43 $175 < R \leq 250$ — 0.54
1.00	5	Horizontal Curve Combined With Sag Vertical Curve	0.54
(use rates for Alignment Conditions 1 through 4)	6	Horizontal Curve Combined With Non-Limited Sight-Distance Vertical Curve	(use rates for Alignment Conditions 1 through 4)
1.00	7	Horizontal Curve Combined With Limited Sight-Distance Crest Vertical Curve (i.e., $K \leq 43$ m/%)	0.54
n/a	8	Sag Vertical Curve on Horizontal Tangent	n/a
n/a	9	Vertical Crest Curve With Non-Limited Sight Distance (i.e., $K > 43$ m/%) on Horizontal Tangent	n/a
1.00	10	Vertical Crest Curve With Limited Sight Distance (i.e., $K \leq 43$ m/%) on Horizontal Tangent	0.54
where: K = rate of vertical curvature, G = grade (%)			
Design Consistency (All Alignment Conditions)			
1.00 to 1.48	Good Design		0.54 to 0.89
1.48 to 2.00	Fair Design		0.89 to 1.25
>2.00	Poor Design		>1.25

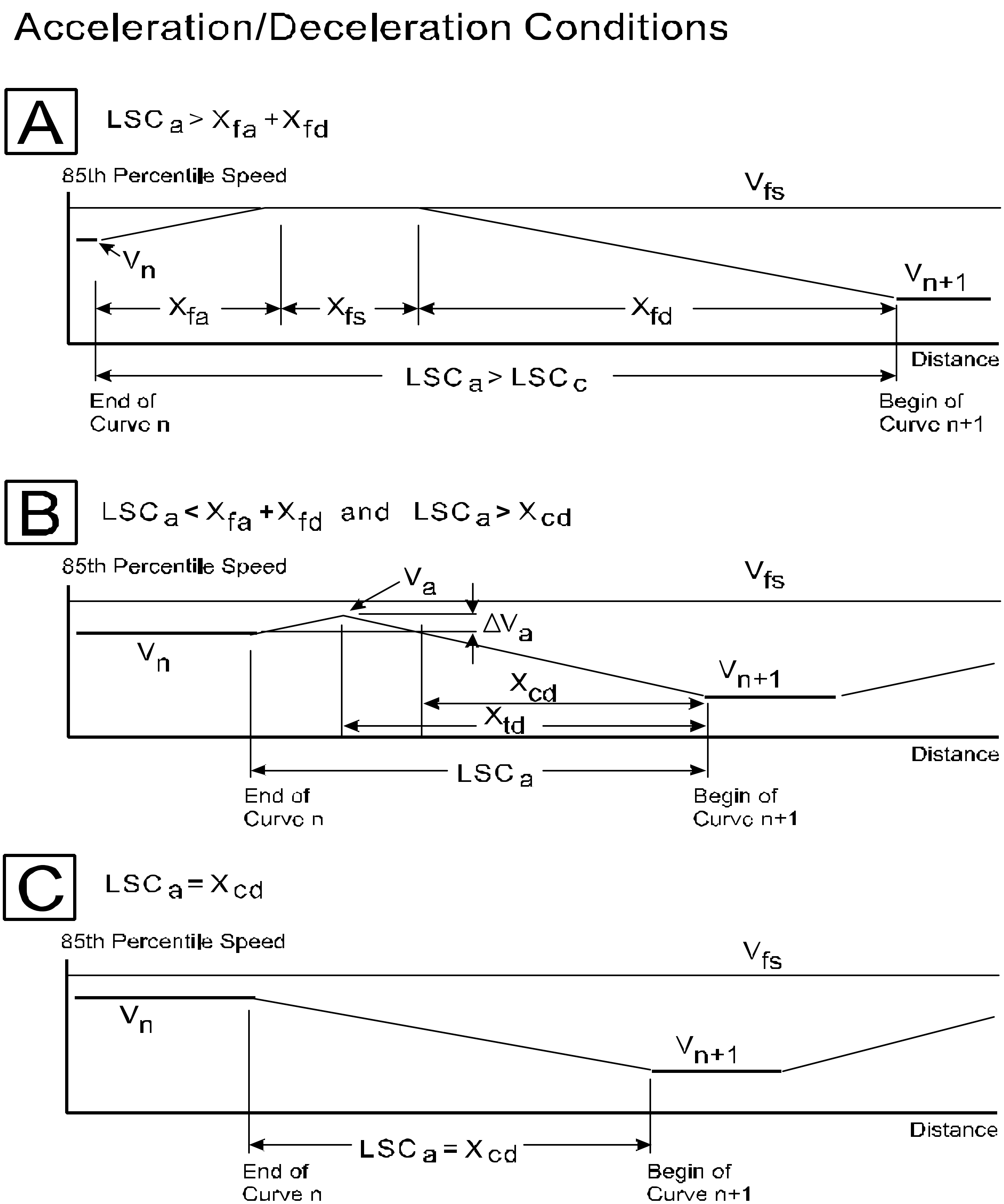


Figure 67. Acceleration/Deceleration Conditions.
(See table 54 for variable definitions.)

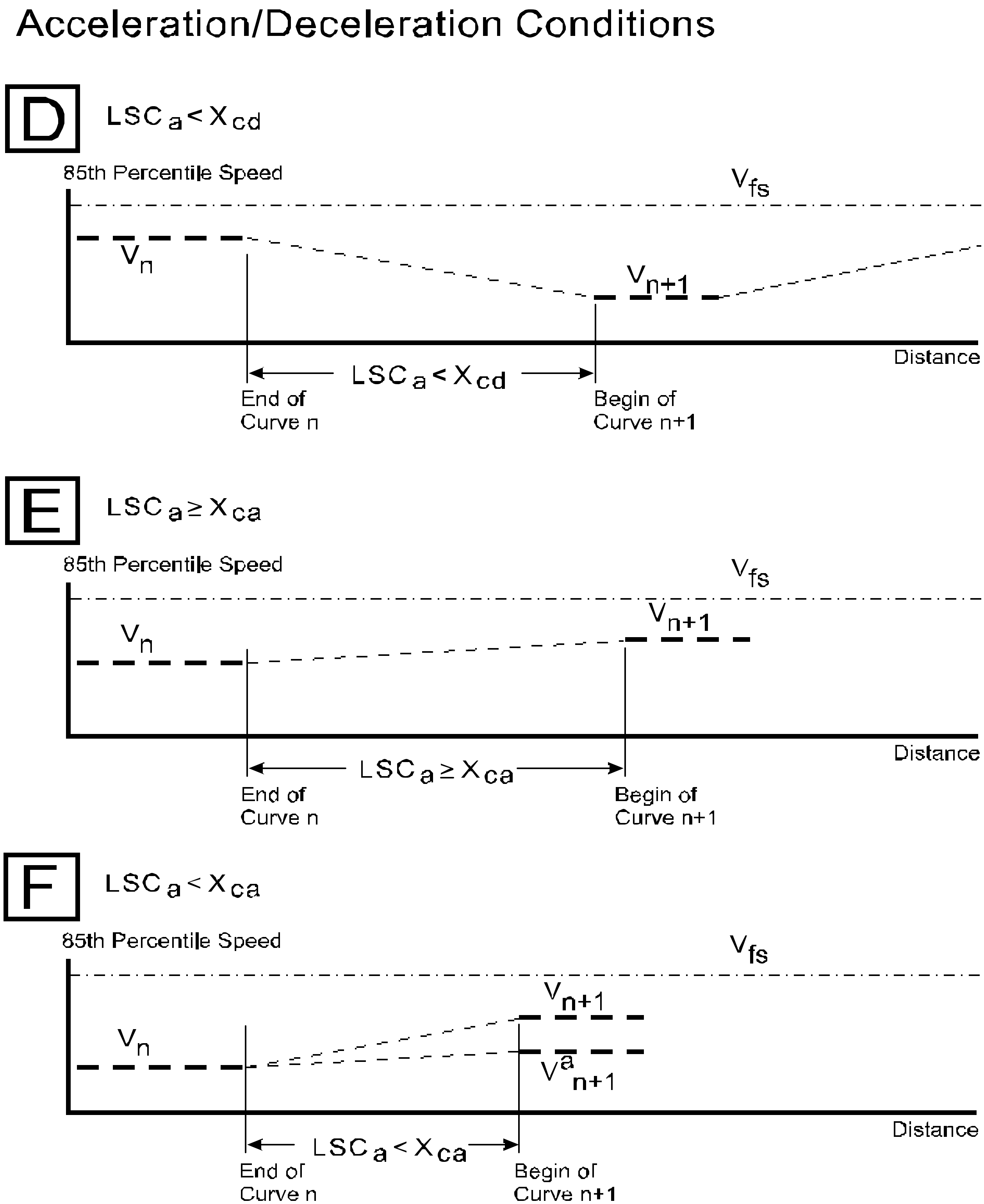


Figure 67. Acceleration/Deceleration Conditions (continued).

Table 54. Equations for Use in Determining Acceleration and Deceleration Distances.

$$LSC_c = X_{fa} + X_{fd} \quad (1)$$

$$X_{fd} = \frac{V_{fs}^2 - V_{n+1}^2}{25.92\, d} \quad (2)$$

$$X_{cd} = \frac{V_n^2 - V_{n+1}^2}{25.92\, d} \quad (3)$$

$$X_{ca} = \frac{V_{n+1}^2 - V_n^2}{25.92\, a} \quad (4)$$

$$X_{fa} = \frac{V_{fs}^2 - V_n^2}{25.92\, a} \quad (5)$$

$$X_{fs} = LSC_a - X_{fd} - X_{fa} \quad (6)$$

$$X_{td} = \frac{V_a^2 - V_{n+1}^2}{25.92\, d} \quad (7)$$

$$V_a = V_n + \Delta V_a \quad (8)$$

Note: When calculating V_a, the curve with the larger radius is to be used.

$$\Delta V_a = \left(\frac{25.92ad(LSC_a) + dV_n^2 + aV_{n+1}^2}{(a+d)} \right)^{\frac{1}{2}} - V_n \quad (9)$$

$$V_{n+1}^a = \left(V_n^2 + 2a(LSC_a)\right)^{\frac{1}{2}} \quad (10)$$

Where:

V_{fs}	=	85th percentile desired speed on long tangents (m)
V_n	=	85th percentile speed on Curve n (km/h)
V_{n+1}	=	85th percentile speed on Curve n+1 (km/h)
V^a_{n+1}	=	85th percentile speed on Curve n+1 determined as a function of the assumed acceleration rate (km/h)
V_a	=	maximum achieved speed on roadway between curves in Condition B (km/h)
ΔV_a	=	difference between speed on Curve n and the maximum achieved speed on roadway between curves in Condition B (km/h)
d	=	deceleration rate, see table 53 (m/s^2)
a	=	acceleration rate, see table 53 (m/s^2)
LSC_c	=	critical length of roadway to accommodate full acceleration and deceleration (m)
LSC_a	=	length of roadway available for speed changes (m)
X_{fd}	=	length of roadway for deceleration from desired speed to Curve n+1 speed (m)
X_{cd}	=	length of roadway for deceleration from Curve n speed to Curve n+1 speed (m)
X_{td}	=	length of roadway for deceleration from V_a to Curve n+1 speed (m)
X_{ca}	=	length of roadway for acceleration from Curve n speed to Curve n+1 speed (m)
X_{fa}	=	length of roadway for acceleration from Curve n speed to desired speed (m)
X_{fs}	=	length of roadway between two speed-limited curves at desired speed (m)

In the previous study, the tangent between horizontal curves was the length of roadway used for acceleration from a curve and deceleration into a curve. If the tangent was sufficiently long, then full acceleration to the desired speed was achieved. When the tangent length was not sufficient for full acceleration, other assumptions concerning acceleration were made. This study adds the vertical component to the speed-profile model. While horizontal alignment influences speed in most cases when vertical and horizontal curves overlap, there are cases when the vertical alignment does affect the speed on the horizontal curve. Also, the vertical alignment can affect speed when no horizontal curvature is present. Given that vertical alignment is now considered in the process, the terms and processes used in determining acceleration and deceleration behavior should be modified from the presentation in the previous study. The following is a summary of the steps needed to determine the acceleration and deceleration behavior of vehicles before and after speed-limiting curves.

The distance between speed-limiting curves is termed the "length of roadway available for speed changes." This length may be the distance between the end of one horizontal curve to the beginning of the next horizontal curve (i.e., the tangent length from PT to PC), if the two speed-limiting curves are horizontal curves. When one of the curves is a vertical curve, then the length of roadway for speed changes will be from the end of the previous curve to the beginning of the following curve. These curves will be called "Curve n" and "Curve n+1" for this procedure.

The procedure begins with the following:

- Calculate length of roadway available for speed changes, LSC_a.
- Calculate critical length of roadway needed to accommodate full acceleration and deceleration, LSC_c (see AC EQ #1 in table 54).
- Calculate length of roadway for deceleration from desired speed to Curve n+1 speed, X_{fd} (AC EQ #2 in table 54).
- Calculate length of roadway for deceleration from Curve n speed to Curve n+1 speed, X_{cd} (AC EQ #3 in table 54).

The determination of the length of roadway available for speed changes (LSC_a) is a critical component in this process. When one grade-limiting curve immediately follows another similar type of curve, then the length of roadway available is the horizontal and/or vertical tangent between the curves. The combination of horizontal and vertical alignment changes that approach. Prior to this, the presence of a vertical curve between two speed-limiting horizontal curves was ignored. In this procedure, one must determine whether the curve limits the speed and if it does, then it must be considered when determining the available length of roadway. In other words, if a crest vertical curve separates two grade-limited curves (say, a short-radius horizontal curve followed by a sag vertical curve), then the length of roadway along the crest is available for acceleration and deceleration.

The equations for critical length of roadway and deceleration lengths are listed in table 54. The calculated values are then compared to determine which condition exists. If the length of roadway available for speed changes is greater than the critical length of roadway ($LSC_a > LSC_c$), then full acceleration and deceleration occurs. When curves are separated by an adequate distance, the acceleration from the initial curve does not affect the deceleration into the following curve. Basically, each curve is beyond the influence of other curves and drivers reach their desired speed on the roadway between curves. This alignment is shown as Condition A in figure 67.

When the length of roadway is not sufficient to allow full acceleration to desired speed (i.e., $LSC_a < LSC_c$), then the length of roadway needed for deceleration (X_{fd}) is compared to the available length (LSC_a). If the length needed for deceleration to Curve n+1 from Curve n is less than the available length (i.e., $LSC_a > X_{cd}$), then some acceleration from the previous curve will occur (see Condition B in figure 67 and the equations listed in table 54). When the distance to decelerate from Curve n to Curve n+1 is equal to the available distance (i.e., $LSC_a = X_{cd}$), vehicles will not accelerate upon departing from Curve n. They will begin their deceleration for Curve n+1 upon departing Curve n as shown in Condition C in figure 67.

In some cases, the procedure will determine a deceleration distance from Curve n to Curve n+1 that is greater than the available distance (i.e., $LSC_a < X_{cd}$, see Condition D in figure 67). In these situations, the IHSDM could raise a flag indicating to the user that greater deceleration than desirable is occurring between two curves (using the values presented in table 53). Note that the deceleration rates for developing a speed-profile model are based on observed behavior, while the design-consistency values are based on comfortable values or friction values for wet pavement. Drivers will accept higher values in certain situations and they will begin deceleration prior to the end of a curve and will continue to decelerate into the next curve. While Condition D should be checked by the user or include additional checks within IHSDM, it may not be as critical as it initially appears.

When the speed on Curve n is less than the speed on Curve n+1, the driver will accelerate between the two curves. If the distance available for acceleration is longer than the distance calculated using the acceleration rates in table 53, then Condition E in figure 67 applies, otherwise Condition F applies. Similar to the deceleration Condition D, the occurrence of Condition F should be checked, but it may not be critical. The acceleration rates listed in table 53 for a speed-profile model were for comfortable, observed levels. Drivers may use higher acceleration rates than shown or they will enter curve n+1 at a speed lower than the value calculated using the speed-prediction equations. It would be unusual for a driver to accelerate past a comfortable rate just because of the geometrics (see V^a_{n+1} in figure 67). The speed-profile model should adjust the downstream speed to match what a driver will reasonably drive rather than assuming a driver will exceed a given acceleration rate to reach the speed predicted by the equations listed in table 52. If condition F exists, then the speed on curve n+1 should be adjusted to V^{a}_{n+1} (see figure 67 and table 54).

Perform Design-Consistency Evaluation

One use of the predicted speed values is to perform an evaluation of the consistency between features. For example, after the speeds are predicted using the equations listed in table 52, the difference in speed between curves can be calculated. If the speed change is greater than (or equal to) a preset value, such as 15 km/h, a design inconsistency can be flagged. If greater knowledge of the quality of design is desired, then the following scale (as proposed by Lamm et al.) could be considered:[70]

Good Design: $\Delta V_{85} \leq 10$ km/h
Fair Design: 20 km/h $\geq \Delta V_{85} \geq$ 10 km/h
Poor Design: $\Delta V_{85} \geq 20$ km/h

The design-consistency evaluation can also be performed after all steps shown in figure 65 are completed. This evaluation will allow for consideration of the acceleration and deceleration needs. Consideration of acceleration and deceleration can occur in several forms. For example, if the distance available for acceleration or deceleration is less than desirable (e.g., deceleration and acceleration conditions D or F in figure 67 and values exceeding those listed in table 53), a flag could be raised informing the user of the situation. To better represent actual operating conditions when this condition occurs, however, IHSDM should include other changes. For example, the current assumption is that all acceleration/deceleration occurs on the tangent; however, observed data demonstrate that some acceleration/deceleration occurs within the curve. Acceptable acceleration/deceleration rates may exist if some acceleration/deceleration is assumed to occur on the curves.

The IHSDM could also adjust the curve speed from the speed determined in a previous step to a speed determined using the acceleration/deceleration rates presented in table 53. For example, the program may want to raise a flag because the predicted speed on curve n was 90 km/h, the speed on curve n+1 was 100 km/h, and these curves are separated by a very short tangent (which results in an acceleration rate greater than the value present in table 53). If this condition existed in the field, a driver would probably begin accelerating from 90 km/h using an acceptable acceleration rate, would traverse curve n+1 at an increasing speed, and may or may not reach the 100-km/h predicted speed within the curve. If these two curves are closely followed by another curve (curve n+2) with a predicted speed of 85 km/h, then the driver may not try to accelerate to 100 km/h on curve n+1, rather the driver would maintain a speed between 90 and 85 km/h on this middle curve. Incorporating this type of check would also serve to verify that a perceived design inconsistency that was flagged because the speed differential between two curves was 15 km/h or greater is really a potential design inconsistency. In the example, the difference in predicted speeds for curve n+1 (100 km/h) and curve n+2 (85 km/h) is 15 km/h, which would raise the design inconsistency flag. When acceleration and deceleration are considered, the curve n+1 speed is adjusted to a value between 90 and 85 km/h and the speed difference between curve n+1 and curve n+2 is now less than the15-km/h limit.

Complete the Speed Profile for the Alignment

In this step, the speeds determined from the above steps would be plotted to illustrate the speed along the alignment. This information is shown within the IHSDM to demonstrate to the users the speed pattern along the alignment.

SPEED-PROFILE EXAMPLE

The following example illustrates how to apply the model to produce a speed profile. The geometry for the sample roadway segment is listed in table 55 and includes three horizontal curves and four vertical curves. The initial step in the procedure is to select a desired operating speed and in this example, a value of 100 km/h was assumed. Next, the geometry of the roadway was used in the speed-prediction equations (see table 52) to determine the speeds for each curve. The geometry was reviewed to determine which feature influences speed at a given point. For example, at 1.45 km, the sag vertical curve is controlling the speed. After the sag curve ends at 1.63 km, the speed predicted for the next feature would normally pass to that assumed for tangents; however, it is determined that the next feature is close and, therefore, acceleration and/or deceleration (A/D) may influence speed and should be checked. Following the sag curve at 1.70 km is a combined curve that results in equation (7) (see table 52) being used to predict a speed of 94 km/h for the feature.

Once the speeds are predicted using the equations listed in table 52, TWOPAS was used to determine locations where speeds are affected by vertical grades for a medium-performance passenger car. The results from the assumed desired speed, the speed-prediction equations, and TWOPAS are then compared, and the lowest predicted speed is chosen for each feature or grade. Figure 63 shows the alignment, desired speed, and predicted speeds for the example. The lowest speed for each feature in this example was the speed calculated using the speed-prediction equations.

Figure 68 shows the predicted speeds for the alignment, but does not include acceleration or deceleration from feature to feature. The acceleration/deceleration rates are calculated based on the alignment and the information in table 53. These rates, along with the predicted speeds, are the inputs into the equations shown in table 54. The equations determine the distance required to decelerate or accelerate, which is the distance prior to or following a curve where deceleration or acceleration begins or ends. Figure 69 shows the speed profile for the sample roadway with the adjustments for acceleration/deceleration.

Table 55. Alignment and Results From Speed-Profile Example.

Alignment							Speed-Profile Results			
Dist. (km)	**Descrip-tion**	**Radius (m)**	**K (m/%)**	**Length (m)**	**Approach Grade (%)**	**Departure Grade (%)**	**Controlling Feature**	**Equation No.[a]**	**Speed (km/h)**	**Tangent Condition[b]**
0.00	BEGIN						TANGENT	TAN	100	
0.50	PVC		26	210	3		CREST	10	99	
0.71	PVT					-5	A/D	A/D	99	C
0.85	PC	250					HC	1	90	
1.10	PT						TANGENT	TAN	100	
1.45	PVC		18	175	-5		SAG	8	100	
1.63	PVT					5	A/D	A/D	100	A
1.70	PC/PVC	400	40	400	5		COMBINED	7	94	
2.10	PT/PVT					-5	TANGENT	TAN	100	
2.50	PVC		33	200	-5		SAG	8	100	
2.70	PVT					1	A/D	A/D	100	A
2.90	PC	275					HC	3	92	
3.18	PT						TANGENT	TAN	100	
4.00	END						TANGENT	TAN	100	

[a] Numbers 1 to 10 = Equations are listed in table 52.
A/D = Acceleration and/or deceleration may control on this feature and should be checked.
TAN = Assumed desired speed for long tangent controls.

[b] See figure 67 and tables 53 and 54.

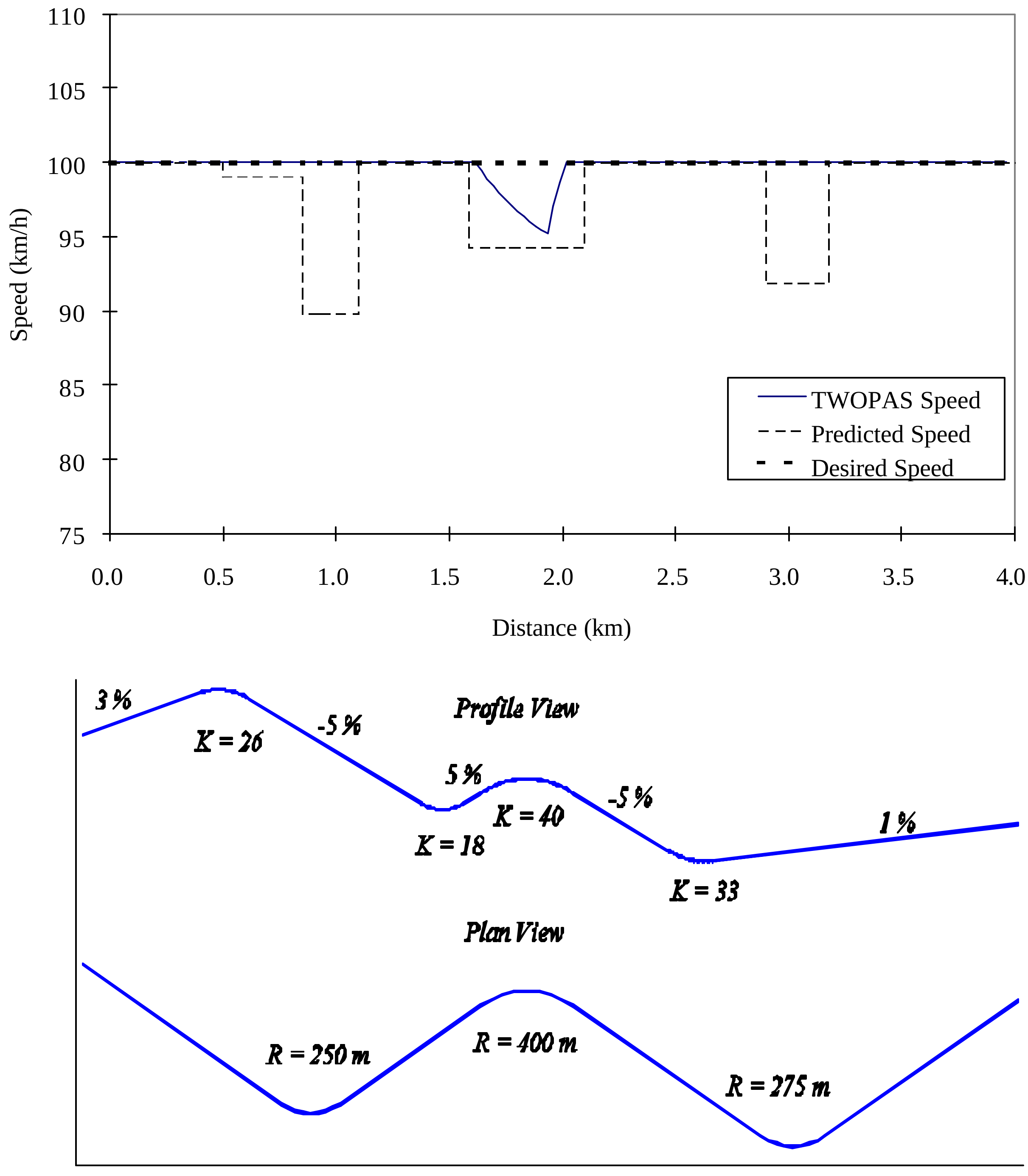


Figure 68. Speed Profile and Alignment.

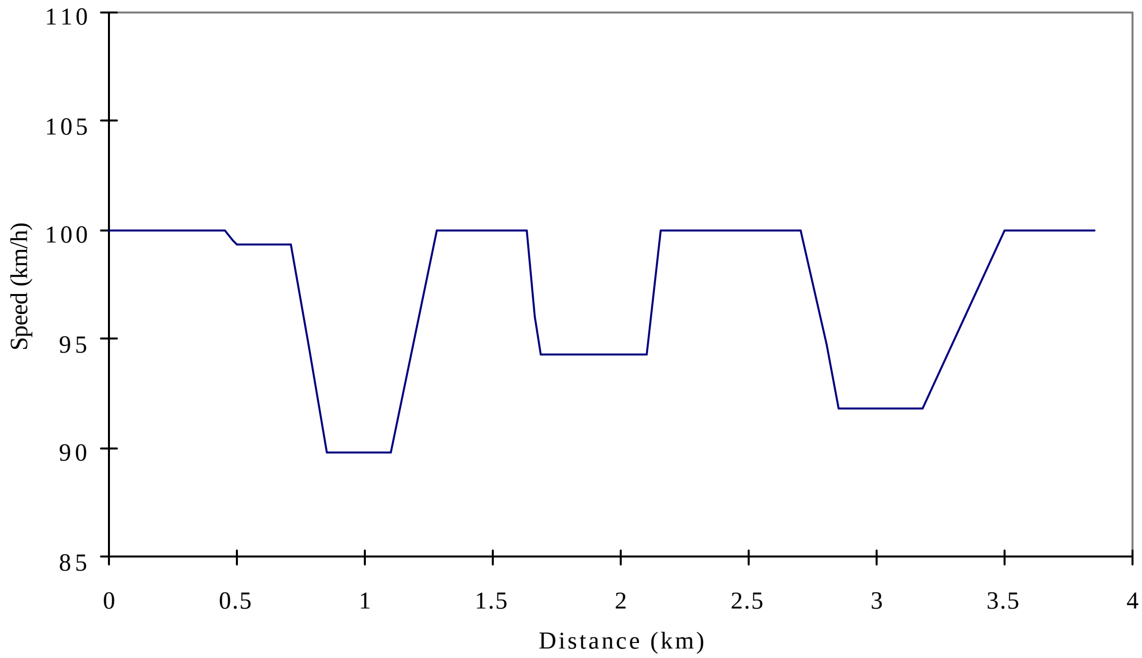


Figure 69. Speed Profile With Acceleration and Deceleration.

DESIGN-CONSISTENCY EXAMPLE

The geometry for a roadway collected as part of this study was used as an example of a design-consistency evaluation. The design-consistency evaluation identifies unacceptable speed changes along an alignment. The roadway was chosen because it had several grade changes along with several horizontal curves. The roadway is slightly more than 17 km in length. The horizontal curvature ranged from 175 m to 1,750 m in radius and the vertical curves ranged from 60 m to 290 m in length. Grades ranged from -6.7 to 6.2 percent, which resulted in the prediction of speeds on horizontal curvatures for all grade conditions. Combinations of horizontal curves and vertical curves were present for non-limited and limited sight-distance crest vertical curves and sag vertical curves. Figure 70 illustrates the speeds predicted along the entire 17.4-km length. The last 1.4 km of the roadway has several speed changes greater than 15 km/h between features. Figure 71 shows that portion of the speed profile while table 56 lists the locations where inconsistencies may exist with the roadway design.

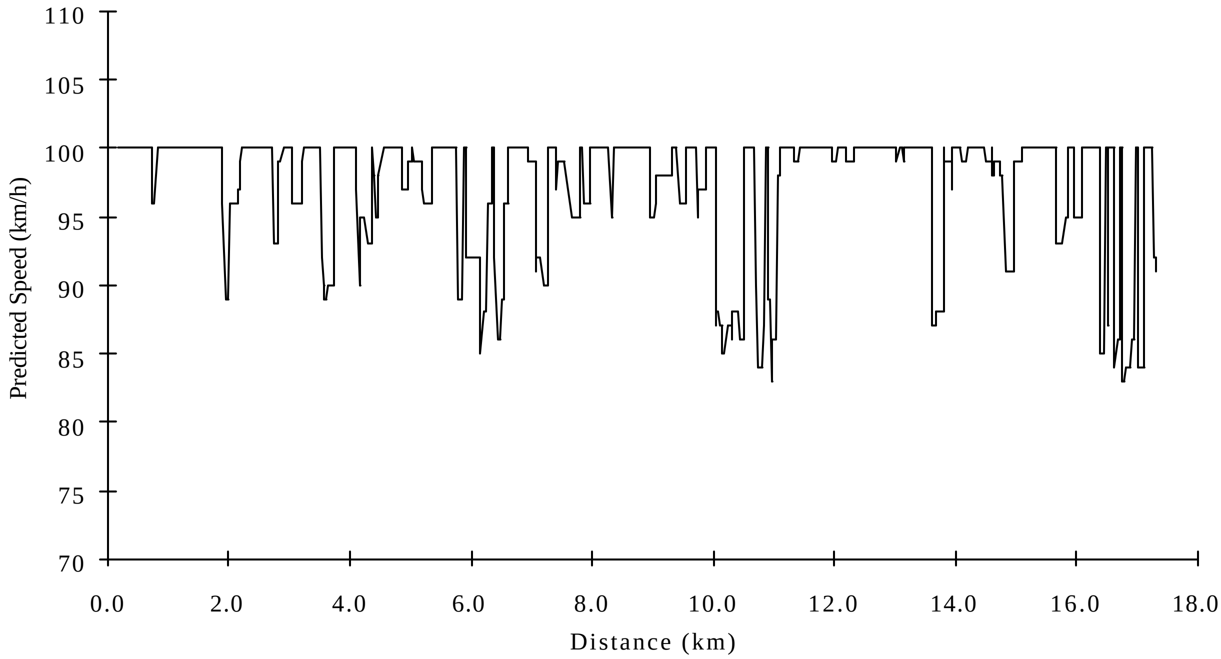


Figure 70. Predicted Speed Profile for Sample Roadway.

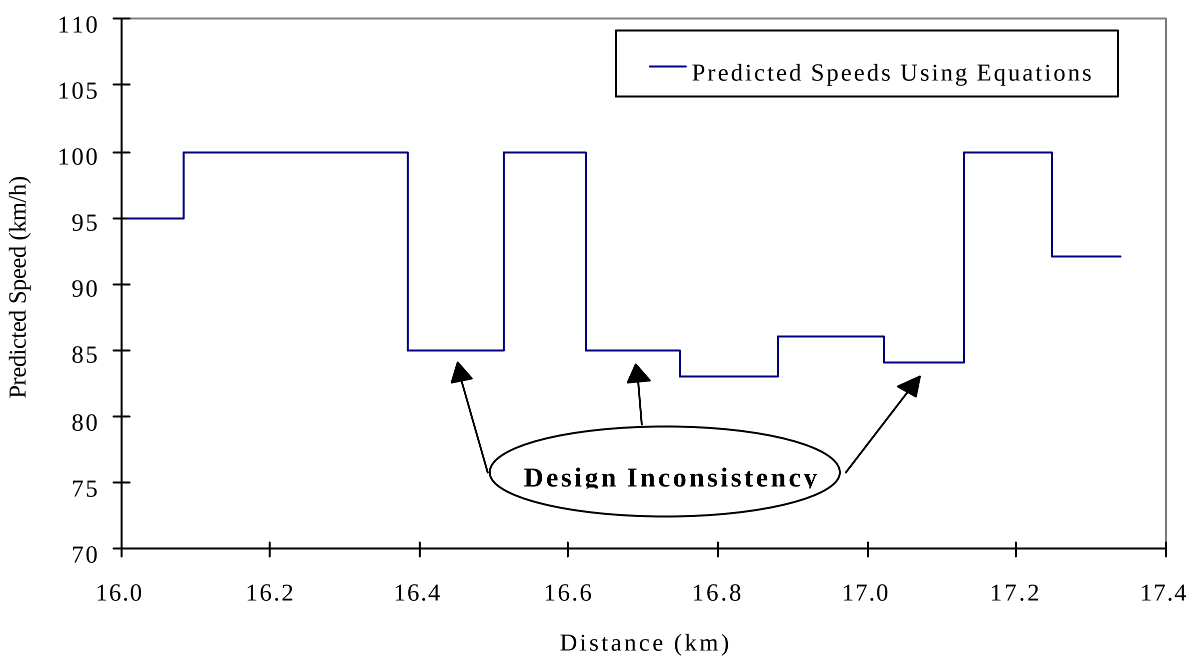


Figure 71. Closeup of a Portion of the Design-Consistency Evaluation.

Table 56. Location of Design Inconsistency for Sample Roadway.

Location (km)	Change in Speed (km/h)
10.7	15
11.0	15
16.4	15
16.6	16
17.1	16

SPEED-PROFILE MODEL ASSUMPTIONS

The speed-profile model developed in this project is to predict the speeds along a continuous alignment using both the vertical and horizontal geometry. The research approach employed to develop the model used either isolated horizontal or vertical curves or, when investigating the horizontal and vertical curve combination, only used curves that overlapped significantly. Data were not collected at sites that did not match the stated criteria. This approach was taken so that a basic understanding could be achieved on the relationships between speed and curve geometry. An advantage to the approach is that we have greater confidence in knowing which elements affect speed along a roadway. A disadvantage occurs when attempting to determine the changing speed along a complex alignment that includes curves that do not fit the criteria developed for the data collection efforts.

The findings from the field-data work were used to develop regression equations to predict speeds for a given set of conditions. Limitations with regression include using the equation beyond its limit and the matching of one regression equation to another. For example, questions are raised on the validity of the predicted values when the regression equation is used beyond the range of the collected data. Another concern is how well regression equations that are developed with different sets of data interact. For example, in this study, we developed speed-prediction equations based on data collected along a set of horizontal curves. We then modified that predicted speed for acceleration and deceleration performance using a regression equation that was developed from other data. In general, the equations fit well, however, there are cases when adjustments will need to be made because of these issues. The following is a discussion on some of the situations identified during the model development that need to be considered in using the speed-profile model.

Vertical Curve Type. Vertical curves are categorized into four types. Type I consists of curves that have an initial upgrade followed by a downgrade. Type III curves have a downgrade that is followed by an upgrade. Type I curves are generally viewed as being the “typical” crest vertical curves, while Type III curves are the typical sag vertical curves. Type II curves are also crest vertical curves; however, they are an upgrade followed by a less step upgrade or a downgrade followed by a steeper downgrade. Type IV curves are sag curves with an upgrade followed by a steeper upgrade or a downgrade followed by a less steep downgrade. This study only collected data at Type I and Type III curves. Therefore, an assumption was made during the model development that all crest curves have a similar speed relationship as the Type I curves, and that all sag curves have a similar speed relationship as found for Type III curves.

Speed on Tangent. The previous study suggested that the tangent speed should be assumed to be 97.9 km/h.[(7)] This research supported that value and also demonstrated that higher speeds may be observed on tangents. The desired speed on roadway value was rounded to 100 km/h for use in the examples.

Maximum Speed on Curves. In some situations, the speeds predicted using the regression equation are higher than the assumed speed on the tangent. For consistency within the speed-profile model, the maximum speed on a curve should be set to the desired speed on the tangent. In the examples, this speed is 100 km/h. Figure 72 illustrates the plots that would have resulted for the speed-profile example if different desired speeds would have been selected.

Minimum Speed on Curves. The smallest radius in the field data set was 100 m. During testing of the model, smaller radii were used to determine how the model would perform. In some cases, the model predicted not only very low speeds, but negative speeds. This is a consequence of using a regression equation beyond its limit. To result in a more realistic speed prediction, the minimum predicted speed is set to 60 km/h when the radius of a horizontal curve is less than 100 m.

Length of Roadway Available for Speed Changes, Formerly Known as Tangent Length. Tangent length is a critical portion of an accurate speed-profile model because it is the area where all acceleration and/or deceleration is assumed to occur for a curve. In the previous work, this length was assumed to be from the PT of the previous horizontal curve to the PC of the following horizontal curve.[7] Vertical alignment was ignored. This research study was to produce a speed-profile model that includes both vertical alignment and horizontal alignment. Given that, one could assume that the length is from the end of a curve to the beginning of the next curve, regardless of whether the curve is a vertical curve or a horizontal curve. The findings to date demonstrate that the speed on a non-limited sight-distance crest curve and on sag curves is not influenced by the characteristics of the

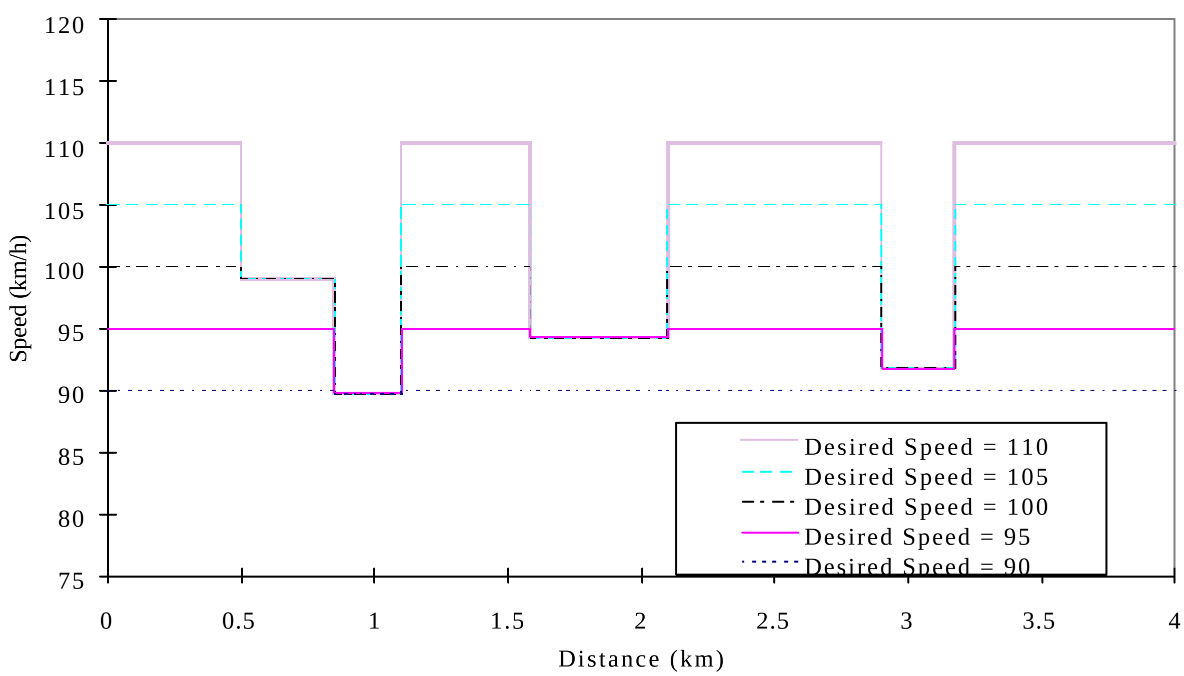


Figure 72. Effect of Different Assumed Desired Speeds on Speed-Profile Example.

vertical curve. Therefore, the determination of the roadway length available for speed changes should include vertical curves of those types.

Acceleration/Deceleration Rates for Vertical and Vertical/Horizontal Alignment. The models developed to predict acceleration and deceleration rates were based on data collected at horizontal curves. Thus, no data were available to predict acceleration/deceleration rates for vertical alignment or for combinations of horizontal/vertical alignment. In these cases, the maximum values selected for the sites studied in the project were assumed. In the cases of crest vertical curves with limited sight distance, the acceleration rate for the departure tangent can be applied starting from the VPI, rather than the VPT, because there the constraint imposed on the driver is removed and the driver can begin accelerating at that point.

Maximum Acceleration/Deceleration Rates. The models developed provide a maximum acceleration rate of 0.54 m/s^2; however, the maximum acceleration observed in the field reached 1.77 m/s^2. The maximum deceleration rate provided by the models is 1.0 m/s^2, while the maximum observed deceleration rate was 1.44 m/s^2. Thus, higher values of acceleration/deceleration may be observed in the field and are recommended for the design-consistency evaluation.

Acceleration/Deceleration Rates Greater Than Assumed Values. In some situations, the length available for speed changes is insufficient for the acceleration or deceleration rates listed in table 53. There are two possible reasons for this discrepancy: (1) the acceleration/ deceleration models developed were based on sites with long approach tangents and horizontal curves only, thus, drivers may apply higher decelerations for more restrictive alignments; and (2) field data showed that there is acceleration and deceleration occurring within the limits of the curve, while the speed-profile model assumes that all acceleration and deceleration takes place prior or after the horizontal curve.

Overlapping Curves. The speed-profile model is based on the first feature encountered. Thus, if a long horizontal curve is combined with a vertical curve, and the vertical curve begins after the PC and ends before the PT, three predicted speeds will result based on that alignment configuration. The speed-profile model examines each feature independently and predicts speeds for the approach grade, the combined curves, and the departure grade based on the horizontal curve radius. If speeds on horizontal curves are constant throughout the curve, then a method must be incorporated into the speed profile to check for potential inconsistencies such as the previous example.

A suggested method is to assume that the lowest speed predicted for any condition controls throughout the horizontal curve. The speed-profile model would need to check each speed predicted within the limits of the horizontal curve to determine the lowest speed. All predicted speeds within the horizontal curve would then be set to the lower values. Currently, this method is not included in the speed-profile model because while the method is logical, it may not reflect observed speeds.

9. RELATIONSHIPS OF GEOMETRIC DESIGN-CONSISTENCY MEASURES TO SAFETY

The preceding chapters of this report have developed several geometric design-consistency measures of potential value to highway designers in assessing the quality of the horizontal and vertical alignment in the designs they develop. These measures include speed reduction, as determined with the speed-prediction equations in table 52, and four of the alignment indices presented in table 19.

These geometric design-consistency measures are conceptually attractive and have been calibrated with actual speed data. However, before a design-consistency methodology is recommended to geometric designers, it would be valuable to demonstrate that the proposed design-consistency measures are, in fact, related to safety. Such a demonstration is presented in this chapter.

The chapter is organized into four major sections. The first section describes the database development and analysis methodology. The second section presents the analysis of speed reduction as a design-consistency criterion for horizontal curves. The third section presents the analysis of roadway alignment indices as design-consistency criteria. The fourth section summarizes the analysis results and presents the conclusions of the chapter.

ANALYSIS METHODOLOGY/DATABASE DEVELOPMENT

The design-consistency methodology using speed reduction as a measure of geometric design consistency is based on a series of speed-prediction models developed through regression analysis. The 10 speed-prediction equations representing the effects of motorist speeds on both horizontal and vertical alignment have been presented in table 52. Taken together, these models allow the prediction of speeds for the sequence of individual features along a two-lane rural highway accounting for the effects of horizontal and vertical alignment. In particular, the models can be used to evaluate the speed reductions between successive tangents and horizontal curves and between successive (i.e., back-to-back) curves.

Databases were developed to test the relationship to safety of the speed-reduction measure and to evaluate the relationship to safety of the roadway alignment indices. To assemble these databases, data were obtained from the FHWA Highway Safety Information System (HSIS) for State-maintained, two-lane rural highways in Washington State. These databases are clearly representative data sets because they include all two-lane rural highways under State jurisdiction in Washington, not just a sample. The Washington HSIS data included roadway inventories, traffic volume, horizontal alignment, vertical alignment, and accidents for 3 years (1993-1995).

Selection of Analysis Sites

Because the available database was so comprehensive, a careful review was required to select sections that were suitable for analysis. A minimum section length of 6.4 km and a maximum section length of 32 km were established for this analysis. These minimum and maximum section lengths were primarily intended to make the database appropriate for the analysis of roadway alignment indices. Sections shorter than 6.4 km might not provide enough individual geometric features to compute a representative value of the alignment indices. Sections longer than 32 km might be so varied that no single value of an alignment index would be representative of the section as a whole (i.e., there is a strong likelihood that any longer section would be non-homogeneous). Following this process, 291 highway sections, with a total length of 4,930 km, were available for analysis.

The first step was to review the data for each route and eliminate portions of the roadway with features that might interfere with the analysis. The following types of locations were eliminated from consideration:

- All locations within 0.8 km of an intersection, with STOP or signal control for traffic on the major road (i.e., the State highway) or within 0.8 km of ramps at freeway-arterial or arterial-arterial interchanges.

- All locations within 0.8 km of a tunnel or a railroad grade-crossing.

- All locations within small towns with speed limits less than 80 km/h, including a buffer area of 0.4 km at either end of the reduced-speed zone.

- All locations within a passing or climbing lane, including a buffer area of 0.4 km at either end of the passing or climbing lane.

Sections were also subdivided at intersections with major changes in average daily traffic (ADT). The remaining sections that were less than 6.4 km in length were then discarded and roadway sections more than 32 km in length were subdivided.

Roadway and Traffic Data

Roadway inventory and traffic data were obtained from the HSIS files for each of these segments. All of the segments analyzed had posted speed limits of 88.5 km/h or more. The traffic volumes of the study sections ranged from 200 to 18,000 veh/day.

Alignment Data

The Washington HSIS files contain data on the geometrics of each horizontal and vertical curve on the highway. For horizontal curves, these data included the location of the point of curvature (PC), the location of the point of tangency (PT), and the radius and length of the curve. For vertical curves, the available data included the approach grade, the departure grade, the algebraic difference in grade, and the length of the curve. The locations of the geometric features were given in the same milepost system used to specify accident locations.

The 291 highway sections were subdivided into individual horizontal curves and tangents and the vertical alignment of each tangent and curve was determined. The predicted 85th percentile speed of motorists on each tangent and curve was determined using the speed-profile models in table 52 and the speed differences between successive features (i.e., tangent/curve, curve/tangent, or curve/curve) were determined. These speed differences generally represented a speed reduction in one direction of travel and a speed increase in the other.

The final database included 5,287 horizontal curves for which the speed differences from adjacent features could be determined.

The alignment data were also used to compute various alignment indices that are discussed later in this chapter.

Accident Data

The accident data from the HSIS files were used to determine the 3-year accident experience of each individual tangent and horizontal curve. The accident analysis considered only non-intersection accidents that involved: (1) a single vehicle running off the road; (2) a multiple-vehicle collision between vehicles traveling in opposite directions; or (3) a multiple-vehicle collision between vehicles traveling in the same direction. These are the same accident types that Zegeer et al. have identified as being "over-represented on curves as compared to tangents."[70] All accidents involving parking, turning, or passing maneuvers; animals in the roadway; or bicycles or motorcycles were excluded.

ANALYSIS OF SPEED REDUCTION AS A DESIGN-CONSISTENCY CRITERION FOR HORIZONTAL CURVES

General Relationship Between Quality of Horizontal Curve Design and Safety

Several guidelines have been recommended for maximum speed reductions from tangents to horizontal curves and for maximum differentials between design and operating speeds on horizontal curves. Lamm et al. suggest that the ranges shown in table 57 represent good, fair, and poor design with respect to the differences among the 85th percentile operating speeds on successive alignment features.[15]

As a first test of the speed-reduction criterion proposed as a measure of design consistency, 5,287 individual horizontal curves from the roadway sections in Washington were classified as good, fair, and poor with respect to design safety using the criteria in table 57. This included all horizontal curves for which the speed reduction for both directions of travel fell in the same design safety level as defined by Lamm et al.[(15)] Table 58 presents a summary of the accident frequencies, exposure, vehicle-kilometers of travel, and accident rates for these 5,287 curves.

The average accident rate is highest for the horizontal curves in the poor category and lowest for the horizontal curves in the good category. These results provide a first indication that horizontal curves that require motorists to make greater speed reductions from the approach tangent are likely to have higher accident rates than horizontal curves requiring lower speed reductions.

Table 57. Design Safety Levels Proposed by Lamm et al. [(15)]

Design Safety Level		
Good	**Fair**	**Poor**
$^{a}V_{85}$ # 10 km/h	10 km/h < $^{a}V_{85}$ # 20 km/h	$^{a}V_{85}$ > 20 km/h

$^{a}V_{85}$ = difference in 85th percentile speed between successive geometric elements (km/h).

Table 58. Accident Rates at Horizontal Curves by Design Safety Level.

Design Safety Level	Number of Horizontal Curves	3-Year Accident Frequency	Exposure (million veh-km)	Accident Rate (accidents/ million veh-km)
Good	4,518	1,483	3,206.06	0.46
Fair	622	217	150.46	1.44
Poor	147	47	17.05	2.76
Combined	5,287	1,747	3,373.57	0.52

Explicit Relationship Between Speed Reduction and Accidents at Horizontal Curves

The initial analysis showed that the quality of design of a horizontal curve does appear to be related to safety. Therefore, an explicit quantitative relationship between speed reduction and safety was developed to test its appropriateness as a design-consistency measure.

The 5,287 curves experienced a total of 1,747 accidents over the 3-year period from 1993 to 1995, with a mean of 0.11 accidents per curve per year. However, a large proportion of the curves

(93.2 percent) experienced one or no accident during the 3-year period. The distribution of these accidents is shown in table 59 and figure 73.

Table 59. Accident-Frequency Distribution for Horizontal Curves.

Number of Accidents in 3 Years	Number of Curves	Percentage of Curves
0	4,152	78.53
1	775	14.66
2	215	4.07
3	84	1.59
4	35	0.66
5	13	0.25
6	10	0.19
7	2	0.04
11	1	0.02

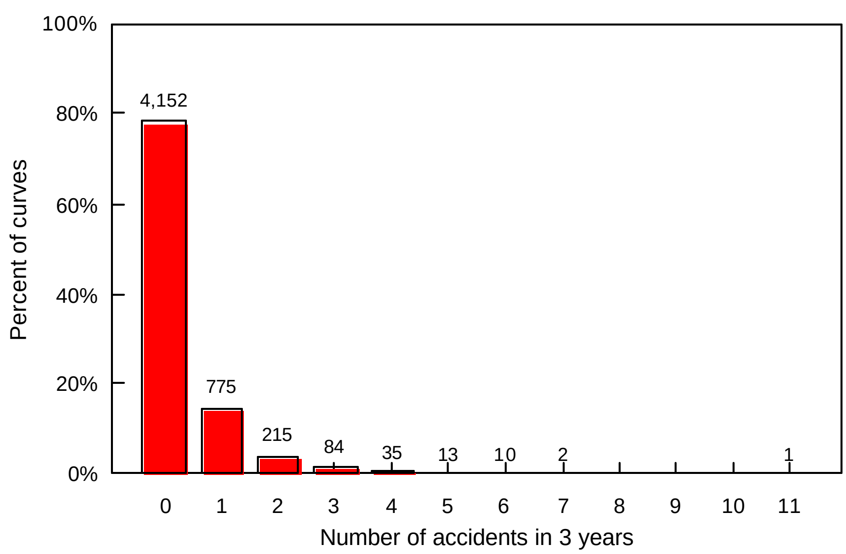


Figure 73. Accident-Frequency Distribution at Horizontal Curves.

Using loglinear regression models, the relationship between accident frequencies and exposure, curve geometries, and speed-reduction variables was investigated. Based on the shape of the accident distribution, it was decided to use Poisson and negative binomial regression models. This approach has been successfully used previously.[(71)] The Poisson distribution is an appropriate choice since accident frequencies are: (1) integers, (2) relatively small numbers, and (3) necessarily non-negative. However, a basic assumption underlying the use of the Poisson distribution is that its mean and variance are equal. This assumption is not always the case. When this assumption is substantially violated (for example, if the distribution has a long tail), then the negative binomial (NB) distribution can provide an improvement over the Poisson distribution. This distribution is chosen whenever the data exhibit either extra variation (overdispersion) or too little variation (underdispersion) relative to that assumed by the Poisson distribution.

The functional relationship between accident frequencies and selected (independent) variables was investigated. The following variables were included in the Poisson models:

- AADT (veh/day).
- Horizontal curve length (km).
- Curve radius (m).
- Speed reduction (km/h).

Two approaches were used to treat exposure. First, annual average daily traffic (AADT) and curve length were both included in the model and their coefficients in the regression model were estimated separately. In this approach, the natural logarithm of AADT and curve length, rather than their untransformed values, were used in modeling. This same approach has been taken by other researchers where accident counts rather than accidents rates are modeled.

The second approach was to combine AADT and curve length into million vehicle-kilometers of travel, a common exposure measure for roadway sections. In this approach, the exposure measure was used in the analysis simply as a scale factor (i.e., its coefficient in the model was forced to be equal to 1.0).

Basic statistics for the independent variables included in the models are shown in table 60.

Table 60. Descriptive Statistics for 5,287 Horizontal Curves.

Parameter	Minimum	Mean	Median	Maximum
Number of accidents in 3 years	0	0.33	0	11
AADT (veh/day)	222	2,283	1,567	18,005
Horizontal curve length (km)	0.016	0.238	0.177	2.977
Exposure (million veh-km)	0.006	0.638	0.321	19.777
Horizontal curve radius (m)	19.5	860.8	582.6	15,250
Speed reduction (km/h)	0	3.91	0.3	32.4

The general functional form selected for this analysis was a multiplicative model relating the expected accident frequencies (Y) at horizontal curves and the independent variables, X_1,ÿ, X_n, as follows:

$$Y = \exp(\text{ß}_0) \exp(\text{ß}_1 X_1) \exp(\text{ß}_2 X_2)\ \text{ÿ}\ \exp(\text{ß}_n X_n) \qquad (34)$$

In the models evaluated here, X_1, X_2, and X_3 represent log(AADT), log(curve length), and speed reduction, respectively. Models of this type were fitted using the GENMOD procedure of the SAS statistical package.[(72)]

When modeling 3-year accident frequencies as a function of log(AADT), log(curve length), and speed reduction, the following statistically significant Poisson regression model was obtained:

$$Y = \exp(-7.1977)\ AADT^{0.9224}\ CL^{0.8419} \exp(0.0662\ SR) \qquad (35)$$

where:
- Y = number of accidents that occurred on the horizontal curve during a 3-year period
- AADT = annual average daily traffic volume (veh/day)
- CL = horizontal curve length (km)
- SR = speed reduction on horizontal curve from adjacent tangent or curve (km/h)

All three parameters were significant at the 95-percent confidence level. In fact, the significance levels associated with all three parameters were less than 0.0001. In order of importance of their contribution explaining the variability in the accident data (as measured by their chi-square value), the three parameters rank as follows: log(AADT) ($?^2$ = 863), log(CL) ($?^2$ = 638), and SR ($?^2$ = 200).

The overall fit of a Poisson (or NB) model can be assessed using the following goodness-of-fit criteria: the deviance (a measure of over- or underdispersion of the data, which under ideal conditions should be close to 1); the Pearson chi-square (another measure similar to the deviance, which should also be close to 1; generally, a value between 0.8 and 1.2 is an indication that the model can be assumed to be appropriate in modeling the data); the ordinary multiple correlation coefficient (R^2); and the Freeman-Tukey correlation coefficient (R^2_{FT}), each correlation with a maximum of 100 percent. A discussion of these goodness-of-fit parameters can be found in SAS documentation and in Fridstrom et al.[72-73]

The deviance for this Poisson model was 0.830 and its Pearson ?^{2} was 1.210. Although these two statistics deviate somewhat from 1.0, the Pearson ?^{2} is just outside the generally acceptable range of 0.8 to 1.2, an indication that the model can be considered appropriate in modeling the data. The model, however, could not be improved using the NB distribution and this same set of independent variables. The two correlation coefficients were R^2 = 19.5 percent and R^2_{FT} = 17.9 percent, indicating that a large proportion in the variation in accident data among the 5,287 curves is not accounted for by this Poisson model.

A second Poisson model in which exposure, as defined above, was used as a scale factor, rather than AADT and curve length as separate variables, provided the following significant model:

$$Y = \exp(-0.8571)\ MVKT\ \exp(0.0780\ SR) \tag{36}$$

where: MVKT = exposure (million veh-km of travel for a 3-year period)

The deviance for this Poisson model was 0.836 and its Pearson ?^{2} was 1.344; the two correlation coefficients were R^2 = 15.6 percent and R^2_{FT} = 16.8 percent, indicating that this model explains slightly less of the variation in accident data than the previous one.

While neither of the models presented in equations 35 and 36 explains a large proportion of the variation in accident data, this observation is often the case with regression relationships between geometric design elements and safety. A key finding is that both models show a strong relationship between the speed reduction on a horizontal curve and the accident frequency on that curve. In both relationships, the speed reduction effect is highly significant, since the significance level in both cases was less than 0.0001. The direction of both relationships indicates that the greater the speed reduction experienced by motorists on a horizontal curve, the greater its accident experience. Thus, speed reduction appears very promising as a design-consistency measure.

Table 61 presents a sensitivity analysis of the accident-prediction model in equation 36. The table shows the sensitivity of predicted accident experience to speed reduction on a horizontal curve. For the typical range of speed reduction shown in the table, the accident rate varies by a factor of 4.0. More specifically, for the middle 50 percent of the speed-reduction range (based on the 25th and 75th percentiles of speed reduction from the actual Washington alignment data), the accident rate varies by a factor of 1.5. These results indicate that speed reduction not only has a statistically significant relationship to accident frequency, but that predicted accident frequency varies with speed reduction in a manner that is realistic and is meaningful in engineering terms. Thus, the sensitivity analysis lends support to the use of speed reduction as a design-consistency measure.

Table 61. Sensitivity of Safety Measures for Individual Horizontal Curves to Speed Reduction.

AADT	Curve Length (km)	Speed Reduction (km/h)	No. of Accidents in 3 Years[a]	Accident Rate (per million veh-km)	Accident Rate (per km per year)
2,000	1	2	1.09	0.50	0.36
2,000	1	5	1.37	0.63	0.46
2,000	1	10	2.03	0.93	0.68
2,000	1	20	4.42	2.02	1.47
5,000	1	2	2.72	0.50	0.91
5,000	1	5	3.43	0.63	1.14
5,000	1	10	5.07	0.93	1.69
5,000	1	20	11.06	2.02	3.69
10,000	1	2	5.43	0.50	1.81
10,000	1	5	6.86	0.63	2.29
10,000	1	10	10.14	0.93	3.38
10,000	1	20	22.11	2.02	7.37

[a]Based on the model for Speed Reduction shown in equation (36).

Regression models analogous to equations (35) and (36) were developed using radius of curvature rather than speed reduction as an independent variable. Radius of curvature was found to be statistically significant in these models, but predicted accident experience was much less sensitive to radius of curvature than to speed reduction; for curve radius, accident rate changed by a factor of 1.2 to 1.3 as curve radius decreased from its 75th to 25th percentile values, as apposed to a factor of 1.5 for a similar change in speed reduction. When both speed reduction and radius of curvature were used as independent variables in the same models, speed reduction was found to have a stronger association with accident rate than curve radius (?^{2} = 190 vs. ?^{2} = 18). These findings indicate that there is potential value in improving the safety of proposed highway design, through use of a design-consistency procedure based on speed reduction, because speed reduction is a better predictor of accident experience than curve radius alone.

ANALYSIS OF ROADWAY ALIGNMENT INDICES AS DESIGN-CONSISTENCY CRITERIA

An alternative method for measuring the design consistency of a roadway alignment is with alignment indices (see chapter 4 and a companion report from this research project).[(8)] Alignment indices are quantitative measures of the general character of a roadway segment's alignment. When the general character of an alignment changes between segments of roadway, geometric inconsistencies rise. A common example is where the terrain transitions from level to rolling or mountainous, and the alignment correspondingly changes from gentle to more severe.

Proposed indicators of geometric inconsistency include the following:

- Large increase/decrease in the values of alignment indices for successive roadway segments.
- High rate of change in alignment indices over some length of roadway.
- Large difference between the individual feature and the average value of the alignment index.

Alignment indices have several advantages for use in design-consistency evaluations. First, they are easy for designers to use, understand, and explain. Second, alignment indices are a function of horizontal and/or vertical alignment elements. Therefore, they provide a mechanism for quantitatively comparing successive geometric elements from a system-wide perspective, which is the basis of design consistency. Third, alignment indices can attempt to quantify the interaction between the horizontal and vertical alignments, a design strategy currently missing from design policy. Under AASHTO design guidelines, horizontal and vertical alignments are usually designed separately to meet certain criteria and then brought together.

The alignment indices proposed as design-consistency measures for entire sections of roadway include: (1) average radius, (2) ratio of maximum radius to minimum radius, (3) average tangent length, and (4) average rate of vertical curvature.(8)

The following discussion addresses the relationship of each of the above alignment indices to the safety of a roadway section with a length of 6.4 to 32 km (4 to 20 m). A later section of this chapter addresses the relationship to safety of alignment indices calculated as ratios of an individual feature to the alignment index for a roadway section. Each of the alignment indices is defined below.

Average Radius

The average radius expresses the sharpness of the curves that motorists typically encounter on a given section of the roadway. A large average radius would indicate curves that are typically not very sharp. Therefore, it is expected that higher speeds would exist for these values compared to smaller average radius values. A small average radius indicates that the curves on a roadway section are quite sharp. The average radius for a roadway section is defined as:

where: AR = average radius (m)
R_i = radius of i^{th} curve on the roadway section (m)
n = number of curves within the section

Maximum Radius/Minimum Radius

The range of the radii along a roadway can be determined by computing the ratio of the maximum to minimum radii. The assumption behind this alignment index is that this ratio can represent the consistency of the design in terms of the use of similar horizontal radii along the road. As this value approaches 1 (i.e., as the consistency of the chosen design radii is increased), a reduced accident rate may be expected. Thus, a section composed of curves of approximately the same radii might be considered a consistent design, even if that curve radius was quite sharp, while an otherwise similar section with a broad range of curve radii might be considered an inconsistent design.

For each roadway section, the ratio of the maximum radius to the minimum radius of the roadway section is computed as:

$$RR = \frac{R_{max}}{R_{min}} \qquad (38)$$

where: RR = ratio of maximum radius to minimum radius
R_{max} = maximum radius for any curve on the roadway section (m)
R_{min} = minimum radius for any curve on the roadway section (m)

While this alignment index appears to be useful, there are three concerns associated with this measure. First, if the maximum radius is much different than all other radii, the curve with the maximum radius may be the inconsistency. Second, ratios have different implications for different situations. For example, a value of two for this index may be the ratio between either a 1,840-m and 920-m curve or a 320-m and 160-m curve. It is likely that motorists would react differently to each situation, thereby making it difficult to determine inconsistencies. A possible solution to this problem would be to use this ratio together with the average or minimum radius for the roadway section. Third, this alignment index

$$AR = \frac{\sum R_i}{n} \qquad (37)$$

does not indicate how many curves are being compared, over what length of roadway, and in what order they occur. A transition from large to small radii might be acceptable if made gradually over several curves, but the alignment index is not sensitive to this.

Average Tangent Length

The average tangent length indicates the length of tangent that is typically available to drivers between curved sections of the roadway. A large value for this index would indicate that the roadway has tangent sections that are typically long, therefore, motorist's speeds would be expected to be higher than for roadways with a smaller value. This value could indicate the possibility of safety problems on a roadway section if a tangent were so long as to become monotonous to the motorist or if a sharp curve were located at the end of the long tangent. On the other hand, a short average tangent length could imply that the roadway consists of a series of short curves and tangents, while a longer average tangent length may indicate a more generous design. Thus, the interpretation of average tangent length as an alignment index appears ambiguous.

The average tangent length is defined as:

$$AT = \frac{\sum TL_i}{n} \qquad (39)$$

where:

	AT	=	average tangent length (km)
	TL_i	=	length of the i^{th} tangent on the roadway section (km)
	n	=	number of tangents within section

Average Rate of Vertical Curvature

Recent vertical alignment research has investigated the relationships between stopping sight distance and speeds or accidents and between grades and speeds or accidents. These studies showed that grades and stopping sight distance have an effect on operating speeds. Thus, an alignment index incorporating vertical alignment seems logical.

The average rate of vertical curvature provides an indication of the amount of change in the vertical alignment of a roadway. The assumption behind this alignment index is that the amount of hilliness on an alignment can have an effect on speeds and accidents.

The average rate of vertical curvature is defined as:

$$AVC = \frac{\sum \frac{L_i}{|A|_i}}{n} \qquad (40)$$

where: AVC = average rate of vertical curvature (m/% grade)
L_i = length of the i^{th} vertical curve on the roadway section (m)
$|A|_i$ = absolute value of the algebraic difference in grade for the i^{th} vertical curve on the roadway section (%)
n = number of vertical curves within a section

Model Development

Table 62 presents descriptive statistics for the 291 roadway sections evaluated, including the 4 alignment indices defined above. The roadway sections include 4,931 km of two-lane rural roadways with an average section length of 17 km. A total of 5,824 accidents occurred on these roadway sections during the 3-year period, ranging from 0 to 42 accidents/year, with an average of 6.7 accidents/year.

Figure 74 illustrates the distribution of the accidents on the sections. Only 4.5 percent of the roadway sections (13 out of 291) experienced either zero or one accident during the 3-year period. With this small percentage of sections with low accident frequencies and with a mean accident count of 20, the assumption of a Poisson distribution for this distribution would be inappropriate. Figure 74 illustrates quite clearly that the shape of the accident distribution is like that of a lognormal distribution; i.e., accident frequencies follow a normal distribution on a logarithmic scale.

Accident frequencies were modeled as a function of exposure (AADT and section length, both on the logarithmic scale) and each of the four alignment indices—average radius, maximum over minimum radius ratio, average tangent length, and average vertical curvature rate—taken one at a time. The modeling was performed using a standard regression model applied to the log-transformed accident frequencies. Thus, the general relationship between accident frequencies and exposure and design-consistency criterion can be expressed using equation (34), except that the error term of that model has a lognormal rather than a Poisson distribution.

Table 62. Descriptive Statistics for Roadway Sections.

Parameter	No. of Sections	Minimum	Mean	Median	Maximum
Number of accidents/3 years	291	0	20	15	127
AADT (veh/day)	291	166	2,911	2,061	18,005
Section length (km)	291	6.44	16.95	16.87	32.07
Average radius (m)	282	185	1,099	871	8,591
Maximum radius/minimum radius	282	1.00	13.13	7.92	179.06
Average tangent length (km)	291	0.10	1.66	0.73	26.06
Average vertical curvature rate (m/% grade)	249	23.9	106.5	85.4	456.2

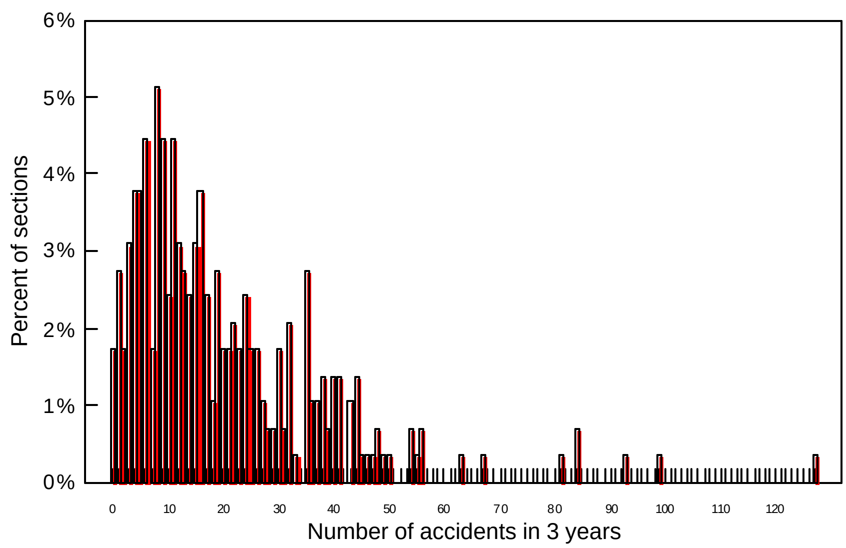


Figure 74. Accident-Frequency Distribution for Roadway Sections.

The results from the four regression models are summarized in table 63. For each of the four models, the table provides the sample size (i.e., number of sections available in the database for which all required data were available), the intercept and regression coefficients for the three independent variables in the model, the model R^2 (i.e., a measure of goodness of fit of the model), the partial R^2 (i.e., the R^2 based only on exposure), and the additional R^2 due to adding the alignment index to the model.

As shown in table 63, all of the models are statistically significant at the 95-percent confidence level. The R^2 values for these models are relatively high for accident-geometric relationships, ranging from 67 to 69 percent. While the R^2 values for these models are substantially higher than those for the models in equations (35) and (36), it should be recognized that the dependent variable being predicted (accident frequency for an extended roadway section) does not vary as much as the accident frequency for individual geometric features modeled in equations (35) and (36). Although the coefficient of each of the four alignment indices is statistically significant, the addition of that criterion to the model only slightly improved the fit of the model as shown by the additional R^2. These values ranged from 0.66 percent to 3.29 percent; thus, most of the variability is explained by AADT and section length, rather than by the alignment index. Nevertheless, all four alignment indices are statistically significant and the effects of three of the four alignment indices—average radius, maximum radius/minimum radius, and average vertical curvature rate—are clearly in the expected direction as indicated in the preceding discussion of the individual alignment indices. Thus, these findings support the use of these three alignment indices as design-consistency criteria.

Table 63. Lognormal Regression Results for Alignment Indices Applied to Entire Roadway Sections.

Parameter	**Coefficient**	**Significant at 95% Level?**
Model 1	**$Y = \exp(ß_0)\ AADT^{ß1}\ \text{Section length}^{ß2} \exp(ß_3\ AR)$**	**282 sections**
Intercept	-7.845	Yes
AADT (logscale)	0.995	Yes
Section length (km) (logscale)	1.108	Yes
Average radius (m)	-0.000137	Yes
Model R^2 (%)[a]	67.26	
Partial R^2 (%)[b]	65.86	
Additional R^2 (%)[c]	1.40	
Model 2	**$Y = \exp(ß_0)\ AADT^{ß1}\ \text{Section length}^{ß2} \exp(ß_3\ RR)$**	**282 sections**
Intercept	-7.859	Yes
AADT (logscale)	0.988	Yes
Section length (km) (logscale)	1.058	Yes
Maximum radius/minimum radius	0.0043	Yes
Model R^2 (%)[a]	66.51	
Partial R^2 (%)[b]	65.86	
Additional R^2 (%)[c]	0.66	

Model 3	$Y = \exp(\beta_0)$ AADT$^{\beta 1}$ Section length$^{\beta 2}$ $\exp(\beta_3$ AT)	291 sections
Intercept	-7.725	Yes
AADT (logscale)	0.978	Yes
Section length (km) (logscale)	1.082	Yes
Average tangent length (km)	-0.049	Yes
Model R^2 (%)[a]	68.46	
Partial R^2 (%)[b]	66.46	
Additional R^2 (%)[c]	2.00	

Table 63. Lognormal Regression Results for Alignment Indices Applied to Entire Roadway Sections (continued).

Model 4	$Y = \exp(\beta_0)$ AADT$^{\beta 1}$ Section length$^{\beta 2}$ $\exp(\beta_3$ AVC)	249 sections
Intercept	-8.297	Yes
AADT (logscale)	1.052	Yes
Section length (km) (logscale)	1.167	Yes
Average vertical curvature rate	-0.0028	Yes
Model R^2 (%)[a]	68.80	
Partial R^2 (%)[b]	65.51	
Additional R^2 (%)[c]	3.29	

[a] Based on full model.
[b] Based on exposure variables only.
[c] Increase from adding design-consistency criterion.

The fourth alignment index—average tangent length—was found to have a negative correlation to accidents, indicating that accidents decrease as the average tangent length increases. As was mentioned above, the interpretation of average tangent length as a design-consistency measure is ambiguous. It is not clear whether the negative correlation of average tangent length to accidents is in the appropriate direction or not. Therefore, a decision was reached not to pursue average tangent length as a design-consistency measure. However, a further analysis presented below addresses the ratio between individual tangent lengths and the average tangent length as a possible design-consistency measure.

Sensitivity analyses were conducted to examine the sensitivity of predicted accident experience to three of the alignment indices-average radius, ratio of maximum radius to minimum radius, and average rate of vertical curvature.

Table 64 illustrates the sensitivity of predicted accident experience to the average radius of curvature of the Washington roadway sections. Over a typical range of average horizontal curve radii, the predicted accident rate varies by a factor of only 1.5, compared to a factor of 4.0 for speed reduction. Similarly, for the range from the 25th to the 75th percentile of average radius, predicted accident rate varies by a factor of only 1.2, compared to 1.5 for the speed-reduction model. This finding suggests that average radius is not as good a predictor of accidents as speed reduction and, therefore, is not as strong a candidate design-consistency measure as speed reduction.

Table 64. Sensitivity of Safety Measures for Entire Roadway Sections to Average Radius of Curvature.

AADT	Section Length (km)	Average Radius (m)	No. of Accidents in 3 Years[a]	Accident Rate (per million veh-km)	Accident Rate (per km per year)
2,000	1	200	0.73	0.34	0.24
2,000	1	800	0.68	0.31	0.23
2,000	1	1,400	0.62	0.28	0.21
2,000	1	3,000	0.50	0.23	0.17
5,000	1	200	1.83	0.33	0.61
5,000	1	800	1.68	0.31	0.56
5,000	1	1,400	1.55	0.28	0.52
5,000	1	3,000	1.24	0.23	0.41
10,000	1	200	3.64	0.33	1.21
10,000	1	800	3.35	0.31	1.12
10,000	1	1,400	3.09	0.28	1.03
10,000	1	3,000	2.48	0.23	0.83

[a] Based on the model for Average Radius (AR) shown in table 63.

Table 64 illustrates the sensitivity of predicted accident experience to the ratio of maximum to minimum radius on a roadway section. For a typical range of ratios, as the ratio of maximum to minimum radius increases, the accident rate increases by a factor of 1.1. Similarly, for the range from the 25th to 75th percentile of ratio of maximum to minimum radius, the predicted accident rate increases by a factor of only 1.03, compared to 1.50 for the speed-reduction model. These results indicate that although the ratio of maximum to minimum radius has a statistically significant relationship to accident experience, accident rate is not very sensitive to the value of the ratio. Thus, it appears that the ratio of maximum to minimum radius is a very poor candidate design-consistency measure.

Table 65 illustrates the sensitivity of accidents to the average rate of vertical curvature. For a typical range of average rate of vertical curvature (AVC), as AVC decreases (i.e., as crest and sag vertical curves become sharper), the accident rate increases by a factor of 1.7. Similarly, for the range from the 25th to the 75th percentile of AVC, the accident rate increases by a factor of 1.3, compared to 1.5 for the speed-reduction model. Thus, accident rate is less sensitive to AVC than to speed reduction and, therefore, AVC may not be as strong a candidate design-consistency measure.

Table 65. Sensitivity of Safety Measures for Entire Roadway Sections to Ratio of Maximum to Minimum Radius of Curvature.

AADT	Section Length (km)	Ratio of Maximum to Minimum Radius	No. of Accidents in 3 Years[a]	Accident Rate (per million veh-km)	Accident Rate (per km per year)
2,000	1	4	0.72	0.33	0.24
2,000	1	8	0.73	0.33	0.24
2,000	1	15	0.75	0.34	0.25
2,000	1	30	0.80	0.37	0.27
5,000	1	4	1.77	0.32	0.59
5,000	1	8	1.80	0.33	0.60
5,000	1	15	1.86	0.34	0.62
5,000	1	30	1.98	0.36	0.66
10,000	1	4	3.52	0.32	1.17
10,000	1	8	3.58	0.33	1.19
10,000	1	15	3.69	0.34	1.23
10,000	1	30	3.93	0.36	1.31

[a] Based on the model for ratio of maximum radius to minimum radius (RR) shown in table 63.

Table 66. Sensitivity of Safety Measures for Entire Roadway Sections to Average Rate of Vertical Curvature.

AADT	Section Length (km)	Average Rate of Vertical Clearance (m/% grade)	No. of Accidents in 3 Years[a]	Accident Rate (per million veh-km)	Accident Rate (per km per year)
2,000	1	50	0.64	0.29	0.21
2,000	1	100	0.56	0.26	0.19
2,000	1	150	0.49	0.22	0.16
2,000	1	250	0.37	0.17	0.12
5,000	1	50	1.69	0.31	0.56
5,000	1	100	1.47	0.27	0.49
5,000	1	150	1.28	0.23	0.43
5,000	1	250	0.96	0.18	0.32
10,000	1	50	3.50	0.32	1.17
10,000	1	100	3.04	0.28	1.01
10,000	1	150	2.64	0.24	0.88
10,000	1	250	2.00	0.18	0.67

[a] Based on the model of average rate of vertical curvature (AVC) in table 62.

ANALYSIS OF RATIOS BETWEEN INDIVIDUAL DESIGN ELEMENTS AND ALIGNMENT INDICES

The alignment indices considered above characterize the quality of the alignment for entire roadway sections. Another use for alignment indices as design-consistency measures is to characterize the relationship of individual geometric features to the alignment index for the roadway section as a whole. Two design-consistency measures proposed for application in this way are:

- Ratio of an individual curve radius to the average radius for the roadway section as a whole.
- Ratio of an individual tangent length to the average tangent length for the roadway section as a whole.

Each of these measures is discussed below.

Ratio of Curve Radius to Average Radius

The assumption behind this alignment index is that when the radius of a given horizontal curve deviates greatly from the average radius along the roadway section, that curve may violate driver expectancy, create a geometric inconsistency, and experience high accident rates. Thus, the radii of individual curves were compared with the average radius of a roadway segment in determining which curve appears to be in conflict with the general character of the horizontal curves of the alignment.

For each horizontal curve, the ratio of the individual curve radius to the average radius of the roadway section was computed and compared to the accident experience at that curve:

$$CRR_i = \frac{R_i}{AR} \qquad (41)$$

where: CRR_i = ratio of radius for an individual horizontal curve to the average radius for the roadway section.

Ratio of Tangent Length to Average Tangent Length

Tangent length influences the necessary speed reduction of motorists as they enter a horizontal curve. The speed motorists reach on a tangent is largely dependent on the length of the tangent. If a tangent is long enough and no conflicting traffic is present, then motorists will drive at their desired speed, which is defined as the speed at which drivers choose to travel under free-flow conditions when they are not constrained by alignment features.[20] Therefore, a common geometric design inconsistency is a long tangent followed by a sharp curve where motorists are driving at a high speed on the tangent and are unable to decrease their speed as needed for the curve.

The assumption behind using this alignment index is that the ratio of tangent length to average tangent length will alert the designer to those tangents that may be much longer than: (1) what drivers expect to find on that roadway section and (2) are appropriate for the types of curves found on that roadway section.

$$RTL_i \; ' \; \frac{(TL)_i}{AT} \tag{42}$$

where: RTL_i = ratio of individual tangent length to average tangent length for the roadway section.

Model Development

The relationship between accident frequencies and exposure and the two design criteria—curve radius/average curve radius at curves and tangent length/average section tangent length—were investigated. The distribution of the 1,747 accident/3 years at horizontal curves is shown in figure 73. The distribution of the 4,074 accidents/3 years on tangents is illustrated in figure 74. In figure 74, as in figure 73, the distribution of accident frequencies appears to follow a Poisson distribution. Of the 5,220 tangent segments, 67 percent experienced no accidents and an additional 17.5 percent experienced only 1 accident in 3 years. An average of 0.78 accidents/3 years occurred on tangents (total length of 3,678 km) compared to an average of 0.33 accidents/3 years at curves (total length of 1,232 km). Table 67 provides basic descriptive statistics of curve- and tangent-related variables. The Poisson models developed for curves and tangents are discussed next.

Horizontal Curves

All three parameters included in the model for horizontal curves were significant at the 95-percent confidence level. In fact, the significance levels associated with all three parameters were less than 0.0001. In order of importance of their contribution in explaining the variability in the data (as measured by their chi-square value), the three parameters ranked as follows: log(AADT) ($?^2$ = 739), log(curve length) ($?^2$ = 604), and ratio of curve radius to average section curve radius ($?^2$ = 104). The deviance for this Poisson model was 0.837 and its Pearson $?^2$ was 1.296. Although these two statistics deviate somewhat from 1, the Pearson $?^2$ is just outside the generally acceptable range of 0.8 to 1.2, an indication that the model can be considered appropriate in modeling the data. However, the model could not be improved considerably using a negative binomial distribution and this set of independent variables. The two correlation coefficients were R^2 = 19.6 percent (the ordinary multiple correlation coefficient) and R^2_{FT} = 17.8 percent (the Freeman-Tukey correlation coefficient), indicating that a large proportion in the accident data (at the 5,302 curves) is not accounted for by the model. The Poisson model is shown below:

$$Y = \exp(-5.932)\ AADT^{0.8265}\ CL^{0.7727}\ \exp(-0.3873\ CRR) \qquad (43)$$

Equation (43) indicates that the relationship to safety of the ratio of individual curve radius to average curve radius is in the direction expected; as the radius of an individual curve increases, its expected accident experience decreases.

Table 68 illustrates the sensitivity of predicted accident experience to the ratio of the radius of and individual horizontal curve for a roadway section as a whole (CRR). For the typical range of CRR shown in the table, the accident rate varies by a factor of 1.8. Similarly, for the range of CCR from the 25^{th} to the 75^{th} percentile of values in the Washington alignment data, accident rate varies by a factor of 1.3, as compared to a factor of 1.5 for speed reduction. While accident experience is not as sensitive to CCR as to speed reduction, CCR is more sensitive to accident experience than either average radius alone or the ratio of maximum radius to minimum radius for a roadway section.

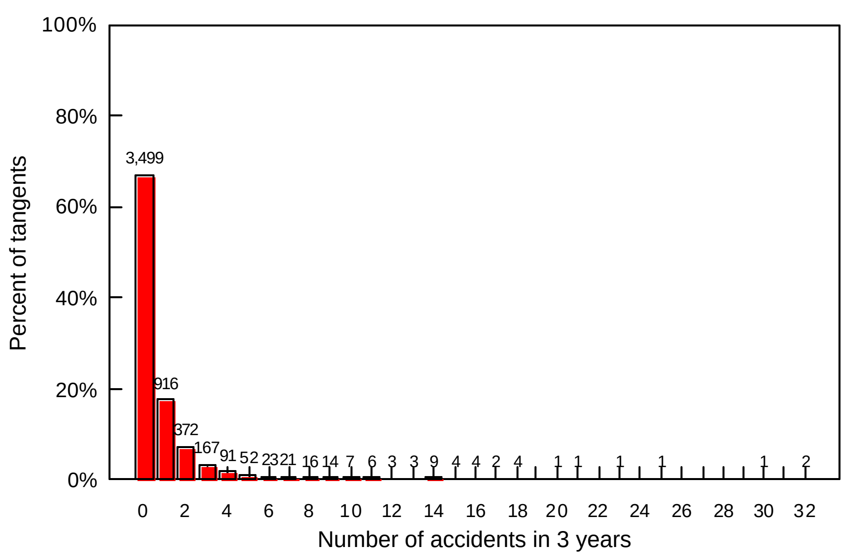


Figure 75. Accident-Frequency Distribution on Tangents.

Table 67. Descriptive Statistics for Individual Curves and Tangents.

Parameter	No. of Features	Minimum	Mean	Median	Maximum
Curves (total of 1,232 km)					
Number of accidents/3 years	5,302	0	0.33	0	11
Curve length (km)	5,302	0.016	0.23	0.18	2.98
AADT (veh/day)	5,302	222	2,280	1,567	18,005
Curve radius/average radius	5,302	0.025	1.00	0.77	10.89
Tangents (total of 3,678 km)					
Number of accidents/3 years	5,220	0	0.78	0	32
Tangent length (km)	5,220	0.016	0.70	0.32	26.06
AADT (veh/day)	5,220	166	2,294	1,567	18,005
Tangent length/average tangent length	5,220	0.002	1.00	0.65	13.99

Table 68. Sensitivity of Safety Measures for Individual Horizontal Curves to the Ratio of the Curve Radius to the Average Radius for the Roadway Section.

AADT	Curve Length (km)	Ratio of Curve Radius to Ave. Radius	No. of Accidents in 3 Years [a]	Accident Rate (per million veh-km)	Accident Rate (per km per year)
2,000	1	0.5	1.17	0.53	0.39
2,000	1	1.2	0.89	0.41	0.30
2,000	1	1.5	0.79	0.36	0.26
2,000	1	2.0	0.65	0.30	0.22
5,000	1	0.5	2.49	0.46	0.83
5,000	1	1.0	2.05	0.38	0.68
5,000	1	1.5	1.69	0.31	0.56
5,000	1	2.0	1.39	0.25	0.46
10,000	1	0.5	4.42	0.40	1.47
10,000	1	1.0	3.64	0.33	1.21
10,000	1	1.5	3.00	0.27	1.00
10,000	1	2.0	2.47	0.23	0.82

[a] Based on the model for Curvature Change Rate (CCR) shown in equation (43).

Tangents

Both parameters included in the model for tangents—AADT and ratio of tangent length (RTL)—were statistically significant at the 95-percent confidence level. Indeed, the significance level for both parameters was less than 0.0001. In order of importance of their contribution in explaining the variability in the data (as measured by their chi-square values), the parameters ranked as follows: log(AADT) ($?^2$ = 2,997) and RTL ($?^2$ = 1,567). The deviance for this Poisson model was 1.538 and its Pearson $?^2$ was 2.413. These goodness-of-fit measures are not satisfactory; other modeling approaches might improve this model. The two correlation coefficients were R^2 = 11.0 percent (the ordinary multiple correlation coefficient) and R^2_{FT} = 26.6 percent (the Freeman-Tukey correlation coefficient), indicating that only a limited proportion of the variance in the accident data (at the 5,220 tangents) is accounted for by the two independent variables. The Poisson model is shown below:

$$Y = \exp(-9.0706)\ AADT^{1.0786}\ \exp(0.3438\ RTL) \qquad (44)$$

While the model in equation (44) would not be very satisfactory as a predictive tool, it does indicate that the ratio of individual tangent length to average tangent length has a statistically significant relationship to accident frequency. This relationship indicates that accident frequency increases as an individual tangent becomes longer relative to the average tangent length. Although RTL is statistically significant in the model in equation (44), RTL is not recommended as a design-consistency measure because of the uncertainty in the interpretation of the appropriate direction of its relationship to accident frequency.

CONCLUSIONS

The statistical models developed in this chapter are not intended for use as accident predictive models, but rather are intended to illustrate the nature of the relationship of candidate design-consistency measures to safety. Of the candidate design-consistency measures, four have relationships to accident frequency that are statistically significant and appear to be sensitive enough that they may be potentially useful in a design-consistency methodology. These four candidate design-consistency measures are:

- Predicted speed reduction by motorists on a horizontal curve relative to the preceding curve or tangent.
- Ratio of an individual curve radius to the average radius for the roadway section as a whole.
- Average rate of vertical curvature on a roadway section.
- Average radius of curvature on a roadway section.

Thus, these measures appear promising for assessing the design consistency of roadway alignments.

Of these candidate design-consistency measures, the speed reduction on a horizontal curve relative to the preceding curve or tangent clearly has the strongest and most sensitive relationship to accident frequency. The evaluation has shown that the speed reduction on a horizontal curve is a better predictor of accident frequency than the radius of that curve. This makes a strong case that a design-consistency methodology based on speed reduction provides a better method for anticipating and improving the potential safety performance of a proposed alignment alternative than a review of horizontal curve radii alone.

Accident frequency is not as sensitive to the alignment indices reviewed as it is to the speed reduction for individual horizontal curves. The alignment index that has the greatest sensitivity to accident frequency is the ratio of an individual curve radius to the average radius for the roadway section as a whole. While this alignment index is potentially useful as a design-consistency measure, accident frequency is not as sensitive to it as to the speed reduction for an individual curve. In other words, it appears to be more useful to compare the speed on horizontal curves to the speed on its immediately adjacent geometric features than to compare its radius to the average radius for the roadway section as a whole.

The average rate of vertical curvature for a roadway section and the average radius of horizontal curves for a roadway section also show promise as design-consistency measures. However, neither of these measures appears to be as appropriate a measure of design consistency as the speed reduction for individual horizontal curves.

The remaining alignment indices evaluated do not appear to be appropriate design-consistency measures because of their weak or ambiguous relationships to accident frequency. Thus, the following alignment indices are not recommended as design-consistency measures:

- Ratio of maximum radius of curvature to minimum radius of curvature for all horizontal curves on a roadway section.

- Average tangent length for a roadway section.

- Ratio of an individual tangent length to the average tangent length for the roadway section as a whole.

10. SUMMARY, FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

Design consistency refers to the conformance of a highway's geometry with driver expectancy. Drivers make fewer errors in the vicinity of geometric features that conform with their expectations. A technique to evaluate the consistency of a design is to evaluate changes in operating speeds along an alignment. If the speed is relatively constant for an alignment with one or more exceptions, such as at a horizontal curve, then that feature should be evaluated and perhaps redesigned. To use operating speed as a consistency tool requires the ability to accurately predict speeds as a function of the roadway geometry. In this research project, several different efforts were undertaken to predict operating speeds for different conditions, such as on horizontal and vertical curves, on tangent sections, and prior to or after a horizontal curve. Speed data were collected at more than 211 two-lane rural highway sites for use in the project (176 simple circular curves, vertical curves, and their approach tangents; 12 spiraled circular curves and their approach tangents; and 21 simple circular curve sites where detailed acceleration/deceleration measurements were obtained).

The data were used to develop speed-prediction equations for horizontal and vertical alignments, to analyze the effects of spiral curves, and to evaluate whether speed is different at the study sites for different vehicle types. Regression equations were developed for passenger cars for most combinations of horizontal and vertical alignment. For some of the combinations, however, sample sizes were not large enough for the equations to be considered definitive. The analysis of spiral curves found that the use of a spiral did not result in a significant difference in speed when compared to similar sites that did not have spirals. Because most of the study sites had few spot-speed observations for trucks and recreational vehicles, a limited analysis was performed for those sites with a minimum of 10 observations. The limited graphical analysis on trucks and recreational vehicles found similar speed trends between the different vehicle types. Therefore, a design-consistency evaluation should use the equations based on passenger car data.

The collected data were also used to determine whether alignment indices can accurately predict speeds on a tangent. In addition, other possible influences of motorists' desired speeds were examined. The findings of this research indicated that combinations of alignment indices and other geometric variables were not able to significantly predict the 85th percentile speeds of motorists on long tangents of two-lane rural highways. For those situations when a vehicle is traveling on an upgrade or downgrade, the equations used in the TWOPAS model can be used to estimate speeds for different vehicle types. The models can be used to predict the speed of the vehicle at any point on the grade if its initial speed at the entry to the grade is known.

Data were collected and analyzed at 21 sites to determine appropriate acceleration and deceleration values prior to and after a horizontal curve. The acceleration and deceleration assumptions and rates involved in the formulation of the original model presented in FHWA-RD-94-034 were examined. New acceleration/deceleration models were constructed from the data to consider the effects of the curve radius.

The speed-prediction models developed in a previous FHWA project and in this project were validated. The objectives of the validation efforts were to evaluate the assumptions and validate the accuracy with which the previous and newly developed speed-profile equations accurately predict speeds as a function of highway geometry; and as necessary, refine, reformulate, and recalibrate the models. The subtasks involved in the validation efforts include performing internal and external validations to compare equation-estimated and observed values.

Based on the results from the validation efforts and the desire to use as many sites as were available, new speed-prediction equations were developed using all data collected in this study. These equations, along with the findings from other tasks, were incorporated into a speed- profile model. The model can be used to evaluate the design consistency of the roadway or can be used to develop a speed profile for an alignment. The model considers both horizontal and vertical curvature and the acceleration or deceleration behavior as a vehicle moves from one feature to another.

Both predicted speed on a facility and alignment indices have been suggested as geometric design-consistency measures. This study included a demonstration of the relationship between these measures and safety. Of the candidate design-consistency measures, the predicted-speed reduction on a horizontal curve relative to the preceding curve or tangent clearly has the strongest and most sensitive relationship to accident frequency.

FINDINGS

The findings from the speed-prediction efforts are subdivided into speeds on curves, alignment indices, TWOPAS, acceleration and deceleration rates, validation of speed-prediction equations, speed-profile model, and relationship of design-consistency measures to safety. The following are the list of findings within each of these categories.

Speeds on Curves

Horizontal Curves on Grades

- For passenger vehicles, radius was the only significant independent variable in predicting 85th percentile speeds for all alignment combinations that included a horizontal curve on grade. The best form of the independent variable in regression equations is 1/R.

- Operating speeds on horizontal curves are very similar to speeds on long tangents when the radius is greater than or equal to approximately 800 m. Under this condition, the grade of the section may control the selection of speeds, and the contribution of the horizontal radius is negligible.
- Operating speeds on horizontal curves drop sharply when the radius is less than 250 m.

Vertical Curves on Horizontal Tangents

- Passenger vehicle speeds on limited sight-distance vertical curves on horizontal tangents could be predicted using the rate of vertical curvature as the independent variable. The best form of the independent variable in regression equations is 1/K.
- A statistically significant regression equation could not be found for crest curves where the sight distance is not limited; therefore, the desired speed for long tangents is assumed.
- For sag curves on horizontal tangents, the plot of the seven available data points and the regression analysis indicate that the desired speed on long tangents should be assumed.

Horizontal Curves Combined With Vertical Curves

- For non-limited sight-distance crest vertical curves in combination with horizontal curves, the lowest speed of the speeds predicted using the equations developed for horizontal curves on grades or the assumed desired speed should be used. The collected speed data for that condition were generally for large horizontal curve radii with several of the speeds being above 100 km/h. Drivers may not have felt the need to reduce their speed in response to the geometry for these large radius horizontal curves.
- For the horizontal curvature combined with either sag or limited sight-distance crest vertical curves, the radius of the horizontal curve was the best predictor of speed.

Spirals

- There is no difference in 85^{th} percentile speeds at the midpoint on circular curves from those with spiral transitions.

Other Vehicle Types

- The data for all truck types and recreational vehicles on horizontal curves display a general speed behavior that is similar to that of passenger vehicles.
- The influence of grade on trucks and recreational vehicles is similar to, but larger than, the effects observed for passenger vehicles.

Alignment Indices

- None of the alignment indices studied were statistically significant predictors of the desired speeds of motorists on long tangents of two-lane rural highways.
- There were significant regional differences in the desired speeds of motorists on long tangents of two-lane rural highways.
- Of the geometric variables examined, only the vertical grade at the tangent site significantly affected the desired speeds of motorists on long tangents of two-lane rural highways.

TWOPAS Vehicle-Performance Equations

- The TWOPAS vehicle-performance equations can be used to determine the speed for specific vehicle types at any point on a grade.

Acceleration and Deceleration Rates for a Horizontal Curve

- The validation results indicate that the acceleration and deceleration assumptions employed in the previous speed-profile model are not valid for the set of study sites selected in this study. The speed values predicted and the speed values measured in the field are statistically different at the 95-percent confidence level.
- The only sites with acceleration and deceleration rates that approached 0.85 m/s^2 were those with curve radii less than 250 m.
- New models were developed to consider the effect of curve radii on the acceleration/deceleration rates. The models were based on maximum acceleration and deceleration rates observed at the study sites.

Validation of the Speed-Prediction Equations

- The validation of the six speed-prediction equations for horizontal and vertical curves (see table 14) was performed by comparing the equation predicted V_{85} to the field observed V_{85} at the midpoint of the curve. The overall mean absolute percent error for the six equations was 5.7 percent.
- Validation was also conducted for the predicted speed change between the midpoint of the approach tangent and the midpoint of the horizontal curve. In general, the models were found to differ from the observed change in speed between the tangent and the horizontal curve by an average of 98 percent.

Speed-Profile Model

- A speed-profile model was developed that can be used to evaluate the design consistency of a facility or to generate a speed profile along an alignment. The design-consistency evaluation consists of identifying undesirable speed changes between features. The speed-prediction equations are used to predict the speeds for the features, and then the differences in speed between successive features would be calculated. To generate a speed profile, the lowest anticipated speed would be obtained by selecting the lowest speed predicted by the selected desired speed on tangent, the speed-prediction equations, and/or the TWOPAS model. This speed would then be adjusted prior to and departing from curves using the acceleration and deceleration values determined in this project.

- The speed-profile model developed in the research appears to provide a suitable basis for the IHSDM design-consistency module.

Relationship of Design-Consistency Measures to Safety

- Of the candidate design-consistency measures, four have relationships to accident frequency that are statistically significant and appear to be sensitive enough that they may be potentially useful in a design-consistency methodology. These four candidate design-consistency measures are:
  - Predicted speed reduction by motorists on a horizontal curve relative to the preceding curve or tangent.
  - Ratio of an individual curve radius to the average radius for the roadway section as a whole.
  - Average rate of vertical curvature on a roadway section.
  - Average radius of curvature on a roadway section.

- Of these candidate design-consistency measures, the speed reduction on a horizontal curve relative to the preceding curve or tangent clearly has the strongest and most sensitive relationship to accident frequency.

CONCLUSIONS

The following general conclusions were developed based on the findings of the study:

- This research produced speed-prediction equations that can be used to calculate the expected speed along an alignment that includes both horizontal and vertical curvature.

- The alignment indices did not explain the variation in measured speeds on long tangents.

- The average 85th percentile speed per State for long tangents ranged from 93 to 104 km/h. Based on the data and engineering judgment, the operating speeds on tangents and the maximum operating speeds on horizontal curves could be rounded to 100 km/h.

- After establishing that current acceleration and deceleration rates and assumptions are not valid, at least for the sites selected in this study, new models were developed that predict acceleration and deceleration rates in the vicinity of a horizontal curve as a function of curve radius. Models were not developed for vertical curves or for horizontal/vertical combined curves.

- The six speed-prediction equations presented in chapter 3 performed well in the validation effort with a range of mean absolute percent error between 4.1 percent and 10 percent. The predicted speed difference between the midpoint of the tangent and the horizontal curve (calculated using the deceleration findings from chapter 6 and the speed-prediction equations presented in chapter 3) did not compare well with the observed speed reduction. The range of mean absolute percent error was 87 percent to 107 percent for the four conditions studied.

- The various findings from this study were used to develop a speed-profile model. This model can be used to evaluate the design consistency of a facility or to generate a speed profile along an alignment.

- Of the different alternatives examined, a design-consistency methodology based on predicted speed reductions was the best identified.

- The IHSDM should contain a design-consistency module based on the speed-profile model developed in this research.

RECOMMENDATIONS

The following recommendations (listed in order of relative priority) are made based on the findings and conclusions of this study:

- Additional insight into the influences of speeds on tangent sections of various lengths and grades is needed. This would greatly enhance the effectiveness of any speed-profile model because it would validate the assumptions currently being made.

- The acceleration and deceleration models developed here were exclusively related to the impact of the horizontal curve. It is recommended that a similar effort be undertaken to assess the impact of vertical curves, as well as horizontal-vertical combinations on acceleration and deceleration profiles.

- Further research should be conducted to extend all aspects of this research, such as speed-prediction equations, acceleration/deceleration behavior, and the design-consistency module speed-profile model, to roadway types other than two-lane rural highways.

- Further refinements should be made to the IHSDM design-consistency module in future research to include a capability of identifying design inconsistencies based on factors other than horizontal and vertical alignment. Such factors might include intersections, driveways, and auxiliary lanes.

- Further research should be conducted on estimating operating speeds of trucks and recreational vehicles for different horizontal and vertical curves. Additional data are required to develop regression models to estimate operating speeds of trucks and RVs.

- Further research is needed on the ability of alignment indices to estimate desired speeds of motorists on long tangents of two-lane rural highways. Gaps in the existing database included roads with posted speeds greater than 88.5 km/h (55 mph) and alignment indices near the maximum values of those in this study.

- Because the safety evaluation demonstrated that predicted-speed reduction has the strongest relationship to accident frequency, speed reduction should be the primary measure in design-consistency methodology for horizontal and vertical curvature. To accomplish this, better methods to predict speeds need to be investigated. Alignment indices may be appropriate measures to supplement speed reduction in a design-consistency methodology, but they should not be considered as the primary measure.