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RECONCILING SPEED LIMITS WITH DESIGN SPEEDS

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Reconciling Speed Limits with Design Speeds

Introduction

The INDOT Design Manual recommends that a design speed is selected based on the functional classification, urban vs. rural environment, terrain, traffic volumes and project scope of work. The design speed should be equal or greater than the legal or anticipated speed limit. According to AASHTO, a design speed should be consistent with the speed a driver is likely to expect on the highway.

By using design speeds, highways are designed in a conservative manner to facilitate the safe motion of vehicles even in adverse but reasonable conditions. Consequently, the 85th percentile of actual free-flow speeds may exceed the design speed. This situation does not have to cause excessive hazard because the majority of drivers adequately perceive the risk.

Some Indiana road sections designed and built in the past do not meet the current design standards. INDOT makes a continuous effort to modernize these sections. Due to prohibitive costs, reduced design speeds and design exceptions are considered. Guidance is needed to help reduce the discrepancy between the economically justifiable design solutions and the design standards expected by the motorists. Predicting the 85th percentile speed on modernized sections would help designers in finding solutions that meet both the motorists' expectations and the current design standards to the possible extent. The objective of the research was the development of a tool for predicting the actual speeds on modernized two- and four-lane roads in Indiana.

Findings

The mean free-flow speed and its variability across drivers are considered important safety factors. The existing speed-predicting models combine the mean speed impacts with the speed dispersion impacts, which make identification of the speed factors and interpretation of the results difficult. Furthermore, the existing models are specialized to selected percentiles and are not able to estimate the entire range of the speed variability at a site. This report presents an advanced method of modeling free-flow speeds that overcomes the limitations of the existing models. This has been accomplished by representing the percentile speed as a linear combination of the mean and the standard deviation.

Free-flow speeds and highway geometry characteristics collected on two-lane rural highways and four-lane suburban and rural highways were used to develop the models. The crash experience on the studied highways was considered to eliminate segments where a high number of crashes indicated that the driver perception of the risk might be incorrect. The models demonstrated their efficiency in identifying relationships between speed and diverse road geometry characteristics, e.g. cross-section dimensions, horizontal curve elements, intersection and driveway densities and median type.

Implementation

The developed speed models were included in a prototype software tool to help highway designers implement the models. The tool generates a profile of the mean speed and any specified percentile speed for the entire project length based on the preliminary highway design values. The tool can be used to evaluate if the predicted speeds meet the desired speeds for the design project, to identify locations in the project with design inconsistencies and to evaluate possible modifications in the design values at any location of the highway project.

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16. Abstract

In recent years, context-sensitive highway design has been promoted to ensure that designers consider the environmental, scenic, aesthetic, historic, community, and preservation aspects of the road. Context-sensitive design may lead to situations where the current design standards cannot be met because of restricting local conditions. Indiana has road sections designed and built some time ago. In a considerable number of roads with the statutory limit of 55 mph (90 km/h), the road geometry does not meet the current standards. At individual intersections and on curves, advisory speeds are posted together with warning signs. Although this solution increases the safety of road users and allows for traveling at reasonably high speeds outside of these segments, the final solution is to upgrade their geometry to the desirable level.

This report presents models that predict user-selected percentile free-flow speeds on two-lane rural and four-lane rural and suburban highways. The percentile speeds are computed as a linear combination of the mean speed and the standard deviation in panel data models with random effects. The developed percentile speed models involve more design variables than typical speed models, and separate the mean speed factors from the speed dispersion factors. These benefits ease the model interpretation and its use in highway design. The study results should help designers bring the predicted speed to the desired speed as close as possible given the budget constraints. Engineering judgment can then be applied to balance safety and construction cost in highway improvement projects.

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The developed speed models were included in a visual basic-based prototype tool to help highway designers implement the models. The prototype tool, named the Highway Speed Prediction Model (HSPM), was developed as a stand-alone, ready-to-use Windows application, as requested by the SAC. The highway design values required to estimate speeds are manually type in by the user. The tool provides default values for most of the variables included in the speed models. The default values correspond to typical values used in highway design or unrestricted base highway conditions. The tool also suggests a range of values for most of the variables based on the field measurements. The tool generates a profile of the mean speed and any specified percentile speed for the entire project length. The user can print the speed profile and the tables with the input design values and add it to the project documents. The help section of the tool includes the user manual containing the instructions. The help section also includes the speed models and the definitions of the variables in the models.

The INDOT Scoping Section of the Environment, Planning and Engineering Division and the Design Division will implement the speed tool in two-lane rural and four-lane rural and suburban highway improvement projects. The tool can be integrated to the highway design process as part of the preliminary design stage. The designers can evaluate if the predicted speeds meet the desired speeds for the design project, identify locations in the project with speed changes that might indicate possible design inconsistencies and evaluate the effect in speeds of any modification in the preliminary design values at any location of the highway improvement project.

The INDOT Design Manual defines the operating speed as the highest overall speed at which a driver can safely travel while not exceeding the design speed. This definition can be modified to concur with the current AASHTO definition that recommends that the operating speed is the speed at which drivers are observed operating their vehicles during free-flow conditions. Typically, the 85th percentile of the free-flow speed distribution is used to represent the highway operating speed, although the use of other percentiles has been also proposed. The speed models included in the speed tool have the capability of predicting any free-flow percentile speed, from the 5th to the 95th percentile, in multiples of five. This attribute is very significant if the current operating speed policy is changed for a percentile other than the 85th percentile. In such circumstances, there will be no need to develop new speed models or to correct the speed tool.

The INDOT Design Manual recommends for new construction/reconstruction projects that the posted speed limit will typically be equal to the design speed used in design, if this does not exceed the legal limit; and that a traffic engineering study may be conducted for various reasons to assist in the determination of the posted speed limit.

By using design speeds, highways are designed in a conservative manner to facilitate the safe motion of vehicles even in adverse but reasonable conditions. The 85th percentile of observed free-flow speeds exceeded the design speed in most situations. The crash experience was added to eliminate cases where the drivers' perception might be incorrect, as represented by a considerably high crash rate for the entire highway segment. The estimated speeds from the developed models will concur with a satisfactory level of safety for modernized highway segments.

All the sites observed in four-lane highways and tangent segments in two-lane rural highways had 85th percentile speeds higher than the posted speed limit. In sites observed inside horizontal curves of two-lane rural highways, 33 percent of drivers, in average, operate at speeds higher than the posted speed limit and 70 percent of drivers operate at speeds higher than the advisory speed. In addition, all the sites observed on horizontal curves had 85th percentile speeds higher than the curve inferred design speed. The difference between the inferred design speeds and the 85th percentile speeds varied from 5.1 to 15.8 mph. The curves without advisory speeds had 85th percentile speeds that exceeded the inferred design speeds in a range of 8.3 mph to 11.4 mph. Following the 85th percentile rule and taking into account the considerable low crash rate in those segments, the posted speed limit may safely exceed the design speed.

The current design policy can be modified to allow setting the posted speed limit at a value higher than the design speed, but according to the 85th percentile speed. The crash experience may be an additional consideration. Engineering judgment can then be applied to balance safety and construction cost in highway improvement projects.

The INDOT Standards Section of the Contracts and Construction Division will adopt the research results to the format consistent with the other departmental policy documents. The adopted text will be added to the Indiana Design Manual – Part V, Road Design. An appropriate INDOT internal committee will facilitate the adoption process.

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CHAPTER 1. INTRODUCTION

1.1. Background

Section 2B.11 of the Manual on Uniform Traffic Control Devices (MUTCD, 2000), the standard for using speed limit signs, reads: "After an engineering study has been made in accordance with established traffic engineering practices, the Speed Limit (R2-1) sign shall display the limit established by law, ordinance, regulation, or as adopted by the authorized agency." The MUTCD also provides guidance that a posted speed limit should be the 85th percentile speed of free-flowing traffic, rounded up to the nearest 5 mph (10 km/h) multiple.

Section 40-3.01 of the Indiana Department of Transportation Design Manual (INDOT, 1994) lists factors to consider during engineering studies for setting speed limits:

- the 85th percentile speed;
- the design speed used during project design;
- road surface characteristics, shoulder condition, grade, alignment and sight distance;
- functional classification and type of area;
- type and density of roadside development;
- the accident experience during the previous twelve months; and
- parking practices and pedestrian activity.

Further the design manual explains "On new construction/reconstruction projects, the posted speed limit will typically be equal to the design speed used in design, if this does not exceed the legal limit. A traffic engineering study may be conducted for various reasons to assist in the determination of the posted speed limit."

Section 40-3.02(01) of the INDOT Design Manual recommends that the selection of a design speed is based on the functional classification, the urban vs. rural environment, the terrain, the traffic volume and the project scope of work. Section 40-3.02(02) directly deals with the relationship between the regulatory speed and the design speed establishing that the design speed should equal or exceed the anticipated posted speed limit after construction or the State legal speed limit on non-posted highways. It also recommends that if the proposed design speed from the Geometric Design Tables is less than the established posted speed limit, one of the following approaches must be selected:

- increase the project design speed to equal or exceed the established or anticipated posted speed limit; or
- seek a design exception for the individual geometric design element(s) (e.g., a horizontal curve) which do not meet the established speed limit."

According to the American Association of State Highways and Transportation Officials (AASHTO, 2001), a design speed is selected to determine the major geometric components of a highway project including the cross-section dimensions, the horizontal alignment, and the vertical alignment. AASHTO also recommends that the design speed should provide safe and continuous operation and should be economically practical and consistent with the speed drivers are likely to expect under normal conditions.

The NCHRP Report 504 (Fitzpatrick et al., 2003) also noticed the inconsistencies between the definitions and the application of the different speed concepts (design speed, operating speed, etc.) used in highway design, as presented in various documents. The report established that the relationship between the design speed and the actual operating speed of the roadway is weak or changes with the magnitude of the design speed. Other limitations in the implementation of the AASHTO design speed concept have been discussed in the past. Fitzpatrick et al. (1999) argued that two flaws of the design speed concept are the use of the design speed of the most restrictive geometric element within the section as the design speed of the road and the non-explicit consideration of operating speeds on tangents or less restrictive curves. It was also questioned how curves with similar radii and superelevation rates can have different design speeds for different maximum superelevation rates; consequently, increasing design inconsistency and crash potential.

Krammes (2000) claimed that the AASHTO design speed policy cannot guarantee uniform operating speeds in rural highway alignments with design speeds of less than 60 mph (100 km/h). He supported his argument by providing evidence that show the disparity between the design speeds and the operating speeds. Speed data collected in 1978 on 12 two-lane rural highways at random points along tangents and curves with design speeds from 50 mph (80 km/h) to 70 mph (110 km/h) showed that drivers exceeded the design speed in sections with a design speed of 50 mph. Speed data collected in 1991 on 28 horizontal curves showed that the 85th percentile speeds exceeded the design speed on all curves with design speeds equal or less than 50 mph. McLean (1981) found similar results in Australia highways. He found that 85th percentile speeds exceeded design speeds on horizontal curves with design speeds equal or less than 55 mph (90 km/h). Islam and Seneviratne (1994) found that the difference between 85^{th} percentile speeds and design speeds of horizontal curves increased rapidly as the degree of curvature increased above 8 degrees.

The disparity between design speeds and operating speeds is not exclusive of two-lane rural highways. Fitzpatrick et al. (1997) measured free-flow speeds on 14 horizontal curves and 9 vertical curves in suburban highways. The observed 85^{th} percentile speeds were greater than the inferred design speed on horizontal curves with design speeds equal or lower than 45 mph (70 km/h) and on vertical curves with design speeds between 30 to 42.5 mph (50 to 65 km/h).

By using design speeds, highways are designed in a conservative manner to facilitate the safe motion of vehicles even in adverse but reasonable conditions. Designing for the worst scenario (e.g. combination of adverse conditions) generates conservative solutions with a built-in large margin of safety. Consequently, the 85^{th} percentile of observed free-flow speeds may exceed the design speed. Following the 85^{th} percentile speed rule and considering the crash experience, the posted speed limit may exceed the design speed of the section.

1.2. Research problem statement

In recent years, context-sensitive design principles have been highly promoted to ensure that all highway design considers the environmental, scenic, aesthetic, historic, community, and preservation impacts of a road project. The application of these principles in road design might lead to situations where the design standards cannot be met because of restricting local conditions. In such cases, horizontal curves have reduced design speeds compared to the adjacent tangent segments, requiring drivers to reduce their speeds to negotiate the curvature change.

A similar dilemma exists for Indiana rural road sections designed and built a long time ago. In a considerable number of sections with speeds controlled by the statutory limit of 55 mph (90 km/h), geometry of these sections does not meet the current design standards. Since individual intersections and curves may not safely carry traffic at the statutory speed, advisory speeds are posted together with warning signs. Although this solution increases the safety of road users and allows for traveling at reasonably high speeds where possible, the final solution is to upgrade the sections to the desirable design level.

INDOT makes a continuous effort to modernize sections that may not meet current design standards. Due to the limited budget, designers of the modernized sections sometimes have to apply a compromise approach. Achieving all major design criteria can be cost prohibitive in some projects. Either a reduced design speed or design exceptions must be considered. A cautious design approach of weighting pros and cons is needed. On one hand, the more expensive solution, the fewer sections are modernized within a certain period. On the other hand, low-cost solutions may not bring the design standard of the modernized sections to the desirable level.

As pointed out previously, the current INDOT design policy, restricted by federal regulations, recommends that the posted speed limit does not exceed the design speed. This requirement may lead to too low speed limits due to the excessive costs involved. Let us consider an example where a design speed of 60 mph (100 km/h) requires buying a developed land along the modernized section to provide a larger clear zone. An alternative to avoid the clear zone widening is using a lower design speed; let's say 40 mph (60 km/h). The third possibility is to design the traveled way for 60 mph and seek a design exception for a clear zone appropriate for 40 mph. Although the third solution seems to be the most rational, it poses a difficult question: What speed limit should be posted on the modernized section? The posted speed limit of 40 mph will meet public disapproval while the 60 mph limit violates the current policy.

Guidance is needed to help reduce the discrepancy between the economically justifiable design solutions and the design standards expected by the public on modernized highway sections. This discrepancy between the expected and the provided standards is manifested through the difference between the speed limit that can be applied on the modernized section (allowed or posted speed limit) and the speed limit expected by the motorists (target or desired speed limit). The desired speed limit can be approximated, in most cases, with the statutory speed limit that applies to the road section considered. Sometimes, the desired speed limit can be set at the current posted speed limit, if accepted by the motorists.

The posted speed limit is the main research issue. This value is a function of design. There are two ways of increasing the posted speed limit: (1) by increasing the design speed, or (2) by allowing the speed limit to exceed the design speed. The ability to increase the design speed strongly depends on the existing highway geometry, the roadside development and topography, and the project budget. Once the highest possible design speed is set and it is still below the desired value, the possibility of increasing the speed limit beyond the design speed has to be considered. This possibility stems from the fact that some design rules may generate too conservative solutions. In other words, the built-in safety margin may result in road geometrics upon which motorists feel comfortable operating vehicles at speeds greater than the designated design speed for a particular road. It should be mentioned that the safety margin might vary from one solution to another. It is even possible that two design alternatives with the same design speed and similar construction costs yield different margins of safety. Therefore, these two solutions are not equivalent since the solution with the wider safety margin would allow setting the speed limit at the value closer to the desired speed.

Design consistency is defined in the NCHRP Report 502 (Wooldridge et al., 2003) as the conformance of the highway's geometry and operational features with driver expectations. This report provides several rules that designers can use to improve the design consistency of high-speed two-lane rural highways regarding changes in cross-section dimensions, horizontal and vertical alignments, sight distance, and other design components. Two consistency issues usually targeted are: the discrepancy between operating speeds and design speeds; and the speed reduction between successive geometric features (Ottesen and Krammes, 2000). The development of speed profiles has been promoted as a practical tool to evaluate the design consistency of new design projects and to assess the impact of improvement projects in existing highways. Several countries have incorporated the use of the expected operating speed on the highway as a basis for selecting design speeds or specific geometric components, such as the superelevation rate and the stopping sight distance, or for detecting design inconsistencies (Polus et al., 1995).

Fitzpatrick et al. (1999) developed a series of speed models for the Interactive Highway Safety Design Model that predict 85th percentile speeds on two-lane rural highways using the radius of horizontal curvature or the rate of vertical curvature for selected combinations of horizontal and vertical alignment conditions. Other studies have developed speed models by evaluating the effects of isolated or restricted alignment conditions on a specific percentile speed, typically the 85th percentile. Although the 85th percentile speed is widely used to approximate highway operating speeds, other percentiles have been suggested to represent a high percentage of drivers in highway design (Polus et al., 1995; Bonneson, 2001). The use of the entire speed distribution has been also recommended (Tarris et al., 1996; Fitzpatrick et al., 2003) to develop speed models instead of focusing on a particular percentile as do the existing models.

A research study is needed to develop refined models that predict speeds along a highway section based on a diverse combination of roadway factors, besides horizontal and vertical curvature factors. Factors like the cross-section dimensions, the roadside development, the clear zone distance, the sight distance and the access density may have a direct effect on speeds. In addition, there is a need to evaluate the effectiveness of other modeling approaches to develop the predictive model, besides simple linear regression; and the use of the entire speed distribution.

A research study is needed to establish guidance that helps designers bring the posted speed limit as close to the desired speed limit in modernized highways. Engineering judgment can then be applied to balance safety and construction cost in highway improvement projects. The last important condition is modifying the current design policy to allow setting the posted speed limit at a value higher than the design speed, but according to the 85th percentile speed and the crash experience in the road section.

1.3. Research objective and scope

The research goal is to help INDOT design modernized highway sections in such a way that the posted speed limits meet as close as possible the desired speed limits. The research objective is to develop a tool useful in selecting design solutions by predicting future speeds on modernized highway sections.

This research focuses on predicting speeds that concur with a satisfactory level of safety. The 85th percentile of free-flow speed is particularly useful as it is a basis for setting speed limits on existing roads. The crash rate on the highway section with the predicted 85th percentile speed should not exceed a critical value. The crash experience is considered to eliminate sections where the driver perception of the risk is too low causing too high speeds or an excessive crash frequency. A research method that applies the above conditions will be proposed in this project.

The research will focus on rural and suburban highway sections without traffic interruptions caused by neither traffic signals nor stop signs. Interstate and local roads will be excluded. The relationship between the operating speeds and the highway components will be investigated by considering sections with different vertical and horizontal curvature characteristics, cross-section dimensions, density of access points (intersections with other roads and driveways), and other factors. The roadway factors to be included in the speed models will be evaluated in this research.

1.4. Expected benefits

First of all, designers will have better guidance to deal with a low design speed where the speed limit expected by motorists is higher. Elimination of too low speed limits will improve the drivers' compliance with traffic regulations at the considered sections and on other sections. Better design consistency with drivers' expectations will promote more adequate and uniform speeds on modernized sections. This effect would increase highway safety in the long run.

Drivers who are involved in crashes may consider lawsuits against INDOT if they realize that the posted speed limit had been set higher than the design speed, even if the actual hazard on the segment was not jeopardized by the geometry conditions. Having the established policy for setting speed limits above design speeds that is based on scientific results will assist INDOT in defending the design solution in court.

The INDOT Standards Section of the Contracts and Construction division will adopt the research results to the format consistent with the other departmental policy documents. The adopted text will be added to the Indiana Design Manual – Part V, Road Design. An appropriate INDOT internal committee will facilitate the adoption process. The Scoping Section of the Environment, Planning and Engineering Division and the Design Division will implement the speed-predicting tool in two-lane rural and four-lane rural and suburban highway improvement projects.

The research results will be published to allow their consideration in future updates of AASHTO, FHWA and ITE design guidelines and manuals.

1.5. Organization of the report

This report is organized into ten chapters. The current chapter discusses the research project background, objectives and scope. Chapter 2 presents a review of the state of the practices of selecting design speeds and posted speed limits. The chapter also reviews the speed factors and the methodologies used in other studies to develop operating speed models.

Chapter 3 presents the research methodology proposed to predict free-flow speeds in two- and four-lane highways. Chapter 4 discusses the data collection process and the relationship between the observed operating speeds and design components on two-lane rural highways. Chapter 5 discusses the data collection process and a similar evaluation for four-lane rural and suburban highways.

Chapters 6 and 7 discuss the calibration process and the evaluation of the speed models developed for two- and four-lane highways, respectively.

Chapter 8 presents a comparison between the posted speed limits, the inferred design speeds and the observed speeds on the highway sections observed in this study. The results show the disparity between the operating speeds and the design speeds in Indiana highways.

Chapter 9 presents the speed predicting tool developed in this study and discusses its prospective operation and implementation in highway design.

Chapter 10 presents the conclusions and recommendations of this research study.

CHAPTER 2. LITERATURE REVIEW

The following chapter presents a review of the state of the practices of selecting design speeds and posted speed limits. A review of recent studies that identified speed factors and developed operating speed models is also presented.

2.1. Practices of selection of design speeds and speed limits

The AASHTO design guide (2001) directly relates the design speed with the horizontal and vertical curvature, the maximum superelevation rate, and the sight distance. Other design components, like the lane and shoulder widths, are not directly related to the design speed. They are considered factors of operating speeds. The AASHTO guide suggests that the operating speed depends upon the capabilities of the drivers and the vehicles, the physical highway characteristics, the amount of roadside interference, the weather, the presence of other vehicles and the speed limitations established by law or traffic control devices.

The AASHTO guide recommends that the selection of the design speed should take into consideration the topography, the anticipated operating speed, the adjacent land use, and the highway functional classification. Fitzpatrick et al. (1997) found out that the most considered factors used by transportation agencies in the United States when selecting design speeds are: urban vs. rural environment, functional class, traffic volume, construction costs, corridor consistency and the agency's design criteria. The study also found that more than 75 percent of the agencies agreed that the expected operating speed on the highway should also be considered when selecting a design speed. The NCHRP Report 504 (Fitzpatrick et al., 2003) presents different considerations used by state departments of transportation in the United States when selecting design speeds, in addition to the factors suggested by the AASHTO guide or the state design manual:

- 0 to 10 mph (0 to 16.1 km/h) above the state-mandated maximum posted speed limit for the functional classification,
- the anticipated operating speed, or
- 5 to 10 mph (8.1 to 16.1 km/h) above the anticipated operating speed.

Several countries have already incorporated in their design procedures the use of the expected operating speed as the basis for selecting design speeds or specific geometric design components, like the curve superelevation rate and the stopping sight distance, and to inspect inconsistencies between the design speed and the operating speed.

The most frequent action taken by transportation agencies when the operating speed is higher than the design speed of the facility is to install warning signs (Fitzpatrick et al., 1997). Typical methods used to set advisory speeds for horizontal curves include ball bank readings, nomographs or calculated directly from the simplified curve formula (ITE, 2001). Studies have proved the ineffectiveness of the practice of setting advisory speeds on curves and the low compliance of drivers with the advisory speed signs. Chowdhury et al. (1998) evaluated the horizontal curve geometry, speeds and ball-bank readings of 28 two-lane highways in three states. Their study found that, in average, 90 percent of drivers exceeded the advisory speed and, in almost half of the sites, nobody obeyed them. In addition, the advisory speeds were set at values lower than the ones suggested by ball-bank readings and AASHTO's friction factors. It was also found that the advisory speeds were not set consistently in the three states.

Lyles (1982) evaluated five different advisory speed sign configurations in two highway locations where speed reduction was necessary to negotiate a horizontal curve. Speed measurements were taken at 200 ft (61 m) intervals starting from 1800 ft (548.6 m) before the curves. The study found that all drivers entered the curves at a speed faster than the one suggested by the advisory speed sign and continued to slow down well inside the curve. An important finding was that all drivers attained their minimum speed at approximately the same point. These findings suggest that most drivers ignore the advisory speed signs and adjust their speed by using their own perception of safety.

Free-flow speed and its variability are considered important safety factors in setting speed limits and in designing roadways. The posted speed limits should reflect a compromise between the travel time and the acceptable risk of crash for a specific highway class. The primary functions of a speed limit are to provide a limiting value and to reduce the dispersion of driving speeds. A policy of 85th percentile free-flow speed for setting speed limits is recommended by the AASHTO design guide (2001) and the MUTCD (FHWA, 2000). The Transportation Research Board (TRB, 1998) suggests that the posted speed limits should represent:

- the maximum speed for a reasonable and prudent driver traveling in free-flowing traffic with good visibility and under fair weather conditions, and
- the speed that will be enforced within some tolerance for minor measurement error.

Fitzpatrick et al. (1997) found that the most common factors used by transportation agencies for setting speed limits are: the 85th percentile speed, the accident experience, the roadside development and the state-mandated maximum speed limit. Some state agencies use the design speed as an initial speed limit, later modifying it with the 85th percentile speed after the facility is under operation. The Traffic Control Devices Handbook (2001) recommends that the 85th percentile speed should be used as a first approximation of the speed limit on a highway segment; attached to other factors like the physical roadway features, the traffic control characteristics, the crash experience and any other condition not readily apparent to drivers like the land use and the access conditions.

There is no well-established basis behind the use of the 85th percentile speed to set the speed limit. The current policy might be the consequence of a 1941 policy that suggested determining the critical or maximum safe speed by observing the 80th or 90th percentile speed under normal weather and daylight conditions (TRB, 1998). The majority of the agencies became accustomed to use the 85th percentile speed to set speed limits, while a few favored the 90th percentile. The rationale behind the use of the 85th percentile was that it approximated the upper limit of the 10 mph (16 km/h) pace speed. The 10 mph pace speed represents the range encompassing the greatest percentage of all the measured speeds and can be used as a surrogate measure of the speed dispersion. Solomon found in 1964 that the crash involvement rate on certain road classes was the lowest for vehicles traveling in a speed range whose upper bound was about one standard deviation above the average traffic speeds, at approximately the 85th percentile speed. The use of other percentiles (Polus et al., 1995; Schurr et al., 2002) and the use of the entire free-flow speed distribution (Tarris et al., 1996; Fitzpatrick et al., 2003) have been proposed to find better estimators of the highway operating speed.

2.2. Highway geometry characteristics as speed factors

Despite a large body of past research on speeds, there is still much to learn about the factors of free-flow speeds. Many factors are believed to be involved in the selection of speeds. These factors can be categorized as roadway characteristics, driver characteristics, vehicle characteristics, trip characteristics, traffic conditions, environmental conditions, speed limit and enforcement level. Assuming that typical drivers can appropriately assess all these factors, their speed selection will depend on an “optimal” decision between increasing safety and reducing travel time.

2.2.1. Speed factors on rural highways

Many studies have dealt with roadway characteristics as speed factors on rural highways. Operating speeds have been found to be related directly to certain elements of the horizontal and vertical curvature. The stopping sight distance, the highway grade and the access density are another factors believed to have some relationship with the operating speed. Some of the most recent studies are reviewed in this section.

Polus et al. (2000) found that the operating speeds on tangent segments in two-lane rural highways depended primarily of the tangent length and the radius of the curves preceding and following the tangent segment. Other elements like the presence of spirals, the speed limit, the enforcement level, the cross-section width, the highway grade, the side slope, the general terrain, the driver attitude and the vehicle's acceleration and deceleration capabilities were identified as less important speed factors. Schurr et al. (2002) analyzed the average, and the 85th and 95th percentile speeds on tangent segments in two-lane rural highways with posted speed limits equal or higher than 55 mph (90 km/h). A positive relationship was found between the posted speed limit and the three speeds, while the traffic volume was found to have a negative relation with the 85th and 95th percentile speeds. No relationship between the cross-section dimensions and any of the three speeds was found.

Fambro et al. (2000) evaluated speeds on crest vertical curves with limited stopping sight distance and on the adjacent tangent segments in multi- and two-lane rural highways. A decrease in the crest design speed (based on the sight distance) was associated with an increase in the difference in mean speed between the crest and the adjacent tangent. No strong relationship was found between the sight distance and the 85th percentile speeds on crest curves, except in two-lane highways without shoulders.

The horizontal curvature is widely documented as a key speed factor in two-lane rural highways, although other curve elements have been found to be somewhat significant. Islam and Seneviratne (1994) found different relationships between the degree of curve and 85th percentile speeds on the beginning, middle and ending points of horizontal curves. Fitzpatrick et al. (1999) found a relationship between the degree of curve and 85th percentile speeds for different highway grades and in combination with vertical curves. Schurr et al. (2002) found that the mean, the 85th and the 95th percentile speeds in the middle of horizontal curves increased with a decrease in the curve deflection angle and with an increase in the curve length. An additional factor was also found to be significant for each speed: the posted speed limit for the mean, the approach grade for the 85th percentile speed and the average daily traffic for the 95th percentile speed.

Similar results have been found in other countries. McLean (1981) claimed that the operating speeds on tangent segments in Australia were influenced by the functional classification, the trip purpose and length, the proximity to urban centers and the overall standard of the alignment. Although no relationship was found, the following ranges for the 85th percentile speed on tangent segments were suggested: 71.5 to 74.5 mph (115 to 120 km/h) for flat terrain, 56 to 68.4 mph (90 to 110 km/h) for rolling terrain, and 43.5 mph (70 km/h) for mountainous terrain. The operating speeds on horizontal curves were found to be strongly dependent of the curvature and the operating speed in the preceding tangent and somewhat related to the sight distance. Kanellaidis et al. (1990) found a similar relationship in Greece highways.

2.2.2. Speed factors on other highway types

Research of speed factors for other highway types is not as extensive as for two-lane rural highways. Fitzpatrick et al. (1997) evaluated the operating speeds on horizontal and vertical curves in suburban highways. A relationship between the 85th percentile speed and the access density was found on tangent approaches to horizontal curves. No relationship was found for tangent approaches to vertical curves. The study suggests that the curvature and the access density are good predictors of the operating speed on horizontal curves and that the inferred design speed based on the sight distance is a good predictor of the operating speed on vertical curves.

Poe and Mason, Jr. (2000) evaluated the operating speeds on tangents and horizontal curves in urban and suburban collector streets. The curvature, the lane width and a roadside hazard rating were found to be significant mean speed factors at the curve midpoint. When the observed speeds on tangents and curves were combined as one sample, only the curvature and the absolute value of the highway grade were found to be significant mean speed factors.

2.3. Available predicting models and research methodologies

It can be said that almost all the existing speed models have the following form:

$$V_i = \sum_k b_k X_{ik} + \varepsilon \quad (2.1)$$

where V_i is the mean speed or the operating speed at site i, X_{ik} is the value of the k exogenous variable at site i, b_k is the regression parameter associated with variable k, and ε is the normally distributed disturbance term. The random disturbance term is generally assumed to have a zero mean value and a constant variance σ^2. Most of the existing studies used a methodology approach based on the effect of isolated horizontal or vertical alignment conditions. Another approach typically used was that the speed is a function of local curve characteristics and a combination of other geometric parameters. This section presents a review of recent studies that developed speed models for different highway geometric features for rural and suburban highways.

Fambro et al. (2000) collected speeds on 41 crest vertical curves with limited stopping sight distance in multi- and two-lane rural highways in three states. Only segments containing crest curves with design speeds less than 60 mph (90 km/h), and consistent cross-section and adjacent land were included. The speeds were measured at the point on the curve with minimum sight distance and at the approach tangent, at least 328 ft (100 m) before the curve. A minimum of 100 speed observations were made using a radar gun. Only the sight distance was found to have a relationship with the crest operating speed in two-lane rural highways without shoulders. The ordinary least squares (OLS) model and its coefficient of determination (R^2) are the following:

$$V_{85,V} = 72.5 + 0.3 \times IDS \text{ , } R^2 = 0.48$$

where:

$V_{85,V}$ = 85th percentile speed on crest vertical curves, km/h

IDS = inferred design speed based on the sight distance, km/h

McLean (1981) measured a minimum of 100 free-flow speed observations on 120 horizontal curves in two-lane rural highways in Australia. Only segments without intersections or unusual highway features were selected. Speeds were also measured on 20 tangent segments to determine an estimate of the tangent operating speed. The traffic volume, the pavement and shoulder widths, the grade, the sight distance, the curve radius and the superelevation rate were recorded for each site. The OLS model and its R^2 value are the following:

$$V_{85,C} = 53.8 + 0.46 \times V_F - \frac{326}{R} + \frac{8500}{R^2} \text{ , } R^2 = 0.92$$

where:

$V_{85,C}$ = 85th percentile passenger car speed on horizontal curves, km/h

V_F = 85th percentile tangent speed based on the design speed and the terrain, km/h

R = curve radius, m

Islam and Seneviratne (1994) developed models to estimate the 85th percentile speeds at the beginning, middle and end points of horizontal curves in two-lane rural highways. A total of 125 speed observations were taken on 8 horizontal curves in Utah. The sites were selected so curve speeds were not affected by limited sight distance, grades higher than 5 percent, defective pavement or adverse alignment. The OLS models and their corresponding R^2 values are the following:

$$V_{85,PC} = 95.4 - 1.48 \times DC - 0.01 \times DC^2, \; R^2 = 0.99$$

$$V_{85,MID} = 103.0 - 2.41 \times DC - 0.03 \times DC^2, \; R^2 = 0.98$$

$$V_{85,PT} = 96.1 - 1.07 \times DC, \; R^2 = 0.90$$

where:

$V_{85, PC}$ = 85th percentile speed at the beginning of a horizontal curve, km/h
$V_{85, MID}$ = 85th percentile speed at the middle of a horizontal curve, km/h
$V_{85, PT}$ = 85th percentile speed at the end of a horizontal curve, km/h
DC = degree of curvature, degrees per 30 meters

A different estimate was found for the degree of curvature in each model. This finding suggests that drivers do not operate at a constant speed inside horizontal curves and that a portion of the deceleration and the acceleration still occurs inside the curve. The models also suggest that the lowest speed occurs at the midpoint for curves of 6 degrees or higher and that the speed at the end of the curve is always the highest. The transition speeds occurring between the tangent and the curve may be influenced by both tangent and curve components. The models also show that the relationship between the speed and the curvature is not strictly linear.

Schurr et al. (2002) developed separate models to predict the mean, 85th and 95th percentile speeds on tangent segments and on the middle of horizontal curves in two-lane rural highways of Nebraska. Free-flow speeds were measured in 50 sites with speed limits ranging from 55 to 65 mph (90 to 105 km/h) and with daily volumes up to 5000 vpd. The sites were selected so pavement conditions, intersections, vertical curves and roadside elements did not affect the curve speeds. Sites with guardrails, traffic control signs, posted speed limits, advisory speed signs or lane widening within 1000 ft (300 m) of the curve were not included. The speeds were measured using two magnetic traffic detectors, one located in the curve midpoint and the other 600 ft (183 m) in advance of the curve. At least 112 speed observations with headways of five seconds or more were recorded during dry and daytime conditions. The roadway width, the posted speed limit and horizontal and vertical curve components were collected. The OLS models developed to estimate the mean and the 85th percentile speeds and their R^2 values are the following:

$$V_{MEAN,T} = 51.7 + 0.51 \times PSL \text{, } R^2 = 0.30$$

$$V_{85,T} = 70.2 + 0.43 \times PSL - 0.001 \times ADT \text{, } R^2 = 0.19$$

$$V_{MEAN,C} = 67.4 - 0.11 \times \Delta + 0.022 \times L + 0.28 \times PSL \text{, } R^2 = 0.55$$

$$V_{85,C} = 103.3 - 0.12 \times \Delta + 0.024 \times L - 1.04 \times G \text{, } R^2 = 0.46$$

where:

$V_{MEAN,\ T}$ = mean passenger car speed at a tangent segment, km/h

$V_{85,\ T}$ = 85th percentile passenger car speed at a tangent segment, km/h

$V_{MEAN,\ C}$ = mean passenger car speed at the middle point of a horizontal curve, km/h

PSL = posted speed limit, km/h

ADT = traffic volume, vehicles per day

Δ = curve deflection angle, decimal degrees

L = arc length of curve, meters

G = approach grade, percent

This study found some different significant variables for each speed. In addition, different estimates were found for those variables shared by the models. The first finding shows the potential for finding different mean speed and speed dispersion factors at a site. The second finding shows that the effect of a specific highway component may be different for the mean speed and the speed dispersion.

Fitzpatrick et al. (1999) collected speed and geometric data in two-lane rural highways from six states. At least 100 speed observations were made during daylight, dry and off-peak conditions using radar guns and traffic classifiers connected to piezoelectric sensors. All sites were located in low-volume segments. Highway components like the grade, the pavement width, the roadside design, the driveway density and the posted speed limit were collected. These components and various alignment indices were evaluated for 88 sites located on tangents, but no relationship was found with the operating speed. An operating speed of 62 mph (100 km/h) was suggested for tangent segments. This value was based on the average value of the observed range from 57.8 to 64.6 mph (93 to 104 km/h) for the 85th percentile speed. A previous study sponsored by the FHWA in 1994 had similar results and suggested a tangent operating speed of 60.8 mph (97.9 km/h) based on the mean value of the observed 85th percentile speeds.

Polus et al. (2000) develop a model to predict operating speeds on tangent segments using the data collected by Fitzpatrick et al. (1999). The sites were divided into four groups based on the tangent length and the radius of the horizontal curves preceding and following the tangent segment. Separate OLS models were developed for each tangent group. The following speed model was suggested for segments having two small curves (radius smaller than 820 ft) and a tangent length of less than 500 ft:

$$V_{85,T} = 101.1 - \frac{3420}{\overline{R}_{1,2}}, \ R^2 = 0.55$$

where:

$\overline{R}_{1,2}$ = average radius of horizontal curves preceding and following the tangent, meters

Fitzpatrick et al. (1999) developed OLS models to estimate 85th percentile speeds for different combinations of horizontal and vertical alignments. Some of the combinations were horizontal curves on upgrades or downgrades, horizontal curves combined with sag or crest vertical curves, and sag or crest vertical curves on tangent segments. The 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 selected three points were the midpoint of the vertical curve, the minimum sight-distance point and the midpoint of the preceding tangent. For sites including horizontal and vertical curves, speeds were measured 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 measured midway between the horizontal curve point of intersection and the vertical curve point of intersection. In all cases, speeds were also measured at the preceding tangent. The following OLS models were suggested for horizontal curves combined with a sag vertical curve and for horizontal curves combined with a sight-limited crest vertical curve, respectively:

$$V_{85} = 105.3 - \frac{3438.19}{R}, \ R^2 = 0.92$$

$$V_{85} = 103.2 - \frac{3576.51}{R}, \ R^2 = 0.74$$

where:

V_{85} = 85th percentile speed, km/h

R = curve radius, meters

This study seems to be one of the first to analyze the effect on operating speeds of specific combinations of vertical and horizontal alignment conditions. Typically, models were developed for isolated horizontal or vertical alignment conditions. This study, however, failed to incorporate cross-section dimensions and other important highway components as factors in the speed models. The speed models developed in this study use only the radius of horizontal curves or the rate of vertical curvature as explanatory variables for a set of alignment combinations and suggest operating speed values for tangents and other alignment combinations. Further refinement is needed to develop speed models with the capability of predicting operating speeds along a roadway segment based on multiple factors than just a fixed set of horizontal and vertical alignment combinations.

Fitzpatrick et al. (2003) collected speed and geometric data in 78 sites from different highway classes in six states. The sites were located at least 0.1 mi (0.16 km) apart from horizontal curves and 0.2 mi (0.32 km) apart from traffic signals or stop signs. The posted speed limit varied from 25 to 55 mph (40 to 90 km/h). The cross-section width, the roadside information, the access density, the speed limit and the pedestrian activity were collected at each site. At least 100 free-flow speeds were measured at each site using a laser gun or traffic classifiers. Speed models were developed for five different highway classes. Except for the posted speed limit and the access density, no other roadway characteristic had a relationship with the operating speeds. Four of the developed OLS models and their R^2 values are the following:

$$V_{85} = 16.1 + 0.83 \times PSL - 0.05 \times AD$$, R^2 = *0.92, for all functional classes*

$$V_{85} = 8.7 + 0.96 \times PSL$$, R^2 = *0.86, for suburban and urban arterials*

$$V_{85} = 14.7 + 0.40 \times PSL - 0.06 \times AD$$, R^2 = *0.58, for suburban and urban collectors*

$$V_{85} = 36.4 + 0.52 \times PSL$$, R^2 = *0.81, for rural arterials*

where:

V_{85} = 85th percentile speed, mph

PSL = posted speed limit, mph

AD = access density, number of intersections and driveways per roadway mile

Fitzpatrick et al. (1997) measured speeds on 14 horizontal curves, 9 vertical curves and the adjacent tangent in suburban highways. The horizontal curves had inferred design speeds from 40 to 75 mph (60 to 125 km/h) and the vertical curves had inferred design speeds from 30 to 40 mph (50 to 60 km/h). The design speed was inferred using the current design policy and the observed variables. Three of the OLS models developed and their R^2 values are the following:

$$V_{85,T} = 74.9 + 22.29 / AD$$, R^2 = *0.71*

$$V_{85,C} = 54.2 + 1.06 \times \sqrt{R}$$, R^2 = *0.72*

$$V_{85,V} = 39.5 + 0.56 \times IDS \text{, } R^2 = 0.56$$

where:

$V_{85,T}$ = 85th percentile speed on tangents, km/h

$V_{85,C}$ = 85th percentile speed on horizontal curves, km/h

$V_{85,V}$ = 85th percentile speed on vertical curves, km/h

AD = approach density, number of driveways and intersections per kilometer

R = curve radius, meters

IDS = inferred design speed, km/h

Poe and Mason, Jr. (2000) measured speeds on tangents and on various points inside and outside horizontal curves in 27 sites located in urban and suburban collector streets. The posted speed limit on the segment was either 25 or 35 mph (40 or 55 km/h). All the segments had curbs and most segments included parking in the direction of travel. The speeds were measured with magnetic detectors at the beginning, middle and end of horizontal curves, at 150 ft (46 m) before and after curves, and at the adjacent tangent segment. Fixed-effects models were developed by using speeds measured on horizontal curves and the approach tangents. The models describe the fixed effects of the geometric characteristics and the random site effects by using a dummy variable for each site. The curvature, the lane width, the highway grade and a roadside hazard rating were significant mean speed factors when each point was used separately to calibrate a model. An additional model was developed using the speeds from three points inside the curve and one point before the curve. Only the curvature and the absolute grade were significant mean speed factors in this model. The models developed for the beginning and the middle point of horizontal curves are the following:

$$V_{MEAN,PC} = 51.1 - 0.10 \times DC - 0.24 \times G - 0.01 \times LW - 0.57 \times RD$$

$$V_{MEAN,MID} = 48.8 - 0.14 \times DC - 0.75 \times G - 0.01 \times LW - 0.12 \times RD$$

where:

$V_{MEAN,PC}$ = mean passenger car speed at the beginning point of a horizontal curve, km/h

$V_{MEAN,MID}$ = mean passenger car speed at the middle point of a horizontal curve, km/h

G = absolute highway grade, percent

LW = lane width, meters

RD = roadside hazard rating, categorical variable

The relevance of this study is the use of a fixed-effects approach to develop the speed models instead of the typical OLS approach. The fixed-effects model is able to explain the fixed effects due to the highway components and the random effects due to the site variation. The modeling approach can be further improved if the sites are considered as a random variable instead of a dummy variable. The observation sites can be considered as a random variable because the selection of the sites in a study comes from a much larger population which we want to make inferences about (Hsiao, 1986).

The existing models estimate a specific speed percentile and they do not distinguish between the mean speed factors and the speed dispersion factors. It makes interpretation of the results difficult because it obscures both factors by using a specific percentile value from the free-flow speed distribution. It is possible that a road with a high mean speed and low speed variability has the same 85th speed percentile as a road with a much lower mean speed but higher speed variability. Modeling the entire free-flow speed distribution, suggested by Tarris et al. (1996) and Fitzpatrick et al. (2003), might rectify this problem. The mean free-flow speed and its variability across drivers are considered important safety factors. It is believed that individual drivers carry out a trade-off between travel time and safety when selecting their desired speed on a trip. The relationship between speed and crashes has been studied with no irrefutable link. There is ongoing discussion as to which factor - the mean speed or the speed dispersion - has an impact on safety. Either opinion is defendable. An increase in mean speed increases the crash severity, while an increase in speed variability increases the frequency of interactions between vehicles.

Solomon concluded in 1964 that the speed dispersion has a U-shaped relationship with the crash involvement rate. The relationship suggested that an increase in the deviation between a motorist's speed and the average speed of traffic is related to a greater chance of involvement in a crash. This relationship has been fairly accepted as a benchmark for later studies (TRB, 1998). Although the Solomon study was criticized for using speed estimates from crash reports and a unrealistic comparison with the traffic mean speed data, other studies have replicated (although in a lesser extent) the U-shaped relationship. Garber and Gadiraju (1989) found evidence that crash rates from different highway types increase with an increase in the speed variance and that an increase in mean speeds is not necessarily related to an increase in accident rates. Their study concluded that minimum speed variance occurs when the difference between the design speed and the posted speed limit is between 5 and 10 mph (10 and 20 km/h). In addition, they found that the speeds increased with better geometric conditions, regardless of the posted speed limit. Collins et al. (1999) found low speed dispersion for horizontal curves with radii values of less than 328 ft (100 m) and, that as the radii increased the range of speed dispersion also increased. It was also found that the speed dispersion decreased with increasing pavement width in segments where the posted speed limit exceeded the design speed or where design inconsistencies were present.

These findings show the potential contribution of design components as speed factors and argue in favor of the development of a model that is able to include the impact of design components as speed dispersion factors to improve the safety of highway design solutions. A methodology that can find significant relationships between highway design components and mean speeds and speed dispersion is needed. Being able to identify separately the mean speed factors and speed dispersion factors would improve the interpretation of the results.

CHAPTER 3. RESEARCH METHODOLOGY

The following chapter discusses the research approach and the methodology proposed to predict free-flow speeds in two- and four-lane highways. The derivation and the calibration process of the percentile free-flow speed are presented as well.

3.1. Research approach

Most of the approaches used in previous studies focused on the isolated effects of horizontal or vertical alignment components on speeds. In addition, the existing models estimate a specific speed percentile and they do not distinguish between the mean speed factors and the speed dispersion factors, which leads to results that are sometimes difficult to interpret. We will try to address these two limitations.

This study considered various highway configurations, including the cross-section dimensions, the roadside clear zone, the available sight distance, the access density, the land development and the roadway alignment. Highway geometry and free-flow speeds were collected at various spots: on tangent segments and before, inside or after horizontal curves, vertical curves and intersections. Highway maps were used to identify candidate segments based on their alignment characteristics. Then, segments with considerable high crash rates were removed. The only restriction imposed on the segment selection was that the speeds must not be affected by traffic signals or stop signs. The sample selection scheme used in this study provides the capability of predicting any percentile speed for a diverse combination of highway components and under the absence of excessive hazard.

3.2. Percentile speed model

An advanced method of modeling free-flow percentile speeds is discussed in this section. The model overcomes the limitations of the existing speed models and is able to model any percentile speed. This is accomplished by representing the percentile speed as a linear combination of the mean and the standard deviation and by using panel data. Two alternate models are discussed: an ordinary least squares (OLS) model for panel data and a generalized least squares (GLS) model that considers random effects.

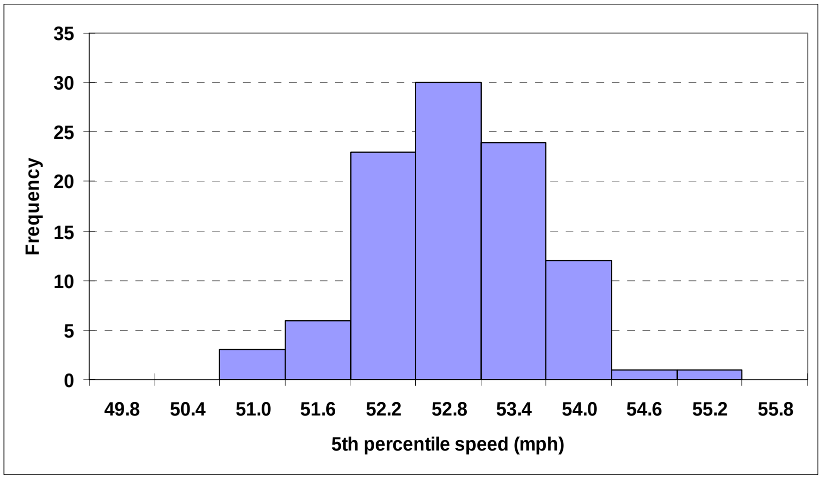
3.2.1. Percentile model without random effects

With the assumption of normally distributed individual vehicle speeds at a site *i*, any p^{th} percentile speed at the site (V_{ip}) can be calculated by multiplying the corresponding Z_p value with the standard deviation of individual speeds (σ_i) and adding that product to the mean speed value (m_i). The Z_p value is the standardized normal variable corresponding to a selected percentile; for example, Z_{50} = 0.0 and Z_{85} = 1.036. This function can be represented as a statistical model by adding an iid normal disturbance term (ε). The OLS percentile model using panel data has the following form:

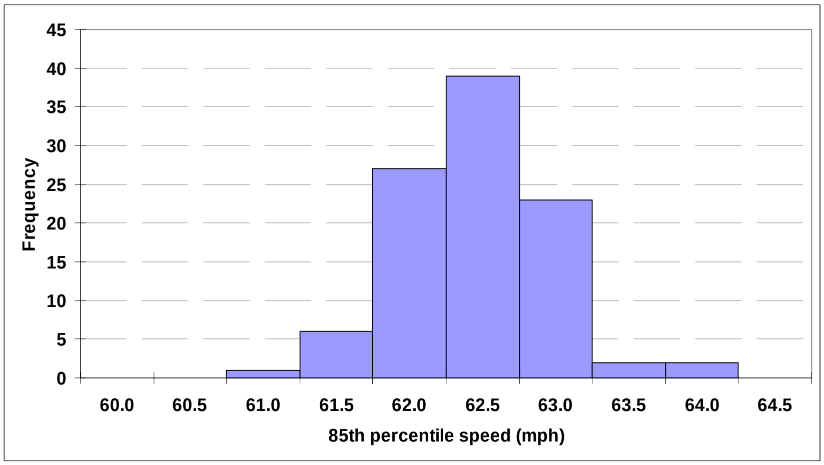
$$V_{ip} = m_i + Z_p \cdot \sigma_i + \varepsilon \tag{3.1}$$

The assumption of ε normality is useful because it leads to regression parameters that are approximately *t* distributed. The assumption of normality for ε is strictly met for the 50th percentile estimates under the assumption of the normally distributed individual speeds. The percentiles distant from the mean may have distribution of their estimates considerably skewed which would make the normality assumption difficult. The normality assumption was evaluated for the 5th, 85th and 95th percentile estimates using the Monte Carlo method. A hundred random numbers were generated for 100 variables using a mean value of 58.55 mph (94.2 km/h) and a standard deviation of 3.55 mph (5.7 km/h). Figure 3-1 presents the frequency histograms for the 5th, 85th and 95th percentile estimates.

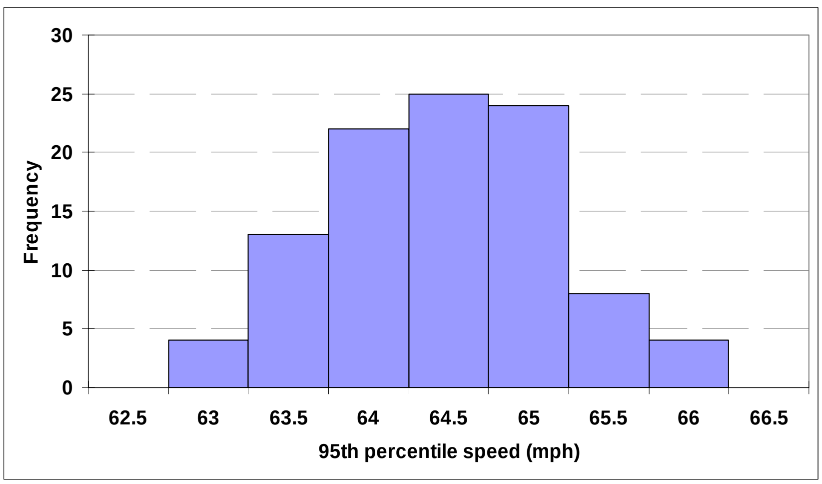
Although the distributions look reasonably symmetrical, the Shapiro-Wilk (*W*) statistic was used to test for departures from normality. The *W* statistic for the 5th, 85th and 95th percentile estimates had p-values of 0.99, 0.19 and 0.82, respectively. The *W* statistics establish that there is no sufficient evidence to conclude that the percentile estimates are not normal with a 95 percent confidence. It was then concluded that the distributions of the three percentile estimates met the normality assumption. The percentile range considered in this study is limited from the 5th to the 95th percentiles to allow the normality assumption to hold. In any case, if predicting future mean responses is the only modeling purpose, then ignoring normality will not hinder the ability of making predictions (Washington et al., 2003).



a) 5th percentile estimates



b) 85th percentile estimates



c) 95th percentile estimates

Figure 3-1 Frequency histograms for the 5th, 85th and 95th percentile estimates.

Assume that the mean speed of free-flowing vehicles in a highway segment is affected by some road characteristics and is represented by the following function:

$$m_i = \sum_j a_j \cdot X_{ij} \tag{3.2}$$

where the a_j coefficient represents the effect of the X_j parameter on the mean speed value.

Furthermore, assume that the standard deviation of individual speeds on a highway segment can be affected by some road characteristics and is represented by the following function:

$$\sigma_i = \sum_k b_k \cdot X_{ik} \tag{3.3}$$

where the b_k coefficient represents the effect of the X_k parameter on the standard deviation of individual speeds.

A linear model to estimate any p^{th} percentile speed can be arranged by inserting the right-hand side of Equations 3.2 and 3.3 into Equation 3.1. The linear model to estimate any p^{th} percentile speed is the following:

$$V_{ip} = \sum_j a_j \cdot X_{ij} + \sum_k b_k \cdot (Z_p \cdot X_{ik}) + \varepsilon \tag{3.4}$$

The model in Equation 3.4 is denoted as OLS-PD to emphasize that ordinary least squares regression is applied to panel data. The panel data approach is frequently used for econometric applications when multiple observations on each individual are present in the data set. Medical studies have used comparable modeling approaches with panel data to obtain percentile estimates. Morgenstern (2002) modeled the mean total body water as a function of patient characteristics while the standard deviation of the individual observations was assumed constant. Morrell (1997) used two-step regression to model separately the mean value and the standard deviation of hearing thresholds. The approach proposed in our study goes a step further by calibrating the mean and standard deviation terms in a single step.

The panel data was created by multiplying all the potential explanatory variables by the standard normal value corresponding to a respective percentile. The percentile speeds from the 5^{th} to the 95^{th} percentile, in increments of five, were calculated for all sites. By having a higher number of observations than typical cross-sectional data sets, panel data sets have more degrees of freedom, reducing collinearity between explanatory variables and improving the efficiency of the parameter estimates (Hsiao, 1986). For example, data from 32 sites will produce a panel of 608 observations, instead of just 32 observations if a cross-sectional data set based only on the mean or the 85th percentile speed is used. The OLS-PD model can be further improved by adding site-specific and percentile-specific random effects to avoid bias in estimating the model parameters caused by unknown factors not incorporated in the regression model.

3.2.2. Percentile model with random effects

The random effects of sites in the linear model shown in Equation 3.4 can be captured with site-specific random shifts μ_i (Greene, 2003) as follows:

$$V_{ip} = \sum_{0}^{J} a_j \cdot X_{ij} + \sum_{0}^{K} b_k \cdot (z_p \cdot X_{ik}) + \varepsilon + \mu_i \tag{3.5}$$

The random effects (RE) model properly grasps both the random (unexplained) and fixed (explained) effects by assigning a portion of the unexplained variability to the sites, *var(μ)*, and the remaining portion, *var(ε)*, to the entire sample. The covariance between the random effects ε and μ is assumed to be equal to zero. The RE model is a generalized least squares regression model based on the assumption that the unexplained site-specific effects are uncorrelated with the included variables in the model. The RE model offers a convenient formulation when *n* cross-sectional units are randomly drawn from a population (Washington et al., 2003), and inferences about that population is the main objective.

The random effects of the percentile dimension in the panel data can be also captured with percentile-specific random shifts ω_p (Greene, 2003) in Equation 3.5 as follows:

$$V_{ip} = \sum_{0}^{J} a_j \cdot X_{ij} + \sum_{0}^{K} b_k \cdot (z_p \cdot X_{ik}) + \varepsilon + \mu_i + \omega_p \tag{3.6}$$

In this model, neither the number of percentiles observed for each group nor the number of sites observed in each period need to be fixed. The data can consist of a sample of observations indexed by both site and specific percentile. The panel created in this study is balanced with all sites having the same number (19) of percentile values.

Previous applications of panel data modeling with random effects are present in dynamic traffic assignment, car ownership and trip generation studies, among others. Poe and Mason, Jr. (2000) used a modeling approach that incorporated the random effect from the individual sites while modeling the fixed geometric effects on mean speeds. Tarris et al. (1996) applied a random effects model by cross-sectioning individual drivers as a group and the location of different speed sensors as a time period. A comparison showing the advantages of the RE model over the OLS and OLS-PD models is presented in Chapter 6.

3.3. Model development and calibration

Figure 3-2 presents a flowchart with the procedure used in this study to develop the speed models for two-lane rural highways. The procedure used to develop the speed models for four-lane highways was significantly simplified and is later explained in this chapter.

The first step was the collection and evaluation of highway geometry characteristics and free-flow speeds from a selected number of observation sites. Chapter 4 discusses the data collection process for two-lane rural highways. Chapter 5 discusses the data collection process for four-lane highways. The second step involved the construction of the panel data as discussed in Section 3.2.1. The panel for two-lane highways is composed of 3002 observations (158 spots multiplied by 19 percentile speed observations per spot); while the panel for four-lane highways is composed of 950 observations (50 spots multiplied by 19 percentile speed observations per spot).

The speed models for two-lane rural highways were developed following an iterative process. Preliminary mean deceleration and acceleration rates were estimated from field observations and the portion of the transition sections on the tangent was initially assumed at the start of the calibration process. Preliminary mean speed models for tangents and horizontal curves were developed using selected sites. Thirty-two sites were selected to develop the model for tangents and twenty sites were selected to develop the model for horizontal curves. An analysis was performed to justify that the speeds on the selected tangent sites were free from the influence of horizontal curves. The speeds from spots located on horizontal curves and the adjacent tangent segment were compared and the length of the transition sections was estimated for the sites using the mean acceleration and deceleration rates. It was concluded that the thirty-two sites selected as tangent sites were located outside the transition section of any horizontal curve.

The preliminary models were used to estimate the mean speeds on tangents and curves for all the sites in the sample. The curve mean speed was estimated only for those sites having curve information. If the estimated curve mean speed was higher than the estimated tangent mean speed, the curve was assumed to have no impact on speeds. In other words, the curve design allows drivers to negotiate the curve at a speed at least equal to the speed influenced by the highway characteristics and cross-section dimensions; therefore speeds can be estimated using the tangent speed model. In addition, it was assumed that the other percentile speeds will follow the same trend as the mean speed (e.g. if the mean speed inside the curve is higher than the mean speed in the tangent segment, the p^{th} percentile speed inside the curve will also be higher than the p^{th} percentile speed in the tangent segment).

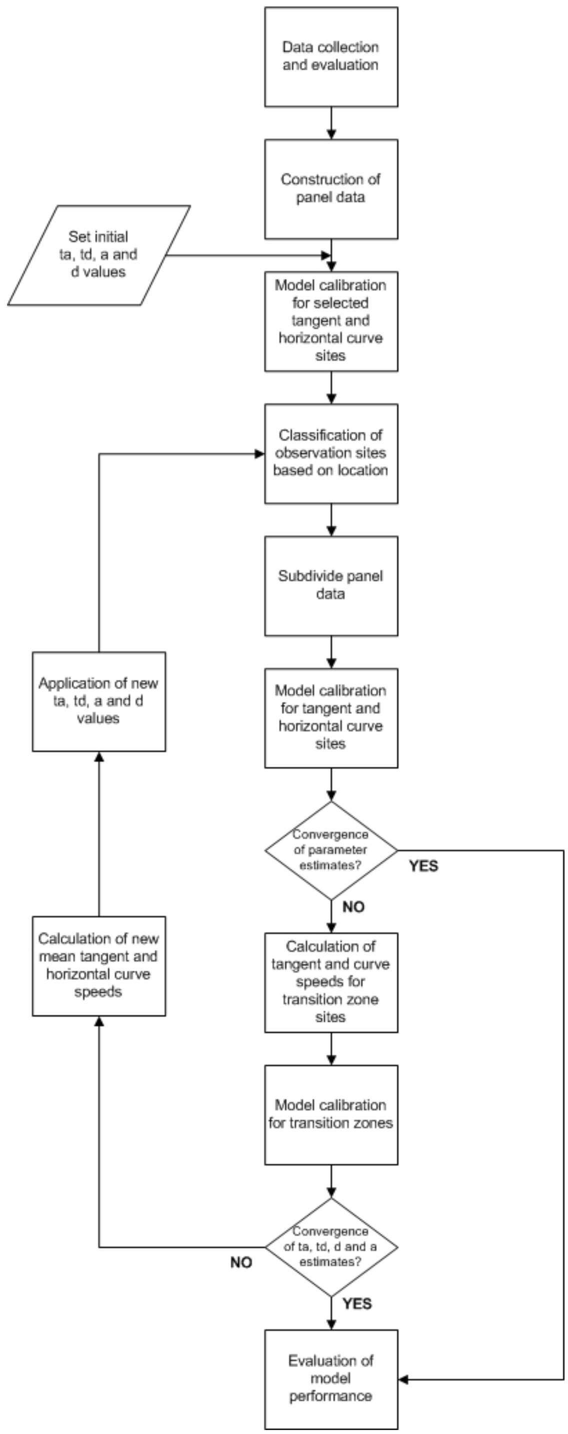


Figure 3-2 Development procedure of speed models

The preliminary mean speeds were used in conjunction with the estimated mean acceleration and deceleration rates to calculate the length of the transition sections for all the sites containing curves. The length of the transition section represents the distance used by drivers to adjust their speeds based on the application of their desired acceleration or deceleration rate. Figure 3-3 presents a schematic of the approach used for the speed modeling in transition sections. The length of the tangent-to-curve transition section, L_d, in feet, was determined as:

$$L_d = \frac{V_C - V_T}{d} \tag{3.7}$$

where:

V_C = speed inside horizontal curves, ft/s

V_T = speed in tangent segments, ft/s

d = mean deceleration rate, (ft/s)/ft

The length of the curve-to-tangent transition section, L_a, in feet, was determined as:

$$L_a = \frac{V_T - V_C}{a} \tag{3.8}$$

where:

a = mean acceleration rate, (ft/s)/ft

The design of the transition section includes the superelevation and the alignment transitions. Indiana design standards do not require the use of spiral curves for the design of transition sections. AASHTO (2001) indicates that the normal practice in such cases is to divide the runoff length between the tangent and curved sections and avoid placing the entire runoff length on either the tangent or the curve following the natural spiral path adopted by drivers. Most agencies use a single value, from 60 to 80 percent, to locate the runoff length on the tangent prior to the curve. Lyles (1982) and Fitzpatrick et al. (1999) found evidence to support that part of the transition occurs inside the curve. Their studies found that drivers decelerate and accelerate inside the horizontal curve, although no estimate of the percentage of the transition length occurring inside the curves was provided.

Theoretical considerations suggest placing a larger portion of the runoff length on the tangent, in a range of 70 to 90 percent, to offer the best operating conditions (AASHTO, 2001). Although the specific proportion depends on the number of lanes rotated and the design speed, the use of a single value for all speeds and rotated widths is considered acceptable by AASHTO. The proportion of both deceleration and acceleration transition section on tangents, t_d and t_a, was initially set at 85 percent. The proportions, as well as the deceleration and acceleration rates, will be calibrated as part of the model development.

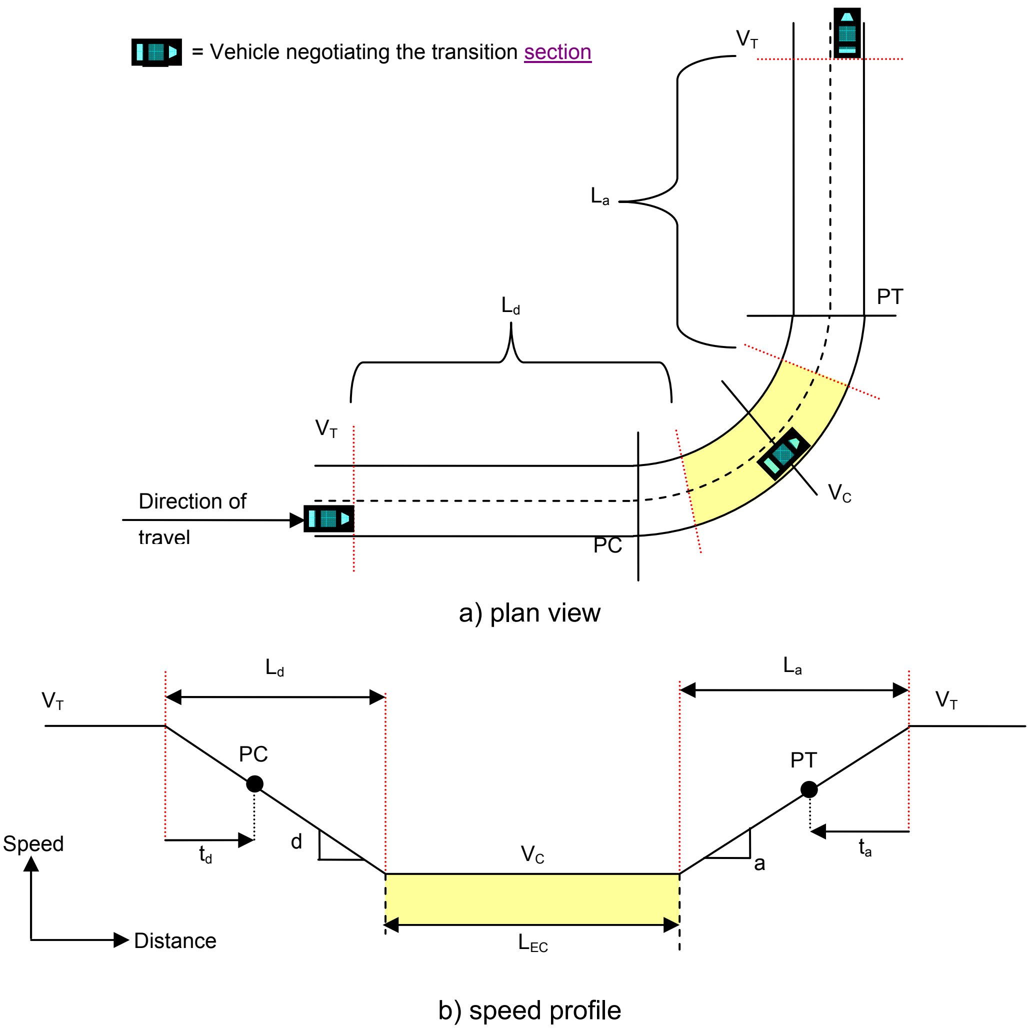


Figure 3-3 Modeling of speeds on curve transition sections

The value for the portion of the transition sections on the tangent was applied to the entire panel and the observation sites were classified based on their location with respect to horizontal curves. The panel was subdivided into four sub-samples: sites on tangents, sites on horizontal curves, sites in deceleration transition sections and sites in acceleration transition sections. These four sub-samples were used to recalibrate new speed models, new mean acceleration and deceleration rates and new values for the portion of the transition section located on the tangent.

In cases where the length of the curve was smaller than the combined length of the deceleration and the acceleration transition sections inside the curve, the estimated mean speed on the curve was not reached (e.g., the length of the effective curve, L_{EC}, is zero). Figure 3-4 presents a schematic of such a case. The effective curve was defined as the section of the horizontal curve where drivers maintain a constant desired speed. The length of the effective curve was calculated as the length of the horizontal curve minus the length of the deceleration and acceleration transition sections inside the curve. When the length of the effective curve is zero, the speed along the curve was determined solely by the transition section models (applying the deceleration or acceleration rate). If the transition sections overlapped inside the curve, the smallest speed reduction due to the deceleration or the acceleration at a specific spot was selected and the site was classify either in the deceleration or acceleration transition sub-sample.

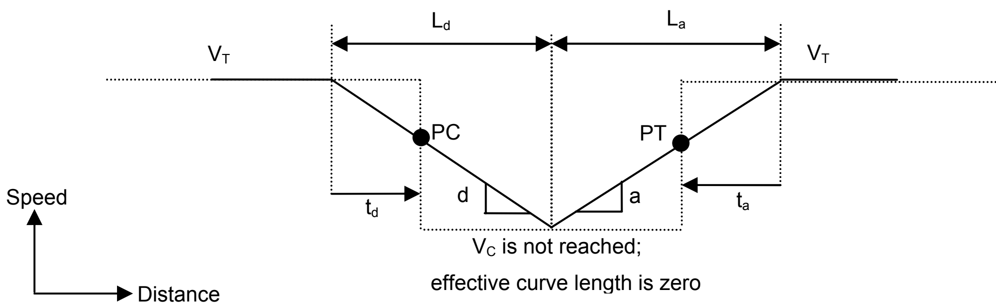


Figure 3-4 Modeling of transition sections for short curves

Speed models were developed independently for the tangent and horizontal curves sub-samples following the model presented in Section 3.2. The parameter estimates calibrated for the tangent and curve models between two successive iterations were compared to check if convergence was reached. If convergence was reached, the iterative process was stopped and the performance of the speed models was evaluated. Otherwise, the iterative process continued.

The new speed models for tangents and horizontal curves were applied to the transition sections sub-samples. The tangent and curve percentile speeds were estimated, from the 5^{th} to the 95^{th} percentiles, for all sites. The speed models for transition sections were calibrated using the estimated and the observed percentile speeds and the distance of the site from the curve. The speed in the transition sections depends on the estimated speeds on the tangent and the curve, the deceleration and acceleration rates, the length of the transition sections and the portion of the transition sections outside the curve.

The speed model for the deceleration transition section has the following form:

$$\begin{aligned} V_p &= V_{Tp} - t_d \times \left(V_{Tp} - V_{Cp}\right) + d \times l_d \\ (V_p - V_{Tp}) &= -t_d \times \left(V_{Tp} - V_{Cp}\right) + d \times l_d \end{aligned} \tag{3.9}$$

where:

V_p = observed percentile speed, ft/s

V_{Tp} = estimated percentile speed on tangent, in ft/s

V_{Cp} = estimated percentile speed on horizontal curve, ft/s

t_d = portion of the deceleration transition section outside of the horizontal curve, ft

d = deceleration rate, (ft/s)/ft

l_d = distance from the site to the beginning of the curve, takes a positive value outside the curve and a negative value inside the curve, ft

The speed model for the acceleration transition section has the following form:

$$
\begin{aligned}
V_p &= V_{Tp} - t_a \times \left(V_{Tp} - V_{Cp}\right) + a \times l_a \\
(V_p - V_{Tp}) &= -t_a \times \left(V_{Tp} - V_{Cp}\right) + a \times l_a
\end{aligned}
\tag{3.10}
$$

where:

t_a = portion of the acceleration transition section outside of the horizontal curve, ft

a = acceleration rate, (ft/s)/ft

l_a = distance from the site to the end of the curve, takes a positive value outside the curve and a negative value inside the curve, ft

The calibration of Equations 3.9 and 3.10 provided new deceleration and acceleration rates and new portions of the deceleration and acceleration transition sections that occur outside the curve. The parameter estimates obtained for the transition models between two successive iterations were compared to check if convergence was reached. If convergence was reached, the iterative process was stopped and the performance of the models was evaluated. Otherwise, the iterative process continued.

The new values for t_d, t_a, d and a were applied to the entire panel along with the new tangent and horizontal curve speed models to perform a new iteration by re-classifying the sites according to their location with respect to horizontal curves. The only restriction imposed in the iteration process was to have a reasonable number of sites assigned to any sub-sample to calibrate the speed model. A minimum of 14 sites was desired in any sub-sample to have enough variability in the values of the explanatory variables. When the number of sites assigned to any sub-sample did not met this constraint the model calibrated and the sites assigned in the last iteration were retained.

The model development process was stopped when there was no change in the classification of sites between two consecutive iterations and the speeds models could not be further improved. The percent change in the number of sites assigned to each sub-sample, the parameter estimates, the coefficient of determination (R^2) values and the t_d, t_a, d and a values were evaluated from iteration to iteration to check for convergence.

The model development procedure was significantly simplified for four-lane highways. A single speed model was only calibrated due to the lack of significant impact of curves. Therefore, no iterative process was needed. The model development for four-lane highways consisted of only four steps: data collection and evaluation, construction of the panel data, speed model calibration and evaluation of the model performance.

Although horizontal and vertical alignment characteristics of four-lane highways were also collected, more emphasis was placed in the diversification of the cross-section dimensions and the access density. In addition, more emphasis was placed in selecting segments located in suburban locations than in rural locations. The design of curves in four-lane highways is more uniform than in two-lane rural highways promoting better speed consistency; therefore spots in four-lane highways where speed changes are forced by adverse curvature conditions are minimal. Segments containing sharp curves were found mostly in urban areas, with posted speed limits lower than 40 mph (60 km/h) or located too close to traffic signals or stop signs. Not enough sites were located in horizontal curves or transition sections to be able to calibrate speed models for those locations in four-lane highways.

CHAPTER 4. DATA COLLECTION IN TWO-LANE RURAL HIGHWAYS

This chapter describes the segment selection and the data collection process used in two-lane rural highway segments. The scope of the data collection process was to collect free-flow speeds and geometry information on highway segments that can be considered by drivers to have a reasonably low crash rate. Figure 4-1 presents a flowchart with the segment selection and the data collection procedures followed for two-lane rural highways. The segment selection process describes the selection criteria and the evaluation of the crash data. The data collection process describes the measurement of the highway geometry characteristics and free-flow speeds. Also, the results from a preliminary analysis performed on the data are discussed in this chapter.

4.1. Data requirements

Highway maps and aerial photos were used to identify candidate highway segments based on their alignment and location. Crash data and traffic volume databases were required to determine the crash exposure rare of those candidate segments. A variety of segment characteristics and horizontal and vertical alignment characteristics were measured on the field to be used as potential explanatory variables in the modeling process. Characteristics like the cross-section dimensions, the roadside clear zone, the highway grade, the sight distance and the posted speed limit were targeted on tangent segments. Horizontal curve components like the degree of curvature and the maximum superelevation rate were targeted on horizontal curves and in transition sections.

4.2. Identification of candidate highway segments

The primary objective of the segment selection process was to include as many highway configurations, based on the cross-section dimensions, the roadside clear zone, the available sight distance, and other geometric features, as possible. Straight highway segments containing vertical curves and segments containing sharp horizontal curves were highly desired.

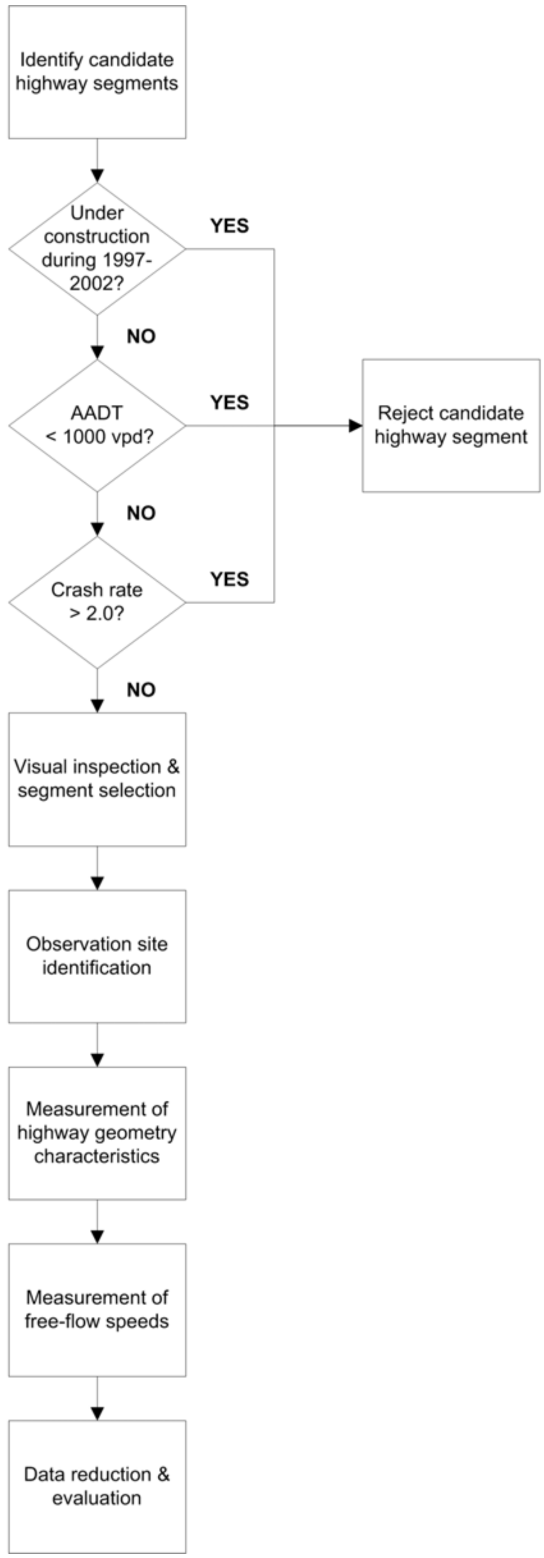


Figure 4-1 Site selection and data collection procedures for two-lane rural highways

The first step in the selection process was the use of highway maps and aerial photos to identify candidate highway segments for data collection based on their horizontal alignment and location. Only U.S. and state highways were evaluated. Interstate highways and local roads were excluded. Table 4-1 presents the general criteria used to identify candidate highway segments. Highway segments with a posted speed limit of 55 mph (90 km/h) were highly desired to capture the effects of the different geometry features on free-flow speeds and to minimize the effect on speeds due to the posted speed limit. For that reason, segments located in rural areas and exhibiting sharp horizontal curvature were targeted as potential candidates. Around a hundred highway segments from the north central and south central regions of Indiana were selected for further review.

Table 4-1 General selection criteria for two-lane rural highway segments

Segment characteristic	Criteria
Terrain	All types
Roadway type	Arterial to collector
Development type	Preferably no development
Pavement surface	PCC to AC
Posted speed limit	55 mph preferred, at least 40 mph (60 km/h)
Annual Average Daily Traffic	Higher than 1000 vpd
Segment length	At least 0.5 mile
Traffic control	No stop signs or traffic signals

The second step was to discard all highway segments involved in reconstruction projects during the years 1997 to 2002. A database of INDOT highway projects was used for this purpose. These segments were excluded to avoid inconsistencies with the crash data and to avoid conflicts during the data collection on the field. A total of 60 non-modernized and 28 modernized highway segments were identified as potential candidates for data collection. These highway segments were divided into 244 shorter sub-segments. Traffic volumes for all sub-segments were obtained from AADT county maps.

Segments with an AADT lower than 1000 vehicles per day were excluded to expedite the speed data collection in the field. Highway maps were used to determine the length of the sub-segments and to identify the names of the intersecting roads inside the sub-segments. The average length of the sub-segments was 1 mile, although shorter and longer segments were also used depending on the location of the intersecting roads at the ends of each sub-segment.

4.3. Calculation of crash occurrence and crash rates

Crash counts from the years 1997 to 1999 were collected to calculate the crash exposure rate for all the highway sub-segments. The objective of determining the crash rate was to exclude those highway segments with likely misperception of the risk which would lead to an excessive crash risk. The crash rate is a measure of the crash risk and was used in this study to identify locations with high crash rates. An upper limit was established for the crash rate for this purpose. The crash rate provides a more balanced measurement of the crash risk in a highway segment than the crash frequency. The crash frequency is highly related to the traffic volume; and consequently, it is very likely that a high-volume segment will also have a high crash frequency because of the higher interaction between vehicles.

It is important to note that the estimated crash rates used to classify the segments as low or high crash locations are representative only of the entire sub-segment, and they are not representative of the safety level of any individual feature (intersection, horizontal curve, etc.), or any combination of those, present on the sub-segment. The available crash database lacks detailed information about the location of the crash to be able to calculate crash rates for specific features like vertical or horizontal curves. In addition, it is not uncommon to find a crash record without a direction or a distance from a reference intersection. The crash database does not provide the information needed to evaluate the impact or the relation that the highway geometry had in a specific crash. Most of the crash contributory information in the records is aimed at the condition of the pavement surface, the drivers or the vehicles.

Crash counts for all sub-segments were obtained by identifying the pseudo code for the main road and for all the crossing roads in the sub-segment, including all county and local roads. Two issues were encountered during the identification of pseudo codes: not all local roads had pseudo codes assigned in the database and not all local names were displayed in the highway maps. The first issue has no major impact because most crashes are referenced to major highways or county roads which are easier to identify. The sub-segments were divided in such a way to ensure that all intersecting roads at the ends of the sub-segments have pseudo codes assigned. Sub-segments with no assigned pseudo codes for the crossing roads were either joined to the adjacent sub-segment or excluded from consideration. The second issue is more critical for urban and suburban highways. It is highly unusual for local or county roads in rural areas to have more than one assigned name.

The total number of crashes and the annual vehicle-miles traveled (VMT) were calculated for 197 sub-segments. The annual VMT value was calculated by multiplying the sub-segment length with the AADT and 365 days per year. The crash rate was calculated by dividing the average number of crashes per year by the annual VMT. Figure 4-2 shows the crash rate for the sub-segments and the upper limit used to identify the low crash rate locations. The upper limit value of 2.0 crashes per million VMT represents the 72th percentile value in the sample. Fifty-six sub-segments with a crash rate higher than 2.0 were removed from the sample.

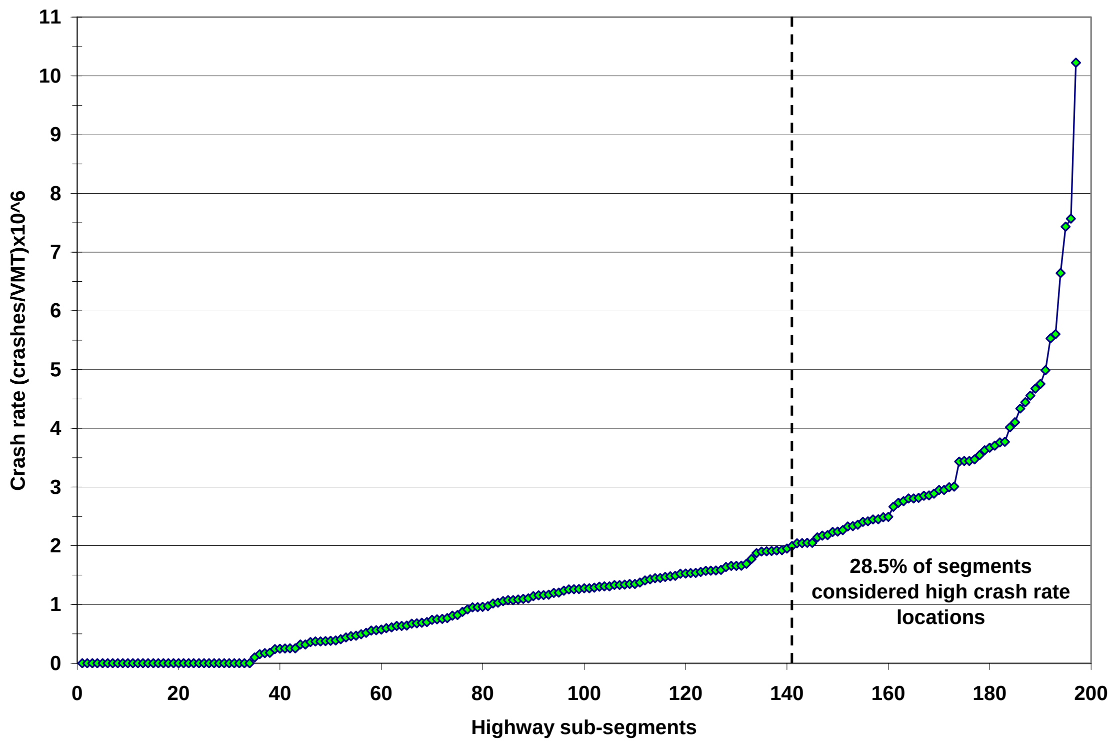


Figure 4-2 Crash rates for highway segments in two-lane rural highways

4.4. Test data collection

The field collection procedure and the speed-collecting equipment were evaluated during a test performed on March 13, 2002. The test was performed on a two-lane rural segment near milepost 37 of State Road 43 in White County. An observation site consisting of two spots located 200 ft (61 m) apart was used for the test. The site was located on a tangent section with the first spot located 155 ft (47m) past a short bridge in the northbound direction. This highway segment was selected because of its low volume, long sight distance and wide clear zone distance available on both directions.

Vehicle speeds were measured for both travel directions using four PEEK ADR-2000 traffic classifiers (one per spot) connected to rubber tubes, a radar gun, a digital video camera and a stopwatch. The clocks on all devices were synchronized in order to track down and identify specific vehicles going through the observation site. Timestamps were manually recorded along with the speeds measured with the radar gun. The time it took a vehicle to cross the speed trap was measured with the stopwatch. The digital video camera was located perpendicular to the spots to record the vehicles going through the observation site.

It was concluded from the test procedure that the distance between spots need to be increased to 300 ft (91 m) in order to be able to capture the change in vehicle speeds more accurately. The digital video camera and the stopwatch were discarded as equipment to measure speeds. Both devices calculate speeds indirectly by measuring the time that it takes a vehicle to traverse the speed trap. The stopwatch does not provide accurate results due to the high error put in by the observer. The digital video camera has a narrow field of view and needs a considerable long distance from the observation site to be able to record the vehicles going through the entire speed trap. Such distance will not be available in most two-lane rural highway locations.

One limitation of the radar gun used in the test was its inability to track down a particular vehicle in high-volume highways because of its wide radar wave. Also, the radar gun emits a constant radar wave which can be easily detected by radar detectors inside the vehicles. A Laser Atlanta ranging laser gun with vehicle tracking and ranging capabilities was obtained after the test. This laser gun has the capability of measuring speed and distance only when the trigger is held reducing the possibility of being detected. The laser gun is the preferred equipment to measure speeds in locations where it can be easily concealed from the drivers' sight. The use of the laser gun expedites the speed data collection compared to the traffic classifiers. The traffic classifiers are the preferred equipment to measure speeds in locations where the laser gun cannot be used. The traffic classifiers are less disruptive to drivers compared to the laser gun, but the setup of the classifiers and the placement and removal of the rubber tubes on the pavement increase substantially the speed collection time for each site.

4.5. Geometric data measurements

The next step taken in the data collection process was to carry out a visual inspection of the highway segments and select adequate observation sites with uniform cross-section dimensions and highway characteristics to measure free-flow speeds and the highway geometry. The data collection took place from May to August 2002. A total of 90 observation sites were selected in two-lane rural highways. Figure 4-3 shows the location of the selected segments in two-lane rural highways. More than one observation site was identified in most of the highway segments. The observation sites were located in eight different counties from the north central region of Indiana.

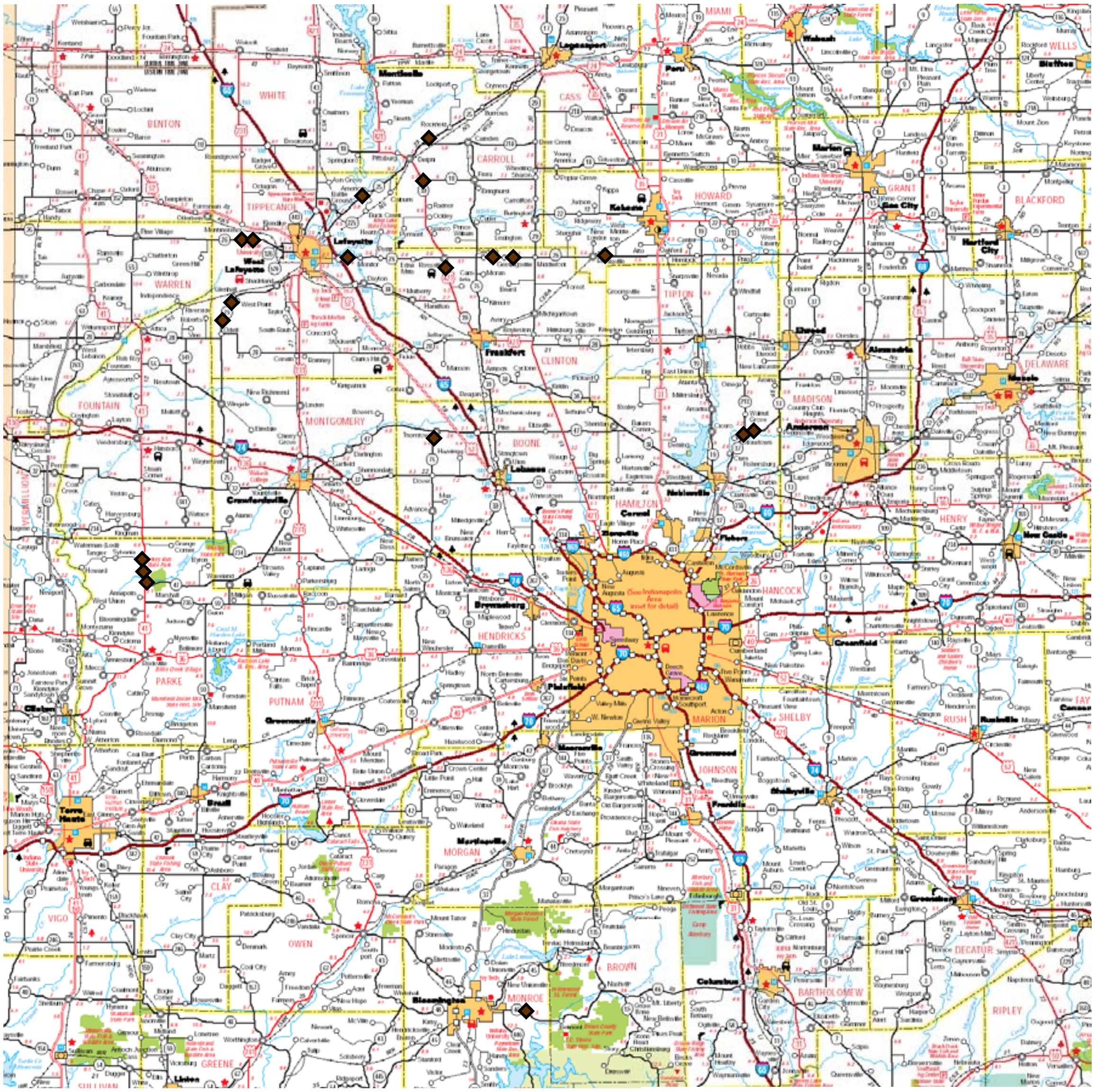


Figure 4-3 Location of the selected two-lane rural highway segments

The segment selection was performed carefully to capture as many cross-section dimensions, shoulder types, roadside clear zone and available sight distance, as possible. Only segments with pavement surface and markings in good condition were selected. Only segments with posted speed limits of 50 or 55 mph (80 or 90 km/h) were selected. Posted speed limits lower than 50 mph were observed in very short segments that serve mostly as transition to small towns or villages. Straight highway segments, as well as segments containing horizontal and vertical curves and intersections were selected. Highway segments containing sharp horizontal curves with advisory speed signs from 35 to 50 mph (55 to 80 km/h) were also included.

The following characteristics and geometric features were collected for each observation site:

- General characteristics: terrain type, pavement surface, level of residential development, and posted speed limit
- Tangents: grade, sight distance, cross-section dimensions, and roadside obstruction
- Horizontal curves: radius, maximum superelevation rate, length, and advisory speed
- Intersections: intersection type and presence of channelization and auxiliary lanes
- Distance from the spot to the beginning of horizontal curves and the middle of intersections, if present

Observation sites were selected on tangent segments, and before, inside and after horizontal curves, vertical curves and intersections. An observation site was defined by two spots located 300 ft apart. Figure 4-4 shows most of the segment characteristics and the cross-section dimensions measured in tangent segments. Appendix A presents a general description of the highway characteristics measured in two-lane rural highways. All the highway characteristics were defined based on the INDOT Highway Design Manual and the AASHTO Green Book. The field form used to record the site information in two-lane rural highways is shown in Appendix A.

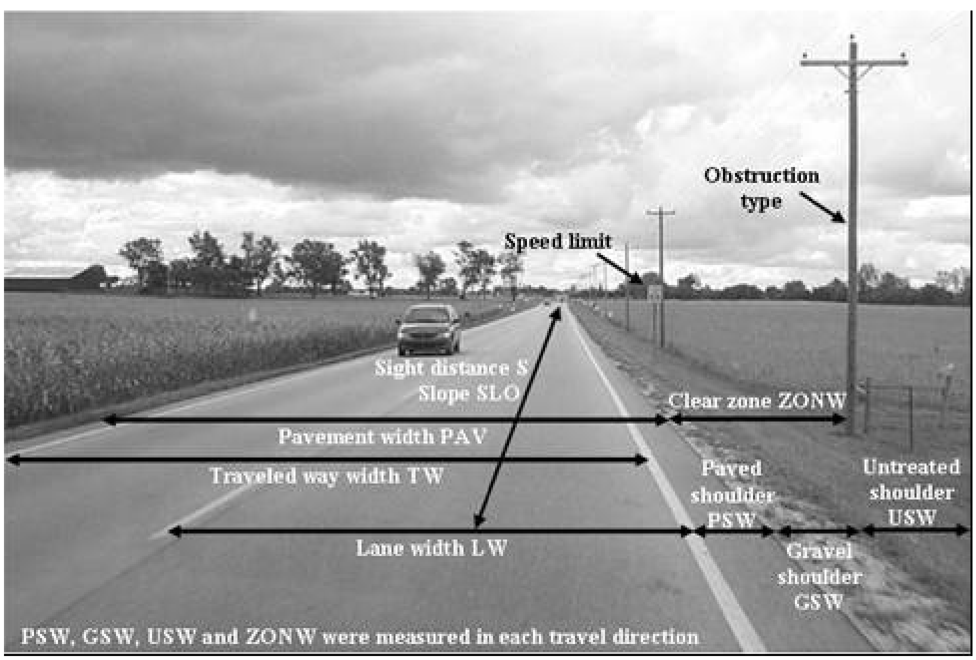


Figure 4-4 Characteristics measured in tangent segments in two-lane rural highways

The segment cross-section was divided into the traveled way and three traversable shoulder types: paved, gravel and untreated. The width of the shoulders was measured using the forgiving roadside concept of the AASHTO Roadside Design Guide (2002). Shoulders must be traversable with no objects likely to cause severe injuries when struck by a motorist. The width of the shoulders was measured for both travel directions. A roadside hazard rating was assigned to each site based on the categories developed by Zegeer et al. (1988). The roadside rating is a seven-point categorical scale from 1 (best) to 7 (worst). A general description for each rating is provided in Appendix A. Any local or isolated feature in the segment cross-section was ignored.

A measuring wheel with a one-inch precision was used to obtain the cross-section dimensions and the distance to intersections and horizontal curves. The ranging laser gun, with a 0.1 ft precision, was used to measure the sight distance at each spot according to the AASHTO standard for stopping sight distance. The middle ordinate of a 100 ft (30.5 m) chord was measured on the pavement edge to estimate the radius of the curves. An electronic level with a 0.1 ft/ft precision was used to measure the highway grade and the maximum superelevation rate of the horizontal curves. The average maximum superelevation rate was calculated by taking two measurements in each travel lane at the midpoint of the horizontal curve.

4.6. Free-flow speed measurements

Speeds were recorded during daylight hours and favorable weather conditions (no heavy rain, no strong wind, and no fog). No speed collection was performed during speed enforcement activities. Headways of five seconds or more were used to identify free-flow vehicles. Speeds were measured for both travel directions. The minimum number of free-flow speed observations taken at any site was 100. Speeds were collected with PEEK ADR-2000 traffic classifiers connected to rubber tubes or with a Laser Atlanta ranging laser gun.

An observation site was composed of two spots located 300 ft (91.4 m) apart. The speeds measured with the traffic classifiers were recorded in separate per-vehicle records (PVR). One traffic classifier was used for two different spots (one in each direction). Figure 4-5 presents the setup of an observation site using the traffic classifiers. Each spot was composed of two 100 ft (30.5 m) long rubber tubes located 16 ft (4.9 m) apart. The middle of the rubber tubes was positioned at approximately the highway centerline. The clocks of the classifiers were synchronized to be able to track down specific vehicles going through the observation site.

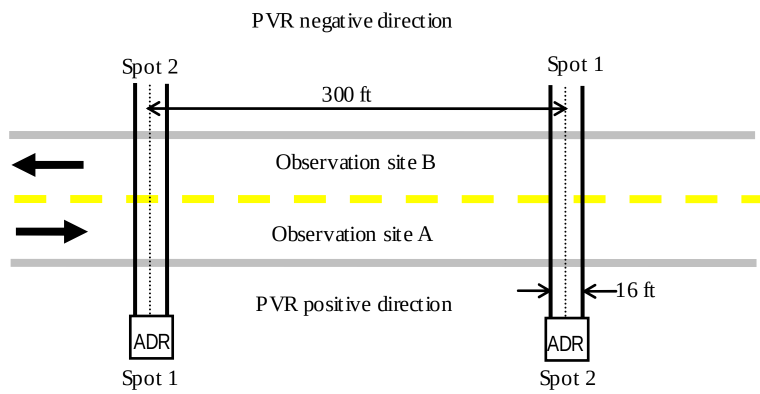


Figure 4-5 Setup of an observation site with traffic classifiers

The PVR data files were reviewed to identify individual free-flowing vehicles and to check inconsistencies in the number of axles and vehicle class. The vehicle class was recorded using the FHWA vehicle classification scheme F. The classification scheme is shown in Appendix A. Motorcycles (class 1) were excluded because they are not considered typical vehicles in highway design. Slow moving vehicles, agricultural machines and postal vehicles going at less than 25 mph (40 km/h), vehicles showing a difference in speed between the two spots of 20 mph (32.2 km/h) or more (vehicles exiting or entering the highway, emergency stops, etc.), and vehicles operating atypically were removed from the data files.

The laser gun was used to measure speeds in segments where the laser gun could be easily concealed from the drivers' sight. The setup of the observation sites was the same as the one shown in Figure 4-5, except that small cones were placed at the end of the shoulders to locate the spots with the laser gun. The speeds measured with the laser gun were adjusted to take into account the angle correction. If the laser gun is not located directly in the vehicle path the measured speed by the laser is lower than the actual vehicle speed. Appendix A shows the layout and the equation used to determine the angle error correction. The target range and the distance from the laser gun to the centerline of the travel lane were measured to adjust the speeds accordingly.

4.7. Summary of highway characteristics and free-flow speeds

The data set is composed of highway characteristics and free-flow speeds for 158 spots. Table 4-2 presents descriptive statistics for some of the observed highway characteristics. The data set was initially divided into 32 spots located on tangent segments free from the influence of horizontal curves, 20 spots located on horizontal curves and 106 spots located in curve transition sections. Although speeds were measured for 180 spots (90 observation sites), the sight distance was not recorded for 22 spots, reducing the sample to 158 spots.

Table 4-2 Descriptive statistics for characteristics in two-lane rural highways

Highway characteristic	Mean	Std. deviation	Minimum	Maximum
Posted speed limit (PSL), mph	54.24	1.80	50.00	55.00
Advisory speed limit (ADV), mph	42.63	4.72	35.00	50.00
Percent trucks (T)	13.52	5.76	3.00	30.00
Sight distance (SD), ft	910.14	475.50	225.55	2179.70
Highway grade (G), percent	-0.11	2.32	-7.10	6.30
Lane width (LW), ft	11.63	0.81	9.25	13.30
Traveled way width (TW), ft	23.27	1.42	18.75	25.67
Pavement width (PAV), ft	28.45	5.96	18.75	44.33
Paved shoulder width (PSW), ft	5.19	5.29	0.00	21.00
Gravel shoulder width (GSW), ft	2.39	2.98	0.00	8.25
Untreated shoulder width (USW), ft	28.48	19.49	0.00	71.00
Clear zone distance (CLR), ft	36.06	20.29	4.83	79.25
Roadside hazard category (RD)	2.87	1.56	1.00	7.00
Degree of curvature (DC), degrees	7.07	3.75	0.86	16.34
Curve radius (R), ft	1244.14	1228.11	350.70	6677.59
Maximum superelevation (SE), percent	6.44	2.36	0.25	10.80
Curve length (HCLEN), ft	796.78	489.31	250.00	3002.00
Mean speed, mph	54.11	5.38	39.98	63.00
85^{th} percentile speed, mph	59.06	5.15	46.02	68.00

The observed mean free-flow speeds have a range of 23 mph (37 km/h) even though the selected highway segments have posted speed limits of 50 and 55 mph (80 and 90 km/h). The observed 85^{th} percentile speeds have a similar range of 22 mph (35.4 km/h). Twenty-four spots were located in highway segments with a posted speed limit of 50 mph. Advisory speed signs for horizontal curves were present in 76 spots varying from 35 to 50 mph (55 to 80 km/h). The large variability of the observed speeds, compared to the small variability in posted speed limits, might be an indication that the geometric components are significant factors of operating speeds in two-lane rural highways.

The AASHTO design guide recommends that vehicles of different sizes and weights with different operating characteristics should be considered in highway design. According to AASHTO, vehicles can be classified as either passenger cars or trucks (class 4 or higher) for uninterrupted traffic flow in rural areas. The average percentage of trucks in the free-flow speed data set is 13.5 percent; however, the percentage per site varies substantially from 3 to 30 percent. The observed speeds from five observation sites were evaluated to examine if there is a significant difference in speeds and speed variance between trucks and other vehicle types. The results of this comparison are presented in Section 4.9.

The amount of available sight distance in a segment is of the utmost importance in the safe and efficient operation of a vehicle on a highway. A wide range of 1954 ft (595.6 m) in sight distance values was obtained for the data set. The sight distance is a function of the terrain type and the change in the horizontal and vertical alignments. Sixteen sites were classified in semi-mountainous terrain, forty-eight sites were classified in rolling terrain and the rest were classified in level terrain. The highway grade also displays a large range of 13.4 percent. Although the grade length was not measured, it was observed that none of the upgrades had the sufficient length to make trucks operate at crawl speeds.

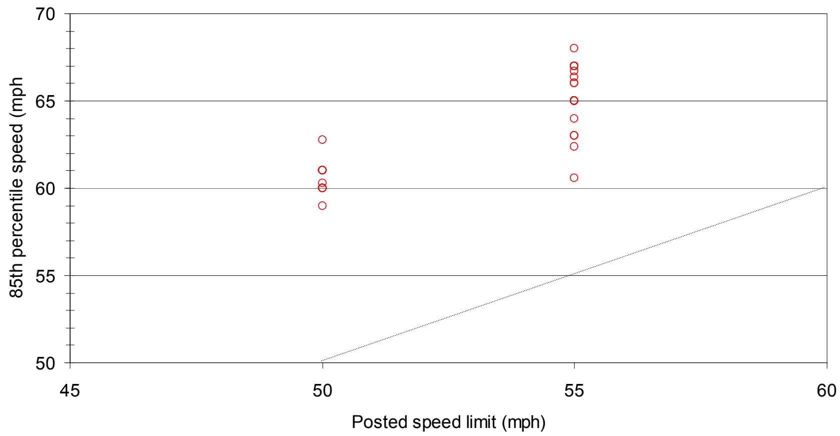
According to AASHTO, the functional advantage of providing access control is the management of the interference with through traffic preserving or improving service and safety. It is well established that a positive correlation exists between the access density and the number of crashes. The intensity of the residential development in the highway segment was recorded by considering the driveway density. Thirty-four spots were located in highway segments having more than 10 residential driveways per mile.

The data set contains a large variability of cross-section dimensions, especially for the three shoulder types. According to AASHTO, the width of the lanes and the shoulders influence the highway level of service, safety and comfort of driving. The range in lane width values is 4.05 ft (1.23 m) and the range in clear zone distance, composed of the width of the three shoulders, is 74.4 ft (22.7 m). All seven roadside hazard ratings were observed in the field. Diverse configurations of shoulder widths and types were included; varying from segments having all three shoulder types and more than 37 ft (11 m) in clear zone in each direction to segments containing a narrow shoulder of 4.8 ft (1.5 m) followed by a guardrail in each direction. The following roadside obstructions were recorded: guardrails, pole line, ditches, and embankments. In some segments there was no definite roadside obstruction between the traveled way and the end of the right-of-way. Fifteen spots were recorded as having guardrails as the roadside obstruction, twelve spots as having pole lines, forty-five spots as having ditches and ten spots as having embankments. Segments with medians, curbs, sidewalks, on-street parking, bus turnouts, bicycle lanes or frontage roads were not observed during the data collection.

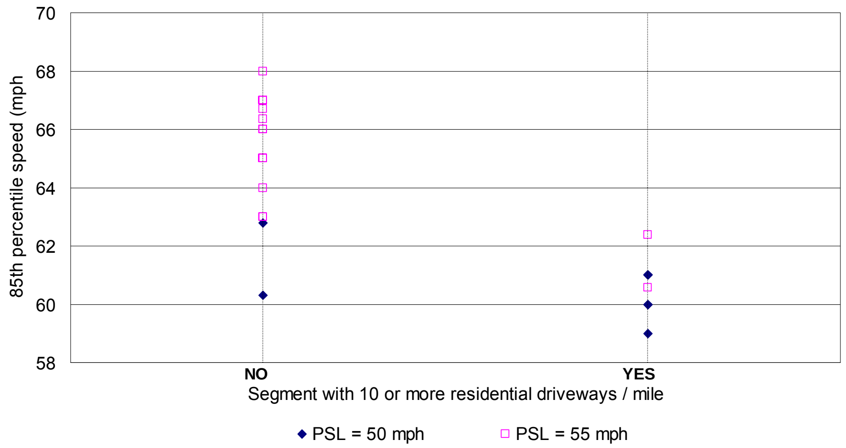
The AASHTO design guide recommends that the design of curves should be based on an appropriate relationship between the design speed and the curvature and their joint relationships with the superelevation rate and the side friction. Although the difference in speed limits is only 5 mph (10 km/h), the observed variability in curvature is very high. Twenty-eight different horizontal curves were observed. The degree of curvature has a range of 15.48 degrees, equivalent to a range in curve radii of 6,326.9 ft (1,928.4 m). The large variability in curvature serves as evidence of the design consistency issues present in Indiana two-lane rural highways. According to AASHTO, design consistency relates to the uniformity of the highway alignment and its associated design element dimensions. A more consistent alignment for similar roadway types promotes lower driver workload and safer conditions as drivers' expectancy is met. The maximum superelevation rate and the curve length also have large variability, 10.55 percent and 2,752 ft (838.8 m), respectively. The selection of the maximum superelevation rate depends of the climate and terrain conditions, the area type and the frequency of slow vehicles. AASHTO recommends a maximum superelevation rate of 8 percent when snow and ice are present. Although both factors are significant for Indiana highway conditions, five different horizontal curves exceed the recommended maximum superelevation rate under those conditions.

4.8. Trends between observed operating speeds and highway characteristics

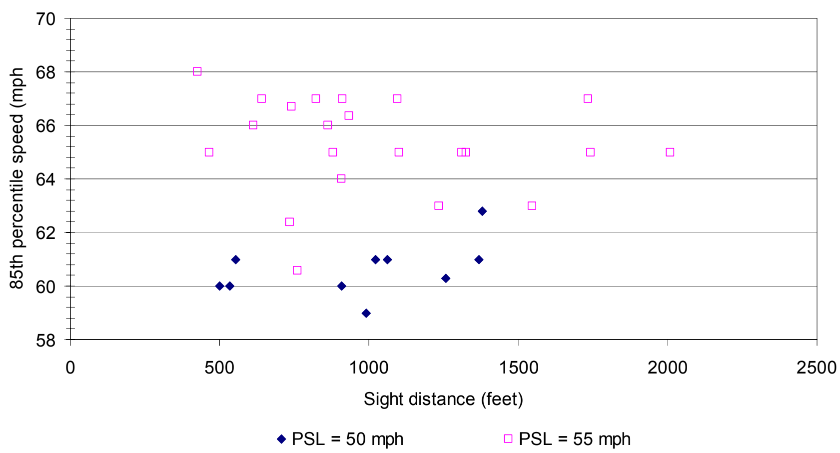
The following section presents a graphical analysis that shows trends between the observed 85th percentile speeds and different characteristics of two-lane rural highway segments. AASHTO defines operating speed as the speed at which drivers are observed operating their vehicles during free-flow conditions. The 85th percentile speed of the observed free-flow speed distribution is typically used to represent the operating speed in a highway segment. This analysis is useful to identify relationships between speeds and highway characteristics. Figure 4-6 presents trends between three segment characteristics and the observed 85th percentile speeds. Figure 4-7 presents trends between three cross-section dimensions in tangent segments and the observed 85th percentile speeds. Figure 4-8 presents trends between two horizontal curve components and the observed 85th percentile speeds. The graphs make a distinction between different posted speed limits and advisory speeds.



a) Posted speed limit



b) Residential development



c) Sight distance

Figure 4-6 Trends between segment characteristics and operating speeds in two-lane highways

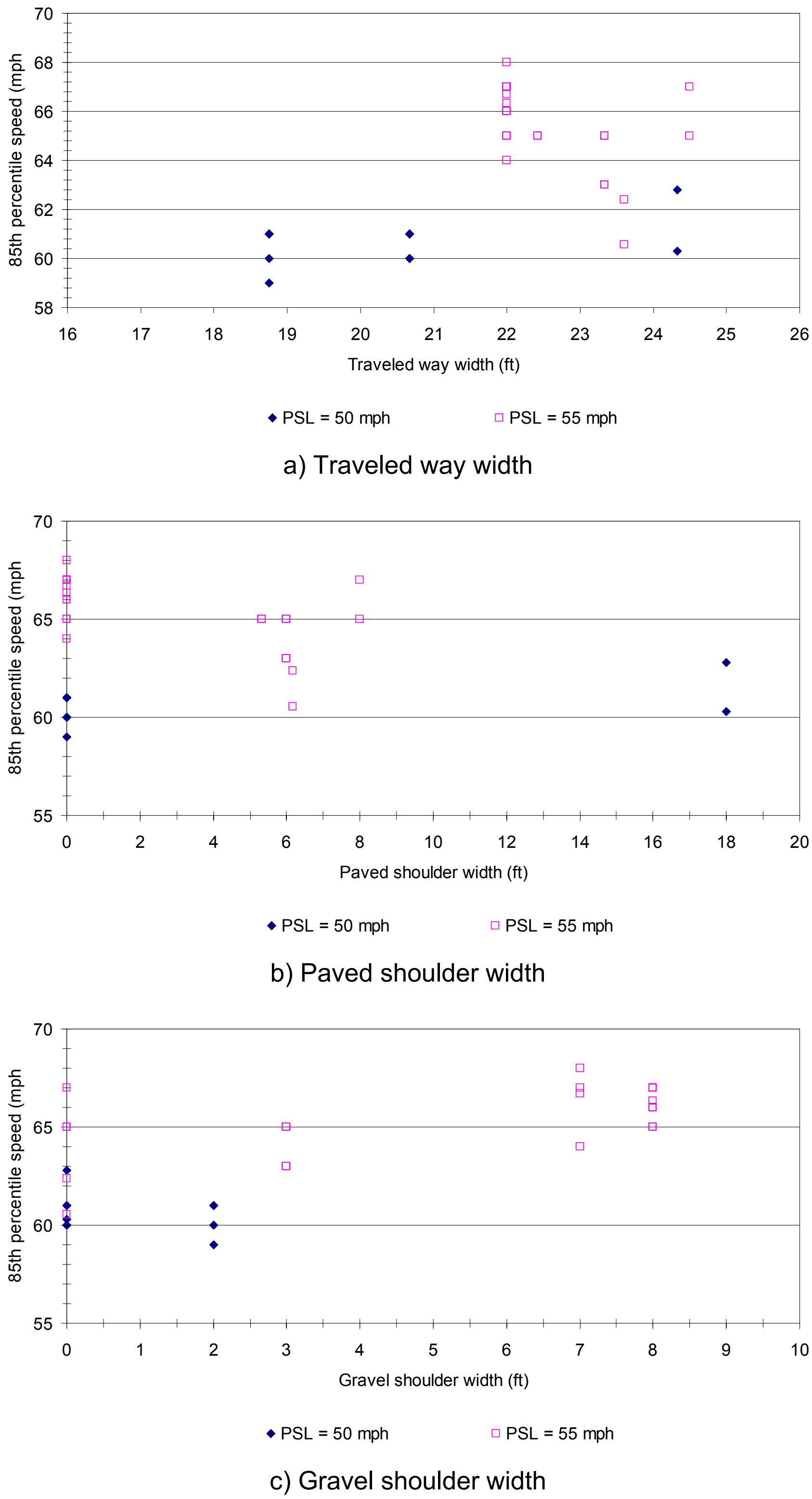
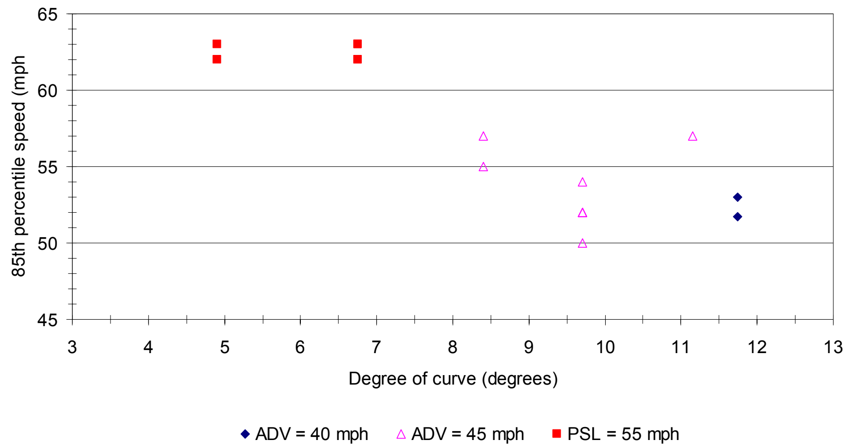
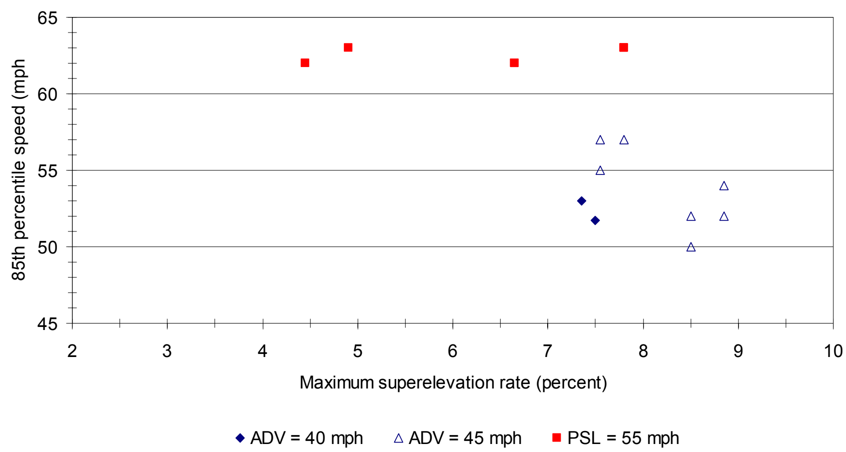


Figure 4-7 Trends between cross-section dimensions and operating speeds in two-lane highways



a) Degree of curve



b) Maximum superelevation rate

Figure 4-8 Trends between curve components and operating speeds in two-lane highways

The posted speed limit (Figure 4-6a) and the presence of high residential development (Figure 4-6b) show the strongest trend with the operating speeds on tangent segments. As expected, a reduction in the posted speed limit decreases the operating speed on the segment. These results are consistent with those found in NCHRP Report 504. All sites have a mean speed higher than 53 mph (85.3 km/h) and an 85th percentile speed higher than 58 mph (93.3 km/h). The posted speed limit on these tangent segments was either 50 or 55 mph (80 to 90 km/h). The observed operating speeds are higher than the posted speed limits in a range of 5.6 to 13 mph (9 to 20.9 km/h). It is interesting that the range of the observed operating speeds for segments with the 55-mph speed limit (7.4 mph) is almost twice the range observed for segments with the 50-mph limit (3.8 mph). This might indicate the effect of other highway characteristics on speeds in tangent segments. One of those factors might be the level of the residential development. The trend indicates that the operating speeds decrease with the presence of high residential development, regardless of the speed limit. The sight distance (Figure 4-6c) appears to have a weak positive relation with the operating speeds when comparing sites with the same speed limit. Very interesting is the fact that the speed variability decreases as the sight distance increases.

The trends observed in Figure 4-7 are peculiar. Increasing the width of the gravel shoulder appears to have a positive effect on speeds when comparing only those sites having a gravel shoulder. The traveled way width and the paved shoulder width do not seem to have an obvious trend with the operating speeds when all sites are analyzed together. This trend was also reported in NCHRP Report 504 for the lane width. When the sites are analyzed taking into consideration the posted speed limit, some trends can be observed. The operating speeds appear to slightly increase with increasing traveled way width (Figure 4-7a) and paved shoulder width (Figure 4-7b) for sites on segments with a 50-mph speed limit and with increasing gravel shoulder width (Figure 4-7c) for sites on segments with a 55-mph speed limit. It is interesting to observe that speeds slightly decrease with increasing gravel shoulder width for sites on segments with a 50-mph speed limit and that speeds are lower for middle values of the traveled way width and the paved shoulder widths on segments with a 55-mph speed limit. A possible explanation for this behavior might be that additional factors are influencing the speeds at those sites or that a relationship exists between some of the cross-section dimensions and the posted speed limits.

The graphs in Figure 4-8 differentiate between sites located in curves with two different advisory speeds from sites located in curves without advisory speeds. All the sites shown in Figure 4-8 are located in segments with a 55-mph (90 km/h) posted speed limit. The degree of curve (Figure 4-8a) and the maximum superelevation rate (Figure 4-8b) show a comparable strong trend with the operating speed. As expected, a reduction in operating speeds occurs on curves with advisory speeds, representative of the increasing curvature. The advisory speed signs are generally used when the curve design is not compatible with the posted speed limit on the segment. Likewise, increasing superelevation rates decrease operating speeds; although the relation between the superelevation rate and the degree of curve need to be evaluated also. A more detailed analysis of the relationships between the highway characteristics and the observed speeds is provided in Chapter 6 using the results of the speed model calibration.

4.9. Speed comparison between different vehicle classes

Individual speed observations from five sites were analyzed to compare the speeds of different vehicle classes. The observed speeds at the selected sites are influenced by different alignment conditions: curve transition sections, a sharp horizontal curve and a tangent segment. The purpose of this comparison was to determine if the speeds from different vehicle classes can be treated as one sample. The selected observation sites are located on flat segments with no vertical curves.

The selected observation sites and their main characteristics are the following:

- 006-075-001: deceleration transition section of horizontal curve with 776 ft (236.5 m) radii
- 006-075-002: acceleration transition section of horizontal curve with 776 ft (236.5 m) radii
- 053-046-001: inside horizontal curve with 682 ft (208 m) radii
- 012-026-013: on tangent with first spot located 106 ft (323 m) after a 4-leg intersection
- 012-026-014: on tangent with first spot located 406 ft (124 m) before a 4-leg intersection

The speed variance and the mean and 85^{th} percentile speeds were calculated for passenger cars (class 2), pick-up trucks (class 3) and all truck classes (classes 4 to 12) for both spots of the observation site. The truck class included buses, single-unit trucks, and combination trucks. Tables 4-3 to 4-7 present the speeds and variance per vehicle class for the observation sites.

Table 4-3 Speed and variance per vehicle class for site 006-075-001

		Speed variance		Mean speed (mph)		85^{th} percentile speed (mph)	
Vehicle class	Count	Spot 1	Spot 2	Spot 1	Spot 2	Spot 1	Spot 2
2	143	33.943	32.212	53.9	47.3	59.0	53.0
3	59	28.769	25.874	53.5	47.8	59.0	53.0
2-3 combined	202	32.301	30.259	53.8	47.5	59.0	53.0
4-9	17	57.610	51.816	52.9	47.8	59.6	54.5

Table 4-4 Speed and variance per vehicle class for site 006-075-002

		Speed variance		Mean speed (mph)		85^{th} percentile speed (mph)	
Vehicle class	Count	Spot 1	Spot 2	Spot 1	Spot 2	Spot 1	Spot 2
2	143	45.531	32.236	49.4	52.5	56.0	58.0
3	57	50.777	33.938	47.6	50.8	55.5	57.0
2-3 combined	200	47.472	33.210	48.9	52.0	56.0	58.0
4-9	17	29.610	29.441	48.1	50.8	53.2	55.8

Table 4-5 Speed and variance per vehicle class for site 053-046-001

		Speed variance		Mean speed (mph)		85th percentile speed (mph)	
Vehicle class	Count	Spot 1	Spot 2	Spot 1	Spot 2	Spot 1	Spot 2
2	210	25.464	22.550	52.3	52.7	57.0	57.0
3	53	25.671	22.407	52.6	53.7	57.0	58.2
2-3 combined	263	25.419	22.375	52.4	52.9	57.0	57.0
4-10	17	12.375	10.559	52.0	53.1	55.0	56.0

Table 4-6 Speed and variance per vehicle class for site 012-026-013

		Speed variance		Mean speed (mph)		85th percentile speed (mph)	
Vehicle class	Count	Spot 1	Spot 2	Spot 1	Spot 2	Spot 1	Spot 2
2	303	40.510	34.620	62.6	62.4	68.0	67.7
3	110	30.830	29.430	61.7	61.5	66.0	65.0
2-3 combined	413	37.99	33.32	62.3	62.1	68.0	67.0
4-10	112	23.00	19.03	60.9	60.9	65.0	65.0

Table 4-7 Speed and variance per vehicle class for site 012-026-014

		Speed variance		Mean speed (mph)		85th percentile speed (mph)	
Vehicle class	Count	Spot 1	Spot 2	Spot 1	Spot 2	Spot 1	Spot 2
2	349	27.480	34.650	61.4	61.2	66.0	66.0
3	101	38.055	44.707	61.8	61.6	67.0	66.0
2-3 combined	450	29.805	36.842	61.5	61.3	66.0	66.0
4-9	135	14.368	20.540	59.9	59.8	63.0	63.0

Two statistical tests were performed to evaluate if a significant difference existed between the mean speeds and the speed variance of different vehicle classes. The comparison between passenger cars and pick-up trucks was performed first. If no significant difference exists between the two mean speeds and the two speed variances of these vehicle classes, this will indicate that the two classes can be combined as one sample, as AASHTO recommends for rural areas. The second comparison was between the combined classes 2 and 3 and all truck classes (4 to 12). If no significant difference exists between the two mean speeds and the two speed variances of these vehicle classes, then all classes can be combined as one sample. A comparison between truck classes was not performed because of the low count for most truck classes.

Random independent samples drawn from two populations can be used to test if two population means are equal. If the samples are large enough, e.g. more than 25 observations, then the distribution of their means can be assumed to be normally distributed using the central limit theorem (Washington et al., 2003). The null hypothesis H_0 in the two-tailed test states that the two population means μ_1 and μ_2 are equal. The test statistic is calculated as:

$$Z = \frac{(\overline{X}_1 - \overline{X}_2) - (\mu_1 - \mu_2)}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}}$$

where (μ_1 - μ_2) is equal to zero under the null hypothesis, ($\overline{X}_1$ - $\overline{X}_2$) is the actual difference in the sample means, s^2_1 and s^2_2 are the sample variances and n_1 and n_2 are the number of observations in the two samples. The denominator in the previous equation is the standard error of the difference between the two sample means and requires two independent samples. The confidence interval for the difference in means is t-distributed with degrees of freedom equal to ($n_1 + n_2 - 2$). The null hypothesis is rejected when the calculated test statistic is larger than the critical value obtained from the *t* distribution tables.

Random independent samples drawn from two populations can be used to test if two population variances are equal. The null hypothesis H_0 in the two-tailed test states that the two variances σ_1^2 and σ_2^2 are equal. The test statistic is calculated as:

$$F_{(n_1-1, n_2-1)} = \frac{s_1^2}{s_2^2}$$

where $F_{(n1\ -1, n2\ -\ 1)}$ is an F-distributed random variable with (n_1 – 1) degrees of freedom in the numerator and (n_2 – 1) degrees of freedom in the denominator. The larger sample variance is placed in the numerator of the previous equation. The null hypothesis is rejected when the calculated test statistic is larger than the critical value obtained from the *F* distribution tables. Both hypothesis tests were performed with a 95 percent confidence level.

The mean speeds for all three vehicle samples in both spots were found to be equal for site 006-075-001 in the deceleration transition section. The speed variance for passenger cars and pick-up trucks in both spots were found to be equal. The speed variance for trucks classes (57.6 for spot 1 and 51.8 for spot 2) were found to be not equal to the variance for combined passenger cars and pick-up trucks (32.3 for spot 1 and 30.2 for spot 2). The null hypothesis for these tests was not accepted by a slight margin. The result of the higher speed variance for truck classes is reflected in the 85th percentile speeds, which are slightly higher than for passenger cars and pick-up trucks. The basis for the higher variance for trucks might be the difference in the available sight distance. Truck drivers are able to see substantially farther down the road because of the higher position of the driver's eye compared to the drivers of the other two vehicle classes.

All mean speeds and variances were found to be statistically equal for the three samples of vehicle classes in site 006-075-001 in the acceleration transition section. In this case, the 85th percentile speed for truck classes (53.2 mph and 55.8 mph) were slightly lower than for the other two classes (56.0 mph and 58.0 mph). The curve is followed by a long tangent segment with no immediate changes in the vertical alignment; therefore the small discrepancy might be due primarily to the different acceleration performance between trucks and the other two vehicle classes.

The mean speeds for all three vehicle samples were found to be statistically equal for both spots of site 053-046-001 inside a horizontal curve. The speed variance for passenger cars and pick-up trucks in both spots were also found to be equal. The speed variance for trucks classes (12.4 for spot 1 and 10.6 for spot 2) were found to be not equal to the variance for combined passenger cars and pick-up trucks (25.4 for spot 1 and 22.4 for spot 2). The null hypothesis for these tests was not accepted by a slight margin. The result of the lower speed variance for truck classes is reflected in the 85th percentile speeds, which are slightly lower than for passenger cars and pick-up trucks. The basis for this behavior might be explained by the difference in vehicle performance. The site was located inside a 2275-ft (693.4 m) long curve with a 682.0 ft (207.9 m) radii and a 7.8 percent superelevation rate. Truck drivers, especially semi-trailer truck drivers, might be more cautious while negotiating a sharp curve with high superelevation to avoid the possibility of roll over or skidding.

The mean speeds of passenger cars and pick-up trucks were found to be statistically equal for the two sites 012-026-013 and 012-026-014 in tangent segments. However, the variances for these two vehicle classes were found to be not equal for the first spot in site 012-026-014 by just a slight margin. The variances for the other three spots were found to be statistically equal between the two samples. The mean speeds and variances of trucks and the combined sample of passenger cars and pick-up trucks were found to be statistically not equal for the two sites. In practical terms, the biggest difference in speeds between the vehicle classes is no more than 3 mph (4.8 km/h). The percentage of trucks in the free-flow traffic sample of these two sites is fairly high at around 22%. There are no immediate changes in the vertical alignment nearby these two sites and sight distances are longer than 800 ft (243.8 m) at all spots. There is no simple explanation for the difference in behavior between the vehicle classes, except to take notice of the difference in braking performance and the probable difference in the drivers' risk perception due to the presence of the 4-leg intersection. Drivers of large trucks might be more cautious when approaching a 4-leg intersection than drivers of smaller vehicles due to the difference in braking performance.

The results of the statistical tests do not provide a definite conclusion that all vehicle classes do not behave similarly in different alignment situations. For most situations, the mean speeds and the speed variances for different vehicle classes can be assumed to be similar. In practical terms, the difference in mean speeds and 85^{th} percentile speeds in the selected sites was no more than 3 mph. Nevertheless, the truck percentage in the traffic flow of the observation sites will be evaluated as an explanatory variable in the modeling process to take into account any possible impact on speeds due to percentage of trucks in two-lane rural highway segments.

CHAPTER 5. DATA COLLECTION IN FOUR-LANE HIGHWAYS

This chapter describes the segment selection and the data collection process used in four-lane highway segments. A collection procedure similar to the one shown in Figure 4-1 for two-lane rural highways was followed. Some adjustments were made to expedite the data collection process. In addition, the results from a preliminary analysis of the collected data are discussed in this chapter.

5.1. Data requirements

The same databases used for two-lane rural highways were employed. Highway maps were required to identify candidate segments and the crash data and the traffic volume databases were required to determine the crash exposure rate of the candidate segments. The horizontal and vertical alignment characteristics were required for the modeling process, with more emphasis placed in the cross-section dimensions and the access density of the highway segments. The design components of horizontal curves were still required, but at a minor scale compared to the effort made for two-lane highways. The curvature design in four-lane highways is more consistent and uniform than in two-lane rural highways; therefore, spots in four-lane highways where speed changes are forced by adverse curvature conditions are minimal.

5.2. Identification of candidate highway segments

The primary objective of the segment selection process was to include as many highway configurations, based on the cross-section dimensions, posted speed limit, sight distance, development type, access density, and geometric features, as possible. Table 5-1 presents the general criteria used to identify candidate segments. Highway segments with a posted speed limit of 55 mph (90 km/h) were highly desired to capture the effects of the different geometry features on the free-flow speeds and to minimize the effect of different speed limits. Straight highway segments with varying cross-section dimensions and intersection and driveway access density, as well as segments with horizontal curves and sight distance restrictions were desired.

Table 5-1 General selection criteria for four-lane highway segments

Segment characteristic	Criteria
Terrain	All types
Location	Suburban to rural
Roadway type	Arterial to collector
Development type	Commercial, residential to no-development
Access control	Full, partial to no control
Pavement surface	PCC to AC
Median type	All types to undivided
Traffic control	No stop sign or traffic signal within 0.5 mile
Posted speed limit	55 mph preferred, at least 40 mph
Curbs	All types to no
Sidewalks	Yes to no
Segment length	At least 1 mile
Annual Average Daily Traffic	Higher than 1000 vpd

The first step in the selection process was the use of highway maps to identify candidate segments based on their location with respect to developed areas. Traffic signals were located on maps to avoid selecting segments having a high density of traffic signals. Only U.S. and state highways in rural and suburban areas were evaluated. Interstate highways and local roads were excluded. The second step was to discard all highway segments involved in reconstruction projects during the years 1997 to 2003 to avoid inconsistencies with the crash data and to avoid conflicts during the data collection in the field. Highway maps were used to determine the length of the sub-segments and to identify the names of the intersecting roads inside the sub-segments. The average length of the sub-segments is approximately 1 mile, although shorter and longer segments were also identified depending on the location of the intersecting roads. The order of the data collection process was altered to expedite the field measurements. The calculation of crash rates was deferred after the selection of highway segments and the collection of the speed and highway information.

5.3. Geometric data measurements

The next step carried out was the visual inspection of the highway segments and the selection of adequate observation sites to measure free-flow speeds and highway characteristics. An observation site was defined by one spot in four-lane highways. Only one spot was used to reduce the data collection time and to increase the number of observed spots. In addition, the second spot was not expected to show any significant deceleration or acceleration since the horizontal alignment in four-lane highways is more consistent than in two-lane rural highways.

The data collection took place from May to September 2003. A total of 67 observation sites were selected. Figure 5-1 shows the location of the selected segments in four-lane highways. More than one observation site was identified in most segments. The observation sites were located in thirteen different counties. The study area covered the central region of Indiana from Miami County in the north to Vigo County in the south. The speed and geometric data was collected on highway segments around large (e.g., Indianapolis, Terre Haute), medium (e.g., Lafayette, Kokomo) and small (e.g., Frankfort, Crawfordsville) scale cities.

The segment selection was performed carefully to capture as many cross-section dimensions, shoulder types, driveway and intersection density and sight distance, as possible. Only segments with pavement surface and markings in good condition were included. No geometric feature with advisory speed signs was found during the collection process. The observation sites were located on tangent segments, and before, inside and after horizontal curves and intersections.

A high diversity of geometric information related to the cross-section, intersections and horizontal curves was collected. Most of the definitions used for the highway characteristics in two-lane rural highways were applied also for four-lane highways. Appendix B presents a general description for the highway characteristics measured exclusively in four-lane highways. All the highway characteristics were defined based on the INDOT Highway Design Manual or the AASHTO Green Book. The field form used for recording the information in four-lane highways is shown in Appendix B.

The following highway characteristics and geometric features were collected:

- General characteristics: terrain type, rural vs. suburban location, pavement surface, and posted speed limit
- Access density: intersection density, driveway density, median opening density, and presence of residential or commercial developments
- Tangents: grade, sight distance, cross-section dimensions, and roadside obstruction
- Roadside features: obstruction type, and presence of auxiliary lanes or sidewalks
- Median: width, type, and surface, and presence of barrier, TWLT lane or auxiliary lanes
- Intersections: intersection type and presence of channelization and auxiliary lanes
- Horizontal curves: radius, maximum superelevation rate, and length
- Distance to the beginning of horizontal curves and the middle of intersections, if present

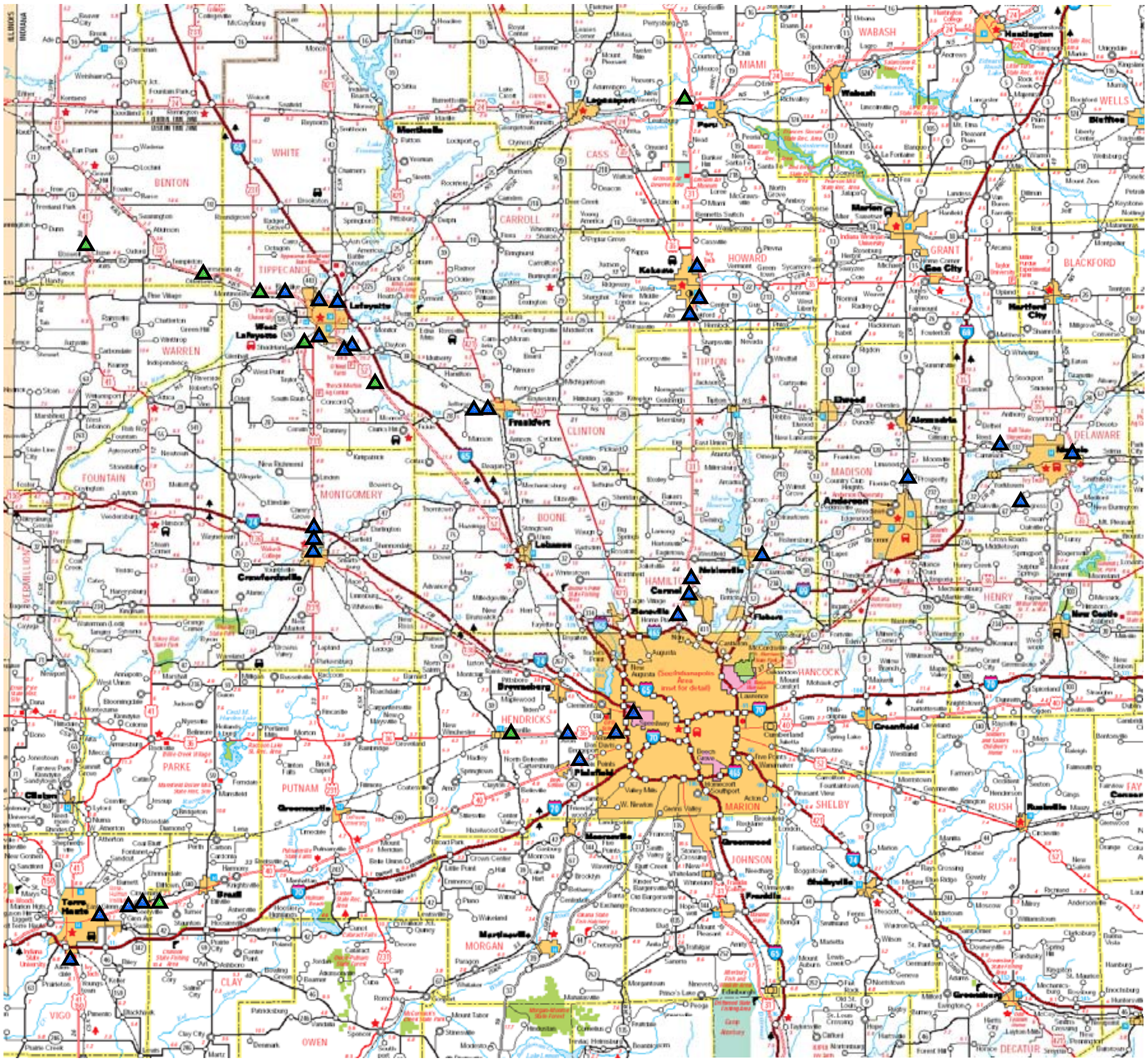


Figure 5-1 Location of selected segments in four-lane highways

Special consideration was given to the cross-section dimensions. The cross section was divided in three parts: the traveled way where speeds were measured, the opposing traveled way and the median. The same cross-section dimensions were measured for both travel directions. The width of the inside and the outside lane were measured separately. The roadside clear zone distance was divided into three traversable shoulder surface types: paved, gravel and untreated. The width of the shoulders was measured using the forgiving roadside concept from the AASHTO Roadside Design Guide (2002). The median width was also divided in three surface types. Any local or isolated feature in the cross-section, like culverts or short guardrails, was ignored.

The access density of the highway segment was estimated by counting the number of intersections and driveways located a quarter mile before and after each site. Three types of intersections were recorded: 4-leg, T and adjacent-T. An adjacent-T intersection has the minor approach leg in the same side as the lanes where speeds were measured. Any crossing road with a stop sign or stop bar was counted as an intersection; otherwise it was counted as a driveway. Driveways were counted separately for each direction. The presence of channelization or auxiliary lanes was also collected for the two closest intersections to the site.

A measuring wheel with a one-inch precision was used to obtain the cross-section dimensions and the distance to intersections and horizontal curves. A ranging laser with a 0.1 ft precision was used to measure the sight distance at each spot according to the AASHTO standard for stopping sight distance. The middle ordinate of a 100 ft (30.5 m) chord was measured on the pavement edge to estimate the radius of the curve. The average maximum superelevation rate was estimated by taking two measurements in each travel lane. An electronic level with a 0.1 ft/ft precision was used to measure the highway grade and the curve superelevation rate.

5.4. Free-flow speed measurements

Speeds were recorded on weekdays during daylight hours and favorable weather conditions (no heavy rain, no strong wind, and no fog). Headways of five seconds or more were used to identify free-flow vehicles. Speeds were collected with a Laser Atlanta laser gun or with rubber tubes connected to PEEK ADR-2000 traffic classifiers. The laser gun was used on locations where the laser gun could be easily concealed from the drivers' sight. Rubber tubes were used in rural highway segments or when the laser gun could not be used. The free-flow speeds were collected in spots located at least a quarter mile away from any traffic interruption like a stop sign or a traffic signal. The minimum number of free-flow speed observations taken at any site was 100.

The same data cleaning procedure used for two-lane highways was followed for four-lane highways. The vehicle class was recorded using the FHWA vehicle classification scheme F, shown in Appendix A. Emergency vehicles, motorcycles and, vehicles turning, braking or exhibiting unusual behavior were ignored. The speeds collected with the laser gun were adjusted to account for the angle correction.

5.5. Calculation of crash occurrence and crash rates

Crash counts from the years 1997 to 1999 were collected to calculate the crash exposure rate for all the selected highway segments. The objective of determining the crash rate was to exclude those highway segments that may cause considerable misperception of the risk which would lead to an excessive crash risk. An upper limit was established for the crash rate to identify the segments with high crash rates. Similar to two-lane highways, the estimated crash rates used to classify segments as low or high crash rate locations are representative only of the entire segment, and they are not representative of the safety level of any individual feature (intersection, horizontal curve, etc.), or any combination of those, present on the segment.

Figure 5-2 shows the crash rates for the selected four-lane highway segments and the upper limit used to identify sites with a high crash rate. The upper limit value of 3.5 crashes per million VMT represents the 75th percentile value in the sample. Seventeen sites having a crash rate higher than 3.5 were discarded.

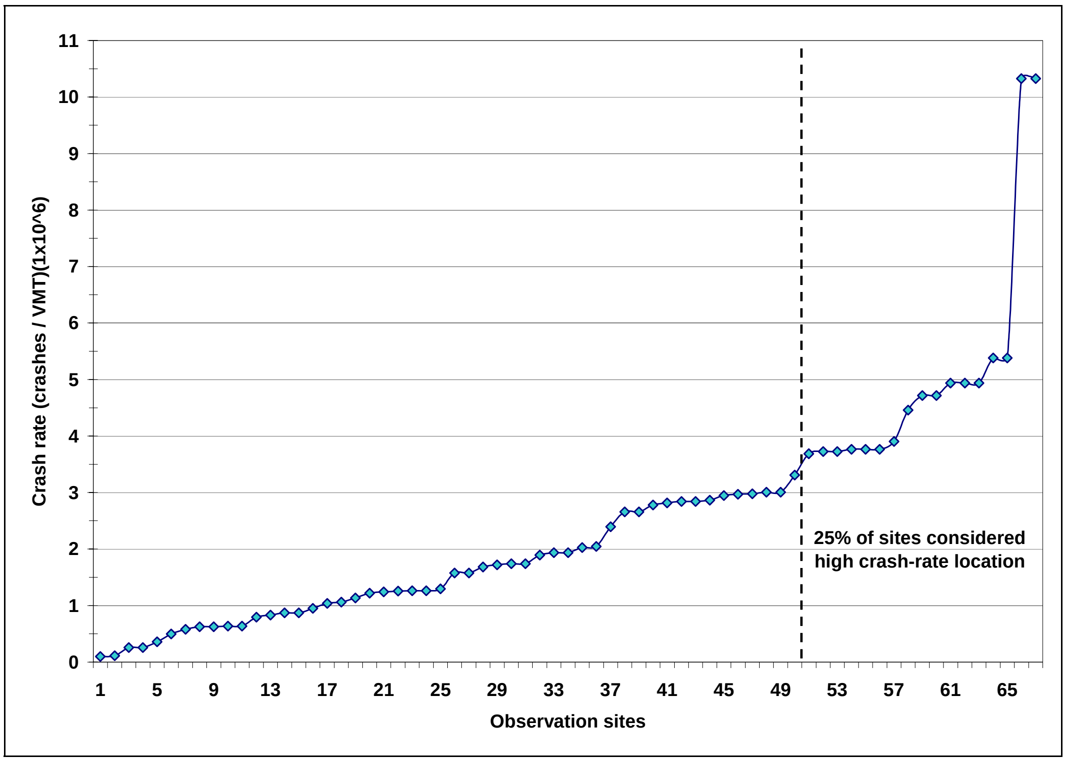


Figure 5-2 Crash rates for highway segments in four-lane rural highways

The same procedure used to collect the crash data for two-lane rural highways was followed for four-lane highways. There were no issues using the crash database to identify pseudo codes for four-lane highways because the names of all minor and local roads were identified beforehand during the data collection in the field.

5.6. Summary of highway characteristics and free-flow speeds

The data set is composed of highway characteristics and free-flow speeds for 50 sites. Table 5-2 presents descriptive statistics for some of the highway characteristics collected. Two sites were located in highway segments with a posted speed limit of 40 mph (60 km/h), 10 sites in segments with 45 mph (70 km/h), 12 sites in segments with 50 mph (80 km/h), and 26 sites in segments with 55 mph (90 km/h). No advisory speed signs were observed in the selected segments. The mean speed had a range of 20.2 mph (32.6 km/h) and the 85th percentile speed had a range of 19.5 mph (31.4 km/h). The range similarity between the observed speeds and the speed limit could be an early indication of the strength of their relationship.

Table 5-2 Descriptive statistics for characteristics in four-lane highways

Characteristic	Mean	Std. deviation	Minimum	Maximum
Posted speed limit (SPL), mph	51.20	4.58	40.00	55.00
Average annual daily traffic (AADT)	20,411	10,275	3550	58,580
Percent trucks (T)	9.27	6.61	1.00	41.50
Sight distance (SD), ft	1391.80	429.75	549.45	2078.00
Highway grade (G), percent	0.03	1.60	-6.20	6.00
Intersection density (INTD), # / mile	3.80	2.95	0	12
Driveway density (DRWD), # / mile	7.16	9.88	0	32
Traveled way width (TW), ft	25.34	0.72	21.41	25.34
Pavement width (PAV), ft	35.04	5.32	24.33	43.50
Total paved shoulder width (PSW), ft	11.49	5.45	0.00	19.58
Total gravel shoulder width (GSW), ft	2.19	3.60	0.00	10.33
Total untreated shoulder width (USW), ft	25.12	18.11	0.00	85.71
External clear zone distance (ECLR), ft	24.52	15.72	0.00	72.92
Median width (ICLR), ft	26.86	20.25	0.00	61.17
Degree of curvature (DC), degrees	3.21	1.37	1.55	5.91
Curve radius (R), ft	2094.22	861.87	969.70	3695.59
Maximum superelevation (SE), percent	3.40	1.94	1.18	6.65
Curve length (HCLEN), ft	975.91	791.88	165.00	2600.00
Mean speed, mph	54.82	4.71	45.09	62.00
85th percentile speed, mph	59.78	4.58	50.10	67.00

The average percentage of trucks in the data set is 9.27 percent; however, the percentage per site varies substantially from 1 to 41 percent. The effect of the truck percentage on speeds will be evaluated by including it as an explanatory variable in the model. It is not necessary to provide passing sight distance on four-lane highways; although adequate stopping sight distance should be provided. A range of 1528 ft (465.9 m) in sight distance was observed. The majority of the segments were located on flat terrain; although several sites were located on rolling terrain. Highway grades display a wide range of 12.2 percent; but only five sites are located in segments having an absolute grade higher than three percent. Although the length of the grade was not recorded, it was observed that none of the upgrades had the sufficient length to make trucks operate at crawl speeds.

Most of the sites were located in suburban areas; only eight sites were located in a rural area. Suburban areas provided more variety in access density values. The intersection density varied from a value of 0 to 12 intersections per mile; while the driveway density varied enormously from 0 to 32 driveways per mile. As expected, most of the segments in rural areas had low access densities. Fifteen sites had high residential development and six sites had high commercial development. High residential or high commercial development was present in segments containing more than 10 residential or commercial driveways per mile, respectively.

In terms of cross-section dimensions, the data set contains a large variability for the three shoulder and median surface types. The observed range in traveled way width was only 3.9 ft. The clear zones, on the other hand, composed of the width of the three shoulders, had a large range. The median width or the inside clear zone (the lateral distance measured from the inside edge of the traveled way to the internal edge of the opposing traveled way, or to the barrier face, if a median barrier was present) had a range of 61 ft. The external clear zone (the lateral distance measured from the outside edge of the traveled way to the roadside obstruction) had a range of almost 73 ft. A diverse combination of cross-section dimensions and access densities were observed. Figure 5-3 shows six typical cross-section configurations observed in four-lane highway segments.

Two different types of cross-section were generally found in rural areas; one with narrow median and clear zones and having frequent access points (Figure 5-3a) and another with median widths of more than 40 ft (12 m), clear zone distances of more than 40 ft and with full access control (Figure 5-3b). AASHTO states that a median width of 40 ft or more promotes to drivers a higher sense of separation from the opposing traffic and the headlight glare is greatly reduced. Two sites in rural areas included a paved median, instead of a grass median, and with a median barrier. The AASHTO Roadside Design Guide suggests that median barriers are not generally used in segments with median widths of more than 30 ft (10 m) or more under the assumption that most errant vehicles could recover within that distance.

a) Rural undivided with narrow median

b) Rural undivided with wide median

c) Suburban undivided with curb

d) Suburban divided with median barrier

e) Suburban divided with TWLT lane

f) Suburban divided with grass median

Figure 5-3 Typical cross-section configurations of four-lane highway segments

Suburban segments provided a higher variety in cross-section dimensions than rural segments. Figure 5-3c shows an undivided segment with unsloped curbs in both directions. Six sites were located in undivided highways; while nine sites were recorded as having curbs in both directions. Curbs are typically used to separate traffic from pedestrians on adjacent sidewalks. Two of the sites containing curbs also had a sidewalk. All the sites located on undivided segments had high residential driveway densities with 20 or more driveways per mile. All, but one of these sites, had a reduced clear zone of 10 ft (3m) or less in each direction. Figure 5-3d shows one of the five sites located in suburban segments with a median barrier. These sites had more access control (less than 15 entry points per mile) and wider clear zones (around 30 ft) than the selected undivided segments, but the traveled way width is very similar, at around 46 ft (14 m).

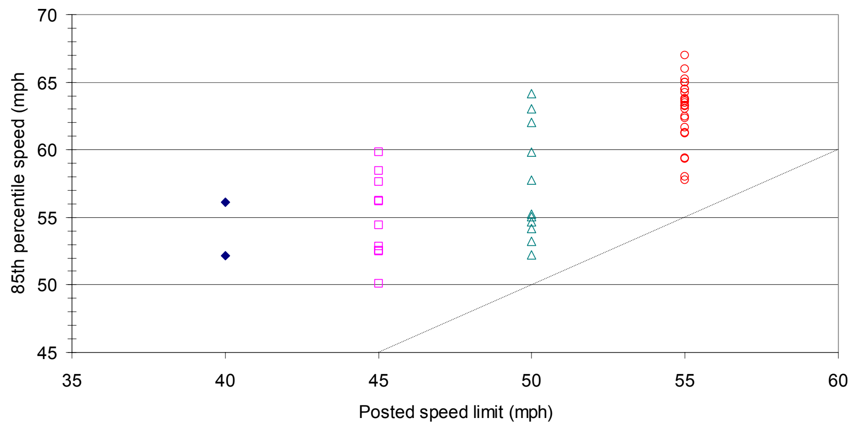
The other two types of suburban cross-sections had wider medians and clear zones while having different median types. Figure 5-3e shows one of fourteen sites located in segments with a two-way left turn (TWLT) median lane. The TWLT median lanes are typically used to provide increased access to closely spaced commercial and residential driveways. Six of the fourteen sites with a TWLT median lane also had high commercial or residential driveway densities. AASHTO suggests that these median lanes help to increase the access to the highway rather than control it. Other advantages awarded to these lanes are reduced travel time, improved capacity, reduced crash frequency and public preference from drivers and property owners. The width of the TWLT median lanes varied from 13.25 to 21 ft (4 to 6.4 m); which is slightly more than what AASHTO suggests for the optimal design of these lanes (3 to 4.8 m). Figure 5-3f shows a segment with a similar cross-section but having a grass median. Nineteen sites were located on segments with either a depressed or leveled grass median, with widths varying as much as 15.5 to 52 ft (4.7 to 15.9 m), or 31.4 to 62 ft (9.5 to 18.9 m) including the internal paved and gravel shoulders. Four other sites were located on suburban segments with paved medians, with widths varying from 12.4 to 15.4 ft (3.8 to 4.7 m). Five different roadside obstructions were observed: curbs, guardrails, pole line, ditches, and embankments. Nine sites had curbs as the roadside obstruction, one site had a guardrail, two sites had a pole line, twenty-eight sites had ditches and four sites had embankments. No segments having on-street parking or bus turnouts were observed.

In terms of horizontal curvature, only eleven sites were located either inside a curve or in the transition section. Sharp curves were only observed in suburban segments with posted speed limits under 40 mph (60 km/h) or to close to traffic signals or stop signs. The design of horizontal curves in four-lane rural highway segments is very consistent. The curves observed during the visual inspection were flat enough to not warrant their inclusion. The model results for two-lane rural highways showed that curves with large radii do not have a bigger impact on speeds than the impact due to the cross-section dimensions and other segment characteristics. Ten different horizontal curves were included. The degree of curvature had a range of 4.36 degrees, corresponding to a range in curve radii of 2725.9 ft (830.8 m). The small range in curvature serves as evidence of the more consistent design in four-lane highways compared to two-lane rural highways. Maximum superelevation rates and curve lengths had ranges of 5.5 percent and 2435 ft (742.2 m), respectively.

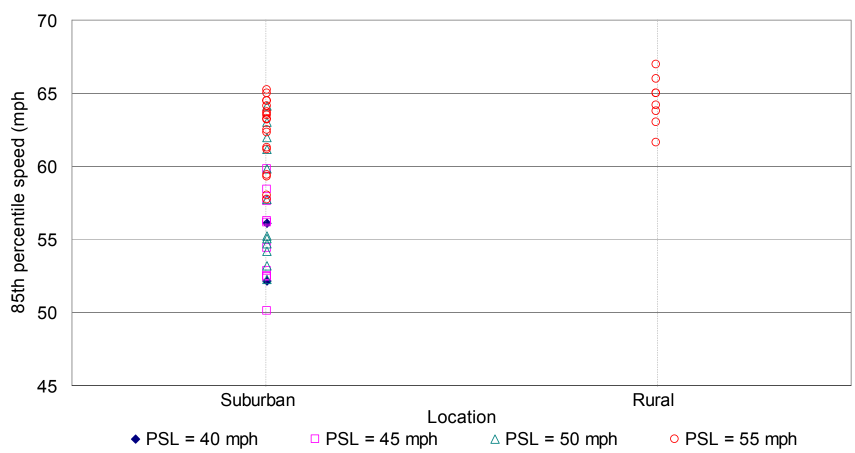
5.7. Trends between observed operating speeds and highway characteristics

The following section presents a graphical analysis that shows trends between the observed 85th percentile speeds and different characteristics of four-lane highway segments. This analysis helps to recognize speed trends and to identify potential relationships with speeds. Figure 5-4 presents trends between three segment characteristics and the observed 85th percentile speeds. Figure 5-5 presents trends between three access control variables and the observed 85th percentile speeds. Figure 5-6 presents trends between three cross-section dimensions and the observed 85th percentile speeds. The graphs make a distinction between the different posted speed limits included in the sample.

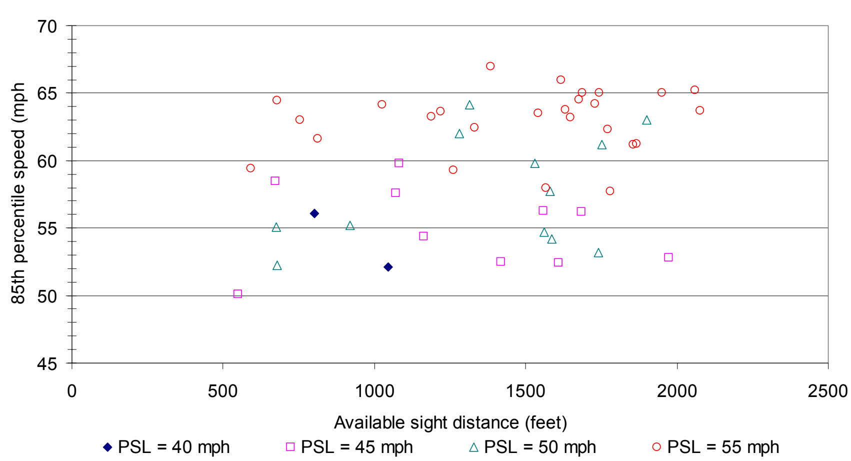
The posted speed limit (Figure 5-4a) and the segment setting (Figure 5-4b) show very strong trends with operating speeds. The observed trend of higher operating speeds in rural areas than in suburban areas is obvious. The observed segments in rural areas had higher speed limits and lower access densities than most suburban segments. Similar to two-lane rural highways, a reduction in the posted speed limit decreases the operating speed. These results are consistent with those found in NCHRP Report 504. The operating speeds are higher than the posted speed limits in all the sites by a margin of 2.2 to 16.1 mph. The variability in operating speed seems to be equivalent for the three highest speed limits. Although some segments had different speed limits, similar operating speeds are observed. This might be a clear indication that the posted speed limit combine with other highway characteristics as significant speed factors. The sight distance (Figure 5-4c) appears to have a weak positive trend with operating speeds, as was the case in two-lane rural highways.



a) Posted speed limit

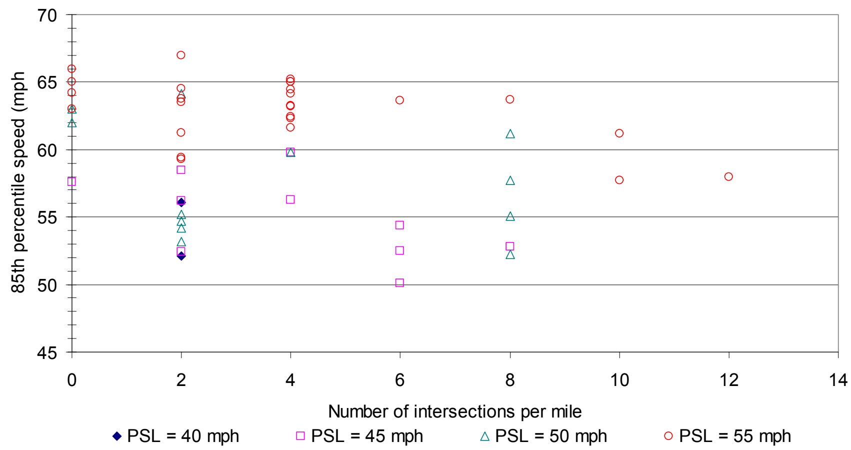


b) Suburban vs. rural setting

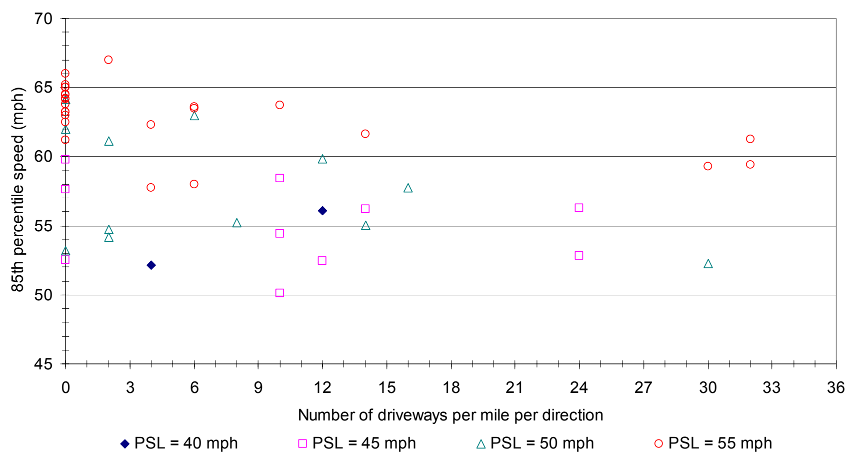


c) Sight distance

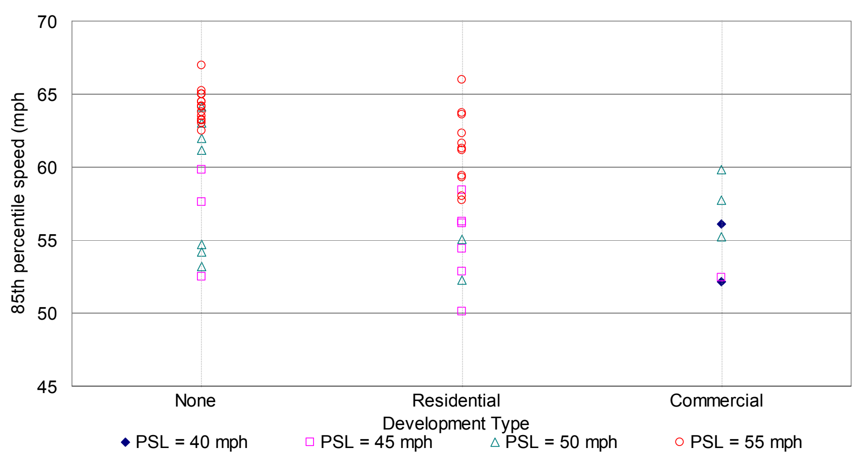
Figure 5-4 Trends between segment characteristics and operating speeds in four-lane highways



a) Intersection density



b) Driveway density



c) Development type

Figure 5-5 Trends between access density and operating speeds in four-lane highways

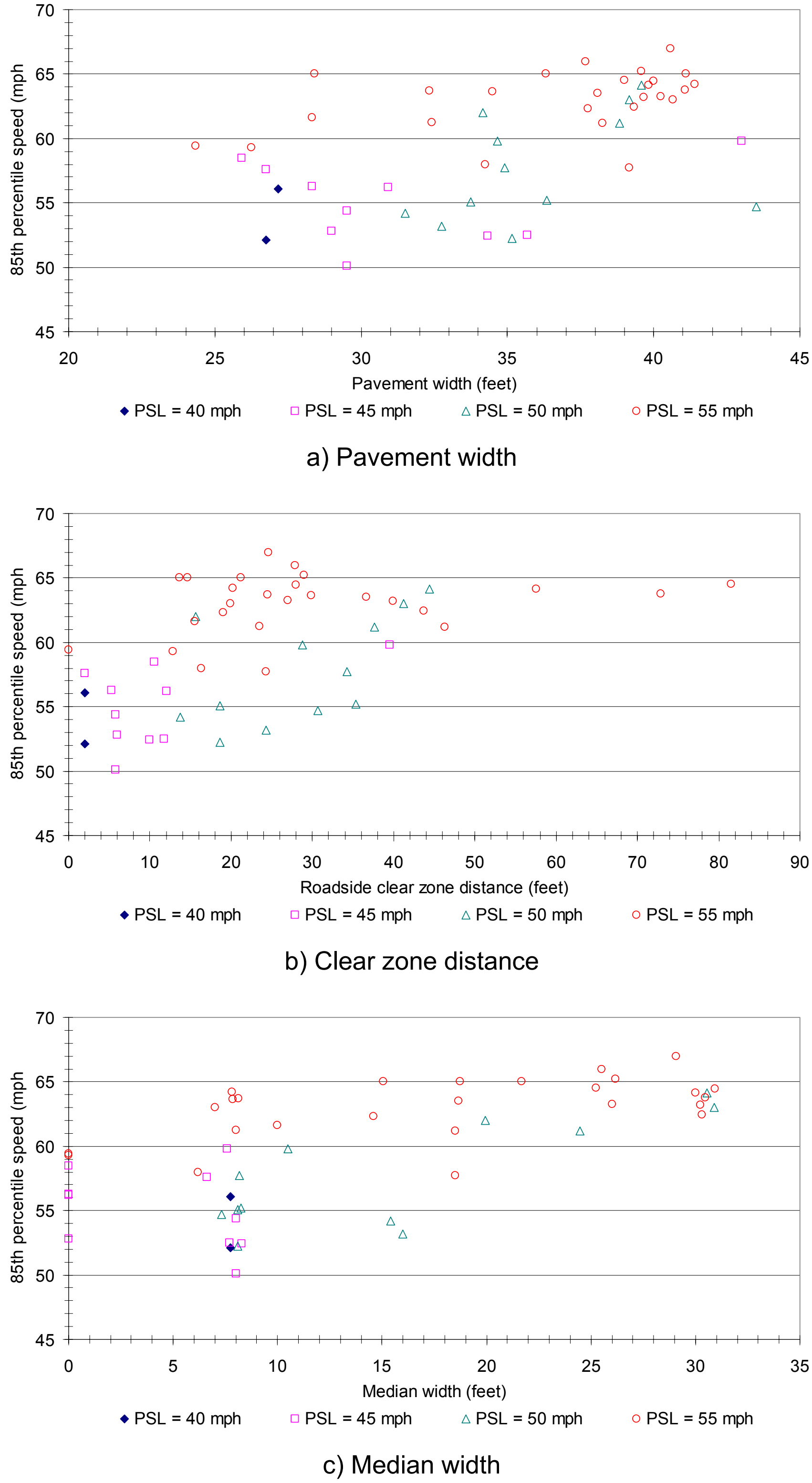


Figure 5-6 Trends between cross-section dimensions and operating speeds in four-lane highways

The intersection (Figure 5-5a) and driveway (Figure 5-5b) densities show negative trends with the operating speeds, as expected. It is noticeable that the speed variability seems to decrease as both densities increase. These trends are consistent with those found for the access density values in NCHRP Report 504. Another possible speed factor might be the presence of high residential or commercial development in the segments (Figure 5-5c). The observed trend indicates that operating speeds decrease with high commercial development; although no segment with high commercial development had a 55 mph (90 km/h) speed limit. Segments with a high residential development have a large range in operating speeds, but the impact of the speed limit on those sites is obvious. It is typical for suburban segments with high development to have speed limits lower than 55 mph. Low speed limits are frequently requested by residents and business owners on suburban segments with the objective of controlling speeds and improving highway safety.

Figure 5-6 shows interesting trends between the cross-section dimensions and the operating speeds that are consistent with those found in NCHRP Report 504. An increase in any of the three cross-section dimensions increases the operating speeds. It is significant that the speed variability is decreasing with increasing roadside clear zone (Figure 5-6b) and median width (Figure 5-6c). The low operating speeds on segments with narrow clear zones might be the result of the lower sense of separation given to drivers from the opposing traffic or the roadside obstructions. It is important to note that the wider cross-section dimensions are present on segments having higher posted speed limits. This trend is generated from design standards where increasing roadway widths are associated with increasing highway design speeds.

A speed analysis for the sites on horizontal curves was not performed because of the low number of sites located in curves selected. A more detailed analysis of the relationships between the highway characteristics and the observed speeds is provided in Chapter 7 using the results of the speed model calibration.

CHAPTER 6. SPEED PREDICTING MODELS FOR TWO-LANE RURAL HIGHWAYS

This chapter presents the results of the speed modeling process for two-lane rural highways. The model development, results and performance evaluation are discussed for the two proposed approaches for modeling panel data: OLS without random effects and GLS with random effects.

6.1. Development of speed models

The speed models were developed following the methodology discussed in Chapter 3. The calibration process used the free-flow speeds and the highway geometry information collected for 158 data points. The observed free-flow speeds were used to calculate from the 5^{th} to the 95^{th} percentile speed, in multiples of five. All the potential explanatory variables were multiplied by the Z_p value corresponding to each percentile to assemble the panel data. The following sections discuss the results for the preliminary values and models used in the calibration process.

6.1.1. Preliminary deceleration and acceleration rates

Speeds from nine observation sites were used to estimate the mean deceleration rate in tangent-to-curve transition sections. The estimated mean deceleration rate was used as an initial value in the iterative calibration process. The speeds at the selected sites were expected to show some deceleration because of their location with respect to horizontal curves. The first spot was located 300 ft (91.4 m) before the curve and the second spot was located at the beginning of the curve. All sites were preceded by long and flat tangent segments. The highest grade in those segments was a 2.6 percent downgrade. All observation sites are composed of two spots located 300 ft (91.4 m) apart.

There are 1,606 individual speed observations in the sample of nine sites. The mean deceleration rate over space and the mean deceleration rate over time calculated for the individual speed observations were -0.0178 (ft/s)/ft and -1.3817 ft/s^2, respectively. This deceleration rate was considered to be too small; therefore, each site was analyzed separately to ignore sites with insignificant or no deceleration. Table 6-1 shows the mean speed and mean deceleration rate per site. The tangent mean speed V_T was estimated using a preliminary OLS regression model. The mean speed V_1 and the mean speed V_2 were calculated for the first and second spot of the observation site, respectively.

Table 6-1 Deceleration rates for sites in tangent-to-curve transition sections

Observation site	Tangent speed V_T (mph)	Speed first spot V_1 (mph)	Speed second spot V_2 (mph)	Mean deceleration rate (ft/s/ft)	Standard deviation (ft/s/ft)	Mean deceleration rate (ft/s^2)	Standard deviation (ft/s^2)
006-075-001	61.45	53.68	47.48	-0.03034 [(b)]	0.0158	-2.2595	1.213
008-421-003	61.14	58.44	55.97	-0.01207	0.0097	-1.0018	0.767
008-421-002	61.40	55.52	53.57	-0.00955	0.0142 [(a)]	-0.7713	1.057
008-025-015	59.85	57.36	56.28	-0.00526	0.0077 [(a)]	-0.4352	0.648
008-025-014	59.73	55.79	54.38	-0.00692	0.0089 [(a)]	-0.5430	0.709
012-421-008	59.56	56.09	54.24	-0.00902	0.0100 [(a)]	-0.7345	0.806
061-041-011	59.71	54.37	48.45	-0.02896 [(b)]	0.0138	-2.2003	1.114
061-041-006	59.63	54.79	48.41	-0.03121 [(b)]	0.0152	-2.3728	1.175
079-025-003	59.70	61.08	59.67	-0.00688	0.0093 [(a)]	-0.6115	0.828
Mean μ				-0.01784		-1.38171	
Standard deviation σ				0.01651		1.25853	
Ratio σ / μ				0.9256		0.9108	

Notes:

a = standard deviation higher than mean value

b = considered significant deceleration rate

The calculated tangent mean speed for site 079-025-003 is 1.38 mph (2.22 km/h) lower than the observed mean speed for the first spot. This might be an indication that the curve by itself does not compel drivers to reduce speeds and that the speed is mostly influenced by the highway characteristics and the cross-section dimensions on the tangent segment. The other sites have estimated tangent mean speeds higher than the observed mean speeds. Therefore, it was assumed that these eight sites were located in the transition section for the curves. The actual length of the transition section was unknown at this point; consequently, the estimated mean deceleration rate per site might be misrepresented due to the short distance between the two spots. Five sites have standard deviation values higher than their respective mean deceleration rates. The high dispersion in the deceleration values of the individual vehicles in those sites indicates that the mean deceleration rate cannot be considered to be significantly different from zero. The preliminary mean deceleration rate was estimated as -0.0301 (ft/s)/ft (-2.2705 ft/s^2), taking only into consideration the three sites that have deceleration rates significantly different from zero.

Speeds from ten observation sites were used to estimate the mean acceleration rate in curve-to-tangent transition sections. The estimated mean acceleration rate was used as an initial value in the iterative calibration process. The speeds at the selected sites were expected to show some acceleration because of their location with respect to horizontal curves. The first spot was located at the end of the curve and the second spot was located 300 ft (91.4 m) after the curve. All sites were followed by long flat tangent segments. The highest grade was a 1.95 percent downgrade. All observation sites are composed of two spots located 300 ft (91.4 m) apart.

There are 1,947 individual speed observations in the sample of ten sites. The mean acceleration rate over space and the mean acceleration rate over time for the individual speed observations were 0.0099 (ft/s)/ft and 0.7357 ft/s^2, respectively. The acceleration rate was also considered to be too small, so any site showing insignificant or no acceleration was therefore ignored. Table 6-2 shows the mean speed and mean acceleration rate per site.

Table 6-2 Acceleration rates for sites in curve-to-tangent transition sections

Observation site	Speed first spot V_1 (mph)	Speed second spot V_2 (mph)	Tangent speed V_T (mph)	Mean acceleration rate (ft/s/ft)	Standard deviation (ft/s/ft)	Mean acceleration rate (ft/s^2)	Standard deviation (ft/s^2)
006-075-002	49.44	52.23	61.40	0.01382 [b]	0.00920	1.0007	0.608
008-421-001	53.18	55.51	61.40	0.01160	0.01447 [a]	0.8272	0.900
008-025-013	55.45	56.96	59.73	0.00730	0.00544	0.5897	0.410
008-025-016	53.16	54.66	59.82	0.00728	0.00956 [a]	0.5362	0.688
012-421-007	57.21	57.98	59.56	0.00388	0.00992 [a]	0.3239	0.807
029-037-001	57.53	58.20	62.01	0.00327	0.00584 [a]	0.2825	0.501
061-041-012	48.12	52.02	59.71	0.01912 [b]	0.01103	1.4046	0.788
061-041-005	46.56	49.90	59.63	0.01647 [b]	0.00881	1.1744	0.643
079-025-001	58.10	58.94	59.74	0.00418	0.00774 [a]	0.3462	0.631
079-025-010	50.95	52.71	60.10	0.00850	0.00712	0.6367	0.528
Mean μ				0.0099		0.7357	
Std. deviation σ				0.0111		0.7947	
Ratio σ / μ				1.1252		1.0801	

Notes:

a = standard deviation higher than mean value

b = considered significant acceleration rate

All ten sites have estimated tangent mean speeds higher than the observed mean speeds. Therefore, it was assumed that the ten sites were located in the transition section for the horizontal curves. Since the actual length of the transition section is unknown, the estimated mean acceleration rates might be misrepresented as well. Five sites have standard deviation values higher than their respective mean acceleration rates. The high dispersion in the acceleration values indicates that the mean acceleration rate cannot be considered to be significantly different from zero. Two additional sites (08-025-013 and 79-025-010) were not included because their acceleration rates might be also considered to be equal to zero by analyzing their standard deviations and mean values. The mean acceleration rate was estimated as 0.0167 (ft/s)/ft (1.2099 ft/s^2), taking only into consideration the three sites that have acceleration rates significantly different from zero.

6.1.2. Preliminary models for tangent segments and horizontal curves

Preliminary speed models were developed using thirty-two sites located on tangent segments free from the influence of curves and twenty sites located on horizontal curves. These two models were used to calculate the mean speeds for the first iteration of the calibration process. These mean speeds and the estimated mean deceleration and acceleration rates were used to calculate the length of the curve transition sections. The results from a correlation analysis performed on the data from these two samples are discussed in this section.

The Pearson correlation coefficient r was calculated to identify the highway characteristics, cross-section dimensions or horizontal curve components that have a linear relation with the mean speed or the 85^{th} percentile speed. The correlation coefficient provides a descriptive measure of the degree of linear association between two random variables in the sample observations; it does not provide an indication that useful predictions can be made (Neter et al., 1996). The coefficient has a value between -1 and 1, inclusive. When r is equal to zero there is no linear relationship between the two variables; when r is equal to 1 or -1 there is a perfect linear relationship. Significant correlations were identified with a 95 percent confidence level.

The posted speed limit, the truck percentage, the traveled way width and the gravel shoulder width have positive linear relationships ($r > 0.42$) with the two speeds in tangent segments. In contrast, the untreated shoulder width and the high residential development variable have negative relationships ($r > 0.37$) with the two speeds. The sign of the relationships with the truck percentage and the untreated shoulder width were not expected. It is generally accepted that the quality of the traffic flow decreases as the number of trucks increases. A wider untreated shoulder presents safer highway conditions because it provides more distance for errant vehicles to avoid a collision with any roadside obstruction. The actual effect on speeds, if any, of these two variables will be further analyzed for the final tangent speed model.

The correlation coefficient can be also used to identify the presence of strong linear relationship between random variables that might indicate the possibility of multicollinearity. Multicollinearity is present in a regression model when explanatory variables are highly correlated or when explanatory variables are correlated with omitted variables that are related to the dependent variable in the model. When explanatory variables are uncorrelated, the effect of those variables in the regression are the same no matter which of the other variables are included in the model (Neter et al., 1996).

Strong correlation between explanatory variables increases the standard deviation of the parameter estimates, but it does not prevent least squares to obtain a best fit to the data, nor does it affect inferences on mean responses or new observations (Washington et al., 2003). In other words, the presence of strong correlation between explanatory variables does not cause any systematic bias of estimation as long as all the correlated variables are present in the model and the inferences are made within the region of observations.

Some cross-section dimensions show significant correlation between each other. The gravel shoulder width is negatively correlated with the paved ($r = 0.52$) and the untreated ($r = 0.70$) shoulder widths. In contrast, the paved shoulder width is positively correlated with the traveled way ($r = 0.70$) and the untreated shoulder ($r = 0.48$) widths. The traveled way width is positively correlated with the untreated shoulder width ($r = 0.41$). The traveled way and paved shoulder widths were combined as a pavement width variable which has a lower correlation with the gravel shoulder width ($r < 0.43$) and no significant relationship with the untreated shoulder width.

In terms of other variables present in tangent segments, the high residential development variable is negatively correlated to the pavement and gravel shoulder widths ($r > 0.38$) and positively correlated to the posted speed limit (PSL_{50}) variable ($r = 0.71$). In other words, tangent segments having high residential development generally have narrower cross-sections and a 50 mph (80 km/h) speed limit. In addition, tangent segments with a 50 mph speed limit generally have narrower traveled way and gravel shoulder widths ($r = 0.53$), but wider untreated shoulder widths ($r = 0.35$).

As expected, the degree of curvature and the maximum superelevation rate have negative relationships with the two speeds in horizontal curves ($r > 0.63$). The degree is positively correlated with the superelevation rate ($r = 0.66$), suggesting that high superelevation rates are generally used in combination with sharp curves. High superelevation rates are typically used to offset the impact of sharp curvature when right-of-way restrictions are present in a design project. In contrast, the curve length is positively correlated with the two speeds ($r < 0.50$). The fact that an increase in curve length generally increases curve speeds might be related to its negative correlation with the degree and the superelevation rate ($r > 0.63$).

The speed models were calibrated using the SAS software. The best specification of the preliminary OLS model to estimate mean speeds in tangent segments, in mph, is the following:

$$V = 58.54 - 4.60 \times PSL_{50} - 0.21 \times G - 1.79 \times RES + 0.30 \times GSW + 0.03 \times USW \qquad (6.1)$$

where:

PSL_{50} = equal to 1 if the speed limit is 50 mph; equal to 0 if the speed limit is 55 mph

G = highway grade, percent

RES = equal to 1 if segment has 10 or more residential driveways per mile; 0 otherwise

GSW = total gravel shoulder width, feet

USW = total untreated shoulder width, feet

The best specification of the preliminary OLS model to estimate mean speeds in horizontal curves, in mph, is the following:

$$V = 50.03 + 0.002 \times SD - 0.12 \times T - 2.67 \times RES - 2.09 \times DC + 7.41 \times SE - 0.62 \times SE^2 \qquad (6.2)$$

where:

SD = sight distance, feet

T = percent of trucks in free-flow speed distribution, percent

DC = degree of curvature, degrees

SE = maximum superelevation rate, percent

The best specification of the preliminary RE model to estimate mean speeds in tangent segments, in mph, is the following:

$$V = 58.63 - 4.63 \times PSL_{50} - 1.81 \times RES + 0.30 \times GSW + 0.03 \times USW \qquad (6.3)$$

The best specification of the preliminary RE model to estimate mean speeds in horizontal curves, in mph, is the following:

$$V = 51.12 - 1.85 \times DC + 6.46 \times SE - 0.55 \times SE^2 \qquad (6.4)$$

The obtained RE models are different from the OLS models by the variables included and their t-statistics. It indicates that omitting the random effects causes some bias in the model estimation. The discussion about the impact of the variables included in the speed models was set aside for the final models. All the variables included in the models are significant with a 90 percent confidence level. The percentile effects in the panel data were also evaluated, but the variance attributed to the percentile dimension was practically insignificant compared to the variance attributed to sites and residuals. Consequently, adding the random effects due to the percentile dimension did not cause any change in the parameter estimates in the RE models.

6.1.3. Percentile speed models without random effects

The estimated mean acceleration and deceleration rates were used in conjunction with the OLS mean speed models for tangent segments and horizontal curves in Equations 6.1 and 6.2, respectively, to calculate the length of the transition sections. The length of the transition sections was used together with the assumed value for the portion of the transition length on the tangent to classify sites and sub-divide the panel data. The iterative calibration process was performed as discussed in Chapter 3 and the final OLS-PD speed models are presented in this section.

Three iterations were completed to achieve convergence in the OLS-PD calibration process. Appendix C shows the final calibration results. Tables 6-3 to 6-6 show the parameter estimates and the percent change in the estimates from consecutive iterations for each one of the four OLS-PD models. In addition, the tables show the number of sites assigned to each sub-sample and the adjusted coefficient of multiple determination, denoted by R^2. The adjusted R^2 value provides a goodness-of-fit measure to compare models with different number of parameters. The coefficient is bounded by 0 and 1; and it is usually interpreted as the amount of variability explained by the independent variables in the regression model. When the adjusted R^2 value is equal to 1, all the variance is explained by the regression model, e.g., all observations fall directly on the fitted regression surface (Neter et al., 1996).

The iterative process was said to converge when there was no change in the site classification between consecutive iterations or when the speeds models cannot be further improved. There was no change in the site classification for the four sub-samples in the third iteration and the curve model was identical to the one developed in the second iteration. Although these two conditions warranted stopping the iterative process, the parameter estimates of the other three models changed slightly and it was decided to continue. Two additional iterations were performed, but were later discarded because the number of sites assigned to the curve sub-sample went below the desired minimum. Anyway, the parameter estimates for the tangent and transition models in the fifth iteration were comparable to those in the third iteration. The biggest percent change for a parameter estimate between the fifth and third iteration was 6.7%, which was considered to be practically insignificant. The results from the third iteration were selected as the final solution. To improve the fit of the models to the data, horizontal curves were divided into flat and sharp curves. All curves having a radius of more than 1700 ft were identified as flat curves. The speeds on flat curves are influenced more by the highway characteristics and the cross-section dimensions than by the curve design itself. Those speeds are estimated using the model for tangent segments with an adjustment factor for the presence of the flat curve.

Table 6-3 Iteration results for the tangent percentile speed OLS-PD model

Parameter	ITERATION 0	1	% change	2	% change	3	% change
PSL_{50}	-4.601	-2.923	-36.5%	-3.183	8.9%	-3.077	-3.3%
GRADE	-0.206	-0.261	26.7%	-0.144	-44.9%	-0.142	-1.4%
PERCENT TRUCKS	-	-	-	-0.051	-	-0.055	8.0%
SIGHT	-	0.005	-	0.005	0.9%	0.005	5.4%
$SIGHT^2$	-	-2.73E-06	-	-2.71E-06	-0.7%	-2.77E-06	2.2%
INTERSECTION	-	-0.558	-	-0.276	-50.5%	-0.384	38.9%
RESIDENTIAL DEVELOPMENT	-1.789	-0.908	-49.2%	-1.051	15.7%	-1.004	-4.5%
PAVEMENT WIDTH	-	0.053	-	0.031	-41.8%	0.032	5.2%
GRAVEL SHOULDER WIDTH	0.302	0.542	79.1%	0.561	3.6%	0.571	1.9%
UNTREATED SHOULDER WIDTH	0.031	0.051	64.3%	0.055	7.6%	0.051	-7.3%
Z_p	5.190	4.716	-9.1%	4.713	-0.1%	4.756	0.9%
Z_p -PSL_{50}	1.856	1.472	-20.7%	1.523	3.4%	1.550	1.8%
Z_p -GRADE	0.122	0.054	-56.0%	0.053	-1.9%	0.053	0.2%
Z_p -TRUCK	-	0.025	-	0.025	2.5%	0.024	-4.5%
Z_p -INTERSECTION	0.355	0.372	4.7%	0.323	-13.2%	0.304	-5.7%
Z_p -CLEAR ZONE	-0.019	-0.015	-21.3%	-0.015	4.0%	-0.016	6.6%
Intercept	58.535	53.026	-9.4%	54.184	2.2%	54.065	-0.2%
Number of sites	32	82	156.3%	85	3.7%	85	0.0%
R^2	94.68	83.03	-12.3%	82.69	-0.4%	82.42	-0.3%

Table 6-4 Iteration results for the horizontal curve percentile speed OLS-PD model

Parameter	ITERATION 0	1	% change	2	% change	3	% change
SIGHT	0.002	0.003	9.6%	0.003	36.5%	0.003	0.0%
TRUCK	-0.116	-0.120	3.5%	-	-	-	-
RESIDENTIAL DEV.	-2.672	-1.989	-25.6%	-2.639	32.7%	-2.639	0.0%
DEGREE	-2.093	-2.092	-0.1%	-2.541	21.5%	-2.541	0.0%
SUPERELEVATION	7.415	7.399	-0.2%	7.954	7.5%	7.954	0.0%
$SUPERELEVATION^2$	-0.620	-0.618	-0.4%	-0.624	0.9%	-0.624	0.0%
Z_p	4.163	4.169	0.1%	4.158	-0.3%	4.158	0.0%
Z_p –DEGREE	0.188	0.188	-0.1%	0.236	25.3%	0.236	0.0%
Z_p-SUPERELEV.	-0.145	-0.145	0.2%	-0.199	37.2%	-0.199	0.0%
Intercept	50.031	49.898	-0.3%	47.664	-4.5%	47.664	0.0%
Number of sites	20	19	-5.0%	14	-26.3%	14	0.0%
R^2	89.56	88.98	-0.6%	93.22	4.8%	93.22	0.0%

Table 6-5 Iteration results for the deceleration transition percentile speed OLS-PD model

Parameter	ITERATION						
	0	1	% change	2	% change	3	% change
t_d	85.00	70.05	-17.6%	65.34	-6.7%	65.53	0.3%
d	0.0301	0.0252	-16.4%	0.0306	21.7%	0.0330	7.8%
Number of sites	-	27	-	30	11.1%	30	0.0%
R^2	-	85.79	-	83.62	-2.5%	84.02	0.5%

Table 6-6 Iteration results for the acceleration transition percentile speed OLS-PD model

Parameter	ITERATION						
	0	1	% change	2	% change	3	% change
t_a	85.00	77.15	-9.2%	71.87	-6.8%	71.64	-0.3%
a	0.0167	0.0182	9.0%	0.0222	22.1%	0.0221	-0.6%
Number of sites	-	30	-	29	-3.3%	29	0.0%
R^2	-	87.12	-	87.70	-0.7%	87.61	-0.1%

The best specification of the OLS-PD model to calculate any percentile speed on tangent segments, in mph, is the following:

$$\begin{aligned} V_p &= 57.137 - 3.082 \times PSL_{50} - 0.071 \times T - 0.131 \times G - 1.003 \times RES \\ &+ 2.38 \times 10^{-3} \times SD - 1.67 \times 10^{-6} \times SD^2 - 0.422 \times INT \\ &+ 0.040 \times PAV + 0.394 \times GSW + 0.054 \times USW - 2.233 \times FC \\ &+ 5.982 \times Z_p + 1.428 \times (Z_p \times PSL_{50}) + 0.061 \times (Z_p \times G) \\ &+ 0.292 \times (Z_p \times INT) - 0.038 \times (Z_p \times PAV) - 0.012 \times (Z_p \times LC) \end{aligned} \tag{6.5}$$

where:

PSL_{50} = equal to 1 if the speed limit is 50 mph; equal 0 if the speed limit is 55 mph

T = percent of trucks in free-flow speed distribution, percent

G = segment grade, percent

RES = equal to 1 if segment has 10 or more residential driveways per mile; 0 otherwise

SD = sight distance, feet

INT = equal to 1 if an intersection is located 350 ft before or after the spot; 0 otherwise

PAV = pavement width, includes the traveled way and both paved shoulder widths, feet

GSW = total gravel shoulder width, includes both directions, feet

USW = total untreated shoulder width, includes both directions, feet

FC = equal to 1 if the spot is located on a flat curve, e.g. radius larger than 1700 ft; 0 otherwise

LC = lateral clearance distance, includes the widths of the total gravel shoulder and the total untreated shoulder, feet;

Z_p = standardized normal variable corresponding to a selected percentile, see Table C-1

The best specification of the OLS-PD model to calculate any percentile speed in horizontal curves, in mph, is the following:

$$V_p = 47.664 + 0.003 \times SD - 2.639 \times RES - 2.541 \times DC + 7.954 \times SE - 0.624 \times SE^2 + 4.158 \times Z_p + 0.236 \times (Z_p \times DC) - 0.199 \times (Z_p \times SE) \tag{6.6}$$

where:

DC = degree of curvature, degrees

SE = maximum superelevation rate, percent

The best specification of the OLS-PD model to calculate any percentile speed in the deceleration transition section, in mph, is the following:

$$V_p = V_{Tp} - 0.6553 \times (V_{Tp} - V_{Cp}) + 0.03299 \times l_d \tag{6.7}$$

where:

V_{Tp} = estimated percentile speed on tangent from Equation 6.5, in ft/s

V_{Cp} = estimated percentile speed on horizontal curve from Equation 6.6, ft/s

l_d = distance from the site to the beginning of the curve, takes a positive value outside the curve and a negative value inside the curve, ft

The best specification of the OLS-PD model to calculate any percentile speed in the acceleration transition section, in mph, is the following:

$$V_p = V_{Tp} - 0.7164 \times (V_{Tp} - V_{Cp}) + 0.02211 \times l_a \tag{6.8}$$

where:

V_{Tp} = estimated percentile speed on tangent from Equation 6.5, in ft/s

V_{Cp} = estimated percentile speed on horizontal curve from Equation 6.6, ft/s

l_a = distance from the site to the end of the curve, takes a positive value outside the curve and a negative value inside the curve, ft

6.1.4. Percentile speed models with random effects

The estimated mean acceleration and deceleration rates were used in conjunction with the mean speed RE model for tangent segments and horizontal curves in Equations 6.3 and 6.4, respectively, to calculate the length of the transition sections. The length of the transition sections was used together with the assumed value for the portion of the transition length on the tangent to classify sites and sub-divide the panel data. The iterative calibration process was performed as discussed in Chapter 3 and the final RE speed models are presented in this section. Three iterations were completed to achieve convergence in the calibration process. Tables 6-7 to 6-10 show the parameter estimates and the percent change from consecutive iterations for each one of the four RE percentile speed models. The tables also show the number of sites assigned to each sub-sample and the log-likelihood or the adjusted R^2 value.

Table 6-7 Iteration results for the tangent percentile speed RE model

Parameter	ITERATION						
	0	1	% change	2	% change	3	% change
PSL_{50}	-4.632	-2.732	-41.0%	-2.759	1.0%	-2.759	0.0%
RESIDENTIAL DEVELOPMENT	-1.806	-	-	-	-	-	-
GRAVEL SHOULDER WIDTH	0.298	0.507	70.2%	0.430	-15.2%	0.430	0.0%
UNTREATED SHOULDER WIDTH	0.031	0.050	62.8%	0.047	-4.7%	0.047	0.0%
Z_p	5.190	7.504	44.6%	7.905	5.3%	7.905	0.0%
Z_p -PSL_{50}	1.856	1.296	-30.2%	1.302	0.5%	1.302	0.0%
Z_p -GRADE	0.122	0.054	-55.6%	0.056	3.1%	0.056	0.0%
Z_p -TRUCK	-	0.018	-	0.018	0.3%	0.018	0.0%
Z_p -INTERSECTION	0.355	0.144	-59.4%	0.227	57.4%	0.227	0.0%
Z_p - TRAVELED WAY WIDTH	-	-0.125		-0.139	11.1%	-0.139	0.0%
Z_p -CLEAR ZONE	-0.019	-0.008	-55.1%	-0.011	35.3%	-0.011	0.0%
Intercept	58.626	55.301	-5.7%	55.491	0.3%	55.491	0.0%
Number of sites	32	82	156.3%	91	11.0%	91	0.0%
Log-likelihood	1707.8	4204.8	146.2%	4838.6	15.1%	4838.6	0.0%

Table 6-8 Iteration results for the horizontal curve percentile speed RE model

Parameter	ITERATION						
	0	1	% change	2	% change	3	% change
DEGREE	-1.846	-2.0496	11.0%	-	-	-	-
SUPERELEVATION	6.458	7.2506	12.3%	-	-	-	-
$SUPERELEVATION^2$	-0.553	-0.6201	12.1%	-	-	-	-
Z_p	4.079	4.4937	10.2%	-	-	-	-
Z_p -TRUCK	0.052	0.0527	1.4%	-	-	-	-
Z_p -GRADE	0.062	-	-	-	-	-	-
Z_p -SIGHT	-0.0006	-0.0008	49.1%	-	-	-	-
Z_p -DEGREE	0.176	0.1939	9.9%	-	-	-	-
Z_p -SUPERELEV.	-0.152	-0.1994	31.1%	-	-	-	-
Intercept	51.119	51.1117	0.0%	-	-	-	-
Number of sites	20	18	-10.0%	10	-44.4%	11	10.0%
Log-likelihood	867.5	793.0	-8.6%	-	-	-	-

Table 6-9 Iteration results for the deceleration transition percentile speed RE model

Parameter	ITERATION						
	0	1	% change	2	% change	3	% change
t_d	85.00	74.246	-12.7%	76.471	3.0%	74.992	-1.9%
d	0.0301	0.02663	-11.5%	0.03080	15.7%	0.02901	-5.8%
Number of sites	-	27	-	28	3.7%	27	-3.6%
R^2	-	85.15	-	88.47	3.9%	88.01	-0.5%

Table 6-10 Iteration results for the acceleration transition percentile speed RE model

Parameter	ITERATION						
	0	1	% change	2	% change	3	% change
t_a	85.00	76.581	-9.9%	77.085	0.7%	77.085	0.0%
a	0.0167	0.01617	-3.2%	0.01760	9.0%	0.01762	0.0%
Number of sites	-	31	-	29	-6.5%	29	0.0%
R^2	-	85.17	-	88.54	4.0%	88.54	0.0%

The calibration process was stopped after the third iteration because there was no change in the site classification for the tangent and acceleration transition zone sub-samples and the speeds models could not be further improved. In addition, the number of sites assigned to the curve sub-sample went below the desired minimum after the first iteration; therefore, the curve model obtained from the first iteration had to be retained for subsequent iterations. This meant that eight sites were misclassified as curve sites for the second iteration and seven sites for the third iteration. Also, the transition zone models were calibrated using the OLS-PD approach because the deceleration and acceleration parameters were insignificant for the RE approach in all iterations. The results for the third iteration were selected as the final solution for the tangent and transition zones, while the curve model obtained from the first iteration was selected as the final solution.

The best specification of the RE model to calculate any percentile speed in tangent segments, in mph, is the following:

$$\begin{aligned} V_p &= 55.491 - 2.759 \times PSL_{50} + 0.430 \times GSW + 0.047 \times USW \\ &+ 7.905 \times Z_p + 1.302 \times \left(Z_p \times PSL_{50}\right) + 0.018 \times \left(Z_p \times T\right) + 0.056 \times \left(Z_p \times G\right) \\ &+ 0.227 \times \left(Z_p \times INT\right) - 0.139 \times \left(Z_p \times TW\right) - 0.011 \times \left(Z_p \times CLR\right) \end{aligned} \quad (6.9)$$

where:

TW = traveled way width, feet

CLR = total clear zone distance, includes the width of the total paved, gravel and untreated shoulder widths, feet

The best specification of the RE model to calculate any percentile speed in horizontal curves, in mph, is the following:

$$\begin{aligned} V_p &= 51.112 - 2.050 \times DC + 7.251 \times SE - 0.620 \times SE^2 \\ &+ 4.494 \times Z_p + 0.053 \times \left(Z_p \times T\right) - 0.001 \times \left(Z_p \times SD\right) + 0.194 \times \left(Z_p \times DC\right) - 0.199 \times \left(Z_p \times SE\right) \end{aligned} \quad (6.10)$$

The best specification of the RE model to calculate any percentile speed in the deceleration transition zone, in mph, is the following:

$$V_p = V_{Tp} - 0.7499 \times \left(V_{Tp} - V_{Cp}\right) + 0.02901 \times l_d \quad (6.11)$$

where:

V_{Tp} = estimated percentile speed on tangent from Equation 6.9, in ft/s

V_{Cp} = estimated percentile speed on horizontal curve from Equation 6.10, ft/s

The best specification of the RE model to calculate any percentile speed in the acceleration transition zone, in mph, is the following:

$$V_p = V_{Tp} - 0.7708 \times \left(V_{Tp} - V_{Cp}\right) + 0.01762 \times l_a \quad (6.12)$$

where:

V_{Tp} = estimated percentile speed on tangent from Equation 6.9, in ft/s

V_{Cp} = estimated percentile speed on horizontal curve from Equation 6.10, ft/s

The obtained RE models are different from the OLS-PD models by the variables included and their t-statistics. It indicates that omitting the random effects causes some bias in the model estimation. All the variables included in the models are significant with a 90 percent confidence level. The percentile effects in the panel data were also evaluated, but the variance attributed to the percentile dimension was practically insignificant compared to the variance attributed to sites and residuals. Consequently, adding the random effects due to the percentile dimension did not cause any change in the parameter estimates in the RE models.

6.2. Discussion of model results

6.2.1. Speed models without random effects

The OLS-PD model for tangent segments in Equation 6.5 includes ten different highway characteristics; six of them representing both mean speed and speed dispersion factors. The first intercept term and the following ten variables apply to the mean speed, while the second intercept (Z_p) and the five variables whose names start with Z_p apply to the standard deviation. A positive sign of a regression parameter in the first group of variables indicates that the variable increases the mean speed, while a positive sign of a regression parameter in the second group of variables indicates that the variable increases the variability of individual speeds.

The adjusted coefficient of determination (R^2) of the tangent model is high, indicating that 84.4 percent of the variability is explained. It must be admitted, however, that generating panel data along the percentile dimension creates speed data variability, which is principally explained with the Z_p factor (the higher the Z_p value, the higher the speed). The model standard deviation is 2.11 mph (3.38 km/h). Sixty-six percent of the mean speed estimates have residuals lower than 2.11 mph and only three percent of the mean estimates have residuals higher than 4.22 mph. This simple evaluation helped us to conclude that the model provides reasonable estimates.

The speed limit is the strongest mean speed and speed standard deviation factor. As expected, the speed limit of 50 mph (PSL_{50}=1) reduces the mean speed by approximately 3 mph (4.8 km/h), but at the same time, increases the variability of the individual speeds. The second finding may indicate that some drivers follow the speed limit closer than others and this difference in compliance further differentiates the individual speeds.

An increase in sight distance in the tangent increases the mean speed up to a specific value, as bounded by the linear and quadratic terms in the equation. Sight distances higher than 712.6 ft will not provide any additional increase in the tangent mean speed.

As expected, the increase in any of the lateral dimensions of the highway cross-section (PAV, GSW or USW) increases the mean speed. It is surprising that the gravel shoulder has the strongest impact. This might be explained with the strong visual contrast between the gravel and blacktop pavement, which improves the roadway delineation. Reducing the distance between the roadside obstructions and the travel lanes (LC) increases the spread of individual speeds. One possible interpretation is that cautious and slow drivers respond to an extra risk (narrow clear zone) stronger than fast and aggressive drivers.

The presence of an intersection within 350 ft (106.7 m) of any spot (INT=1) in the tangent segment slightly reduces the mean speed by 0.4 mph (0.64 km/h) while increasing the dispersion of the individual speeds. One interpretation for this impact is that cautious drivers respond to the extra risk presented by vehicles entering and exiting the intersection stronger than fast and aggressive drivers. An analogous interpretation applies to the 1 mph reduction in mean speeds due to a high residential development (RES=1) in a segment.

The effect on the speeds of the remaining variables in the model is easy to explain. As expected, an increase in the truck percentage reduces the mean speeds and an upgrade reduces the mean speed and increases the dispersion while a downgrade increases the mean speed and reduces the dispersion. The model provides an additional reduction in mean speeds when the spot is located on a flat curve.

The OLS-PD model for horizontal curves in Equation 6.6 includes four different highway and curve characteristics, two of them repeated as mean speed and speed dispersion factors. The first intercept term and the following five variables apply to the mean speed, while the second intercept (Z_p) and the two variables whose names start with Z_p apply to the standard deviation. The R^2 value of the OLD-PD model is quite high, indicating that 93.2 percent of the variability is explained.

All the sites included in the sample have a 55 mph (90 km/h) posted speed limit; therefore the speed limit cannot be included as a factor in the model. Anyway, the 55 mph limit is the highest state-mandated speed limit allowed for two-lane rural highways in Indiana; therefore, any speed reduction forced by the adverse curvature conditions in the sample can be considered to be the highest. In other words, a curve with sharp radii in a segment with a 55 mph posted speed limit is expected to compel a bigger speed reduction to negotiate the curvature than a comparable curve in a segment with a 40 mph posted speed limit.

The curve design elements are the strongest mean speed and speed standard deviation factors. As expected, an increase in the degree of curve (DC) reduces the mean speed and increases the speed dispersion. The impact mechanism for the maximum superelevation rate is not as clear. A linear and a quadratic factor for the maximum superelevation rate were included in the curve model, similar to the factors found for the sight distance in tangent segments. In this case, superelevation rates higher than 6.4 percent compel drivers to reduce mean speeds; although the net impact on mean speeds has to be studied along with the degree of the curve. It was already established by the results of the correlation analysis discussed in Section 6.1.2 that the degree of curve and the superelevation rate were positively correlated indicating that high superelevation rates were used to offset sharp curvature. AASHTO recommends that the design of curves should be based on an appropriate relationship between design speed and curvature and on their joint relationships with superelevation and side friction. Therefore, it is recommended that any change in the superelevation rate used to evaluate its impact on curve speeds needs to include the corresponding change in the degree of curve.

An increase in the available sight distance (SD) in the curve increases the mean speed. The increase in speed, in this case, is not bounded by a maximum value in sight distance, like in tangent segments. It has to be noted that the maximum sight distance observed in the sample inside a horizontal curve was around 1500 ft (467.2 m) and it is not recommended to use a higher sight distance value to predict speeds with the model.

A reduction in mean speed occurs in curves due to a high residential development in this segment. The impact is similar as the one found for tangent segments, but the reduction is 1.6 mph (2.57 km/h) higher.

The implication of the parameter estimates in the transition models are easy to explain. The deceleration transition model in Equation 6.7 establishes that 65.53 percent of the deceleration transition length occurs on the tangent segment prior to the curve. The model also establishes that the mean deceleration rate used by drivers in horizontal curves is 0.033 (ft/s) / ft. The acceleration transition model in Equation 6.8 establishes that 71.64 percent of the acceleration transition length occurs on the tangent segment following the curve. The model also establishes that the mean acceleration rate used by drivers in horizontal curves is 0.022 (ft/s) / ft.

6.2.2. Speed models with random effects

The RE model for tangent segments in Equation 6.9 includes eight different highway characteristics, only one is presented as both a mean speed and speed dispersion factor. The first intercept term and the following three variables apply to the mean speed, while the second intercept (Z_p) and the six variables whose names start with Z_p apply to the standard deviation. The truck percentage, sight distance, highway grade, pavement width and the binary variables for intersections and residential development were all removed as mean speed factors. The traveled way width was added as a speed dispersion factor.

The posted speed limit is again the strongest mean speed and speed standard deviation factor. This time, a speed limit of 50 mph (PSL_{50}=1) reduces the mean speed by approximately 2.8 mph (4.4 km/h), but at the same time, increases the variability of the individual speeds, although in a lower magnitude than in the OLS-PD model.

As expected, an increase in two of the shoulder dimensions (GSW or USW) increases the mean speed. The gravel shoulder width still has the strongest impact. Reducing the distance between the roadside obstructions and the travel lanes (CLR) and the width of the travel lanes (TW) increases the spread of individual speeds. The impact mechanism was already explained for the OLS-PD model. The grade, truck percentage and the intersection variable have the same effect on the dispersion of the individual speeds, although smaller, compared to the OLS-PD.

The RE model for horizontal curves in Equation 6.10 includes four different highway and curve characteristics, two of them representing both mean speed and speed dispersion factors. The first intercept term and the following three variables apply to the mean speed, while the second intercept (Z_p) and the four variables whose names start with Z_p apply to the standard deviation. All the sites included in the RE sample have a 55 mph posted speed limit; therefore the speed limit cannot be included as a factor in the model. An increase in the available sight distance (SD) in the curve reduces the speed dispersion; while an increase in the truck percentage (T) increases it.

As before, the curve design elements are the strongest mean speed and speed standard deviation factors. Although the impact is smaller than in the OLS-PD, an increase in the degree of curve (DC) reduces the mean speed and increases the speed dispersion. The impact mechanism for the maximum superelevation is the same as in the OLS-PD; although in this case, superelevation rates higher than 5.8 percent compel drivers to reduce mean speeds.

The implication of the parameter estimates in the transition models is comparable. The deceleration transition model in Equation 6.11 establishes that 74.99 percent of the deceleration transition length occurs on the tangent segment prior to the curve and the mean deceleration rate is approximately 0.029 (ft/s)/ft. The acceleration transition model in Equation 6.12 establishes that 77.08 percent of the acceleration transition length occurs on the tangent segment following the curve and the mean acceleration rate is approximately 0.018 (ft/s)/ft.

6.3. Evaluation of speed models

The performance of the speed models was evaluated by analyzing the model residuals and the sensitivity of the parameter estimates. The residual analysis involved comparing the observed mean speeds with the mean speeds estimated by the prediction models. The residuals were calculated by subtracting the estimated model values from the observed values. The sensitivity analysis included calculating the 10th percentile and 90th percentile values for the variables included in the models and comparing the partial effects of those values on the 85th percentile speed.

Figure 6-1 present the performance of the two speed models developed for tangent segments. The diagonal line in the graph represents a perfect correspondence between the speeds estimated by the models and the observed values. As the points get closer to the diagonal line, the closer the estimated value is to the observed value.

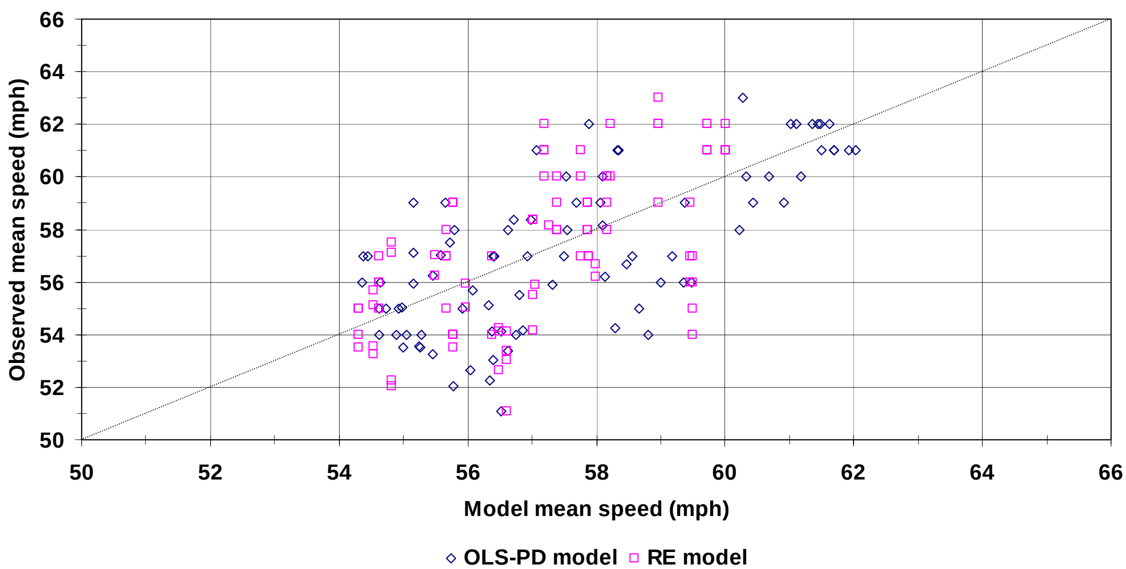


Figure 6-1 Performance of speed models for tangent segments

It can be observed that both models provide similar mean speed estimates and there is no apparent bias from the model estimates. The residual standard deviation, also known as the root mean square error (MSE), for the OLS-PD model is 3.62 km/h (2.25 mph). Six out of the 85 sites (7 percent) in the OLS-PD sample have residuals higher than two standard deviations (4.50 mph). Sixty sites (70.6 percent) in the OLS-PD sample have residuals smaller than 2.25 mph. SAS does not provide MSE values for the RE model, therefore the 4.5 mph value will be used for the comparison. Only 2 out of the 91 sites (2.2 percent) in the RE sample have residuals higher than 4.5 mph, while fifty-nine sites (64.8 percent) have residuals smaller than 2.25 mph. It can be concluded that although the RE model has a smaller range of residuals than the OLS-PD model, the OLS-PD model has a slightly higher percent of the estimates closer to the observed values.

The sensitivity of the estimated 85th percentile speed obtained from both models was calculated using the mean speed factors and the dispersion factors. The sensitivity represents a partial measure of the difference in the 85th percentile speed estimate by using extreme values in the variables included in the model. Table 6-11 presents the sensitivity evaluation for the OLS-PD model. Table 6-12 presents the sensitivity evaluation for the RE model. The tables show the 10th and 90th percentile values for the variables in both models. These values were set as 0 and 1 for all binary variables.

Table 6-11 Sensitivity of the speed estimate in the OLS-PD tangent model

Parameter	Estimate	10th perc. value	Partial effect	90th perc. value	Partial effect	Speed sensitivity
PSL_{50}	-3.0818	0	0.00	1	-3.08	-3.08
T	-0.0710	6.20	-0.44	23.69	-1.68	-1.24
SD	0.0024	514.58	1.22	712.60	1.70	-
SD^2	-1.670E-06	264792.58	-0.44	507798.76	-0.85	-
$SD - SD^2$			0.78		0.85	0.07
G	-0.1307	0.10	-0.01	4.29	-0.56	-0.55
RES	-1.0338	0	0.00	1	-1.03	-1.03
INT	-0.4216	0	0.00	1	-0.42	-0.42
PAV	0.0401	21.00	0.84	40.25	1.62	0.77
GSW	0.3941	0.00	0.00	7.00	2.76	2.76
USW	0.0544	10.00	0.54	48.00	2.61	2.07
Z_p-PSL_{50}	1.4280	0	0.00	1.04	1.48	1.48
Z_p-G	0.0608	0.10	0.01	4.45	0.27	0.26
Z_p-T	0.2917	0	0.00	1.04	0.30	0.30
Z_p-INT	-0.0383	21.76	-0.83	41.72	-1.60	-0.76
Z_p-CLR	-0.0118	17.62	-0.21	63.22	-0.75	-0.54

The speed limit and the gravel and untreated shoulders are the only mean speed factors in both models that have speed sensitivities equal or higher than 1.8 mph (2.9 km/h). The sensitivities of these three parameters are somewhat lower in the RE model. The gravel shoulder width provides a mean speed sensitivity of 2.7 mph in the OLS-PD and 3 mph (4.8 km/h) in the RE model. Each one of the other six mean speed factors in the OLS-PD has a sensitivity of 1 mph (1.6 km/h) or less. These six mean speed factors were not included in the RE model. This can be interpreted as that the variance attributed to these six variables in the OLS-PD is now attributed to the site random variable in the RE model.

Table 6-12 Sensitivity of the speed estimate in the RE tangent model

Parameter	Estimate	10^{th} perc. value	Partial effect	90^{th} perc. value	Partial effect	Speed sensitivity
PSL_{50}	-2.759	0	0.00	1	-2.76	-2.76
GSW	0.430	0.00	0.00	7.00	3.01	3.01
USW	0.047	10.00	0.47	48.00	2.28	1.80
Z_p-PSL_{50}	1.302	0	0.00	1.04	1.35	1.35
Z_p-G	0.056	-2.49	-0.14	3.01	0.17	0.31
Z_p-T	0.018	6.72	0.12	24.08	0.42	0.31
Z_p-INT	0.227	0	0.00	1.04	0.24	0.24
Z_p-TW	-0.139	21.76	-3.02	25.70	-3.56	-0.55
Z_p-CLR	-0.011	17.62	-0.20	63.22	-0.72	-0.52

The combined cross section elements in both models have the highest sensitivity in the 85^{th} percentile speed estimate with over 4 mph (6.4 km/h), considering only the gravel and untreated shoulder widths. Variables that are included as both mean and dispersion factors need to account for the sensitivity of both factors in the 85^{th} percentile speed estimate. For example, the posted speed limit reduces 85^{th} percentile speed between 1.4 mph and 1.6 mph. This value in the OLS-PD was calculated by adding the mean speed sensitivity of -3.08 mph and the dispersion sensitivity of 1.48 mph. The other dispersion factors in both models have a sensitivity of 1 mph (1.6 km/h) or less in the 85^{th} percentile speed estimate.

Figure 6-2 present the performance of the two speed models developed for horizontal curves. The OLS-PD mean speed estimates seem to be closer to the observed values. It can also be observed that both models do not show any apparent bias in their estimates. The residual standard deviation for the OLS-PD model is 1.75 mph (2.83 km/h). Only one out of the 14 sites (7 percent) in the OLS-PD sample has a residual higher than two standard deviations (3.50 mph). Eleven sites (78.6 percent) in the OLS-PD sample have residuals smaller than 1.75 mph. Although only one site (5.6 percent) in the RE sample has a residual higher than two standard deviations (3.50 mph); only 10 out of the 18 sites (55.6 percent) have residuals smaller than 1.75 mph. The OLS-PD model provides slightly better speed estimates than the RE model. The lesser performance of the RE model might be the result of not being able to calibrate the curve model for the third iteration and having to keep the model obtained in the first iteration.

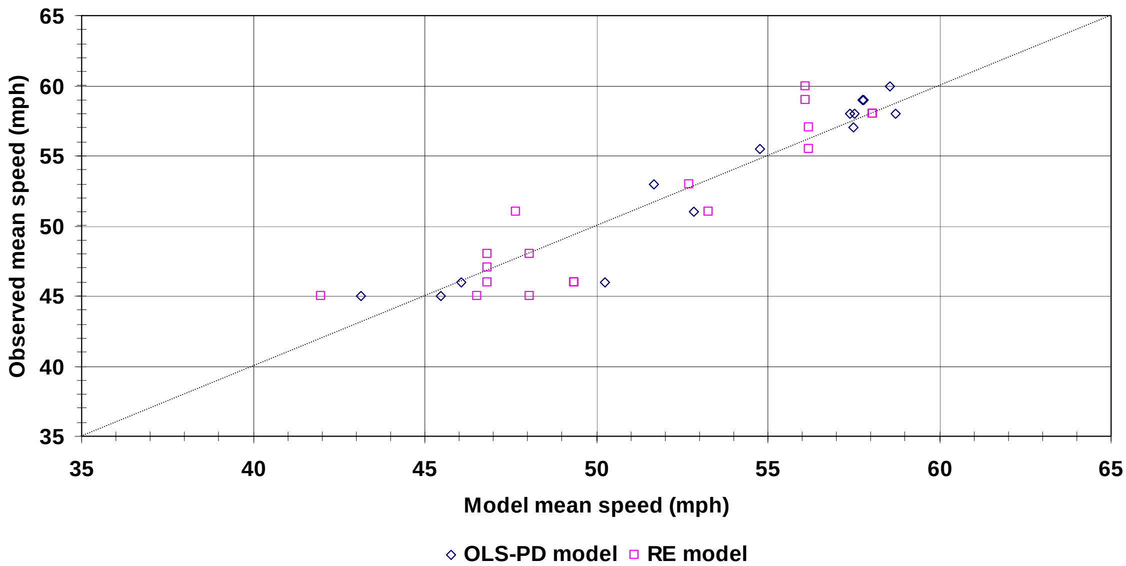


Figure 6-2 Performance of speed models for horizontal curves

The sensitivity of the speed estimates in both models was calculated using the mean speed and the dispersion factors included in the models. Table 6-13 presents the sensitivity evaluation for the OLS-PD model. Table 6-14 presents the sensitivity evaluation for the RE model.

Table 6-13 Sensitivity of the speed estimate in the OLS-PD horizontal curve model

Parameter	Estimate	10^{th} perc. value	Partial effect	90^{th} perc. value	Partial effect	Speed sensitivity
SD	3.44E-03	357.75	**1.23**	1114.50	**3.83**	**2.60**
RES	-2.639	0	**0.00**	1	**-2.64**	**-2.64**
DC	-2.541	3.84	**-9.77**	10.74	**-27.29**	**-17.52**
SE	7.954	2.74	**21.75**	8.92	**70.95**	**-**
SE^2	-0.624	7.48	**-4.67**	79.57	**-49.65**	**-**
$SE – SE^2$	-	-	**17.09**	-	**21.30**	**4.21**
Z_p-DC	0.236	3.98	**0.94**	11.13	**2.62**	**1.69**
Z_p-SE	-0.199	2.83	**-0.56**	9.24	**-1.84**	**-1.27**

As expected, the degree of curve and the superelevation rate provide the highest sensitivity of the mean speed factors in both models with more than 4 mph (6.44 km/h). The degree of curve by itself has a sensitivity of more than 11 mph (17.7 km/h). The other two mean speed factors in the

OLS-PD, sight distance and high residential variable, also have notable sensitivities with more than 2.5 mph (4 km/h). These two mean speed factors were not included in the RE model.

Table 6-14 Sensitivity of the speed estimate in the RE horizontal curve model

Parameter	Estimate	10^{th} perc. value	Partial effect	90^{th} perc. value	Partial effect	Speed sensitivity
DC	-2.050	5.74	**-11.77**	11.33	**-23.21**	**-11.44**
SE	7.251	5.26	**38.10**	9.40	**68.16**	**-**
SE^2	-0.620	27.62	**-17.12**	88.36	**-54.79**	**-**
$SE – SE^2$	-	-	**20.98**	-	**13.36**	**-7.61**
Z_p-T	0.053	7.61	**0.40**	16.84	**0.89**	**0.49**
Z_p-SD	-8.2E-04	371.45	**-0.30**	1169.97	**-0.96**	**-0.65**
Z_p-DC	0.194	5.95	**1.15**	11.74	**2.28**	**1.12**
Z_p-SE	-0.199	5.45	**-1.09**	9.74	**-1.94**	**-0.86**

The degree of curve and the superelevation rate are the only speed dispersion factors included in the OLS-PD model. The sensitivity in the 85^{th} percentile speed estimate of the degree of curve is 15.8 mph (25.5 km/h) in the OLS-PD and 12.5 mph (20.1 km/h) in the RE model. The sensitivity in the 85^{th} percentile speed estimate of the superelevation rate is 2.9 mph (4.7 km/h) in the OLS-PD and 6.8 mph (10.9 km/h) in the RE model. The other two dispersion factors included in the RE model, sight distance and percent trucks, have sensitivities of less than 1 mph (1.6 km/h).
The sensitivity analysis of both models showed that the cross-section components and the curve elements provide the biggest opportunity for improving operating speeds in tangent segments and horizontal curves, respectively. There are additional highway characteristics in the speed models that provide designers a minor improvement in operating speeds.

Figure 6-3 present the performance of the two speed models developed for the tangent-to-curve transition section. The OLS approach was used to calibrate the models in both cases. It can be observed that the model developed for the RE process provides better mean speed estimates. The adjusted R^2 value for the OLS-PD and the RE is 0.84 and 0.88, respectively. The residual standard deviation for the OLS-PD model is 6.16 mph (9.9 km/h) while the residual standard deviation for the RE model is near 5 mph (8 km/h).

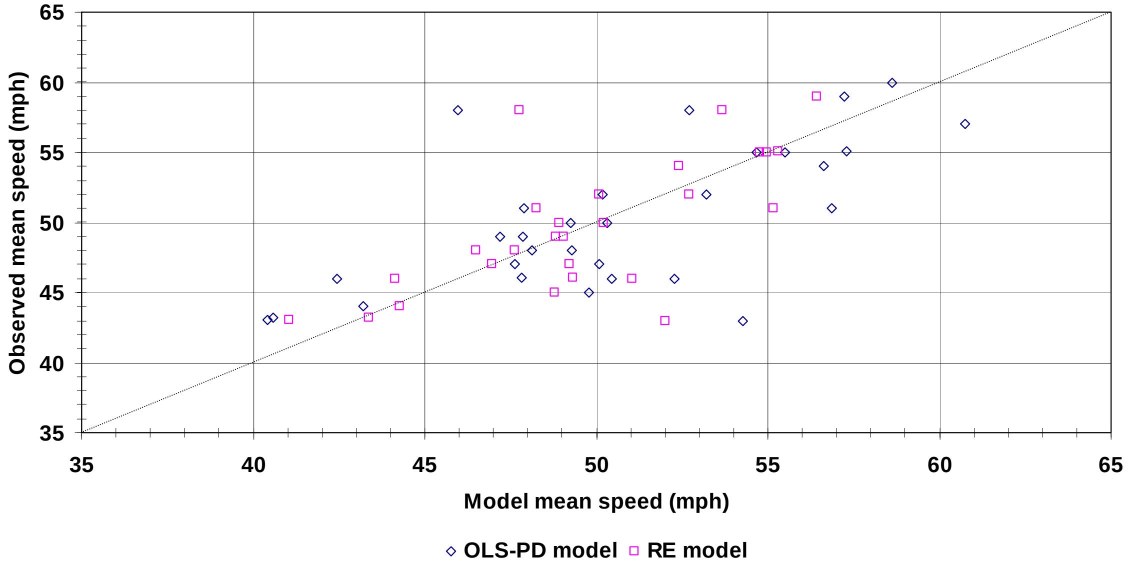


Figure 6-3 Performance of speed models for deceleration transition zones

Figure 6-4 present the performance of the two speed models developed for the curve-to-tangent transition sections. Both models provide comparable mean speed estimates. The adjusted R^2 values for the OLS-PD and the RE models are almost identical, 0.876 and 0.885, respectively. The residual standard deviation for the OLS-PD model is 5.8 mph (9.3 km/h) while the residual standard deviation for the RE model is near 5.1 mph (8.2 km/h).

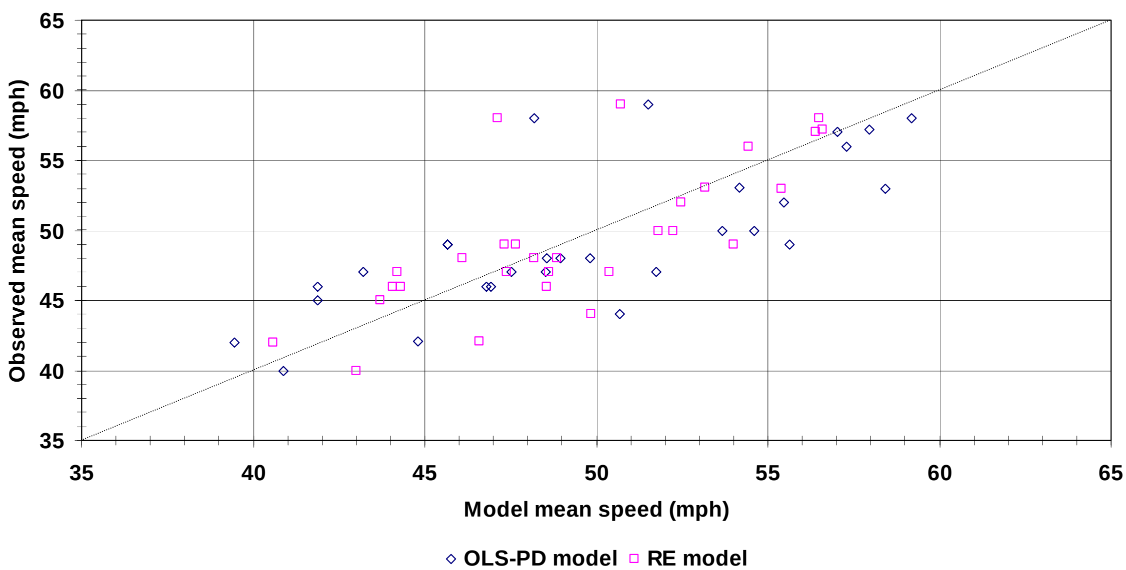


Figure 6-4 Performance of speed models for acceleration transition sections

6.4. Comparing the traditional and proposed models

The results presented in previous sections showed that the two speed models proposed provide reasonable estimates and have comparable performance and parameter sensitivities. The results of a comparison between the two proposed models and the traditional OLS model, shown in Equation 2.1, are discussed in this section. The data from 32 tangent sites were used to develop the three different speed models. All variables included in the models are significant at a ten percent level.

The best specification of the traditional OLS 85th percentile speed model, in mph, is the following:

$$V_{85} = 63.983 - 3.592 \times PSL_{50} + 0.273 \times GSW$$

The traditional OLS model consists only of the binary variable PSL_{50} and the gravel shoulder width (GSW). A 50-mph posted speed limit reduces the 85th percentile speed by nearly 3.6 mph (5.8 km/h); while an increase in the GSW increases the 85th percentile speed, as expected. The adjusted R^2 value of the traditional OLS model is 0.74.

The best specification of the OLS-PD model, in mph, is the following:

$$\begin{aligned} V_p &= 56.733 - 5.430 \times PSL_{50} - 0.209 \times G + 0.036 \times PAV + 0.422 \times GSW + 0.037 \times USW \\ &+ 5.235 \times Z_p + 1.908 \times \left(Z_p \times PSL_{50}\right) + 0.115 \times \left(Z_p \times G\right) - 0.018 \times \left(Z_p \times CLR\right) \end{aligned}$$

The best specification of the RE model, in mph, is the following:

$$\begin{aligned} V_p &= 57.980 - 5.694 \times PSL_{50} + 0.371 \times GSW + 0.036 \times USW \\ &+ 6.035 \times Z_p + 1.739 \times \left(Z_p \times PSL_{50}\right) + 0.115 \times \left(Z_p \times G\right) \\ &- 0.022 \times \left(Z_p \times PAV\right) - 0.061 \times \left(Z_p \times GSW\right) - 0.021 \times \left(Z_p \times USW\right) \end{aligned}$$

The first notable difference between the proposed models and the traditional OLS model is the number of variables included. The OLS-PD model includes five mean speed factors with two of them working also as speed dispersion factors; while the RE model includes five speed dispersion variables with three of them working also as mean speed factors. This difference emphasizes the value of the proposed models in estimating the impacts of different design parameters on speeds.

The second difference is the amount of variability explained by the proposed models. The adjusted R^2 value for the OLD-PD model is quite high, indicating that 93.8 percent of the variability is explained. The adjusted R^2 value for the RE model is similar, indicating that 93.4

percent of the variability is explained. It must be admitted, however, that generating panel data along the percentile dimension creates speed data variability, which is explained with Z_p.

The third difference is the ability of the proposed models to easily quantify the impacts of the variables on mean speeds and on the speed standard deviation. The OLS model combines the effects on the mean speed and its standard deviation, obscuring the impact mechanism. In the proposed models, the mean speed factors may be considered as crash severity factors (severe crashes happen at high speeds), while the speed dispersion factors may be considered as crash frequency factors (speed variability increases interactions between vehicles). For example, the first intercept term and the following five variables in the OLS-PD apply to the mean speed, while the second intercept (Z_p) and the three variables whose names start with Z_p apply to the standard deviation. The impact mechanism was already explained for the models in Section 6-2.

Figure 6-5 presents a residual plot grouped by percentiles for the OLS-PD. Near-symmetric distribution of points around zero indicates that the distribution of normality is approximately met and no systematic bias is associated with specific percentiles. Figure 6-6 presents a residual plot grouped by sites. This time, there are obvious upward and downward shifts of the residuals. These shifts are caused by unknown factors not incorporated in the OLS-PD model that are apparently related to site characteristics omitted in the model. This calls for an improvement in the OLS-PD model by adding site-specific random effects in the RE model to avoid bias in estimating the model parameters.

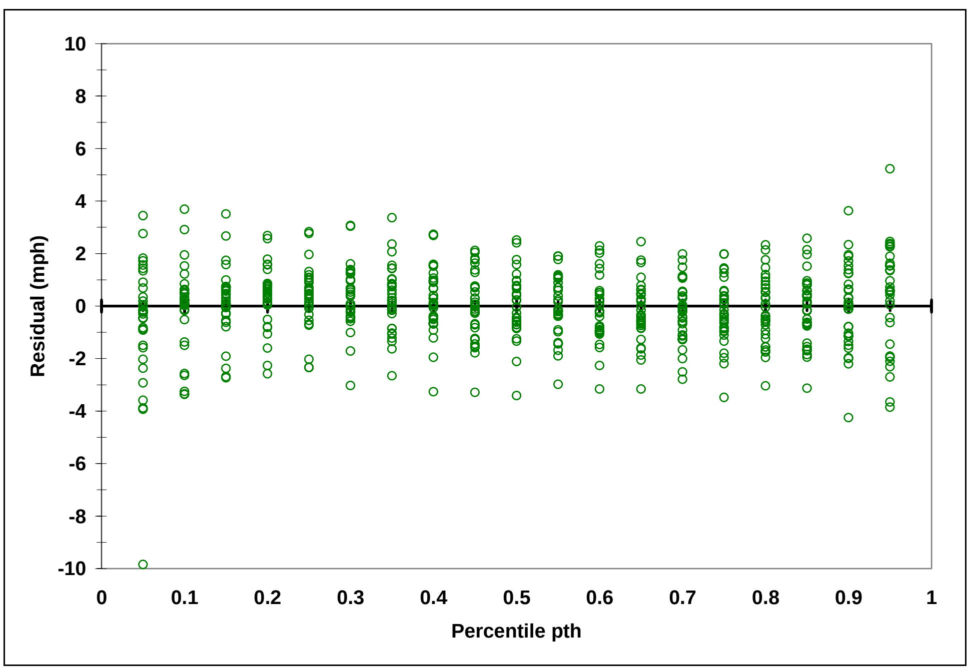


Figure 6-5 Residuals of OLS-PD model arranged by percentiles

The obtained RE model is different from the OLS-PD model by the mean speed and speed dispersion variables included and their t-statistics. It indicates that omitting the random effects causes some bias in the model estimation. The signs in front of the terms representing cross-section dimensions are positive as expected, but the PAV and GR variables became insignificant. The Z_p-ZONW variable included was replaced with the three shoulder type variables.

The Lagrange multiplier statistic tests if adding the random effect term is justified. The large value of the Lagrange statistic (with a p-value of approximately zero) argues in favor of the RE over the OLS-PD model. Most of the unexplained variance (1.58) is attributed to the unknown site-specific factors (μ) and a small portion of the remaining unexplained variance (0.79) is attributed jointly to residuals and percentiles (ε). The percentile effects in the panel data were evaluated, but the variance attributed to the percentile dimension was practically insignificant (0.05) compared to the variance attributed to sites (1.58) and residuals (0.74). Also, adding the random effects due to the percentile dimension did not cause any significant change in the parameter estimates in the model. This finding supports the inference made related to the OLS-PD residual behavior shown in Figure 6-5 as to having no bias associated to the percentile dimension.

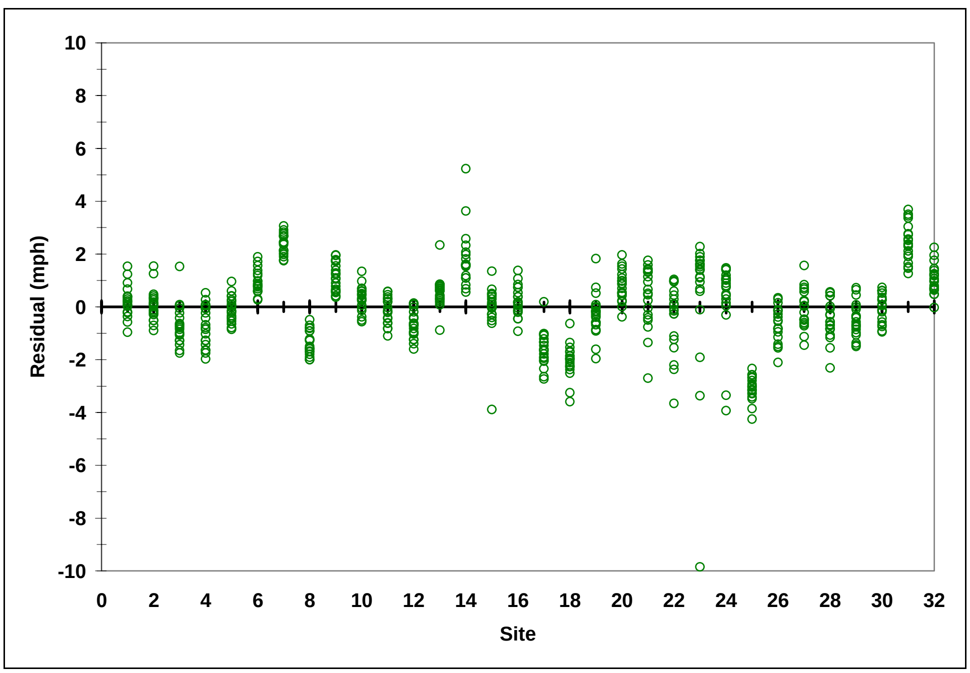


Figure 6-6 Residuals of OLS-PD model arranged by sites

Figure 6-7 presents a plot of measured versus estimated 85th percentile speeds using both models. The plot shows that both models provide similar 85th percentile speed estimates. Even though the adjusted R^2 value for the RE model is higher than for the traditional OLS model, 0.934

and 0.740 respectively, the error sum of squares (SSE) was used to evaluate the performance of the models. If all the Y_i observations fall on the fitted regression, SSE is equal to zero and the model is a perfect fit. The larger SSE is, the greater the variation of the Y_i observations around the fitted regression. The traditional OLS model has practically the same SSE value (59.25) as the RE model (59.51) when estimating only the 85th percentile speeds; therefore the performance of both models is comparable.

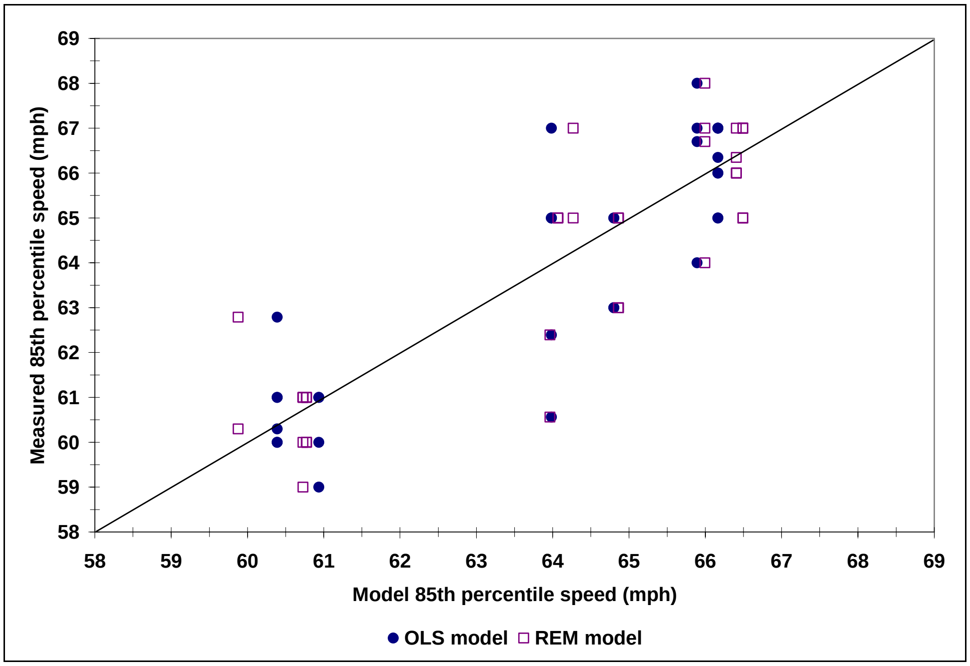


Figure 6-7 Performance of the traditional OLS and RE models in estimating 85th percentile speed

The proposed models have the same capabilities as a traditional OLS model in predicting the 85th percentile speeds. Their three main advantages include predicting any user-specified percentile, involving more design variables than traditional OLS models, and separating the impacts on mean speed from the impacts on speed dispersion.

CHAPTER 7. SPEED PREDICTING MODELS FOR FOUR-LANE HIGHWAYS

This chapter presents the results of the speed modeling for four-lane highways. The model development, results and performance evaluation are discussed for the two proposed approaches for modeling panel data: OLS without random effects and GLS with random effects.

7.1. Development of speed models

The model development procedure was simplified for four-lane highways. Not enough highway segments were found with horizontal curves that induce drivers to reduce their speeds to be able to calibrate speed models for those locations. Therefore, only a single model for four-lane highway segments was calibrated. The calibration process used the free-flow speeds and the road geometry information collected for 50 sites. The speed data was used to calculate from the 5^{th} to the 95^{th} percentile, in multiples of five. All the potential explanatory variables were multiplied by the Z_p value corresponding to each percentile to assemble the panel data.

The Pearson correlation coefficient r was calculated to identify which highway characteristics, cross-section components and curve design elements have a linear relation with the mean speed or the 85^{th} percentile speed. Significant correlations were identified with a 95 percent confidence level. The posted speed limit and the right-of-way width have the strongest positive linear relationships with the two speeds ($r > 0.59$). In addition, the truck percentage, the rural area variable and the sight distance have smaller positive relationships with the two speeds ($r < 0.45$). In contrast, the access density, the high residential and commercial development variables and the presence of curbs, sidewalks and TWLT median lane variables have a negative relationship ($r > 0.30$) with the two speeds. The degree of curve and the superelevation rate do not have a significant linear relationship with the speeds.

The total cross-section width is positively correlated with the posted speed limit ($r = 0.52$), as expected, while it is negatively correlated ($r > 0.33$) with the access density, high residential development, curbed segments and on the presence of TWLT median lanes. On the other hand, the posted speed limit is positively correlated with rural areas ($r = 0.36$) and negatively correlated with high commercial development, curbs, sidewalks and TWLT median lanes (r > 0.27). In terms of other variables, the access density tends to increase with the presence of curbs, high residential development, undivided highways and TWLT median lanes (r > 0.30). As expected, the access density tends to be lower in rural areas (r = -0.28).

The SAS output for the developed speed models for four-lane highway segments is shown in Appendix C. The best specification of the OLS-PD model to calculate any percentile speed in four-lane highway segments, in mph, is the following:

$$\begin{aligned} V_p &= 53.884 - 4.753 \times PSL_{50} - 5.481 \times PSL_{45} - 7.432 \times PSL_{40} + 2.045 \times RUR \\ &+ 8.711 \times 10^{-4} \times SD - 0.279 \times INTD - 0.023 \times DRWD + 1.732 \times PS \\ &+ 0.020 \times ECLR + 0.046 \times ICLR - 2.102 \times RAIL - 1.193 \times DITCH \\ &+ 6.051 \times Z_p - 0.496 \times (Z_p \times PSL_{45-40}) - 0.585 \times (Z_p \times RUR) \\ &- 4.194 \times 10^{-4} \times (Z_p \times SD) - 0.011 \times (Z_p \times ECLR) - 0.430 \times (Z_p \times TWLT) \end{aligned} \tag{7.1}$$

where:

PSL_{50} = equal to 1 if the posted speed limit is 50 mph; 0 otherwise

PSL_{45} = equal to 1 if the posted speed limit is 45 mph; 0 otherwise

PSL_{40} = equal to 1 if the posted speed limit is 40 mph; 0 otherwise

PSL_{45-40} = equal to 1 if the posted speed limit is 45 or 40 mph; 0 otherwise

RUR = equal to 1 if the segment is in a rural area; 0 otherwise

SD = sight distance, feet

INTD = intersection density; number of intersections per mile

DRWD = driveway density; number of adjacent driveways per mile

PS = equal to 1 if the highway segment has a paved shoulder; 0 otherwise

ECLR = external clear zone, lateral clearance distance measured from the exterior edge of the traveled way to the face of the roadside obstruction, feet

ICLR = internal clear zone, lateral clearance distance measured from the interior edge of the traveled way to the inside edge of the opposing traveled way or to the median barrier face, if a barrier is present in the median, feet

RAIL = equal to 1 if a guardrail is located 20 feet or less from the outside edge of the traveled way; 0 otherwise

DITCH = equal to 1 if the middle of a ditch is located 20 feet or less from the edge of the traveled way; 0 otherwise

TWLT = equal to 1 if a two-way left turn median lane is present; 0 otherwise

Z_p = standardized normal variable corresponding to a selected percentile, see Table C-1

The best specification of the RE model to calculate any percentile speed in four-lane highway segments, in mph, is the following:

$$\begin{aligned} V_p &= 54.027 - 4.764\times PSL_{50} - 4.942\times PSL_{45} - 6.509\times PSL_{40} + 1.652\times RUR \\ &+ 1.281\times 10^{-3}\times SD - 0.320\times INTD + 0.034\times ECLR + 0.056\times ICLR \\ &+ 5.899\times Z_p - 0.423\times\left(Z_p\times PSL_{45-40}\right) - 0.464\times\left(Z_p\times RUR\right) - 4.800\times 10^{-4}\times\left(Z_p\times SD\right) \\ &+ 0.042\times\left(Z_p\times INTD\right) - 4.220\times 10^{-3}\times\left(Z_p\times CLR\right) - 0.477\times\left(Z_p\times TWLT\right) \end{aligned} \tag{7.2}$$

where:

CLR = total clear zone, includes the median width and external clear zone, feet

The obtained RE model is very similar to the OLS-PD model; most of the variables are included in both models, although their estimates and t-statistics are different. It indicates that omitting the random effects causes some bias in the model estimation. All the variables included in the models have t-statistic values higher than 1. The significance requirements were relaxed for the RE model to include variables that have a practical estimate value. The percentile effects in the panel data were also evaluated, but the variance attributed to the percentile dimension was insignificant compared to the variance attributed to sites and residuals. Consequently, adding the random effects due to the percentile dimension did not cause any change in the parameter estimates in the RE model.

7.2. Discussion of model results

The OLS-PD in Equation 7.1 includes fourteen different highway characteristics, five of them included as both mean speed and speed dispersion factors. The first intercept term and the following twelve variables apply to the mean speed, while the second intercept (Z_p) and the five variables whose names start with Z_p apply to the standard deviation. The impact mechanism explained in Chapter 6 for the models in two-lane rural highways applies also for four-lane highways. The R^2 value of the OLD-PD model is high, indicating that 86 percent of the variability is explained.

The posted speed limit is the strongest mean speed factor; but not as strong as a speed standard deviation factor. Three binary variables were used to represent the speed limits lower than 55 mph. As expected, as the speed limit decreases the mean speed also decreases. It is surprising that the net impact on the mean speed diminishes with each additional 5-mph reduction in speed limit. At the same time, segments with speed limits lower than 50 mph have reduced dispersion of individual speeds. This finding somewhat contradicts the positive impact on dispersion found for the 5-mph speed limit reduction in two-lane rural highways. This might be indicative that drivers may be insensitive to the 5-mph reduction (from 55-mph to 50-mph) and may not perceive the need of reducing their speed for comparable suburban highway segments. The reduction in dispersion may indicate that drivers behave more uniformly on suburban highways with the lower speed limits due to more restricted highway conditions (narrower cross-section) or to the higher likelihood of police enforcement.

Another important mean speed factors on four-lane highways are the intersection (INTD) and driveway (DRWD) densities. The mean speed decreases as the number of intersections and driveways per mile increases in the highway segment. The impact on the speed is obvious as drivers respond to the extra risk presented by the vehicles entering and exiting the highway.

As expected, an increase in the external and internal clear zones increases the mean speed, and at the same time, roadside obstructions (guardrails and ditches) located at 20 feet or less from the outside edge of the traveled way impact negatively the mean speed. Reducing the external clear zone increases the spread of the individual speeds as cautious and slow drivers respond to the extra risk represented by the narrower clearance distance more strongly than fast and aggressive drivers. The median type also has an important effect on the speed standard deviation. The presence of TWLT median lanes reduces the speed dispersion. The TWLT median lanes provide some sense of separation between opposing traffic lanes and also allow vehicles to enter and exit the traveled way in a more effective and safe way, thus reducing the impact on the quality of the traffic flow.

The impact of the rest of the variables in the model is easy to understand. An increase in the available sight distance (SD) in the segment increases mean speeds while reducing the dispersion. Highway segments in rural areas (RUR) have higher mean speeds and lower dispersion mainly because of the higher posted speed limits, wider cross-section dimensions and lower intersection and driveway densities compared to segments in suburban areas.

The RE model in Equation 7.2 includes ten different highway characteristics; seven of them are included as both mean speed and speed dispersion factors. The first intercept term and the following eight variables apply to the mean speed, while the second intercept (Z_p) and the six variables whose names start with Z_p apply to the standard deviation. The impacts of the RE variables are comparable to those found in the OLS-PD model, with some variations.

Similar to the OLS-PD, the posted speed limit is the strongest mean speed factor; but not as strong as a speed standard deviation factor. The same three binary variables were used to represent the speed limits. As before, speed limits set below 55 mph reduce the mean speed, but this time the reduction is smaller for the posted speed limits lower than 50 mph. A similar impact in dispersion was found. Speed limits set lower than 50 mph reduce the dispersion of the individual speeds, although to a smaller amount.

The impact in mean speed by the intersection density (INTD) is similar to the OLS-PD; although the intersection density was found to also increase the speed dispersion in the RE model. The explanation is the same as cautious drivers respond to the extra risk presented by vehicles entering and exiting the intersection stronger than fast and aggressive drivers.

Again, an increase in any of the two clear zone distances of the highway cross-section increases the mean speed, although now both clear zones are combined in the RE model as the total clear zone distance (CLR). The impact of the individual clear zones is lost, but the impact mechanism is the same. Reducing the width of the total clear zone distance increases the spread of individual speeds as cautious and slow drivers respond to the extra risk represented by a narrower highway segment. As in the OLS-PD model, the presence of TWLT median lanes has a similar important effect on the speed standard deviation in the RE model.

The impact of the rest of the variables in the model is obvious. An increase in the available sight distance (SD) in the segment increases the mean speed and reduces the dispersion. Highway segments in rural areas (RUR) have higher mean speeds and lower dispersion because of the higher posted speed limits, wider cross-section dimensions and lower intersection and driveway densities compared to segments in suburban areas.

7.3. Evaluation of speed models

The performance of the speed models was evaluated by analyzing the mean speed residuals and the sensitivity of the 85th percentile speed estimates. The sensitivity analysis included calculating the 10th percentile and 90th percentile values for the variables included in the models and comparing the partial effects of those values on the estimated 85th percentile speed.

Figure 7-1 presents the performance of the mean speed estimates for the two models developed for four-lane highways. The diagonal line in the graph represents a perfect correspondence between the mean speeds estimated by the model and the observed mean speeds. It can be observed that both models provide similar mean speed estimates and there is no apparent bias from the model estimates. The residual standard deviation for the OLS-PD model is 2.35 mph (3.8 km/h).

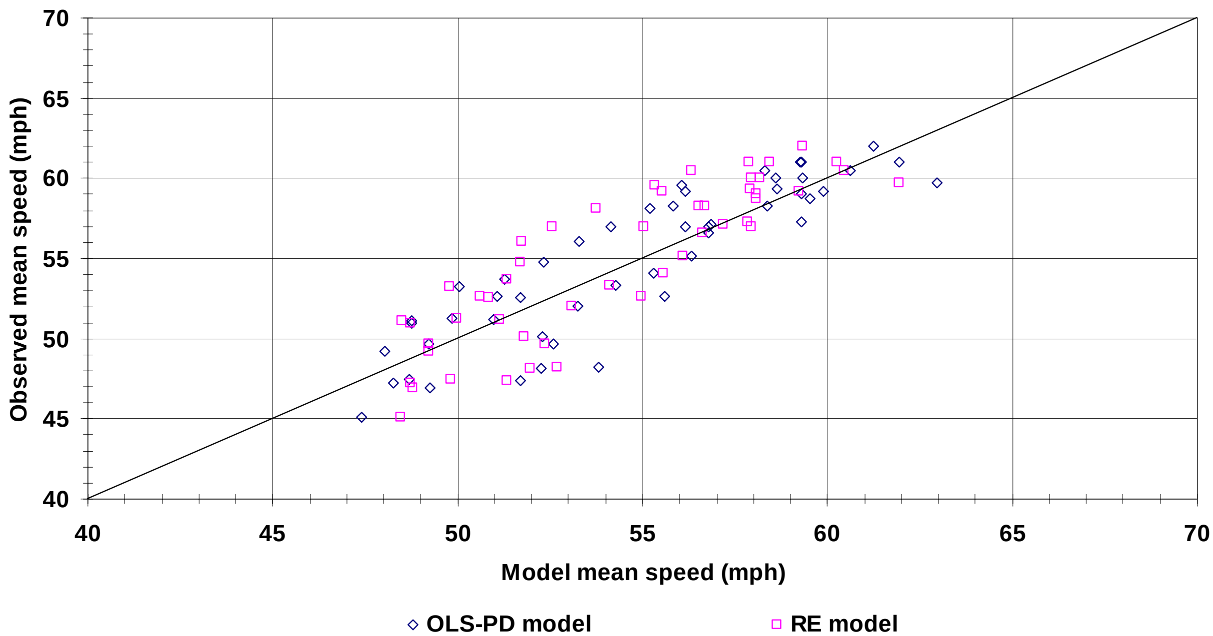


Figure 7-1 Performance of speed models for four-lane highways

Seventy percent of the mean speed estimates in the OLS-PD sample have residuals smaller than one standard deviation (2.35 mph). SAS does not provide MSE values for the RE model; therefore the same value was used to make the residual comparison. Sixty-four percent of the mean speed estimates in the RE sample have residuals smaller than 2.35 mph. It was concluded that both models provide comparable mean speed estimates, but the RE model does a better predicting job than the OLS-PD model, by keeping all residuals to less than 4.5 mph.

The sensitivity of the 85th percentile speed estimates was calculated using the mean speed and the speed dispersion factors included in both models. Table 7-1 presents the sensitivity evaluation for the OLS-PD model. Table 7-2 presents the sensitivity evaluation for the RE model. The tables present the 10th and 90th percentile values for all the variables in both models. These threshold values were set as 0 and 1 for all binary variables.

The three speed limit variables are the mean speed factors with the highest sensitivity in both models. The mean speed sensitivities of the posted speed limit variables range from 4.75 to 7.43 mph. The mean speed sensitivity of the two lowest speed limit variables is lower in the RE model. In terms of the 85th percentile speed sensitivity of the OLS-PD estimate, a 50-mph limit reduces the speed by 4.75 mph, a 45-mph limit reduces it by almost 6 mph and a 40-mph limit reduces it by almost 8 mph. For the RE 85th percentile speed estimate, a 50-mph limit reduces the speed by almost 4.76 mph, a 45-mph limit reduces it by 5.37 mph and a 40-mph limit reduces it by almost 7 mph.

Table 7-1 Sensitivity of the speed estimate in the OLS-PD model

Parameter	Estimate	10th perc. value	Partial effect	90th perc. value	Partial effect	Speed sensitivity
PSL_{50}	-4.753	0	0	1	-4.753	-4.753
PSL_{45}	-5.481	0	0	1	-5.481	-5.481
PSL_{40}	-7.432	0	0	1	-7.432	-7.432
RUR	2.045	0	0	1	2.045	2.045
SD	0.000871	678.8	0.591	1833.2	1.597	1.006
INTD	-0.279	0.0	0	8.0	-2.232	-2.232
DRWD	-0.023	0.0	0	24.0	-0.552	-0.552
PS	1.732	0	0	1	1.732	1.732
ECLR	0.02	5.4	0.108	42.6	0.852	0.744
ICLR	0.046	0.0	0	60.5	2.783	2.783
RAIL	-2.102	0	0	1	-2.102	-2.102
DITCH	-1.193	0	0	1	-1.193	-1.193
Z_p-$PSL_{45\text{-}40}$	-0.496	0	0	1	-0.514	-0.514
Z_p-RUR	-0.585	0	0	1	-0.606	-0.606
Z_p-SD	-0.00042	678.8	-0.295	1833.2	-0.796	-0.502
Z_p-ECLR	-0.011	5.4	-0.062	42.6	-0.485	-0.424
Z_p-TWLT	-0.43	0	0	1	-0.446	-0.446

The cross-section dimensions also show high sensitivities. The external and internal clear zone distances included in the OLS-PD model account for a mean speed sensitivity of 0.7 mph and 2.8 mph, respectively; while the same elements in the RE model account for a mean speed sensitivity of 1.3 mph and 3.4 mph, respectively. The 85th percentile speed sensitivity of the external clear zone distance, included as a speed dispersion factor in the OLS-PD, increases that speed by just 0.3 mph. The sensitivity might be indirectly affected by the inclusion of the binary variables related to the presence of paved shoulders, and guardrails and ditches on the roadside. All of those binary variables affect the mean speed by more than 1 mph.

The sensitivity of the intersection density variable in both models accounts for a reduction in the 85th percentile speed of 2.2 mph. In contrast, the sensitivity of the driveway density variable accounts for a reduction in 85th percentile speed of just 0.55 mph in the OLS-PD model. The driveway density was not included as a significant variable in the RE model. The rural area binary variable is the other variable in the OLS-PD and the RE models that has a sensitivity of more than 1 mph in the 85th percentile speed estimate. The sensitivity of the sight distance variable estimate model accounts for an increase in the 85th percentile speed of just 0.5 mph in the OLS-PD model and 0.9 mph in the RE model.

Table 7-2 Sensitivity of the speed estimate in the RE model

Parameter	Estimate	10th perc. value	Partial effect	90th perc. value	Partial effect	Speed sensitivity
PSL_{50}	-4.764	0	0	1	-4.764	-4.764
PSL_{45}	-4.942	0	0	1	-4.942	-4.942
PSL_{40}	-6.510	0	0	1	-6.510	-6.510
RUR	1.652	0	0	1	1.652	1.652
SD	0.00128	678.8	0.869575	1833.2	2.348	1.479
INTD	-0.320	0.0	0	8.0	-2.563	-2.563
ECLR	0.034	5.4	0.185058	42.6	1.460	1.275
ICLR	0.056	0.0	0	60.5	3.400	3.400
Z_p-PSL_{45-40}	-0.423	0	0	1	-0.423	-0.423
Z_p-RUR	-0.464	0	0	1	-0.464	-0.464
Z_p-SD	-0.00048	678.8	-0.32584	1833.2	-0.880	-0.554
Z_p-INTD	0.042	0.0	0	8.0	0.338	0.338
Z_p-CLR	-0.00422	9.8	-0.04115	68.57	-0.289	-0.248
Z_p-TWLT	-0.477	0	0	1	-0.477	-0.477

The sensitivity analysis for both models showed that the clear zone dimensions, the absence of roadside obstructions and the control of access points provide the biggest opportunity for improving operating speeds in four-lane highways. Improvements in sight distance and the incorporation of TWLT median lanes provide designers with a less significant chance of increasing operating speeds. Although both models proved that they are capable of providing good speed estimates, the implementation of the RE model is more appropriate, based on the theoretical discussion presented in Chapter 3, and was the selected model for the implementation of the research results.

CHAPTER 8. SPEED LIMITS, DESIGN SPEEDS AND OBSERVED SPEEDS

AASHTO defines the operating speed in its latest design guide as the speed at which drivers are observed operating their vehicles during free-flow conditions. The 85th percentile value of the observed free-flow speed distribution is typically used to represent the operating speed, as recommended in the MUTCD. On the other hand, the current INDOT design manual presents a definition of operating speed as the highest overall speed at which a driver can safely travel while not exceeding the design speed. This definition, along with the requirement that posted speed limits cannot exceed the design speed on a highway, may lead to too low speed limits due to the discrepancy between the economically justifiable design solutions and the design standards expected by the public on modernized highway sections. This discrepancy between the expected and the provided standards is manifested through the difference between the speed limit that can be applied on the modernized section (allowed or posted speed limit) and the speed limit expected by the drivers (target or desired speed limit). The desired speed limit can be approximated, in most cases, with the statutory speed limit that applies to the road section considered. Sometimes, the desired speed limit can be set at the current posted speed limit, if accepted by the motorists.

The following chapter presents the results from an evaluation of the observed 85th percentile speeds and the design speeds for the highway segments selected in this study. The evaluation was made using the sites observed on tangent segments and horizontal curves in two-lane rural highways and for all sites observed in four-lane highways. The purpose of the evaluation is to identify trends that demonstrate the discrepancy between the operating level provided by the design standards and the observed speeds in highway segments and to provide evidence to support changes in the design guidelines that restrict the selection of posted speed limits higher than the design speed of a highway facility. The crash experience on the highway segments was considered to eliminate segments where a considerable high number of crashes indicated that the driver perception of the risk on the segment might be incorrect. Therefore, the observed speeds in the selected highway segments are believed to concur with a satisfactory level of safety.

8.1. Speeds in two-lane rural highway segments

Free-flow speeds measured in thirty-two sites located on tangent segments free from the influence of horizontal curves were selected for this evaluation. Figure 8-1 shows the cumulative percentages for the mean and 85th percentile speeds for the sites located in those tangent segments. The figure shows clearly the high speed trends observed in tangent segments; all sites have a mean speed higher than 53 mph (85.3 km/h) and an 85th percentile speed higher than 58 mph (93.3 km/h). The posted speed limit on these segments was either 50 or 55 mph (80 or 90 km/h).

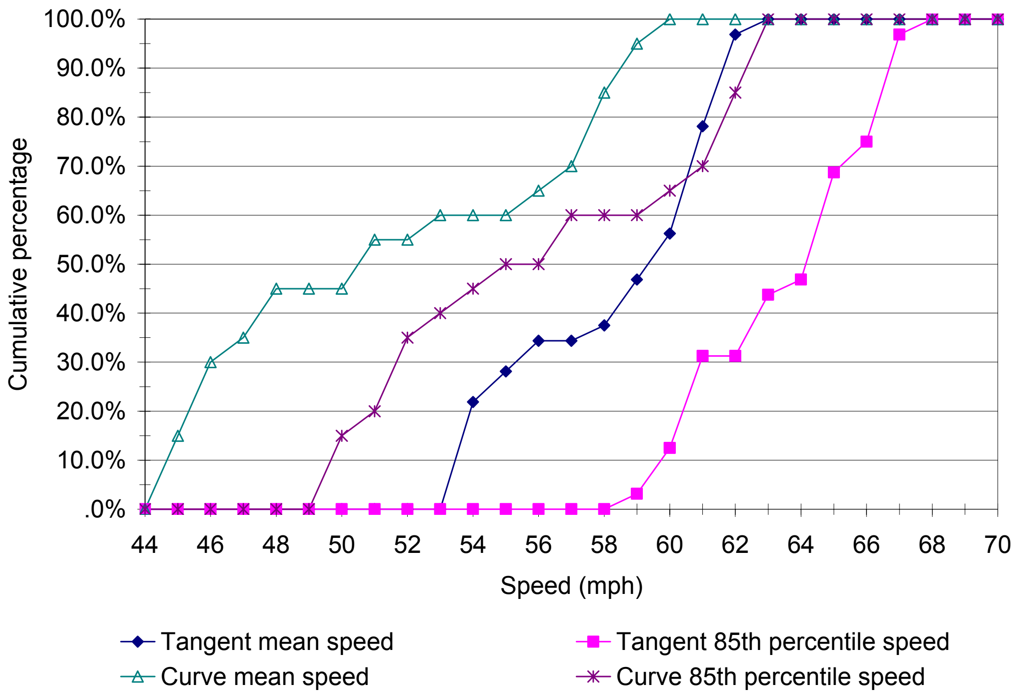


Figure 8-1 Cumulative percentages of the mean and 85th percentile speeds in two-lane highways

The INDOT design table for two-lane rural arterial 3R projects set the design speed for the general controls equal to the posted speed limit. The design table also recommends that the minimum design speed should equal the anticipated posted speed limit after construction, or the state legal limit of 55 mph on non-posted highways. Figure 4-6a already showed that the observed 85th percentile speeds are 5.6 to 13 mph (9 to 20.9 km/h) higher than the posted speed limits. These results are consistent with those published for tangent segments of different highway functional classifications in the NCHRP Report 504. The range of the observed 85th percentile speeds for segments with the 55-mph speed limit (7.4 mph) is almost twice the range observed for segments with the 50-mph limit (3.8 mph). If the posted speed limit rule is taken into consideration, then all the sites located on tangent segments have an operating speed higher than its design speed.

The INDOT design table also recommends three typical design speeds, from 50 to 60 mph (80 to 100 km/h), for the alignment elements in a highway segment based on the minimum stopping sight distance (SSD) of the segments. The minimum sight distance measured in the observed sites located on tangents was 426.5 ft (130 m). All but three of the sites meet the minimum required SSD for the 60 mph design speed. Using this method all, but one, of the sites had 85th percentiles speeds equal or higher than the design speed. It has to be noted that the measured sight distance on each site might not the actual minimum SSD on the segment; therefore the design speed for all segments might be misrepresented and another approach is preferred to infer the design speed.

Two additional methods to infer the design speed of the tangent segments are available in the AASHTO Roadside Design Guide and the AASHTO Green Book. The AASHTO Roadside Design Guide provides typical design speed values based on the clear zone distance and the traffic volume. The design speeds suggested by the AASHTO Roadside Design Guide vary from 40 to 60 mph (60 to 100 km/h). Figure 8-2 shows the inferred design speed based on the roadside design of each tangent site versus its corresponding 85th percentile speed. Using this method, all sites had 85th percentile speeds equal or higher than their inferred design speeds. All, but two, of the sites had 85th percentile speeds higher than their inferred design speeds by 5 mph or more; twenty-eight sites had a difference of more than 10 mph (16 km/h). As the inferred design speed decreases the difference between the operating speeds and the design speeds increases considerably. The difference between the 85th percentile speed and the inferred design speed varies from 0 mph (for a design speed of 60 mph) to 28 mph (for a design speed of 40 mph).

The rural arterials section in the AASHTO Green Book provide typical design speeds based on the terrain type, the sight distance, or the traveled way width and design volume. Figure 8-3 shows the inferred design speed based on the traveled width and the traffic volume for each tangent site versus its corresponding 85th percentile speed. The maximum design speed of 75 mph (125 km/h) was assigned to a site when the minimum width was met for the different design speeds. That was the case for ten of the sites that met the minimum traveled way width of 24 ft (7.2 m) for segments with more than 2000 vehicles per day. In contrast, the other 22 sites did not meet the minimum width required for the lowest design speed, regardless of the traffic volume. The minimum design speed of 40 mph (60 km/h) was assigned to those sites. The difference between the 85th percentile speed and the inferred design speed in those cases is excessive, varying from 19 to 28 mph (30.5 to 45 km/h). There might be two plausible explanations for the high difference between the two speeds from the standpoint of the design standards; these segments might have been build for a lower traffic volume than the current one in operation, or that older design standards were applied, failing to meet the current design values suggested by AASHTO. Another reason from the standpoint of driver behavior might be that the width of the traveled way does not have a real strong impact on speeds for sites located on tangent segments, as was observed from the speed evaluation presented in Chapter 4 and in the NCHRP Report 504. The latter explanation is corroborated by the low sensitivity of 0.62 mph (1 km/h) in the estimate of the 85th percentile speed shown by the pavement width variable (includes the traveled way and the paved shoulder widths) in the OLS-PD speed model. The pavement width variable was not even considered as a speed factor in the RE model.

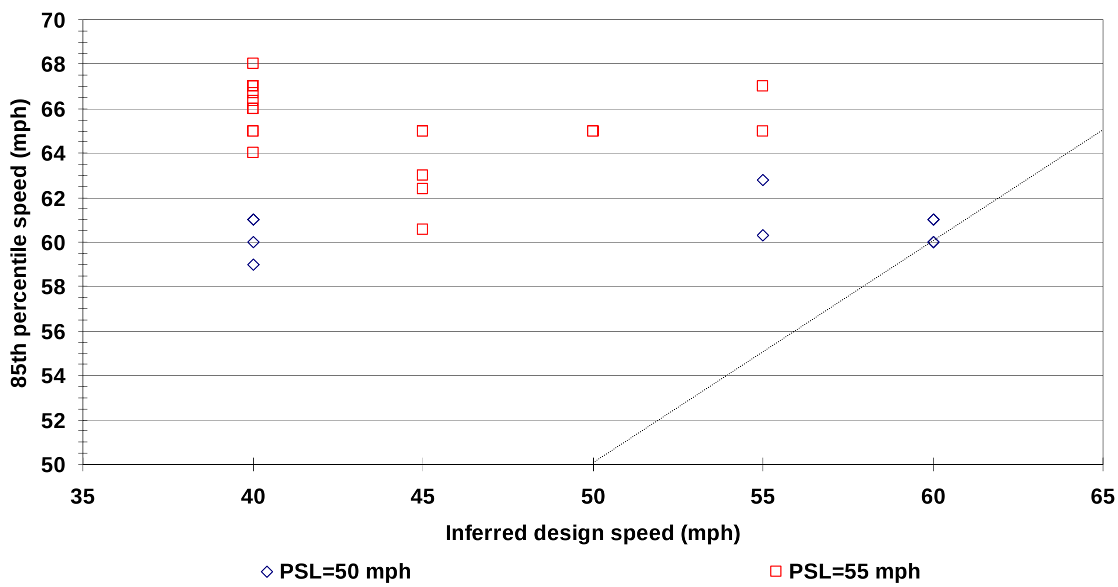


Figure 8-2 Inferred design speeds based on the roadside design versus 85th percentile speeds on tangent segments

It was already showed in Chapter 4 that a reduction in the posted speed limit of the segment from 55 mph to 50 mph decreases the 85th percentile speed. That trend is a good indicator of the effect on speeds allocated to the posted speed limit. Thirty of the thirty-two sites (93.8 percent) in the sample have a difference between the 85th percentile speed and the posted speed limit of 8 mph (12.9 km/h) or more. Only six sites in the sample have a difference of less than 10 mph. This difference might be also influenced by speed enforcement tolerance. It may be inferred that most drivers consider that the design of straight alignments allow for high operating speeds with a reasonable minimal risk of crash and assume as acceptable to drive at about 10 mph above the speed limit with a minimal risk of getting a speeding ticket. The NCHRP Report 504 seems to concede the same line of reasoning when it concluded that the road design might have a minimal influence on speeds unless restricted horizontal or vertical alignment conditions are present.

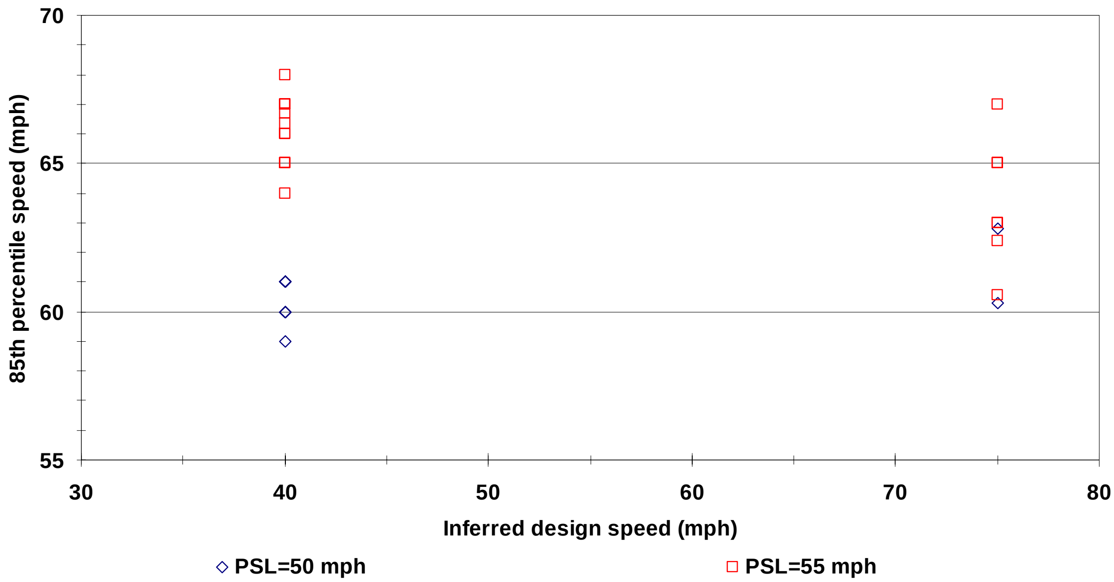


Figure 8-3 Inferred design speeds based on the traveled way width and volume versus 85th percentile speeds on tangent segments

Figure 8-4 shows the observed percentage of vehicles going at speeds higher than the posted speed limit on tangent segments. Regardless of the posted speed limit, the minimum percentage of vehicles traveling at a speed higher than the speed limit for all thirty-two sites on tangents is 55 percent. The MUTCD and the AASHTO design guide recommend that a posted speed limit should be the 85th percentile speed of free-flowing traffic. The 85th percentile rule is applied under the premise that is tolerable that 15 percent of the vehicles in the free-flow speed distribution operate at speeds higher than the posted speed limit. In average, 82.8 percent of all drivers are going at speeds faster than the posted speed limit on all sites. Thirty sites (94 percent) have 70 percent of more of the drivers going at speeds higher than the posted speed limit. If the 85th percentile rule is applied literally, fourteen sites (43.8 percent) can have a posted speed limit equal to 60 mph and seventeen sites (48.6 percent) can have a posted speed limit equal to 65 mph.

Figure 8-1 also shows the cumulative percentages for the mean and 85th percentile speeds for twenty sites located on the effective section of horizontal curves. The effective section of a horizontal curve excludes the portion of the transition section inside the curve and represents the section of the curve where drivers maintain a constant speed. The sites were located on nine different curves with posted speed limit of 55 mph. Twelve sites were located on curves showing advisory speed signs of 40 mph (60 km/h) or 45 mph (70 km/h). Forty percent of the sites on curves have mean speeds higher than 55 mph and fifty percent of the sites have 85th percentile speeds higher than 55 mph.

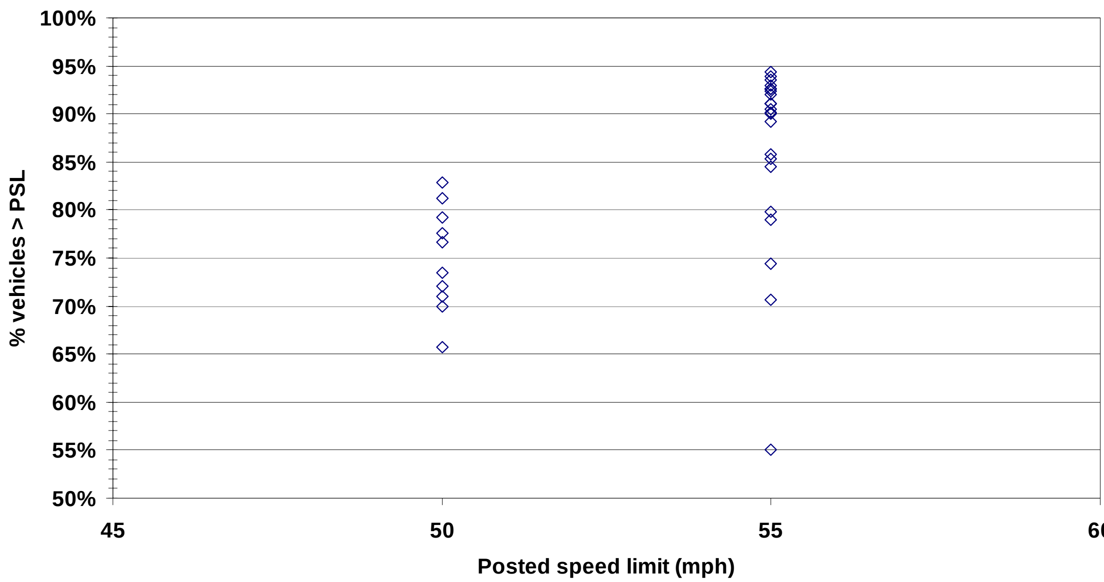


Figure 8-4 Percentage of vehicles going at a speed higher than the speed limit on tangents

The design speed of the curves has an obvious impact on the observed speeds as already illustrated by the speed models developed in this study. The basic curve formula and the observed radii and superelevation rates were used to infer the design speed of all curves. There are a total of twenty-four different horizontal curves in the sample; although speeds were observed on the effective curve section on only nine curves. The speeds on the curves were measured on sites that were located on the transition sections. Figure 8-5 shows the inferred design speed versus the advisory speed or the posted speed limit for the twenty-four curves. For the most part, the advisory speeds or the posted speed limits are set practically close or over the corresponding inferred curve design speed. Only one curve in the sample was found to have a posted speed limit (55 mph) of more than 5 mph over its corresponding inferred design speed. In contrast, there are eight curves (two with an advisory speed of 45 mph and six with a posted speed limit of 50 or 55 mph) that have inferred design speeds of at least 10 mph higher than the curve advisory speed or the posted speed limit. The arbitrary use of curve advisory speeds where conditions allow for higher speeds without increasing the risk of crash might lead to situations where drivers will ignore those signs in more restrictive conditions. The MUTCD emphasizes that regulatory and warning signs should be used conservatively because these signs, if used to excess, tend to lose their effectiveness. The use of other control devices (alignment, arrows or chevron signs, delineators, etc.) might be a more effective approach in such cases to warm drivers of restrictive conditions when negotiating the curve. The 85th percentile speeds for the sites located on the effective section of the curves that show advisory speed signs or posted speed limits close to their corresponding inferred design speeds (within 5 mph) were targeted in the following evaluation.

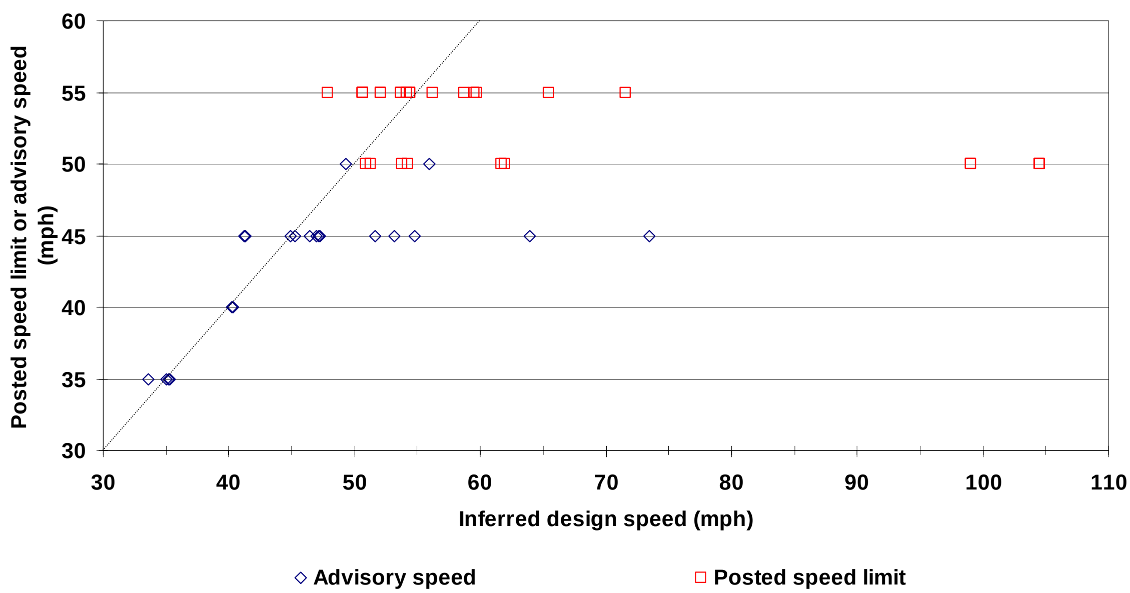


Figure 8-5 Inferred design speeds for horizontal curves

Figure 8-6 compares the inferred design speeds versus the 85th percentile speeds of the sites located on the effective section of a horizontal curve. All fifteen sites have 85th percentile speeds higher than the curve inferred design speed. The range of the difference between the inferred design speeds and the 85th percentile speed is between 5.1 and 15.8 mph. The curves without advisory speeds have 85th percentile speeds that exceed the inferred design speeds in a range of 8.3 mph to 11.4 mph. It is important to observe that although the posted speed limit is the same for all sites, drivers adjust their speeds depending on the conditions presented by the curve geometry. The results observed in our study are fairly consistent with those found in previous studies and the NCHRP Report 504 that showed that drivers exceeded the design speed in rural sections with design speeds of 55 mph (90 km/h) or less. The observed trend is present on curves with and without advisory speed signs. The results published in the NCHRP Report 504 are reproduced in Figure 8-7 for convenience. The trend shows large variability in operating speeds for a given inferred design speed. Based on their results, the NCHRP Report 504 concluded that the use of design speeds higher than 50 mph will not result in higher operating speeds. That statement disagrees with our results, although it is important to make the distinction that the speeds evaluated in our study come from segments with low crash rates.

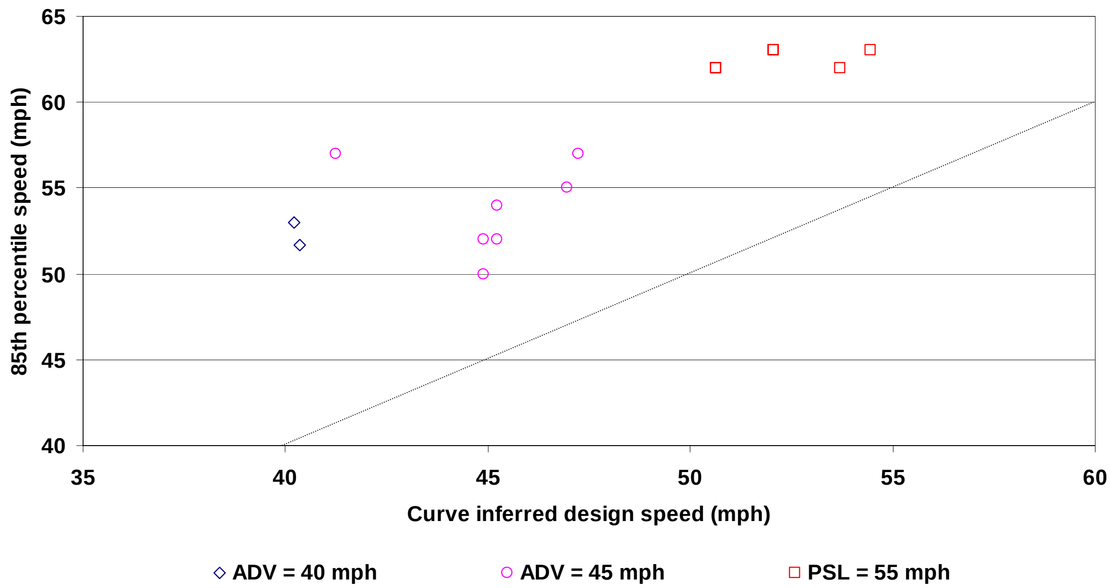
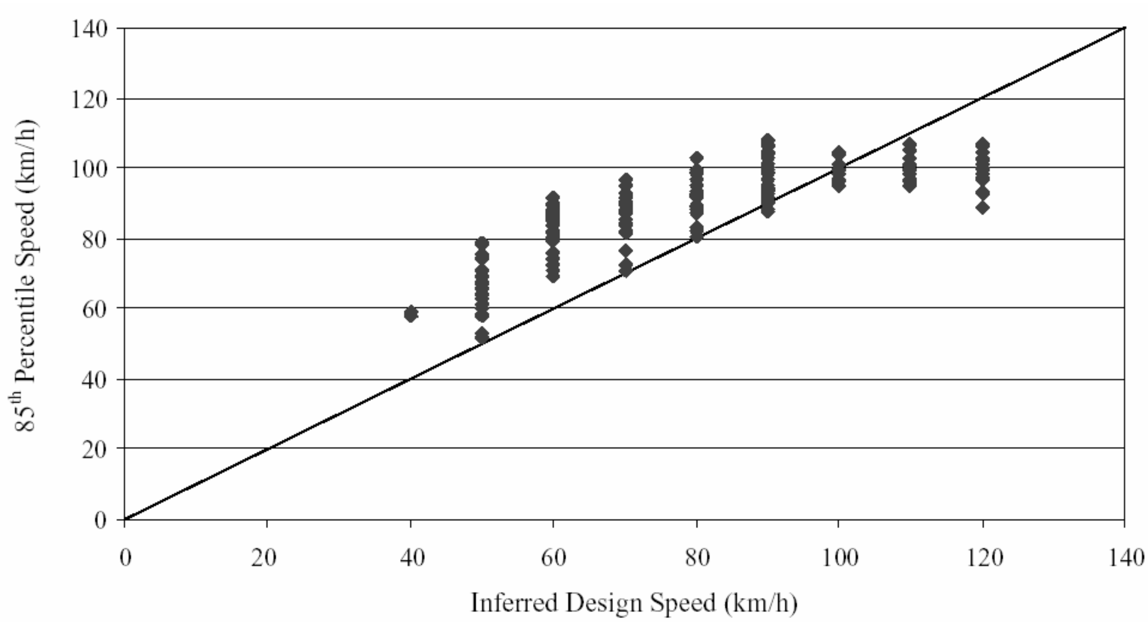


Figure 8-6 Inferred design speeds versus observed 85th percentile speeds in horizontal curves



(Source: NCHRP Report 504)

Figure 8-7 Inferred design speeds versus 85th percentile speeds in horizontal curves of two-lane rural highways

The speed dispersion at a site is typically accepted as a contributing factor in accident potential. Accident frequency is believed to increase with an increase in speed dispersion at a site because of the increasing frequency of interactions between vehicles. It is fairly accepted that an increase in the deviation between a motorist's speed and the average speed of traffic is related to a greater chance of involvement in a crash. Garber and Gadiraju (1989) concluded that minimum speed variances will occur when the difference between the design speed and the posted speed limit is between 5 and 10 mph for different highway types. In addition, their study found that speed variance increased as the difference between the design speed and the posted speed limit increased and that speeds increased with better geometric conditions, regardless of the posted speed limit. Figure 8-8 shows the speed variance observed on each site versus the difference between the curve inferred design speed and the posted speed limit. Minimum speed variance is observed on curves which the posted speed limit is about 3 mph above the inferred design speed. In addition, the graph confirms the trend of increasing variance with an increasing difference between the inferred design speed and the posted speed limit. It would be interesting to add sites on curves in high crash rate segments to confirm the U-shaped relationship and that large speed variability can be associated directly to high crash rate segments. The observed trend also supports the third conclusion of the Garber and Gadiraju study. Figure 8-6 already proved this. As expected, 85th percentile speeds are increasing with better geometric conditions on the curve, as represented by an increase in the curve design speed.

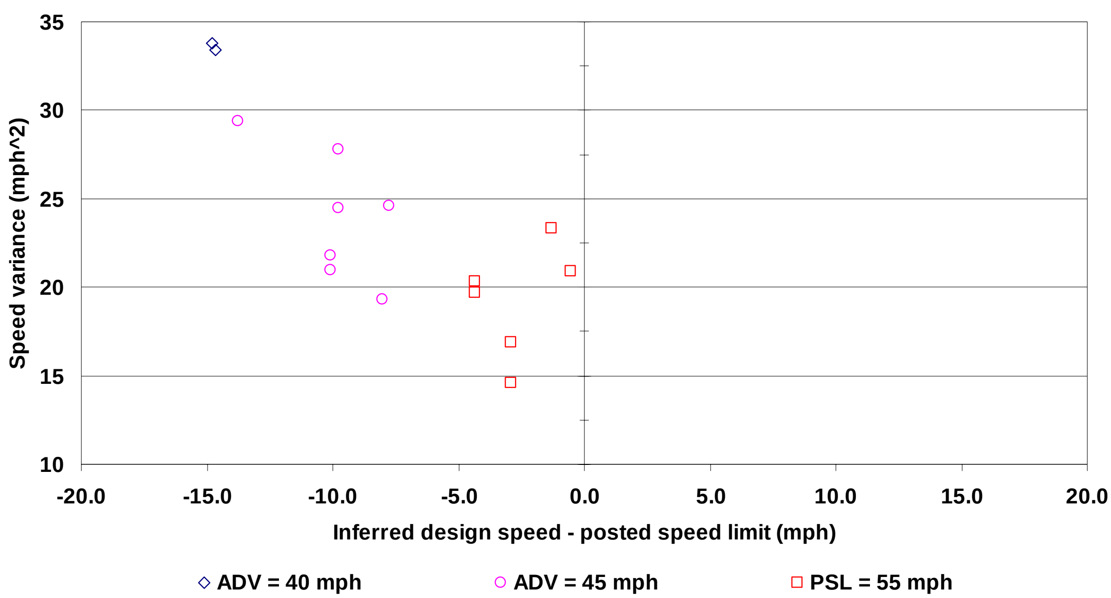


Figure 8-8 Speed variance observed on horizontal curves

Figure 8-9 presents the percentage of drivers going at a speed higher than the 55-mph speed limit or the advisory speed, if present, in the horizontal curves. In average, 77 percent of drivers are traveling at speeds higher than the posted speed limit or the advisory speed on the curve. These results are more conservative than those found by Chowdhury et al. (1998). Their study found that, in average, 90 percent of drivers exceeded the curve advisory speed in two-lane rural highways, and, in almost half of the sites, nobody obeyed them.

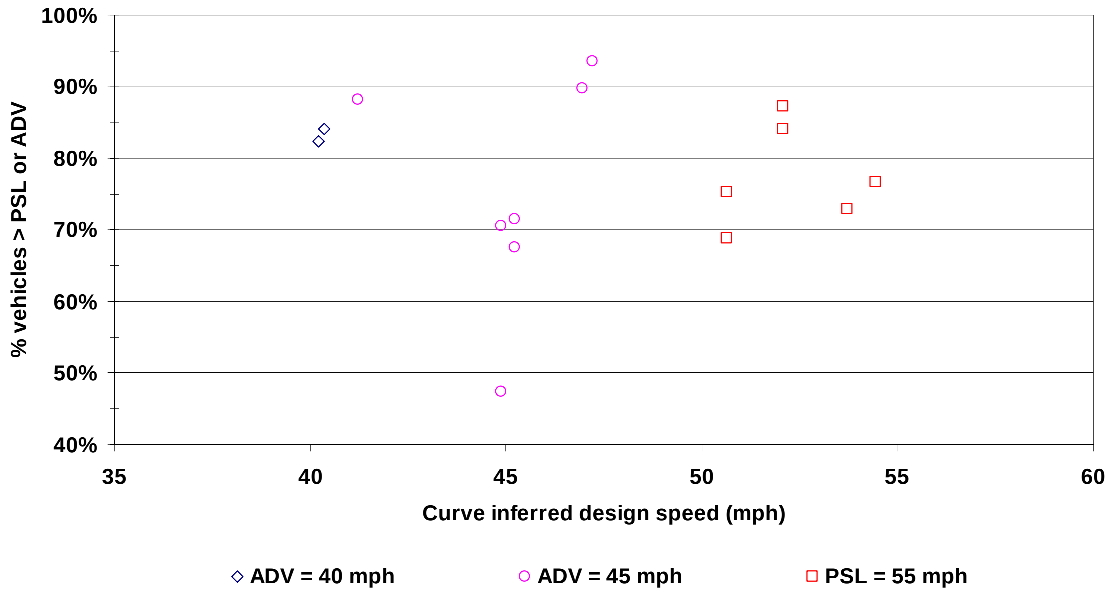


Figure 8-9 Percentage of vehicles going at speeds higher than the speed limit or the advisory speed on curves

The speed trends observed in two-lane rural highways show that the posted speed limit can be set at around 5 to 10 mph above the design speeds without causing excessive hazard to drivers. Although the speeds observed in this study come from two-lane rural highway segments that have a considerable low crash rate, a more comprehensive analysis has to be carried out to characterize the safety level of any individual design feature, like intersections and horizontal curves. The available crash database lacked the required detailed information about the location and the cause of the crash in order to calculate crash frequencies or crash rates for individual design features on the segments. The observed speeds identify trends that show the current discrepancy between the operating level provided by the design standards and the observed operating speeds in the selected two-lane rural highway segments that concur with a satisfactory level of safety.

8.2. Speeds in four-lane highway segments

Free-flow speeds measured in fifty sites located on four-lane highway segments were used to evaluate trends between the operating speed and the design speed. Figure 8-10 presents the cumulative percentages for the mean and 85th percentile speeds for sites in four-lane highway segments. The cumulative percentage curve shows that all the sites had 85th percentile speeds higher than 50 mph (80.5 km/h). There are four different posted speed limits in the sample, from 40 to 55 mph (60 to 90 km/h). No advisory speed was observed in any of the segments. It was shown in Chapter 5 that a reduction in the posted speed limit decreases the 85th percentile speeds on the sites; although the variability in 85th percentile speeds was comparable for the three highest speed limits. Higher operating speeds were observed in rural areas. The observed segments in rural areas had higher speed limits and lower access densities than most suburban segments. The observed speeds show a similar trend as the one found in two-lane rural highways: 82 percent of the sites had mean speeds above the posted speed limit and all the sites had 85th percentile speeds higher than the posted speed limit. The nine sites with mean speeds lower than the posted speed limit were located in suburban areas. The 85th percentile speeds are higher than the posted speed limits by a margin of 2.2 to 16.1 mph.

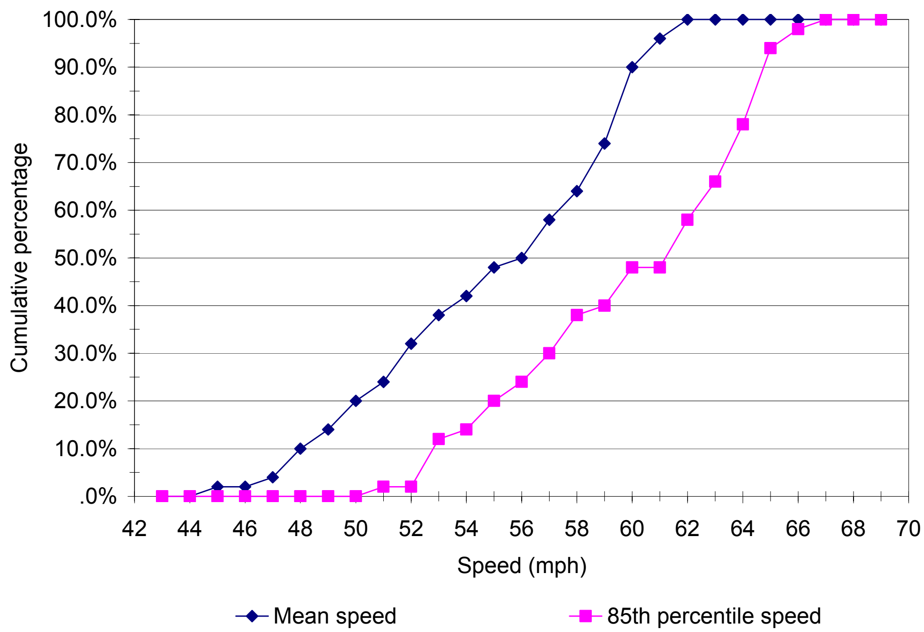


Figure 8-10 Cumulative percentages of the mean and 85th percentile speeds in four-lane highways

The INDOT design table for multi-lane rural and urban arterial 3R projects recommends that the design speed for the general controls should be equal to the posted speed limit. The design table also recommends that the minimum design speed should equal the anticipated posted speed limit after construction, or the state legal limit of 55 mph (90 km/h) on non-posted rural highways. The design table also recommends that the legal limit for urban arterials is 30 mph (50 km/h) and with an engineering study may be raised to a maximum of 55 mph (90 km/h). If the posted speed limit rule is taken into consideration, then all the sites have an operating speed higher than its design speed.

The design tables also recommend three typical design speeds, from 50 to 60 mph (80 to 100 km/h), for the alignment elements in rural arterials, and five typical design speeds, from 30 to 55 mph (50 to 90 km/h), for the alignment elements in multi-lane urban arterials. The minimum sight distance measured in the field was 549 ft (167 m). Forty-seven sites located outside horizontal curves met the required minimum sight for the highest design speeds; 60 mph for rural highways and 55 mph for urban highways. In contrast to two-lane rural highways, the sight distance remains fairly constant throughout most of the four-lane segments; therefore the observed sight distance values can be used to infer the design speed of the four-lane highway segments. Figure 8-11 shows the trend between the inferred design speeds and the posted speed limit for all sites. All, but one, of the fifty sites on four-lane segments have inferred design speeds equal or higher than the posted speed limit on the segment. One site, located on a curve, has a posted speed limit set at 1 mph above its design speed. This serves as evidence that other factors, besides the sight distance, are considered when setting speed limits on four-lane highways.

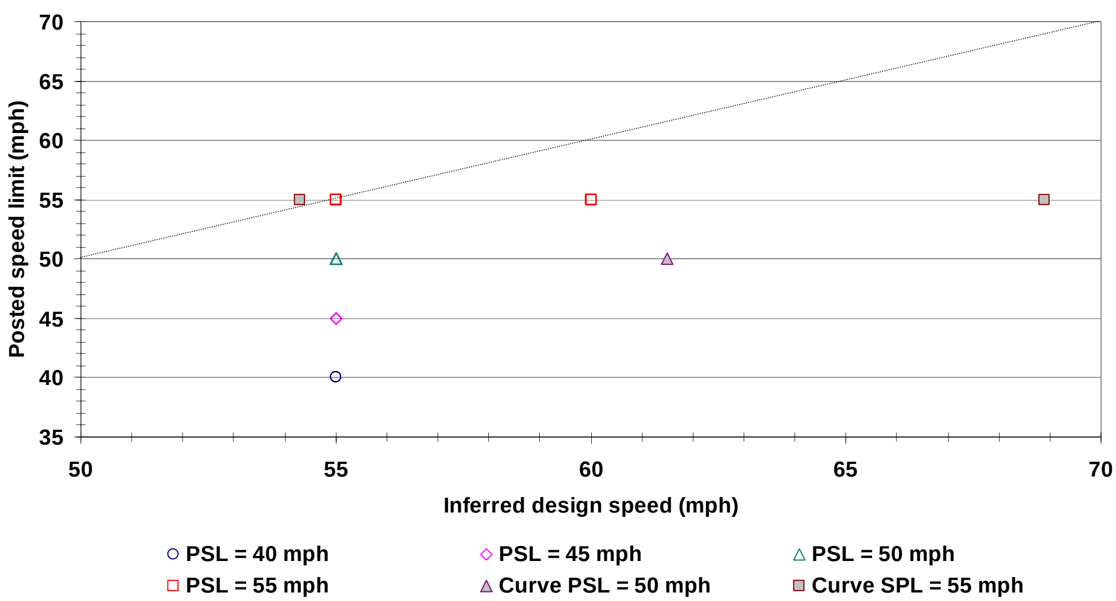


Figure 8-11 Inferred design speeds versus posted speed limits in four-lane highway segments

Only three sites were located on horizontal curves. The design speed for these curves was calculated using the basic curve formula with the measured values of radii and superelevation rates. The inferred design speeds for the curves were 54, 62 and 69 mph (87, 99 and 110 km/h); resulting in a difference between the 85^{th} percentile speed and the design speed of 10.7, -0.3 and -4.8 mph (17.2, -0.5 and -7.7 km/h), respectively. Although the sample is too small to reach an irrefutable conclusion; the observed trend was expected based on previous studies found in the literature. The NCHRP Report 504 observed that the operating speeds on curves in suburban highways were higher for design speeds of 43.5 mph (70 km/h) or lower.

Figure 8-12 shows the inferred design speed versus the 85th percentile speed for all sites. All sites located in segments with a 55 mph speed limit have 85th percentile speeds equal or higher than the inferred design speed. The range of the difference between the 85th percentile speed and the inferred design speed in those sites is between 1.6 and 10.2 mph. As the posted speed limit decreases in the segment, the difference between the two speeds also decreases. The range of the difference between the 85th percentile speed and the inferred design speed is between -2.8 and 9.2 8 mph in sites with a 50 mph speed limit and between -2.9 and 1.1 mph in sites with a 45 mph speed limit. The trend clearly presents the impact on 85th percentile speeds caused by the change in posted speed limit on the segments, although other factors like the intersection and driveway densities and the rural environment also play a major role on speeds. It was already observed that the drivers on rural areas tend to go even faster that on suburban areas, regardless of the posted speed limit.

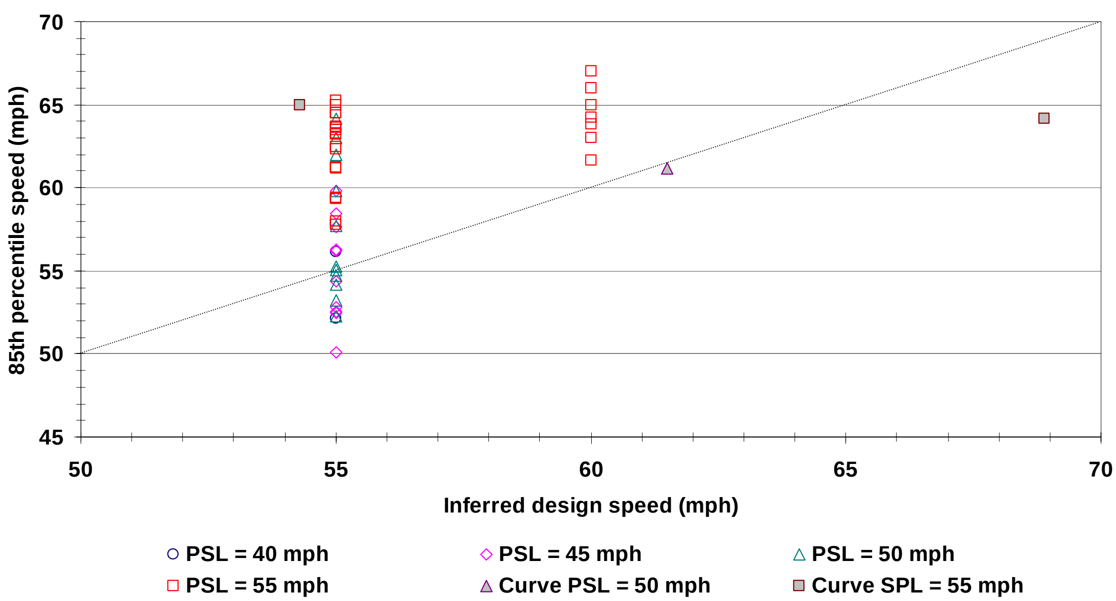


Figure 8-12 Inferred design speeds versus 85th percentile speeds in four-lane highway segments

The speed model developed for four-lane highways already illustrated the point that the posted speed limit had a strong impact on mean speed; although its impact on speed dispersion was not as strong. Figure 8-13 shows the percentage of drivers that are going at speeds higher than the posted speed limit in the four-lane highway segments. Regardless of the speed limit, the minimum percentage of vehicles going at a speed higher than the posted speed limit in any site is

27 percent. Thirty-six sites (72 percent) exhibit a more extreme behavior with 70 percent of more of the vehicles going at speeds higher than the posted speed limit. It is obvious that there are other factors involved in the selection of speeds, besides the posted speed limit and the inferred design speed, especially in segments with the two highest speed limits. This is verified with the huge variability in the percentage of vehicles going at speeds higher than the posted speed limit. More drivers appear to accept the 50- and 55-mph speed limits as reasonable values for most highway conditions on suburban areas. The level of the police enforcement may be another reason for this behavior. The intensity of the speed enforcement in suburban areas is certainly higher than in rural areas. This may be demonstrated with the higher percentage of vehicles going at speeds over the posted speed limit in sites located in rural areas. Most drivers might feel that the design of most of the four-lane suburban highway segments is generous enough to allow traveling at speeds about 50 or 55 mph with a reasonable low risk of crash. An enforcement tolerance issue comparable to the one observed in two-lane rural highways might be present; with 85th percentile speeds about 10 mph higher than the posted speed limit for the two highest speed limits and about 15 mph for the two lowest speed limits.

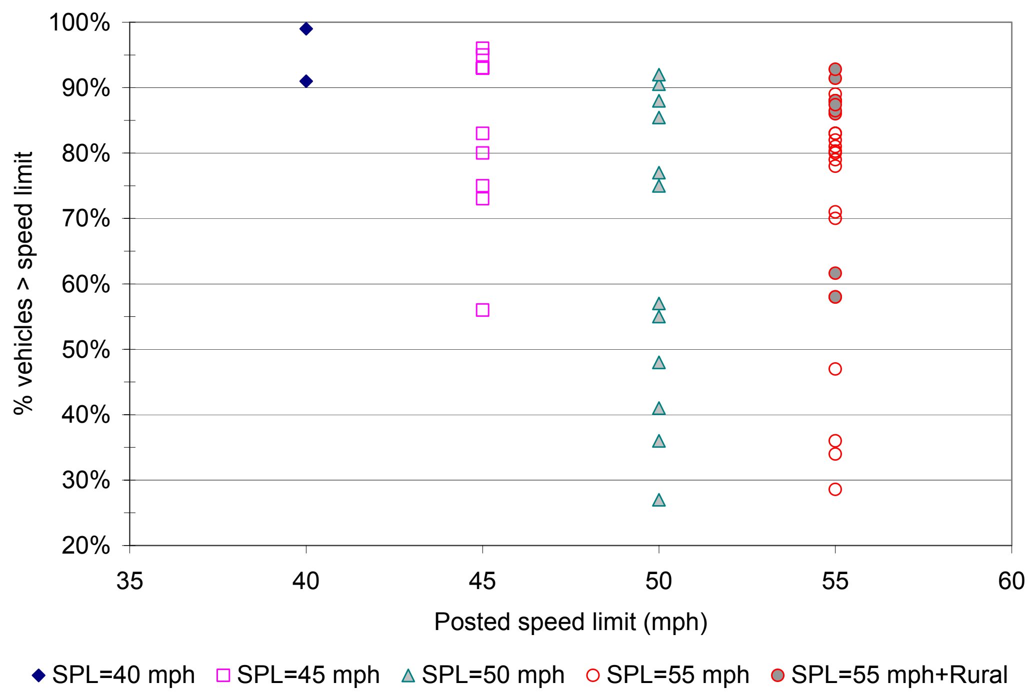


Figure 8-13 Percentage of individual speeds higher than posted speed limit in four-lane highways

The calculated crash rates in four-lane highway segments are representative of the safety level of the entire segment. The design of four-lane highway segments is more consistent than in two-lane rural highways. In other words, highway characteristics are more uniform throughout longer sections and the presence of inconsistent features like sharp curves or sections with inadequate sight distance are not common. Therefore, the speeds measured in four-lane highways may be associated with a reasonably minimal crash rate.

By using design speeds, highways are designed in a conservative manner to facilitate the safe motion of vehicles even in adverse but reasonable conditions. The speed observations made in this study prove that the 85^{th} percentile of the observed free-flow speeds exceeds the design speed in most situations. The crash experience was added to eliminate cases where the drivers' perception might be incorrect, as represented by a considerably high crash rate for the entire highway segment. It may be concluded that the observed free-flow speeds concur with a satisfactory level of safety for highway segments.

All sites observed in four-lane highways and tangent segments in two-lane rural highways have 85^{th} percentile speeds higher than the posted speed limit. In the observed sites on horizontal curves of two-lane rural highways, 77 percent of drivers, in average, operate at speeds higher than the advisory speed or the posted speed limit. Following the 85^{th} percentile rule and taking into account the considerable low crash rate in those segments, the speed limit may exceed the design speed by 5 to 10 mph. The discrepancy between the two speeds does not cause excessive hazard because the majority of drivers adequately perceive the risk. The current design policy can be modified to allow setting the posted speed limit at a value higher than the design speed, but according to the 85^{th} percentile speeds and the crash experience. Once the design policy is modified, the speed models developed in this research project can be used to predict safe operating speeds for improvement projects, context-sensitive projects or design exceptions. Engineering judgment can be applied to balance safety and construction cost in highway improvement projects. The estimated operating speeds can be used to set posted speed limits that concur with the expected speed by drivers on the modernized sections.

CHAPTER 9. SPEED PREDICTION TOOL FOR TWO- AND FOUR-LANE HIGHWAYS

This chapter presents the speed prediction tool that implements the speed models developed in this study. The properties and the possible implementation of the tool are also discussed in this chapter.

9.1. PURPOSE OF THE SPEED PREDICTING TOOL

The developed speed models were included in a visual basic-based prototype tool to help highway designers implement the models. The prototype tool, named the Highway Speed Prediction Model (HSPM), was developed as a stand-alone, ready-to-use Windows application, as requested by the SAC.

The purpose of the tool is to help highway designers predict any percentile speed in two-lane and four-lane highway improvement projects. Figure 9-1 presents a flowchart with the proposed implementation of the speed tool in the highway design process. Specific information about the cross-section dimensions, the horizontal curve design, the sight distance, the highway grade, the residential driveway density, the percent of trucks and the location of intersections and curves is required to predict speeds in two-lane rural highways. Specific information about the roadside clear zone distance, the median width and type, the sight distance, the intersection and driveway densities, and the rural vs. suburban setting is required to predict speeds in four-lane highways. The highway design information is used by the tool to predict the percentile speeds based on their impact on the mean speed and the speed dispersion.

The tool provide users with a profile showing the mean speed and any specified percentile speed for the entire project length based on the preliminary highway design values. The tool can be used to evaluate if the predicted speeds meet the desired speeds for the design project. The profile can be used to identify segments on the project with design inconsistencies (e.g. excessive speed changes). The tool provides the opportunity to incorporate possible modifications to the design values at any location of the project that might increase or reduce the predicted speed until it reaches the desired speed. The user can print the speed profile and the tables with the input design values and add them to the project documentation.

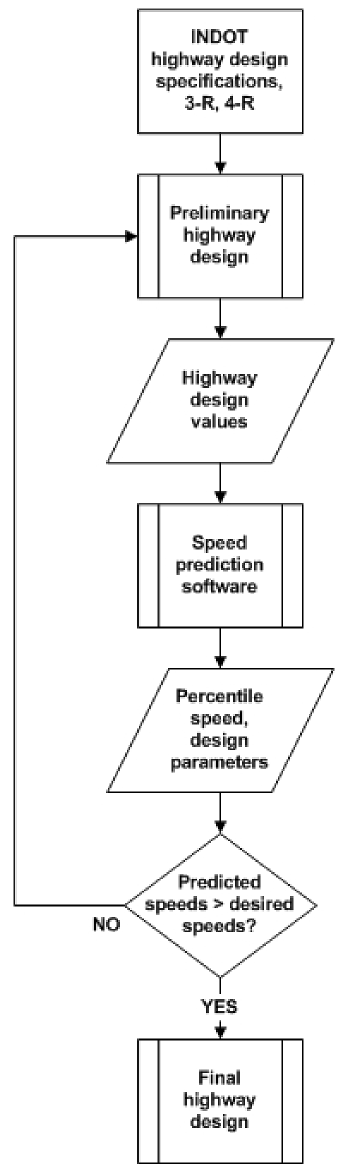


Figure 9-1 implementation of the speed tool in highway design

9.2. Speed predicting models

The OLS-PD models were suggested for implementation to predict the free-flow percentile speeds in two-lane rural highways. The tool uses four different equations to predict the speeds on tangents, on horizontal curves and in curve transition sections. Equation 6.5 shows the model developed to predict speeds on tangent segments and Equation 6.6 shows the model developed to predict speeds on horizontal curves. Equations 6.7 and 6.8 show the models developed to predict speeds on deceleration and acceleration rates in curve transition sections, respectively.

The tool calculates the speed of spots located inside horizontal curves using Equations 6.5 and 6.6 separately. The tool compares both speeds and selects the lower speed as the predicted speed for the spots located inside curves not identified as flat curves. If the speed calculated with Equation 6.5 is lower than the speed calculated with Equation 6.6, the speed at the curved section is controlled primarily by the cross-section dimensions and the highway characteristics and there is no major impact on speeds due to the horizontal curve. If the speed calculated with Equation 6.5 is higher than the speed calculated with Equation 6.6, the speed at the curved section is controlled primarily by the horizontal curve. In this case, the speeds for the deceleration and acceleration transition sections are calculated.

Some assumptions were made in order to predict the speed change in the transition sections. It was assumed that drivers apply the same mean deceleration and acceleration rates regardless of their desired operating speed on the tangent segment. In addition, the portion of the transition sections located outside the horizontal curve remains constant. Fast drivers will start decelerating at an earlier point in the alignment than slow drivers, but in average, drivers will have a common point to start decelerating and accelerating in transition sections.

The single RE model, shown in Equation 7.2, was suggested for implementation to predict free-flow percentile speeds in four-lane highways. The tool creates the speed profile for the entire project length with this equation and any transition speed through the project is calculated by applying the mean deceleration and acceleration rates developed for two-lane rural highways.

9.3. INPUTS TO SPEED TOOL

The following section presents the data requirements of the speed tool. The tool consists of a main container screen, where all the options are accessed through the use of menus. Figure 9-2 shows the main screen of the prediction tool. The main screen contains six menus: FILE, EDIT, INSERT, CALCULATE, CHART and HELP. The application menus allow quick access to the different tool operations. Some submenus are enabled or disabled depending on the type of project to indicate the available options. The menus are used to perform the following tasks:

- FILE menu: to create, open and save projects, print files and exit the tool.
- EDIT menu: to easily transfer the information of a form, from one record location to another.

- INSERT menu: to show and activate the project forms to enter the information.
- CALCULATE menu: to request the speed calculations.
- CHART menu: to show the speed profile chart.
- HELP menu: to access the help section and the copyright information.

The project information is entered by using two different set of forms. The input is organized into four forms for two-lane rural highways and into three forms for four-lane highways. The forms provide default typical values and suggest range of values for most of the variables included in the speed models. The tool also includes a help section that has the user manual and the instructions on how to use the tool. The help section also presents the speed models and the definitions of the variables included in the models.

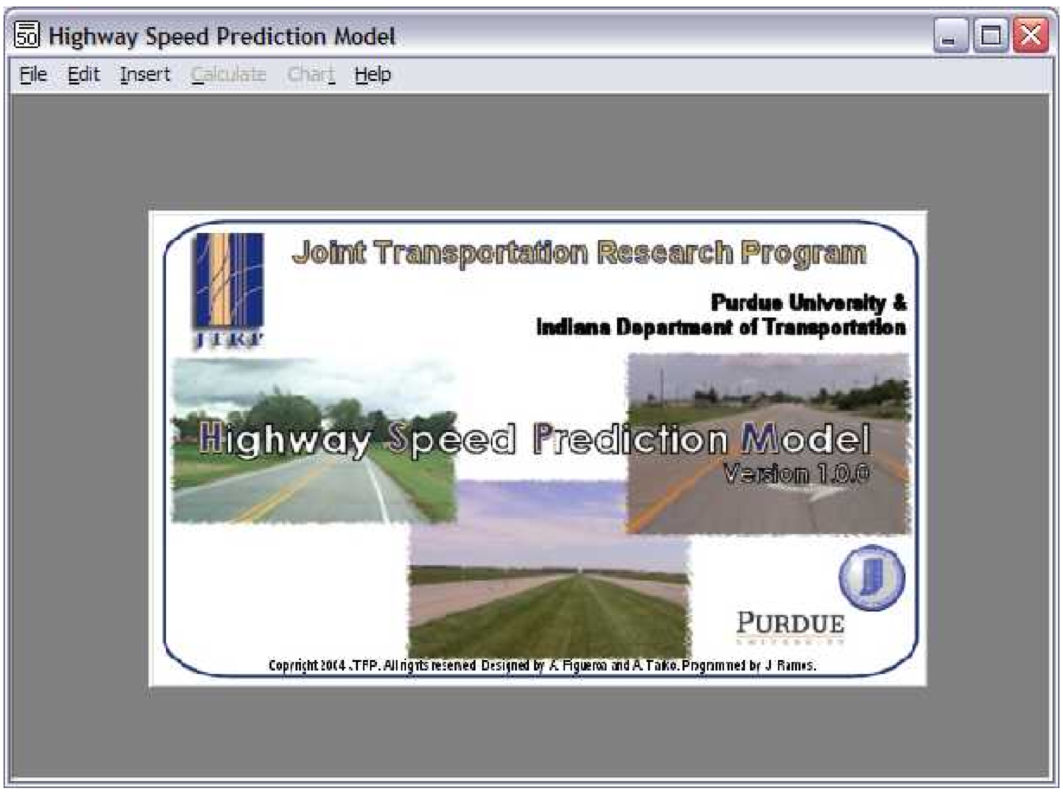


Figure 9-2 Main screen of the prediction tool

Figure 9-3 shows the project general information form. This form is used to enter the general information about the highway design project and can be used to divide the project into segments. A project can be divided to reflect changes in the county location, the number of lanes, the percentage of trucks, or the posted speed limit. The segments are identified by their starting and ending mileposts. The user can request the tool to calculate any percentile speed from the 5th to the 95th, in increments of five, to be shown in the profile. The default speed is the

85^{th} percentile speed. The mean speed is automatically calculated. The user can also request any distance interval to perform the speed prediction. A distance interval of 100 ft is provided as the default value.

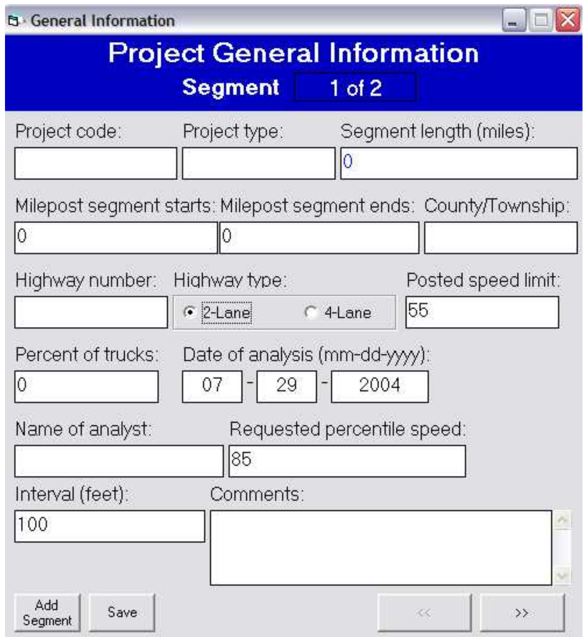


Figure 9-3 Project general information form

If consecutive segments in the project have different number of travel lanes (two-to-four or four-to-two), the tool estimates the transition speed by applying the mean deceleration and acceleration rates. If the transition is from two to four lanes, the mean acceleration rate will be applied starting at the end of the two-lane segment. If the transition is from four to two lanes, the mean deceleration rate will be applied ending at the beginning of the four-lane segment.

Figure 9-4 shows the cross-section information form. This form is used to input the cross-section dimensions for any specified number of segments in the project by specifying the starting and ending mileposts. The form includes default values corresponding to typical values used in highway design or unrestricted base highway conditions. The default values are applied to the entire project length unless otherwise specified. The form requires separate inputs for two-lane and four-lane segments. For two-lane segments, the user must enter the widths of the traveled way and three traversable shoulders. The shoulder widths must represent the total for both directions. The tool assumes a symmetric cross-section with respect to the highway centerline. For example, entering a total gravel shoulder width of 8 ft indicates that a gravel shoulder of 4 ft is used in each travel direction. For four-lane segments, the user must enter the widths of the traveled way, the median (or the internal clear zone if a barrier is present) and the roadside clear zone distance. The cross-section widths must represent the total width for one travel direction.

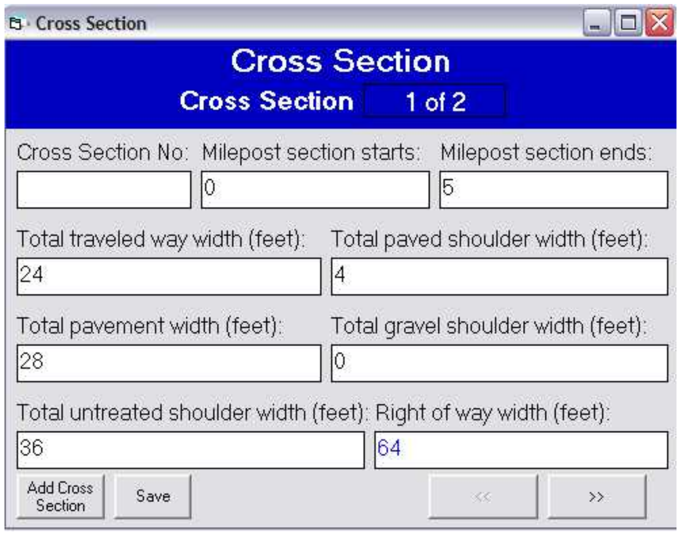


Figure 9-4 Cross-section information form

Figure 9-5 shows the horizontal curve information form. This form is active only for two-lane rural highway projects. The form provides the option of entering the design information for any specified number of curves in the project. No default values are provided for the curve components. The user must enter the radius, the maximum superelevation rate, and the starting and ending mileposts for each curve. A maximum sight distance of 2200 ft is used as a default value for all curves unless otherwise specified by the user.

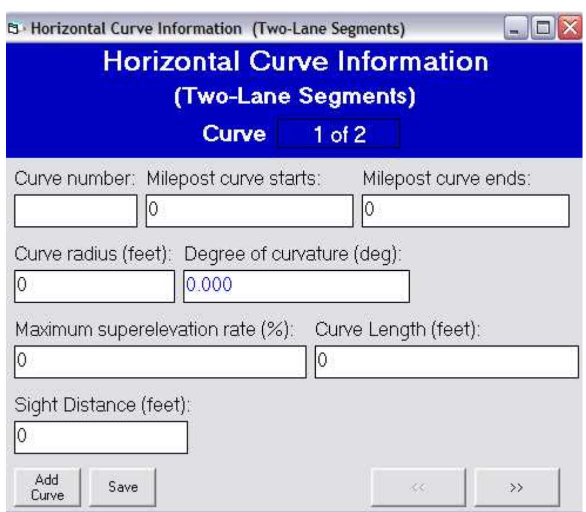


Figure 9-5 Horizontal curve information form

Figure 9-6 shows the additional highway information form for two-lane rural highway projects. The additional information required is the segment grade, the sight distance on tangent segments, and the location of high residential developments and intersections along the project. The form allows the user to identify any specified number of segments having different values in grade and sight distance. A grade of zero percent is provided as a default value. Engineering judgment must be exercise with caution by the user when entering high grade values for considerable long distances. Grades of considerable length that might cause a significant speed reduction for trucks must not be analyzed with the speed models provided with this tool. The tool will assume no restriction in sight distance in the entire project, but the impact on speeds of the sight distance is different for tangent segments than for curves in two-lane rural highways. A maximum sight distance of 712.6 ft is used as a default value for tangent segments for the entire project unless a lower value is otherwise specified. This default value corresponds to the maximum sight distance value used by the speed model to calculate speeds on tangents. Sight distance values higher than the default will not provide any additional increase in speeds due to the curvilinear relationship between the sight distance and the observed speeds in tangent segments. The segments having a residential driveway density of more than 10 driveways per mile can be identified in the project by providing their starting and ending mileposts. All intersections are identified by providing the milepost for the center of the intersection. The tool assumes that there are no segments along the project having high density of residential driveways or intersections unless otherwise specified.

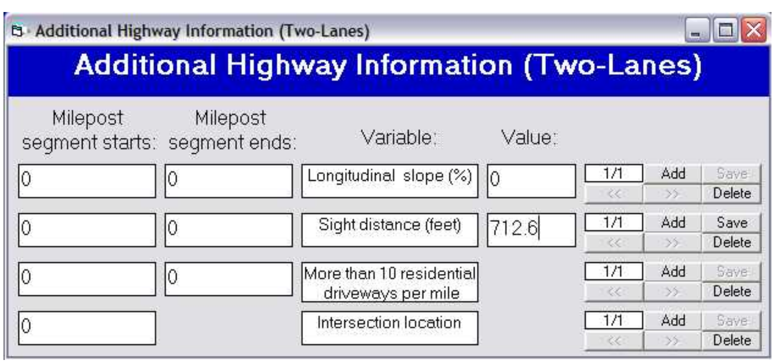


Figure 9-6 Additional highway information form for two-lane rural highway projects

Figure 9-7 shows the additional highway information form for four-lane highway projects. The additional information required is the sight distance and the intersection density along the project. The tool will assume no restriction in sight distance for the entire project. A maximum sight distance of 2078 ft is used as a default value for the entire project unless a lower value is otherwise specified by the user. The default sight distance corresponds to the maximum value observed in the field. The form allows the user to identify any specified number of segments with different intersection densities by providing the starting and ending mileposts. The intersection density must represent the total number of intersections per segment mile. The default density value is 0 intersections per mile. The form can be also used to identify segments located in rural or suburban settings and the location of segments that include a TWLT median lane. The tool assumes that the project is located in a suburban area unless otherwise specified.

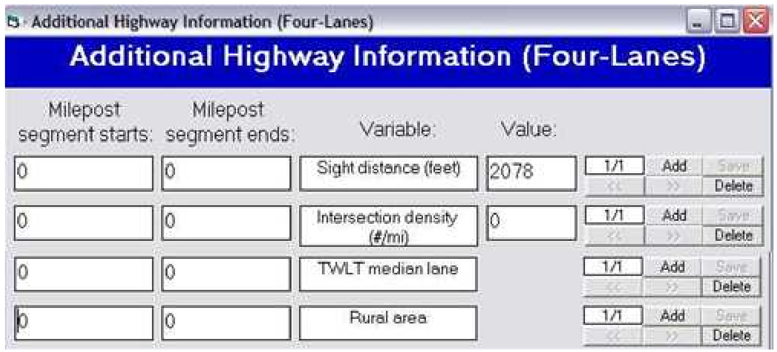


Figure 9-7 Additional information form for four-lane rural highway projects

The speed calculations are made based on the input information for the project. The calculations are fairly detailed and involve taking into account the information of various forms and segments, after the input is saved in the tool. The speed calculations are started by selecting the CALCULATE menu on the main screen (Figure 9-2).

9.4. SPEED RESULTS

The results are provided as a speed profile graph for the entire project length. Operating speed profiles has been promoted widely as a practical tool to evaluate the design consistency of new design projects and to assess the impact of improvement projects in existing highways. Two issues are usually targeted: the discrepancy between the operating speeds and the design speeds and the speed reduction between successive geometric features. Several countries have already incorporated the expected operating speed as a basis for selecting design speeds or specific geometric components or for detecting design inconsistencies.

Figure 9-8 shows an example of the speed profile for a highway project. The example profile shows the speeds along a 2-mile long 2-lane rural segment with a horizontal curve. To view the speed profile chart, the user must select the CHART menu on the main screen (Figure 9-2). The profile shows the mean speed and the 85th percentile speed for the example project. The user can check the milepost and the predicted speed at any point by directly clicking on the speed profile graph. The chart provides many options to display different regions of interest on the speed profile. The scroll bar located on the bottom of the chart can be used to browse through the project. The options on the right side of the chart can be used to change the size of the window, the minimum milepost, and the major and the minor unit of the horizontal scale. The UPDATE WINDOW button can be used to reflect the desired changes in the chart. The vertical scale is automatically adjusted by the tool.

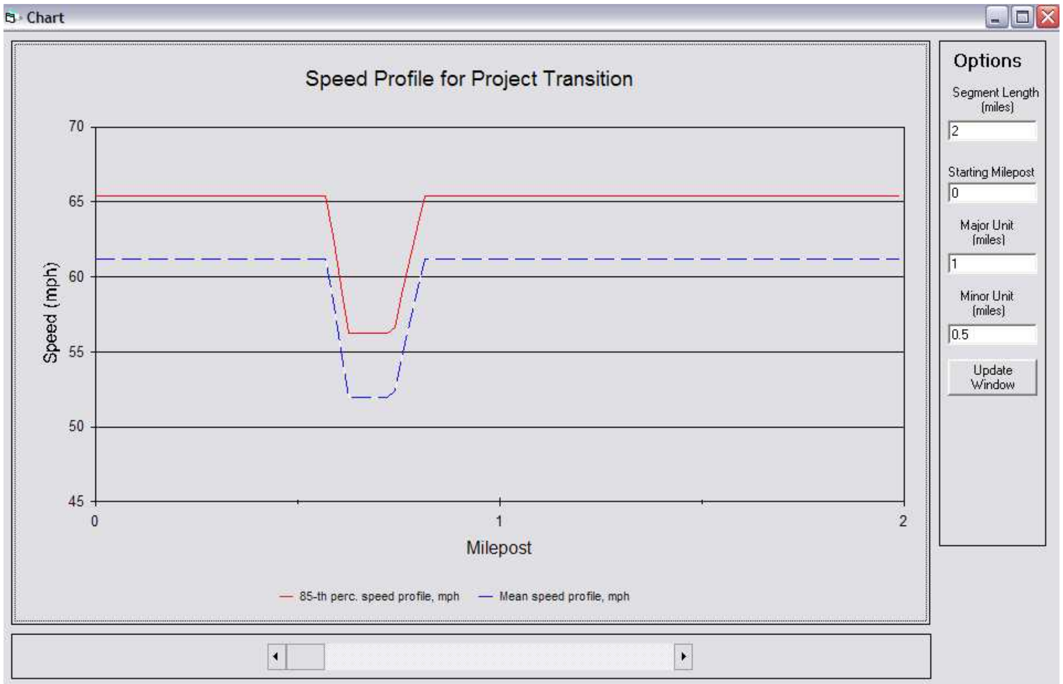


Figure 9-8 Speed profile example

The speed profile can be used to identify locations where the predicted speed is lower than the desired speed for the highway project. The profile can also be used to evaluate the consistency of the proposed design by identifying locations where speed changes are higher than a desired value. The user can request printouts of the entire speed profile and the tables with the input values used to predict speeds for the highway project. The chart printout will be an exact copy of the speed profile as displayed on the chart window. The chart options allow the user to get different snapshots of a project profile in cases when the project length requires more than one graph to observe the speed graphs. The tool allows the user to make modifications to the design project values to evaluate how speed changes in the profile. The user must open the desired input form to make any change in the input values by selecting it from the INSERT menu on the main screen (Figure 9-2). The tool allows the user to have multiple forms open at the same time to make it easy to visualize the changes made in the project. The changes made in each form must be saved by using the SAVE button. The user must request the tool to perform again the speed calculations before the chart can show the updated speed profile. The impact of any modification made to the design values will be reflected immediately on the speed profile.

CHAPTER 10. CONCLUSIONS AND RECOMMENDATIONS

The mean free-flow speed and the variability of speeds across drivers are important safety factors in setting speed limits and designing roadways. Despite a large body of past research on speeds, there is still much to learn about the factors of free-flow speeds, especially for tangent segments and suburban highways. The existing models estimate a specific speed percentile and they do not distinguish between the mean speed factors and the speed dispersion factors, which leads to results that are sometimes difficult to interpret. It is possible that a road with a high mean speed and low speed variability has the same 85^{th} speed percentile as a road with a much lower mean speed but higher speed variability. In addition, most of the models were developed using an approach based on the effect of isolated horizontal or vertical alignment conditions.

This report presents an advanced method of modeling free-flow speeds that overcomes the limitations of the existing models. The entire speed distribution was utilized as an innovative approach to develop the speed models instead of focusing on a particular percentile. This was accomplished by representing the percentile speed as a linear combination of the mean and the standard deviation. This model is possible due to the normality of individual free-flow speeds distribution at a site. Two alternate models were evaluated: an ordinary least squares model for panel data (OLS-PD) and a generalized least squares model that considers random effects (RE).

Free-flow speeds and geometry characteristics collected from two-lane rural and four-lane rural and suburban highways were used to develop the speed models. The models demonstrated their efficiency in identifying relationships between diverse road geometry characteristics and speeds. Existing models for tangent segments on two-lane rural highways have not been able to identify significant relationships between speeds and the cross-section dimensions. In contrast, most of the cross-section dimensions were retained in the models. The interpretation of the model results was straightforward. It was equally easy to identify the impacts of the variables on the mean speed and on the speed standard deviation.

The OLS-PD models were suggested for implementation to predict free-flow speeds in two-lane rural highways. Four models were developed to predict the free-flow speeds depending on the location: tangents, horizontal curves, or curve transition sections. The model for tangent segments includes ten different highway characteristics; six of them included as both mean speed and speed dispersion factors. The posted speed limit and the widths of two of the traversable shoulders were identified as the strongest mean speed and speed standard deviation factors. There are additional characteristics (sight distance, highway grade and residential driveway density) included in the model that have smaller impacts on speeds. The following speed factors, and their impacts, were identified for tangent segments in two-lane rural highways:

- reducing the speed limit decreases the mean speed and increases the speed dispersion
- increasing any of the cross-section dimensions increases the mean speed
- reducing the traveled way and the distance between the roadside obstructions and the travel lanes increases the speed dispersion
- increasing the highway grade reduces the mean speed and increases the speed dispersion
- increasing the sight distance up to 712.6 ft increases the mean speed; higher sight distance values do not provide any additional increase in speeds
- an increase in the truck percentage in the free-flow speed distribution decreases the mean speed
- a high density of residential driveways (10 or more per mile) reduces the mean speed
- the presence of intersections reduces locally the mean speed

The model for horizontal curves includes four different highway and curve characteristics, two of them included as both mean speed and speed dispersion factors. The degree of curve and the superelevation rate were identified as the strongest mean speed and speed standard deviation factors. The following speed factors, and their impacts, were identified for horizontal curves in two-lane rural highways:

- increasing the degree reduces the mean speed and increases the speed dispersion
- increasing the superelevation rate increases mean speed up to a certain value. The impact mechanism for the superelevation rate is not as evident due to its curvilinear relationship with speeds and its correlation with the degree. Any change in the superelevation rate must include the corresponding change in the degree of curve in order to evaluate its impact on speeds
- increasing the sight distance increases the mean speed
- a high density of residential driveways (10 or more per mile) reduces the mean speed

The results of the transition speed models are easy to explain. The transition models established that 65.5 percent of the deceleration transition length occurs on the tangent segment prior to the curve with a mean deceleration rate of 0.033 (ft/s)/ft, and that 71.6 percent of the acceleration transition length occurs on the tangent segment following the curve with a mean acceleration rate of 0.022 (ft/s)/ft.

A single RE model was suggested for implementation to predict free-flow speeds in four-lane rural and suburban highways. The RE model includes ten different highway characteristics; seven of them included as both mean speed and speed dispersion factors. The posted speed limit, the intersection density and the median width were identified as the strongest speed factors in four-lane highways. There are additional characteristics (sight distance, presence of a TWLT median lane and the rural vs. suburban setting) included in the model that have smaller impacts on speeds. The following speed factors, and their impacts, were identified for four-lane highways:

- reducing the posted speed limit decreases the mean speed
- setting low speed limits (40 – 45 mph) decreases the speed dispersion
- increasing the median width or the roadside clear zone increases the mean speed while reducing them increases the spread of individual speeds
- including a TWLT median lane decreases the speed dispersion
- increasing the intersection density decreases the mean speed and increases the speed dispersion
- increasing the sight distance increases the mean speed and decreases the speed dispersion
- a suburban setting decreases the mean speed and increases the speed dispersion

The developed models have the same prediction capabilities as the existing percentile-specific models. The advantages of the developed models include predicting any user-specified percentile speed, involving more design variables than traditional OLS models, and separating the impacts on mean speed from the impacts on speed dispersion. The crash experience was considered to eliminate cases where the drivers' perception might be incorrect, as represented by a considerably high crash rate for the highway segments. The speed estimates from the proposed models will concur with a satisfactory level of safety for modernized highway segments.

The models were implemented in a visual basic-based tool that will help highway designers to predict any percentile speed for improved two- and four-lane highway projects. The tool can be integrated into the highway design process to evaluate if the estimated free-flow speeds meet the desired speeds for the highway project. The tool calculates a profile of the mean and any specified percentile speed for the entire project length based on the highway design values. The speed profile is a practical tool to evaluate the design consistency of new design projects and to assess the impact of improvement projects in existing highways. The speed profile will aid designers to identify segments with discrepancies between operating speeds and design speeds and segments showing excessive reduction in operating speeds between successive geometric features. The tool can be easily used to evaluate modifications in the design values that will increase or reduce the expected speed until it reaches the desired speed.

The results from this research study can be used as basis to ask for changes in the INDOT highway design policies. The INDOT definition of operating speed as “the highest overall speed at which a driver can safely travel while not exceeding the design speed” is incompatible with the current definitions in the MUTCD or the AASHTO Green Book. AASHTO relaxed their operating speed definition as “the speed at which drivers are observed operating their vehicles during free-flow conditions”. The 85th percentile value of the free-flow speed distribution is typically used to represent the operating speed on a highway (although higher percentiles have been also suggested), and to set posted speed limits.

The INDOT design manual also presents fundamental conflicts in the application of the different speed concepts (design speed, operating speed, etc.) in highway design. The major conflict is the requirement for new construction and reconstruction projects that the posted speed limit should be set equal to the design speed used in design, if this does not exceed the legal limit. Using the design speed of the project to set the posted speed limit on the highway might result, in some cases, on a too low value that might not meet the drivers’ expectations on the highway. Liability concerns are the most likely reasons behind this requirement, even though drivers can exceed the design speed without any obvious safety hazard.

An evaluation of the observed free-flow speeds proved that the 85th percentile speeds exceed the inferred design speed in most cases. The difference between the 85th percentile speed and the inferred design speed was exceptionally high, varying from 19 to 28 mph (30.5 to 45 km/h), on two-lane rural tangent segments with a 40-mph design speed. The 85th percentile speeds were 5.6 to 13 mph (9 to 20.9 km/h) higher than the posted speed limits on tangent segments in two-lane rural highways. In average, 82.8 percent of all drivers were going at speeds faster than the posted speed limit on all sites. If the 85th percentile rule is applied literally, fourteen sites (43.8 percent) could have a posted speed limit equal to 60 mph and seventeen sites (48.6 percent) could have a posted speed limit equal to 65 mph, without presenting obvious safety problems.

All sites observed on horizontal curves had 85th percentile speeds higher than the curve inferred design speed. The difference between the inferred design speeds and the 85th percentile speeds varied from 5.1 to 15.8 mph. The curves without advisory speeds had 85th percentile speeds that exceeded the inferred design speeds in a range of 8.3 mph to 11.4 mph. In average, 77 percent of the drivers operated at speeds higher than the advisory speed or the posted speed limit on horizontal curves.

All the sites observed in four-lane highways had 85th percentile speeds higher than the posted speed limit. All, but one, of the observed sites on four-lane highway segments had inferred design speeds equal or higher than the posted speed limit on the segment. Regardless of the posted speed limit in the segment, the minimum percentage of vehicles going at a speed higher than the posted speed limit in any site is 27 percent. Thirty-six sites (72 percent) exhibited a more extreme behavior with 70 percent of more of the vehicles going at speeds higher than the posted speed limit.

By using design speeds, highways are designed in a conservative manner to facilitate the safe motion of vehicles even in adverse but reasonable conditions. Designing for the worst scenario (e.g. combination of adverse conditions) generates conservative solutions with a built-in large margin of safety. Consequently, the 85th percentile of observed free-flow speeds may exceed the design speed. The current design policy can be modified to allow setting the posted speed limit at a value higher than the design speed, but according to the operating speeds and the crash experience. Engineering judgment can then be applied to balance safety and construction cost in highway improvement projects. Once the design policy is modified, the speed models developed in this research study can be used to predict safe operating speeds for improvement projects, context-sensitive projects or design exceptions.

The INDOT Standards Section of the Contracts and Construction division might adopt the research results to a format consistent with the other departmental policy documents. The adopted text will be added to the Indiana Design Manual – Part V, Road Design. The Scoping Section of the Environment, Planning and Engineering Division and the Design Division might implement the speed-predicting tool for two-lane rural and four-lane rural and suburban highway improvement projects.

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APPENDIX

APPENDIX A. DESCRIPTION OF GEOMETRIC DATA IN TWO-LANE HIGHWAYS

The definitions and descriptions presented here were used to collect data in two-lane rural highway segments. Figure A-1 shows the form used to record the highway information.

OBSERVATION SITE LOCATION AND CHARACTERISTICS

A. General Highway Characteristics

Number		Weather		Terrain	L / R / M	
County		Observers		Pavement	AC / PCC	
Date		Milepost		Speed limit		mph
Time		From / to		Adv. speed		mph
				Equipment	Laser / ADR	

B1. Observation Site Geometric Data

Site Id	000-000-000	# of lanes		TW width		ft
Direction	NB / SB / WB / EB			Lane width		ft

	SPOT 1			SPOT 2	
Sight distance		ft	Sight distance		ft
Mean	#DIV/0!	ft	*Mean*	#DIV/0!	ft
% slope			% slope		
Mean	#DIV/0!	%	*Mean*	#DIV/0!	%
Shoulder surface			Shoulder surface		
PAV		ft	PAV		ft
GRA		ft	GRA		ft
UNT		ft	UNT		ft
NOT		ft	NOT		ft
Clear Zone	0	ft	Clear Zone	0	ft
Obstruction			Obstruction		
Curb	YES / NO		Curb	YES / NO	
Curb type	MOUNT / BARRIER		Curb type	MOUNT / BARRIER	

C1. Observation Site Geometric Features

Tangent	YES / NO		Vertical curve	YES / NO		Intersection	YES / NO
			Curve type	SAG / CREST		Int. Type	T/4-leg/T-adj
			Curve length			Channelization	YES / NO
Hor. Curve	YES / NO					Auxiliary Lane	YES / NO
Measurements	IN / OUT						
Curve length		ft					
Superelevation	IN	OUT		IN, in	OUT, in		Adjusted, ft
			Middle ordinate				0.000
					Distance to CL		Adjusted, ft
Mean	#DIV/0!	#DIV/0! %	Radius	#DIV/0!			#DIV/0!
	Distance to						
PC Station		ft					
PT Station		ft					

D1. Features Ahead from Second Spot

Tangent	YES / NO		Vertical curve	YES / NO		Intersection	YES / NO
			Curve type	SAG / CREST		Int. Type	T/4-leg/T-adj
			Curve length			Channelization	YES / NO
Hor. Curve	YES / NO					Auxiliary Lane	YES / NO
Measurements	IN / OUT						
Curve length		ft					
Superelevation	IN	OUT		IN, in	OUT, in		Adjusted, ft
			Middle ordinate				0.000
					Distance to CL		Adjusted, ft
Mean	#DIV/0!	#DIV/0! %	Radius	#DIV/0!			#DIV/0!
	Distance to						
PC Station		ft					
PT Station		ft					

Figure A-1 Data collection form for two-lane highway segments

A. General Highway Characteristics

- *Highway number*: state road or U.S. highway number as it appears in highway maps.
- *Milepost*: closest highway mile post to the observation site.
- *Terrain type*: prevailing terrain type in the highway segment, if it is not clear which terrain type to use, the flatter should be selected.
    - *L (LEVEL)*: terrain is considered to be flat, sight distances are long and the impact on vehicular performance is minimal.
    - *R (ROLLING)*: natural slopes consistently rise above and fall below the roadway grade and, occasionally, steep slopes present some restriction to the desirable highway alignment. In general, generates steeper grades, causing trucks to reduce speeds below those of passenger cars.
    - *M (MOUNTAINOUS)*: longitudinal and transverse changes in elevation are abrupt and benching and side hill excavations are frequently required to provide the desirable highway alignment. It aggravates the performance of trucks relative to passenger cars, resulting in some trucks operating at crawl speeds.
- *Pavement*: pavement surface of the travel lane monitored; AC for asphalt concrete or PCC for Portland cement concrete.
- *Speed limit*: posted speed limit for the highway segment.
- *Adv. speed*: advisory speed sign posted for horizontal curves or intersections.

B. Observation Site Geometric Data

- *Site Id*: unique ten-digit control number assigned to each observation site, for example, 079-025-001_1. The first three digits represent the county number, the middle three represent the highway number, the next three identifies a specific observation site in that highway and the last digit identifies the first or second spot of the observation site.
- *TW (traveled way) width*: portion of the roadway for the movement of vehicles, exclusive of shoulders.
- *Lane width*: width of the travel lane in the direction monitored.
- *Sight distance*: available sight distance at the observation site. It is the distance along a roadway throughout which an object of specified height is continuously visible to the driver. This distance was measured with the ranging laser gun; two measurements were taken at each spot.
- *% slope*: segment grade, in percent. The grade was measured with the electronic level at each spot of the observation site; two measurements were taken at each spot.
- *Shoulder surface*: portion of the cross-section contiguous with the traveled way for accommodation of stopped vehicles, for emergency use and for lateral support of subbase, base and surface course. The widths of three types of traversable shoulder

surfaces were measured. Another roadside surface that was not stabilized and it was not suitable for the safe accommodation of stopped vehicles, or containing obstacles like trees, boulders, etc., was not measured.

- *PAVED*: usable shoulder paved with an AC or PCC surface.
- *GRAVEL*: usable shoulder paved with gravel, shell, crushed rock, mineral or chemical additives or any other sealed aggregate surface.
- *UNTREATED*: usable shoulder with a stabilized turf growth or earth surface.

- *Clear zone*: unobstructed, relatively flat area provided beyond the edge of the traveled way for the recovery of errant vehicles, it includes any shoulders or auxiliary lanes.
- *Obstruction*: lateral obstruction that set apart the clear zone, e.g. pole line, guardrail, bridge parapet, curb, property line, etc.
- *Curb type*: type of curb present in the highway segment. Curbs were classified as:
  - *MOUNTABLE* curbs are low, no more than 6 in. in height, with flat sloping faces and are designed so that vehicles can cross them readily when required.
  - *BARRIER* curbs are relatively high and steep-faced, 6-9 in. in height, designed to inhibit or at least discourage vehicles from leaving the roadway.
- *Roadside rating*: roadside hazard rating system developed by Zegeer et al. (1988) to characterize the accident potential for roadside designs on two-lane highways. The roadside hazard rating is based on a seven-point categorical scale from 1 (best) to 7 (worst). Zegeer et al. (1988) provides illustrations for each rating. The criteria used to identify the ratings were the following:
  - *Rating 1*: clear zone greater than or equal to 9 m (30 ft) from the pavement edge and a recoverable roadside.
  - *Rating 2*: clear zone between 6 and 7.5 m (20 and 25 ft) from the pavement edge and a recoverable roadside.
  - *Rating 3*: clear zone of about 3 m (10 ft) from the pavement edge and a marginally recoverable rough roadside surface.
  - *Rating 4*: clear zone between 1.5 and 3 m (5 to 10 ft) from the pavement edge, may have a guardrail 1.5 to 2 m (5 to 6.5 ft) from the pavement edge, may have exposed trees, poles, or other objects about 3 m (10 ft) from the pavement edge and a marginally forgiving roadside, but with an increased chance of a reportable roadside collision.
  - *Rating 5*: clear zone between 1.5 and 3 m (5 to 10 ft) from the pavement edge, may have a guardrail 0 to 1.5 m (0 to 5 ft) from the pavement edge, may have rigid obstacles or embankment within 2 to 3 m (6.5 to 10 ft) of the pavement edge and virtually non-recoverable roadside.

- o *Rating 6*: clear zone less than or equal to 1.5 m (5 ft), no guardrail with exposed rigid obstacles within 0 to 2 m (0 to 6.5 ft) of the pavement edge and non-recoverable roadside
- o *Rating 7*: clear zone less than or equal to 1.5 m (5 ft), cliff or vertical rock cut with no guardrail and non-recoverable roadside with high likelihood of severe injuries from roadside collision.

C. Observation Site Geometric Features

- *Tangent*: record YES if the observation site is located on a straight segment.
- *Vertical curve*: record YES if a vertical curve is nearby the observation site. Vertical curves were classified as sag or crest curve.
- *Intersection*: record YES if an intersection is nearby the observation site. Intersections were classified as 4-leg, T-intersection or adjacent-T intersection.
- *Channelization*: record YES if the intersection has some sort of channelization. Channelization provides separation or regulation of conflicting traffic movements into definite paths of travel by traffic island or pavement marking to facilitate the orderly movements of both vehicles and pedestrians.
- *Auxiliary lane*: record YES if the intersection has auxiliary lanes for some or all turning movements. Auxiliary lanes are used preceding median openings and at intersections preceding right-turning movements, it includes left and right turn lanes, acceleration and deceleration lanes, and climbing lanes.
- *Distance to*: distance from the observation site to the center of the intersection.
- *Horizontal curve*: record YES if a horizontal curve is nearby the observation site.
- *Curve length*: measure the length of the horizontal curve from PC to PT.
- *Superelevation*: measure the maximum superelevation rate of the horizontal curve. The maximum superelevation rate was measured with the electronic level in four spots (two in each travel lane) at about the middle of the curve.
- *Middle ordinate*: the middle ordinate was measured as the perpendicular distance from the middle of a 100-foot long chord to the circular curve. The curve radius *R*, in feet, was calculated using the chord and the middle ordinate in the same units as:

$$R = \frac{C^2}{8 \times M} + \frac{M}{2}$$

The degree of curvature *DC* was calculated by dividing 5,729.578 with the radius, in feet.

Section D was used to record the information for a second vertical curve, horizontal curve or intersection, if present, nearby the observation site.

Figure A-2 shows the FHWA Type F classification scheme used to record the vehicle class. The classification scheme is separated into categories depending on whether the vehicle carries passengers or commodities. Non-passenger vehicles are further subdivided by number of axles and number of units, including both power and trailer units. Note that the addition of a light trailer to a vehicle does not change the classification of the vehicle. The classification is based on the axle spacing (in feet) and the number of axles for each vehicle class.

Type	Description	1-2 Axles	2-3 Axles	3-4 Axles	4-5 Axles	5-6 Axles	6-7 Axles
1	Motorcycle	0.1-6.0					
2	Car	6.1-10.2					
2	Car w/1 Axle Trlr	6.1-10.2	6.0-18.0				
2	Car w/2 Axle Trlr	6.1-10.2	6.0-18.0	0.1-6.0			
3	Pickup/Van	10.3-13.0					
3	Pickup/Van w/1A Trlr	10.3-13.0	6.0-18.0				
3	Pickup/Van w/2A Trlr	10.3-13.0	6.0-18.0	0.1-6.0			
4	Bus	20.0-40.0					
4	Bus	20.0-40.0	0.1-6.0				
5	2-Axle -- Six Tire	13.1-20.0					
6	3-Axle -- Single Unit	6.1-23.0	0.1-6.0				
7	4-Axle -- Single Unit	6.1-23.0	0.1-9.0	0.1-9.0			
8	2S1	6.1-17.0	14.0-40.0				
8	3S1	6.1-20.0	0.1-6.0	6.1-40.0			
8	2S2	6.1-17.0	14.0-40.0	0.1-6.1			
9	3S2	6.1-22.0	0.1-6.0	6.1-40.0	0.1-9.0		
9	3-Axle w/Trlr	6.1-22.0	0.1-6.0	6.1-23.0	1.1-23.0		
10	6-Axle -- Single Trlr	6.1-22.0	0.1-6.0	0.1-40.0	0.1-11.0	0.1-11.0	
10	7-Axle -- Single Trlr	6.1-22.0	0.1-6.0	0.1-40.0	0.1-13.0	0.1-13.0	0.1-13.0
11	5-Axle -- Multi Trlr	6.1-17.0	11.1-25.0	6.1-18.0	11.1-25.0		
12	6-Axle -- Multi Trlr	6.1-22.0	0.1-6.0	1.1-25.0	6.1-18.0	11.1-25.0	
13	7-Axle -- Multi Trlr	0.1-40.0	0.1-40.0	0.1-40.0	0.1-40.0	0.1-40.0	0.1-40.0
15	Unclassified Vehicles						

Figure A-2 FHWA vehicle classification scheme F

Figure A-3 shows the layout used to determine the cosine error correction. The laser gun measures the relative speed that a vehicle is approaching the gun. If the laser gun aims directly to the vehicle path, the alpha (α) angle is zero degrees and the measured speed is equal to the actual vehicle speed. If the laser gun is not located directly in the vehicle path the measured speed is lower than the actual vehicle speed. The measured speed is directly related to the cosine of the alpha angle between the laser and the vehicle travel direction. If the alpha angle is maintained below 20 degrees, the measured speed will be more or equal than 95 percent of the actual speed. As alpha increases, the larger the error and the lower the displayed measured speed.

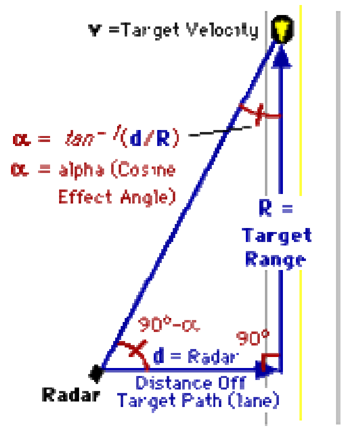

Figure A-3 Cosine effect correction layout

On a straight highway segment, the distance (*d*) of the laser from the lane centerline and the range (*R*) of the vehicle determine alpha. The greater the distance the laser is off the road and/or the closer the target, the larger the angle and the error. The following equation was used to adjust the speeds measured with the laser gun:

$$V_{actual} = \frac{V_{laser}}{\cos\alpha} = \frac{V_{laser}}{\left(\dfrac{R}{\sqrt{R^2 + d^2}}\right)}$$

APPENDIX B. DESCRIPTION OF GEOMETRIC DATA IN FOUR-LANE HIGHWAYS

The definitions and descriptions presented here were used to collect data in four-lane highway segments. Some of the highway characteristics were already defined for two-lane rural highways in Appendix A. Figure B-1 shows the data collection form used to record the highway information.

OBSERVATION SITE LOCATION AND CHARACTERISTICS

A. General Highway Characteristics

Site Id		Date		Terrain	L / R / M	
Highway		Time		Pavement	PC / AC	
Direction		County		Speed limit		mph
From / to		Milepost		Adv Speed		mph

B. Observation Site Cross-section Data

Opposite Direction			*Median Information*			*Direction Measured*		
TW width		ft	Divided Hwy	YES / NO		TW width		ft
TW lanes			Barrier type			TW lanes		
In lane width		ft	Median type			In lane width		ft
Out lane width		ft	Median width		ft	Out lane width		ft

Shoulder Info.	*Opposite outside*	*Opposite inside*	*Inside measured*	*Outside measured*
PAVED	ft	ft	ft	ft
GRAVEL	ft	ft	ft	ft
UNTREATED	ft	ft	ft	ft
NO				
Clear Zone	0 ft	0 ft	0 ft	0 ft
Obstruction				
Only Lane	YES / NO	YES / NO	YES / NO	YES / NO
Only Lane width	ft	ft	ft	ft
Curb	YES / NO	YES / NO	YES / NO	YES / NO
Curb type	Sloped / Unsloped	Sloped / Unsloped	Sloped / Unsloped	Sloped / Unsloped
Parking allowed	YES / NO	YES / NO	YES / NO	YES / NO
Bus stop	YES / NO	YES / NO	YES / NO	YES / NO
Sidewalk	YES / NO	YES / NO	YES / NO	YES / NO

		Mean				*Mean, %*
Sight (ft)		#DIV/0!	ft	Slope (%)		#DIV/0!

	T	4-LEG	T-ADJ	Median Openings		Opposite Side	Measured Side
# Intersections					# Driveways		
Int. Density	0	0	0	0	Driveway Density	0	0
Total Intersection Density	0				Total Driveway Density		0

*if crossing road has a stop sign it is consider an intersection

* measured 1/4 mile before and after

C. Observation Site Geometric Features

Tangent	YES / NO	Hor. Curve 1	YES / NO		Middle Ordinate	IN, in	OUT, in
		Measurement	IN / OUT				
Intersection 1	YES / NO	Curve length	ft		Middle Ordinate Adj., ft		0.000
Int. Type	4-LEG / T / T-ADJ	Radius	#DIV/0! ft		Dist. to CL, ft		#DIV/0!
Channelization	YES / NO		IN	OUT			
Auxiliary Lane	YES / NO ft	Superelevation				PC Station	PT Station
Distance to					Distance to, ft		
		Mean, %	#DIV/0!	#DIV/0!	#DIV/0!		
Intersection 2	YES / NO						
Int. Type	4-LEG / T / T-ADJ	Hor. Curve 2	YES / NO		Middle Ordinate	IN, in	OUT, in
Channelization	YES / NO	Measurement	IN / OUT				
Auxiliary Lane	YES / NO	Curve length	ft		Middle Ordinate Adj., ft		0.000
Distance to		Radius	#DIV/0! ft		Dist. to CL, ft		#DIV/0!
			IN	OUT			
		Superelevation				PC Station	PT Station
					Distance to, ft		
		Mean, %	#DIV/0!	#DIV/0!	#DIV/0!		

Figure B-4 Data collection form for four-lane highway segments

A. General Highway Characteristics

The same definitions used for two-lane highways were applied.

B. Observation Site Cross-section Data

The cross section was divided in three parts: the travel direction where speeds were measured, the opposite travel direction and the median. The cross-section dimensions were recorded separately for the observed direction and for the opposite direction. The same definitions for lane and shoulder widths used in two-lane highways were applied. Additional information recorded in four-lane highways:

- *Divided highway*: record YES if a median is separating the opposing travel directions. Record NO if the opposing travel directions are divided only by pavement markings.
- *Barrier type*: record the type of median barrier (concrete barrier or guardrail) present in the highway segment.
- *Median type*: a median is the portion of a highway separating opposing directions of the traveled way. The types of median were classified as: two-way left turn lane, flush grass or paved, or depressed grass median.
- *Median width*: the dimension between the edges of traveled way and includes the left shoulders, if any.
- *Only lane*: record YES if auxiliary lanes are present.
- *Only lane width*: record the width(s) of the auxiliary lane(s), if present.
- *Parking allowed*: record YES if on-street parking is allowed.
- *Bus stop*: record YES if a bus stop or bus turnout is present.
- *Sidewalk*: record YES if a sidewalk is present.
- *Intersections*: record the number of intersections located 0.25 miles before and after the observation site. Intersections were classified as 4-leg, T-intersection or adjacent-T intersection. Any crossing road with a stop sign or stop bar was considered as an intersection.
- *Median openings*: record the number of median openings located 0.25 miles before and after the observation site.
- Driveways: record the number of driveways per travel direction located 0.25 miles before and after the observation site.

C. Observation Site Geometric Features

The same definitions used in two-lane highways were applied.

APPENDIX C. SAS OUTPUT FOR SPEED MODELS

The REG Procedure
Model: MODEL1
Dependent Variable: SPDPI (MPH)

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	17	39248	2308.72278	515.29	<.0001
Error	1597	7155.27566	4.48045		
Corrected Total	1614	46404			

Root MSE	2.11671	R-Square	0.8458
Dependent Mean	57.04110	Adj R-Sq	0.8442
Coeff Var	3.71085		

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Intercept	1	57.13716	0.60190	94.93	<.0001
SPL50	1	-3.08181	0.14037	-21.96	<.0001
TR	1	-0.07103	0.01087	-6.54	<.0001
GRA	1	-0.13066	0.02482	-5.26	<.0001
SD	1	0.00238	0.00063261	3.76	0.0002
SQSD	1	-0.00000167	2.565983E-7	-6.51	<.0001
INT	1	-0.42156	0.12335	-3.42	0.0006
RES	1	-1.03382	0.13680	-7.56	<.0001
PAV	1	0.04013	0.00948	4.23	<.0001
GSW	1	0.39408	0.03257	12.10	<.0001
USW	1	0.05442	0.00473	11.50	<.0001
FC	1	-2.23289	0.15769	-14.16	<.0001
Zp	1	5.98158	0.27862	21.47	<.0001
ZSPL50	1	1.42801	0.14984	9.53	<.0001
ZGRA	1	0.06082	0.02832	2.15	0.0319
ZINT	1	0.29168	0.13891	2.10	0.0359
ZPAV	1	-0.03825	0.00829	-4.62	<.0001
ZUNSW	1	-0.01180	0.00479	-2.46	0.0140

UNSW = GSW + USW

Figure C-5 SAS output for OLS-PD model of tangent segments in two-lane highways

The REG Procedure
Model: MODEL1
Dependent Variable: SPDPI (MPH)

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	8	11276	1409.53011	456.72	<.0001
Error	257	793.14748	3.08618		
Corrected Total	265	12069			

Root MSE	1.75675	R-Square	0.9343
Dependent Mean	53.53045	Adj R-Sq	0.9322
Coeff Var	3.28178		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Intercept	1	47.66393	0.70375	67.73	<.0001
SIGHT	1	0.00344	0.00038581	8.91	<.0001
RESIDENTIAL	1	-2.63882	0.37769	-6.99	<.0001
DEGREE	1	-2.54087	0.07225	-35.17	<.0001
SUPERELEV	1	7.95351	0.25641	31.02	<.0001
SQ SUPERELEV	1	-0.62395	0.01916	-32.57	<.0001
Zp	1	4.15760	0.40492	10.27	<.0001
Z-DEGREE	1	0.23579	0.06700	3.52	0.0005
Z-SUPERELEV	1	-0.19873	0.06797	-2.92	0.0038

Figure C-6 SAS output for OLS-PD model of horizontal curve in two-lane highways

The REG Procedure
Model: MODEL1
Dependent Variable: Vp (MPH)

NOTE: No intercept in model. R-Square is redefined.

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	2	113627	56814	1499.42	<.0001
Error	568	21522	37.89029		
Uncorrected Total	570	135149			

Root MSE	6.15551	R-Square	0.8408
Dependent Mean	12.77525	Adj R-Sq	0.8402
Coeff Var	48.18308		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Td	1	0.65534	0.01216	53.91	<.0001
DEC RATE	1	-0.03299	0.00175	-18.84	<.0001

Figure C-7 SAS output for OLS-PD deceleration transition zone model in two-lane highways

The REG Procedure
Model: MODEL1
Dependent Variable: VT_V

NOTE: No intercept in model. R-Square is redefined.

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	2	131988	65994	1949.11	<.0001
Error	549	18588	33.85853		
Uncorrected Total	551	150577			

Root MSE	5.81881	R-Square	0.8766
Dependent Mean	13.89950	Adj R-Sq	0.8761
Coeff Var	41.86344		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Ta	1	0.71637	0.01165	61.50	<.0001
ACC RATE	1	-0.02211	0.00151	-14.60	<.0001

Figure C-8 SAS output for OLS-PD acceleration transition zone model in two-lane highways

The Mixed Procedure
Convergence criteria met.
Covariance Parameter Estimates

Cov Parm	Estimate	Standard Error	Z Value	Pr Z
SITE	5.8934	0.8993	6.55	<.0001
Residual	0.7177	0.02513	28.56	<.0001

Fit Statistics

-2 Res Log Likelihood	4838.6
AIC (smaller is better)	4842.6
AICC (smaller is better)	4842.6
BIC (smaller is better)	4847.6

Effect	Estimate	Standard Error	DF	t Value	Pr > |t|
Intercept	55.4910	0.7491	87	74.08	<.0001
SPL50	-2.7589	0.6211	1631	-4.44	<.0001
GRAVEL SHO	0.4297	0.09759	1631	4.40	<.0001
UNTREATED SHO	0.04743	0.02040	1631	2.32	0.0202
Zp	7.9050	0.4037	1631	19.58	<.0001
Z-SPL50	1.3021	0.06985	1631	18.64	<.0001
Z-%TRUCK	0.01763	0.004332	1631	4.07	<.0001
Z-SLOPE	0.05582	0.01104	1631	5.05	<.0001
Z-INTERSECT	0.2270	0.05399	1631	4.20	<.0001
Z-TRAVEL WAY	-0.1387	0.01737	1631	-7.98	<.0001
Z-CLEAR ZONE	-0.01143	0.001851	1631	-6.17	<.0001

Figure C-9 SAS output for RE model of tangent segments in two-lane highways

The Mixed Procedure
Convergence criteria met.
Covariance Parameter Estimates

Cov Parm	Estimate	Standard Error	Z Value	Pr Z
SITE	6.7101	2.5441	2.64	0.0042
Residual	0.3985	0.03155	12.63	<.0001

Fit Statistics

-2 Res Log Likelihood	793.0
AIC (smaller is better)	797.0
AICC (smaller is better)	797.1
BIC (smaller is better)	798.8

Effect	Estimate	Standard Error	DF	t Value	Pr > |t|
Intercept	51.1117	3.5218	14	14.51	<.0001
DEGREE	-2.0496	0.3886	319	-5.27	<.0001
SUPERELEV	7.2506	1.5771	319	4.60	<.0001
SQ SUPERELEV	-0.6201	0.1189	319	-5.21	<.0001
Zp	4.4937	0.2621	319	17.15	<.0001
Z-%TRUCK	0.05269	0.01190	319	4.43	<.0001
Z-SIGHT	-0.00082	0.000145	319	-5.67	<.0001
Z-DEGREE	0.1939	0.02145	319	9.04	<.0001
Z-SUPERELEV	-0.1994	0.02636	319	-7.56	<.0001

Figure C-10 SAS output for RE model of horizontal curves in two-lane highways

The REG Procedure
Model: MODEL1
Dependent Variable: VT_V

NOTE: No intercept in model. R-Square is redefined.

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	2	93093	46546	1883.26	<.0001
Error	511	12630	24.71584		
Uncorrected Total	513	105723			

Root MSE	4.97150	R-Square	0.8805
Dependent Mean	11.94287	Adj R-Sq	0.8801
Coeff Var	41.62736		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Td	1	0.74992	0.01250	59.99	<.0001
DEC RATE	1	-0.02901	0.00145	-20.03	<.0001

Figure C-11 SAS output for RE deceleration transition section model in two-lane highways

The REG Procedure
Model: MODEL1
Dependent Variable: VT_V

NOTE: No intercept in model. R-Square is redefined.

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	2	111150	55575	2128.54	<.0001
Error	549	14334	26.10945		
Uncorrected Total	551	125484			

Root MSE	5.10974	R-Square	0.8858
Dependent Mean	12.58584	Adj R-Sq	0.8854
Coeff Var	40.59911		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Ta	1	0.77085	0.01200	64.25	<.0001
ACC RATE	1	-0.01762	0.00131	-13.40	<.0001

Figure C-12 SAS output for RE acceleration transition section model in two-lane highways

The REG Procedure
Model: MODEL1
Dependent Variable: SPDPI

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	18	32502	1805.68794	325.34	<.0001
Error	931	5167.13335	5.55009		
Corrected Total	949	37670			

Root MSE	2.35586	R-Square	0.8628
Dependent Mean	54.81397	Adj R-Sq	0.8602
Coeff Var	4.29792		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Intercept	1	53.88387	0.66720	80.76	<.0001
PSL50	1	-4.75280	0.21291	-22.32	<.0001
PSL45	1	-5.48068	0.27999	-19.57	<.0001
PSL40	1	-7.43243	0.47862	-15.53	<.0001
RURAL AREA	1	2.04530	0.27275	7.50	<.0001
SIGHT DIST.	1	0.00087110	0.00020453	4.26	<.0001
INTD	1	-0.27922	0.03172	-8.80	<.0001
DRIVEWAY DENS	1	-0.02315	0.01131	-2.05	0.0410
PAVED SHOULDER	1	1.73209	0.66148	2.62	0.0090
EXT.CLEAR ZONE	1	0.02003	0.00689	2.91	0.0037
INT CLEAR ZONE	1	0.04607	0.00590	7.81	<.0001
GUARDRAIL	1	-2.10242	0.43722	-4.81	<.0001
DITCH	1	-1.19337	0.27417	-4.35	<.0001
ZP	1	6.05144	0.38640	15.66	<.0001
Z-PSL45-40	1	-0.49657	0.25017	-1.98	0.0474
Z-RURAL	1	-0.58538	0.25767	-2.27	0.0233
Z-SIGHT DIST	1	-0.00041937	0.00022206	-1.89	0.0593
Z-EXT.CLR ZONE	1	-0.01109	0.00621	-1.79	0.0744
Z-TWLT	1	-0.43003	0.21883	-1.97	0.0497

Figure C-13 SAS output for OLS-PD model for four-lane highways

```
                Covariance Parameter Estimates

                             Standard         Z
Cov Parm      Estimate          Error     Value        Pr Z
SITE            5.0572         1.0179      4.97      <.0001
Residual        0.6179        0.02913     21.21      <.0001

                        Fit Statistics
         -2 Log Likelihood              2491.3
         AIC (smaller is better)        2527.3
         AICC (smaller is better)       2528.1
         BIC (smaller is better)        2561.7

                        Standard
Effect         Estimate        Error       DF    t Value    Pr > |t|
Intercept       54.0267       1.5104       42      35.77      <.0001
PSL50           -4.7644       0.8522      892      -5.59      <.0001
PSL45           -4.9419       1.0380      892      -4.76      <.0001
PSL40           -6.5094       1.8626      892      -3.49      0.0005
RUR              1.6520       1.0160      892       1.63      0.1043
SD             0.001281     0.000798      892       1.61      0.1087
INTD            -0.3204       0.1196      892      -2.68      0.0075
ECLR            0.03427      0.02611      892       1.31      0.1897
ICLR            0.05620      0.02168      892       2.59      0.0097
ZP               5.8994       0.1384      892      42.62      <.0001
ZPSL4540        -0.4231      0.08597      892      -4.92      <.0001
ZRUR            -0.4645      0.08867      892      -5.24      <.0001
ZSD            -0.00048     0.000075      892      -6.46      <.0001
ZINTD           0.04219      0.01087      892       3.88      0.0001
ZCLR           -0.00422     0.001091      892      -3.87      0.0001
ZTWLT           -0.4770      0.07440      892      -6.41      <.0001
```

Figure C-14 SAS output for RE model for four-lane highways

Table C-1 Standardized normal variables

Percentile, p	Standard normal variable, Zp	Percentile, p	Standard normal variable, Zp
0.05	-1.6452	0.55	0.1254
0.10	-1.2817	0.60	0.2529
0.15	-1.0364	0.65	0.3849
0.20	-0.8415	0.70	0.5240
0.25	-0.6742	0.75	0.6742
0.30	-0.5240	0.80	0.8415
0.35	-0.3849	0.85	1.0364
0.40	-0.2529	0.90	1.2817
0.45	-0.1254	0.95	1.6452
0.50	0.0000		