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OPERATING SPEED PREDICTION MODELS FOR HORIZONTAL CURVES ON RURAL FOUR-LANE NON-FREEWAY HIGHWAYS

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ABSTRACT OF DISSERTATION

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ABSTRACT OF DISSERTATION

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in the College of Engineering at the University of Kentucky

By
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Lexington, Kentucky

Director: Dr. Nikiforos Stamatiadis, Professor of Civil Engineering

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One of the significant weaknesses of the design speed concept is that it uses the design speed of the most restrictive geometric element as the design speed of the entire road. This leads to potential inconsistencies among successive sections of a road. Previous studies documented that a uniform design speed does not guarantee consistency on rural two-lane facilities. It is therefore reasonable to assume that similar inconsistencies could be found on rural four-lane non-freeway highways. The operating speed-based method is popularly used in other countries for examining design consistency. Numerous studies have been completed on rural two-lane highways for predicting operating speeds. However, little is known for rural four-lane non-freeway highways.

This study aims to develop operating speed prediction models for horizontal curves on rural four-lane non-freeway highways using 74 horizontal curves. The data analysis showed that the operating speeds in each direction of travel had no statistical differences. However, the operating speeds on inside and outside lanes were significantly different. On each of the two lanes, the operating speeds at the beginning, middle, and ending points of the curve were statistically the same.

The relationships between operating speed and design speed for inside and outside lanes were different. For the inside lane, the operating speed was statistically equal to the design speed. By contrary, for the outside lane, the operating speed was significantly lower than the design speed. However, the relationships between operating speed and posted speed limit for both inside and outside lanes were similar. It was found that the operating speed was higher than the posted speed limit.

Two models were developed for predicting operating speed, since the operating speeds on inside and outside lanes were different. For the inside lane, the significant factors are: shoulder type, median type, pavement type, approaching section grade, and curve length. For the outside lane, the factors included shoulder type, median type, approaching section grade, curve length, curve radius and presence of approaching curve. These factors indicate that the curve itself does mainly influence the driver's speed choice.

KEYWORDS: Operating Speed, Prediction Model, Horizontal Curve, Rural Four-Lane Non-Freeway Highway, Geometric Feature

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TO MY PARENTS

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CHAPTER 1: INTRODUCTION

1.1 Problem Statement

Traditionally design speed has been selected to determine the radii of horizontal curves for roadway design. One of the significant weaknesses of the design speed concept is that it uses the design speed of the most restrictive geometric element within the roadway section, usually the horizontal or the vertical curve of the alignment, as the design speed of the entire road. Therefore, the speeds that motorists travel on tangents are not explicitly taken into consideration in the design speed concept. This leads to potential inconsistencies among successive sections of a road. These inconsistencies might result in a sudden change in three aspects of the roadway environment: the characteristic of the roadway, driver workload, and driver operating speed.

A sudden change in the roadway characteristic might surprise motorists, and such sudden changes might violate driver's expectancy. Driver's expectancy is formed by driving experience, and it has a significant influence on driving task. Its increase might result in an increase of driver's mental workload (Messer et al., 1981). The lack of consistency in roadway geometric design has also been identified as an apparent potential cause of increasing driver's mental workload, which can lead to driver error (Kanellaidis, 1996). It has been found that driver error is one of the leading contributors to crashes (Alexander, 1986).

A requirement placed on roadway design is to meet driver expectations by creating a consistent roadway design. Studies have examined the relationships between design speed and operating speed on rural two-lane highways (Fitzpatrick et al., 1995; Ottesen and Krammes, 2000; Polus et al., 2000; Schurr et al., 2000; Fitzpatrick et al., 2003). Most studies concluded that operating speeds and design speeds are often not in agreement indicating that roadway design does not always meet driver expectations. These studies also indicate that design inconsistency exists on those roads. It is therefore reasonable to assume that similar inconsistencies could be found on rural four-lane non-freeway highways.

Currently, the two speed-based approaches, which are design speed approach and operating speed approach, are mainly used for evaluating design consistency. The design speed-based method is used by AASHTO for evaluating design consistency involving the selection of a design speed. It has been documented that the design speed-based method does not guarantee consistency (Krammes and Glascock, 1992). The operating speed-based method is popularly used in Europe, Australia, and Canada, and has also been proposed for use in the United States. This method has two different ways to evaluate design consistency: (1) predict the value of the operating speed differential between two successive sections of a road; (2) predict the difference between the operating speed and design speed values for a specific section of a road. Combination of both these ways is also used. The core of the operating speed-based method is the operating speed prediction. Again most of the work completed is for rural two-lane facilities and little is known for rural four-lane non-freeway highways.

Several studies indicate that horizontal curvature is highly related to crashes. It was reported that the average crash rate for highway segments including horizontal curves is about three times the average accident rate for tangent segments (Glennon, 1987). Crash rates on horizontal curves are 1.5 to 4.0 times greater than on tangents on rural two-lane highways (Zegeer et al., 1992). Data from the 2005 Kentucky Traffic Collision Facts Report show that the percent of fatality to crashes on curves was 1.37% and 0.5% on tangents (KTC, 2005). This indicates that in Kentucky, the percent of fatality to crashes on curves is approximate 2.7 times higher than on tangents.

Examination of fatal crash by highway type showed that rural two-lane highways had the highest rates. Data from Kentucky traffic crash analysis (Green et al., 2006) showed that the fatal rate in recent five years (2001-2005) on rural undivided four-lane highways was the second highest rate (1.6/100MVM[1]). Moreover, if comparing the crash rates by the number of lanes, it could be found that the crash rate on rural undivided four-lane highways was the highest among the crash rates on rural two-way highways, which was slightly greater than on rural two-lane highways (Figure 1-1). Other studies have observed that four-lane highways have higher crash rates than two-lane highways as well (Zegeer et al., 1992; Ikeda and Mori, 2005).

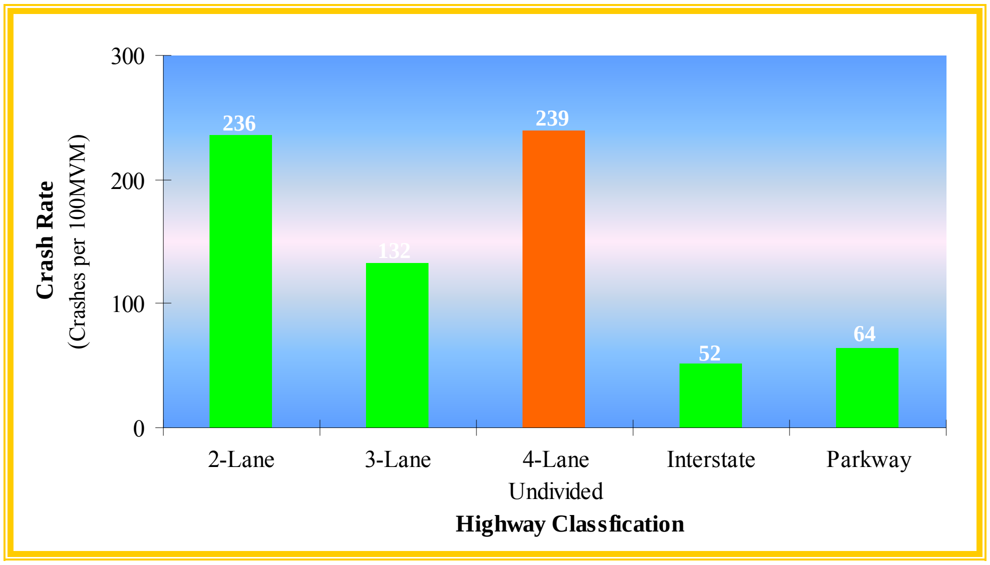

Figure 1-1: Average Crash Rates on Rural Highways in Kentucky (2001-2005)

Numerous studies have been completed on rural two-lane highways for predicting operating speeds and evaluating design consistency. However, few studies have considered these issues for rural four-lane non-freeway highways. Therefore, prediction models for rural four-lane non-freeway highways are needed.

1.2 Study Objective

This dissertation focuses on operating speed prediction for horizontal curves on rural four-lane non-freeway highways. The operating speed refers to the 85th percentile vehicle speed observed under free-flow conditions. This is a term that has been defined as

[1] 100MVM is defined as 100 million vehicle-miles.

the standard by the transportation profession for a longtime and thus it was considered appropriate for use here as well. The vehicle speed stands for the speed of passenger-car class vehicles, which include passenger cars of all sizes, sport/utility vehicles, minivans, vans, and pick-up trucks. These passenger-car class vehicles are in agreement with the classification of design vehicles defined in the book "A Policy on Geometric Design of Highways and Streets 2001" (AASHTO, 2001). The purpose of conformity to the manual is for facilitating designers to apply the results of this study in highway design. The rural four-lane non-freeway highways used here do not include interstate four-lane highways or parkways. Operating speeds only on horizontal curves are considered in this study.

The objective of this study is to develop models to predict operating speeds for horizontal curves on rural four-lane non-freeway highways. The primary steps to achieve this are the following:

1) Study driver's speed characteristics in horizontal curves on rural four-lane non-freeway highways. Since the roadways selected in this study are rural four-lane non-freeway highways, speed characteristics are much more complicated than those on rural two-lane highways. Driver's speed choices on inside and outside lanes in each direction of travel are studied. Speed characteristics at the beginning, middle, and ending points of a curve in each lane are studied. Speed characteristics in each direction of travel are compared as well.
2) Identify the potential factors affecting driver's speed choice in horizontal curves on rural four-lane non-freeway highways. These factors include: geometric features, pavement type, and traffic volumes.
3) Develop models for predicting operating speeds in horizontal curves on rural four-lane non-freeway highways. The models are developed based on roadway pavement, geometrics and traffic volumes using only a portion of the data.
4) Validate the models developed for predicting operating speeds in horizontal curves on rural four-lane non-freeway highways using the remaining data.
5) Provide recommendations for highway design.

1.3 Study Contributions

This dissertation advances the state of art in modeling operating speed for horizontal curves on rural four-lane non-freeway highways. The contributions to highway design and further research are the following:

1) Provide understanding of speed characteristics on rural four-lane non-freeway highway horizontal curves.

Up to date, little has known about the operating speed characteristics at horizontal curves on rural four-lane non-freeway highways. Previous research has focused on free-flow speed on basic segments, such as the procedure used in the "Highway Capacity Manual" (HCM, 2000). This study presents the operating speed characteristics on each lane in each direction of travel. The results could be used for highway design, traffic control, and further research.

2) Reveal the relationships between operating speed and geometric features, pavement, and traffic.

Numerous studies have conducted on rural two-lane highways to establish the relationships between operating speed and geometric features, pavement, and traffic. The results have been broadly used in highway design, rehabilitation, reconstruction, management of such facilities. The results from this study could also be useful in similar applications. For instance, when designers are determining the right shoulder width of a section, they may apply the results of this study to estimate the general impact of right shoulder width on operating speeds.

3) Provide prototype operating speed prediction models for examining design consistency.

The operating speed-based methodology for examining highway design consistency has been widely used in Europe, Canada, and Australia, and has also been proposed for use in the United States. The prerequisite of this methodology is that operating speed prediction models exist. Models developed in this study could be applied in highway design process or highway geometrics evaluation process for examining design consistency.

4) Provide prototype operating speed prediction models for evaluating highway safety.

Previous models for identifying hazardous sites on highways other than four-lane highways have related crash rates to operating speeds or operating speed differentials. These models employ the operating speed prediction models to estimate operating speed, and then input it into the models to estimate crash rates. The operating speed prediction models presented in this study could also be applied for identifying potentially hazardous sites on rural four-lane highways. Moreover, these models could be used for further crash prediction modeling.

5) Provide directions for highway design.

During the highway design process, models developed for operating speed prediction for horizontal curves on rural four-lane non-freeway highways could be used for estimating operating speeds and evaluating highway geometric features. The proposed procedure could be used for improving consistency in highways.

6) Decrease potential crash rates.

The ultimate purpose of the study is to improve highway design so as to reduce potential crashes. The models developed in this study could be used to evaluate design consistency and to identify potential hazardous sites, achieving improvements on highway safety.

1.4 Dissertation Organization

This dissertation is organized into ten chapters, including introduction, literature review, data collection, speed characteristics, relationships related to operating speed, design speed, and posted speed limit, design speed and geometric elements, operating speed and geometric elements, model development, validation of the models, and findings and recommendations.

The introduction chapter states the problem and describes the dissertation scope, dissertation objectives, and dissertation contributions. The literature review chapter reviews previous research conducted in this field. The data collection chapter depicts the site selection process, geometric data collection method, and the operating speed data collection. The speed characteristics chapter analyzes the speed characteristics on horizontal curves based on the valid speed data. The next chapter, relationships related to operating speed, design speed, and posted speed limit, discusses the relationships among operating speed, design speed, and posted speed limit. The design speed and geometric elements chapter examines the trends between design speed and the geometric elements. The operating speed and geometric features chapter analyzes the impacts of geometric features on operating speeds. The operating speed model development chapter develops two operating speed prediction models. Next, the chapter validation of the operating speed models validates the models developed in this dissertation. The final chapter, findings and recommendations, summarizes the study efforts and findings, and presents recommendations for highway design. Further research is also discussed in this chapter.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter summarizes the major results of the literature review. The literature review focuses on the following aspects: horizontal curve and crashes; speed and safety; factors influencing speed; design speed selection; driver expectancy, workload, and error; operating speed characteristics on rural highways; design speed, operating speed, and speed limit; operating speed predicting models developed; design consistency evaluation; data collection technology, and a summary of the literature review and its implications on this study.

This literature review provides a valuable insight on the research conducted to date in regard to these aspects. The basic purpose of this comprehensive review is to understand the previous research undertaken in this field and to take advantage of the relative information to accomplish this study. Articles in journals and publications from state departments of transportation, Federal Highway Administration, Transportation Research Board, American Association of State Highways and Transportation Officials, and other research institutes are searched. Databases such as the Transportation Research Information Services (TRIS), American Society of Civil Engineers (ASCE), and Engineering Village 2 are utilized.

2.2 Horizontal Curve and Crashes

This section reviews the relationships between horizontal curves and crashes. The purpose of this review is to understand the impact of horizontal curves on crashes, and to document the contribution of horizontal curve to crashes.

2.2.1 *Crash Rates on Horizontal Curves*

The road environment has been identified as an important cause of crashes. Past accident studies have indicated that horizontal curves experienced a higher crash rate than tangents.

Raff (1953) studied the crash rates in fifteen states. The data covered one year's crash experience on about 5,000 miles of highway. The crash rates were compared by crash location (curve and tangent). It was showed that, for two-lane, three-lane, and four-lane divided and undivided highways, generally the crash rates on curves were higher than the rates on tangents. When degree of curvature increased, the differences of crash rates between curves and tangents became greater.

Smith et al. (1981) analyzed the crash database named "Skid Reduction", which consisted of crash data for two one-year periods in fifteen states. It was reported that, the crash rate was 2.329/MVM on rural two-lane highway curves with a degree of curvature less than 5.5, while 2.199/MVM on tangents. The figures indicated that the crash rate on curves was slightly higher than on tangents.

Glennon et al. (1983) conducted a study to compare the accident experience on 3,304 curves and 253 tangent segments which were located on rural two-lane highways in four states. It was concluded that the average crash rate for horizontal curves was about three times the average crash rate for tangent segments.

Zegeer et al. (1992) summarized the findings in three studies which aimed to establish the relationships between safety and geometric elements. It was found that accident rates on horizontal curves were 1.5 to 4.0 times greater than on tangents on rural two-lane highways. In another study, aiming to identify crash factors on curves as they compared to tangents, they used 3,427 curve/tangent pairs in Washington State (Zegeer et al., 1991). They found that the percentage of severe crashes (fatal and A-type injury) on curves was generally higher than on tangents.

2.2.2 *Impact of Horizontal Curves on Crashes*

Hauer (1999) analyzed the experimental models predicting accidents with curve elements. It was concluded that, the choice of degree of curve strongly affected safety when the deflection angle was large. For any given deflection angle, smaller degree of curve led to a much safer curve. The change in radius length resulted in a proportional change in accidents whenever the radius was small or large.

Ikeda and Mori (2005) performed an analysis of correlation between roadway alignment and traffic accidents on two-lane and four-lane highways using the Integrated Traffic Accident Database and the Road Management Database in Japan. It was observed that accident rate increased as radius of curve decreased.

Data from United Kingdom Department of Highways indicated that reducing the radius of a curve could result in continually increasing accident rate. This increase became significant at curve radii less than 200 meters (UKDOH, 1990).

The effect of the transition section on safety has been a focus point of some studies. Curve transition has two important safety-related functions. It can direct drivers into a safe path and provide a space to accomplish superelevation. Zegeer el al. (1990) conducted a large-scale analytical effort for identifying the horizontal curve features that affect safety and traffic operations. The authors found that the presence of the spiral section reduced total crashes by 2% to 9%, depending on degree of curve and central angle. However, this effect varied unsystematically. Glennon et al. (1985) found that spirals would have been beneficial to safety too.

Unfortunately, conflicting results were obtained in other studies. According to North Carolina police crash reports, approximately 62% of fatal crashes and 49% of nonfatal curve-related crashes occurred at the beginning or end of the curve rather than in the curve center (Zegeer et al., 1990). Based on laboratory experiments, Stewart (1977) concluded that spiral curves would give drivers a false picture of the true curvature than a true circular curve.
To verify the safety benefits of spiral transition on horizontal curves on rural two-lane highways, Council (1998) used 2,108 spiral curve and 6,136 nonspiral curve ends. The study concluded that in level terrain, spiral curves benefited sharper curves (>3 degrees), while in rolling terrain, the benefits were present when the ADT was greater than 3,600.

In contrast, in mountain terrain crash probability increased when the transition segment (i.e. spiral curve) was present.

2.3 Speed and Safety

Safety implications due to high speed exist because speeding reduces the available reaction time and this could result to a crash. Stuster and Coffman (1998) conducted a synthesis of safety research related to speed and speed management. In this synthesis they examined various studies that related crash rates with change in mean speeds, change in speed at impact and change in posted speed limits. This section reviews the impacts of speed on safety, including the impact of speed on crashes and the influence of speed on severity of crashes.

A landmark study used 10,000 crashes to examine and define a relationship between vehicle speed and crash incidence on rural highways (Solomon, 1964). A relationship was identified in the form of a U-shape curve between the deviation from the average travel speed and crash rate per 100 million miles. According to this curve, crash rates were the lowest when the travel speeds are close to the mean speed of the traffic. However, as the deviation of the travel speed from the mean speed increases in excess of 15 mph, the likelihood of being involved in a crash also increases. One other important observation from this curve is that crash rates decrease with an increase in speed, but this fact only holds good as long as the speed of the vehicle is not above 65mph. Later, Cirillo (1968) confirmed Solomon’s research by conducting a similar analysis on 2,000 vehicles involved in daytime crashes on Interstate freeways. This is illustrated in Figure 2-1. The analysis was limited to two or more vehicles traveling in the same direction.

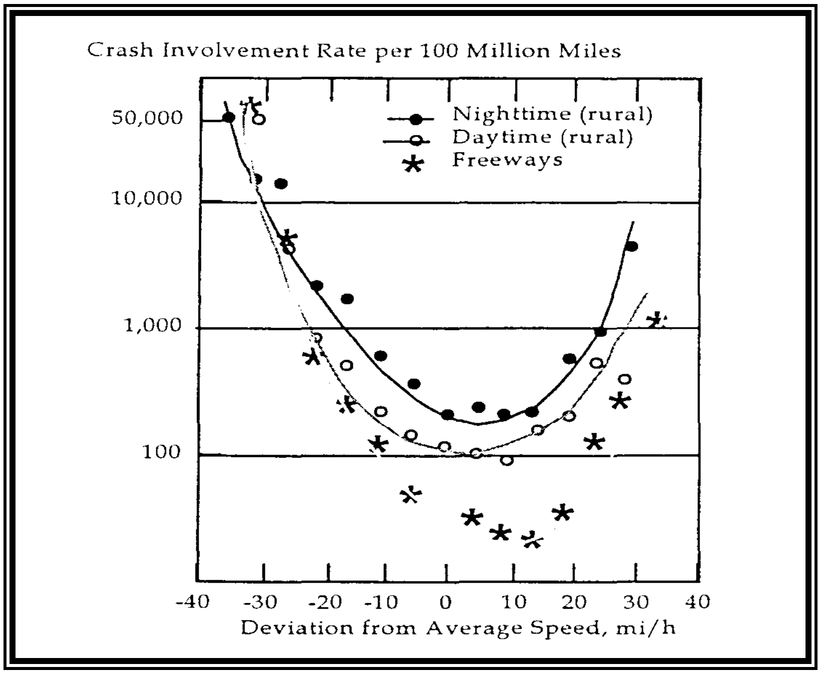

Figure 2-1: Crash Involvement Rate by Deviation from Average Travel Speed (from Solomon, 1964 and Cirillo, 1968)

In defense to earlier studies, researchers emphasized speed variance, rather than absolute speed, as the primary culprit in the incidence of crashes. Speed variation is defined as a vehicle's deviation from the mean speed of free-flowing traffic.

The speed of the vehicle also influences the severity of the crash. An early study showed that the severity of a crash on rural roads increased with an increase in speeds on rural roads (Solomon, 1964). This happened at a faster rate at speeds over 60 mph. The crashes occurring at speeds more than 70 mph mostly resulted in fatal injuries. Another study revealed that chances of injury in a crash depended on the change in impact speeds (Bowie and Waltz, 1994). The study noted that when the change in speed at impact was less than 10 mph, the chances of a moderate or more serious injury to occur were less than 5 percent. This probability increased to 50 percent when the difference in speed at impact exceeded 30 mph. Joksch (1993) noticed that the probability of a car driver being killed in a crash increased with the change in speed to the fourth power.

2.4 Factors Influencing Speed

Several factors could influence vehicle speed. Oppenlander (1966) reviewed 160 items and found that over 50 specific factors influence vehicle speed. Bennett (1994) classified such factors into six based on the study conducted by Wahlgren (1967). The six classifications are: road condition (curvature, gradient, roughness, sight distance, and width); driver; vehicle; traffic conditions; road environment; and other factors. The European Transport Safety Council (1995) listed 32 factors and divided them into three categories: road and vehicle, traffic and environment, and driver related factors. This review concentrates on the following respects: road characteristics, driver characteristics, vehicle, road environment, and traffic control.

2.4.1 *Road Characteristics*

Warren (1982) reported the most significant road characteristics contributed to the operating speeds include curvature, grade, length of grade, number of lane, surface condition, sight distance, lateral clearance, number of intersections, and built-up areas near the roadway. In another study, Tignor and Warren (1990) found that the number of access points and nearly commercial development have the greatest influence on vehicle speeds. However, Fildes et al. (1987, 1989) found road width and number of lanes are the factors having the greatest influence on speed choice.

The review conducted by the European Transport Safety Council (1995), reported that road width, gradient, alignment and layout and their consistency are important determinants of vehicle speed on a particular stretch of road. Their interaction appears to play a more significant role than any individual feature does.

2.4.2 *Driver Characteristics*

Several factors that could affect speed are related to the driver (age, gender, attitude, income, perceived risks, and so forth). As was observed by Solomon (1964), the mean speeds of young drivers, out of state vehicles, buses and latest model passenger vehicles were higher. A similar study conducted by Fildes et al. (1991) found that younger drivers, drivers without passengers, drivers of new cars, drivers traveling for business purposes and high mileage drivers were more likely to drive faster than average speeds and exceed the speed limit.

Mustyn and Sheppard (1980) found that more than 75% of drivers claimed to have driven at speeds greater than the posted speed limit as the roadway was permitting them to do so. According to the participants of the study, crossing the speed limit by 10 mph was not an unlawful thing to do but they considered driving in excess of 20 mph as a serious offense.

Based on a study of speed selection behavior of Korean drivers, Kang (1998) concluded that male drivers with higher income tended to drive faster and experienced drivers drive at a higher speed than others. Trip distance and frequency of use of a road were also found as important factors for speed selection behavior.

Smiley (1999) found that peripheral vision is a primary cue for speed choice. When peripheral vision is eliminated, drivers utilize only the central field of view to determine speed. If peripheral stimuli are close, drivers feel that they are going faster.

Drivers might realize that their behaviors may influence the driving patterns of others, and then they might adjust their own speeds in accordance with their estimation of the behavior of other drivers (Haglund, 2000). It was found that in most situations experienced divers can take advantage of knowledge of a task to enhance their driving performance (Elslande et al. 1997).

2.4.3 *Vehicle*

Vehicle characteristics such as type, power/weight ratio, maximum speed, and comfort might influence speed (ETSC, 1995). Vehicles are classified into four general types based on weight, dimensions, and operating performance. The physical characteristics of vehicles are the key controls in geometric highway design. For example, trucks and buses generally require more generous designs than do passenger-car vehicles since the boundaries of the turning paths of each vehicle type are different (AASHTO, 2001).

Therefore, on a sharp curve, speeds should be adjusted to balance centrifugal force according to vehicle type. Winfrey (1969) found that, the level of driving comfort provided by a vehicle can be a factor, and noisy vehicles with high vibrations will often travel slower than quiet and smooth vehicles.

2.4.4 *Road Environment*

In addition to the preceding factors, the environmental conditions are always present and can influence speeds. Kanellaidis (1995) investigated factors determining choice of speed on interurban road curves from the driver's standpoint. The 207 drivers investigated were grouped into two groups: violators who exceeded the posted speed limit and nonviolators who did not exceed the posted speed limit. It was reported that violators determined their choice of speed primarily based on road environmental elements while nonviolators based on signing.

Reduced visibility due to light rain caused a 2 km/h drop in free-flow speed on a freeway in Canada, while a 3 km/h drop was observed due to light snow (Ibrahim and Hall, 1994). Reduced visibility due to fog has been found to cause a 9.65 km/h (6 mph) decline in mean speeds on a freeway in Minnesota (cited in Stuster and Coffman, 1998) and an 8 km/h reduction on I-84 in Southeast Idaho (Liang et al., 1998). Greater speed reductions were observed when weather conditions worsened. Ibrahim and Hall (1994) reported that heavy rain caused a 5 to 10 km/h decline and heavy snow caused a 38 to 50 km/h decline in free-flow speed on a Canadian freeway. Even windy weather plays a vital role in slowing down vehicles. This is exactly what Liang et al. (1998) have found out in a study that showed that drivers reduced their speeds by 0.7 mph for every mile when the wind speed exceeded 25 mph.

Brilon and Ponzlet (1996) studied 15 sites in Germany to assess the effects of different weather conditions. Based on the comparison of speeds in daylight and darkness, it was concluded that darkness reduced driver speed by 5 km/h.

Kyte et al. (2000) compared two cases to assess the effects of snow or ice-covered pavement. In each case, visibility was good and there was no precipitation; wind speed was low as well. It was found that the presence of snow or ice on the pavement caused a drop of 23 km/h and 21 km/h, respectively.

Cooper et al. (1980) found that average vehicle speeds increased by 2 km/h after resurfacing of major roadways in the United Kingdom but no change occurred in some locations where surface unevenness remained the same after resurfacing. Parker (1997) found no change in speeds between "before" and "after" resurfacing on two rural highways.

Lamm et al. (1990) investigated 24 curved sections on rural two-lane highways in New York State under both dry and wet conditions. In both conditions, the visibility was not affected. The statistical analysis indicated that the difference in the operating speeds between the two conditions was not significant.

2.4.5 *Traffic Control*

2.4.5.1 Transitional speed zone

To force drivers to travel at the posted speed limit, the concept of transitional speed zones has been implemented. Hildebrand et al. (2004) reviewed studies that have examined the effectiveness of transitional speed zones. At 13 selected sites, 11 percent of drivers who were in transitional speed zones were within the speed limits and 37 percent were on either side of the transitional zone. The mean speed dropped in the transitional zone but, mean speeds at the start of the lowest speed zone were higher than the speed limit. Another observation that was made is that the speed dispersion in transitional zones did not increase. The transitional zones are able to reduce operating speeds at the onset of the lower speed zone but there was little difference compared to those sites without a transitional zone.

2.4.5.2 Speed enforcement

Enforcement is often required to assure that drivers adhere to posted speed limits. Past research showed that the presence of a police vehicle forced drivers to drive at speeds that are more compliant with posted speed limits (Shinar and Stiebel, 1986; Benekohal et al., 1992; Hauer et al., 1982). Aerial enforcement has been proven to be positive in reducing highway speeds but as observed by Saunders (1979), it showed negative results when it was deployed and removed. Blackburn et al. (1989) found that aerial enforcement was significantly more effective than radar in detecting and apprehending drivers, who exceeded the posted speed limits and used radar detectors and CB radio. Research by Teed and Lund (1991) found the use of laser guns to be more effective than radar guns in identifying speeding drivers. The use of cameras has also been proven to be an effective means of enforcing speeding laws. Rogerson et al. (1994) found that a speed reduction greater than 15km/h occurred within 1 km of a speed camera. Freedman et al. (1993) found drone radar was related to a 1 mph reduction in average vehicle speed but Streff et al. (1995) reported little significance in speed reductions due to the drone radar deployment. Dart and Hunter (1976) evaluated the effects of speed indicator and they found that the speed indicator had no significant effect on operating speeds. On the contrary, Casey and Lund (1990) found that the presence of a speed indicator reduced traffic speeds at the placement sites and for a short distance past the site. Perrillo (1997) observed 2-3 mph reductions in the vicinity of the speed feedback trails in Texas. Public information and education played no significant role in the reduction of speed, speeding, crashes, and crash severity.

2.5 Design Speed Selection

The definition of design speed has seen several changes since Barnett offered the concept in 1936 (Fitzpatrick et al, 2003). Before 1988, the definitions of design speed had been related to operating speed directly. The current concept of design speed has been defined as “a selected speed used to determine the various geometric design features of the roadway” (AASHTO, 2001).

This section summarizes the current design speed selection procedures used by US Departments of Transportation and other countries based on some previous reviews. The procedures proposed in previous studies are also included in this section. This review helps to identify shortcomings of the current design speed selection procedures.

2.5.1 *Current Design Speed Selection Procedures*

To provide safe and economical roads, design speed has been used as the controlling factor in determining the radii of horizontal curves in USA (AASHTO, 2001). According to a survey concentrated on the application of design speed, several other countries also use design speed as the criteria for curve radius selection (Polus et al, 1998). Defects of the current design selection procedures were discussed in some studies.

The Green Book suggests the use of design speed as a guiding factor in the design of any roadway section. Recently, designers are reexamining this view mainly due to lack of consistency in its use. In a recent study, Fitzpatrick and Carlson (2002) examined the selection of design speed values by DOT's and they found that several factors are used by states. These include legal speed limit, legal speed limit plus a value (5 or 10 mph), anticipated operating speed, terrain, accident history and incremental costs in addition to the design guidelines suggested by AASHTO. Other studies (Fitzpatrick et al., 1995, 1996, 1997) also reported that the above factors were taken into consideration for determining the design speeds.

Fitzpatrick et al. (2003) also examined the order in which various factors were prioritized by state DOT's to determine the design speed. For a roadway most DOT's start with functional classification, legal speed limit, legal speed limit plus 5 or 10 mph, traffic volume, and end with anticipated operating speed. It is important to note that the anticipated operating speed is at the bottom of the list and it has not been seriously considered.

In regard to the adoption of design speeds, Krammes (2000) reported that AASHTO's minimum design speeds for arterials on rolling terrain and for collectors on level and rolling terrain underestimated the desired speed of today's drivers. He observed that AASHTO's policy will not guarantee a full compliance between design speed and operating speed if the design speed is less than 62.1 mph. To correct for this discrepancy, Fitzpatrick and Carlson (2002) recommended design speed values for rural two-lane highways, which were different from those recommended by AASHTO. They suggested the use of anticipated operating speed or posted speed plus 10 mph as the design speed.

Polus et al. (1998) observed that the AASHTO design policy controls only the minimum values for design speed and encourages the use of above minimum values. This may currently underestimate the driver's desired speeds. Also, in the classical design speed concept the policies adopted for maximum superelevation rates vary from state to state and from roadway to roadway. These variations might influence driver's speed selection on horizontal curves and may increase the disparity between design and operating speeds. US engineers have a range of design speeds to select among those recommended by AASHTO which are based on functional classification. However, there is a tendency for selecting high speeds, a practice that often disregards driver's desired or operating speeds. Also AASHTO's policy on design speed selection lacks a feedback loop in which the driver speed behavior resulting from the designed alignment can be estimated and compared with the assumed design speed.

After reviewing the standards of international design speeds for roadway geometric design, Polus et al. (1998) concluded that AASHTO should conduct further research on the distribution of driver's desired speeds on rural highways to recommend changes for the suggested minimum design speeds. Research should also be undertaken to fully develop and validate the speed profile procedures for evaluating alignment inconsistencies.

Polus et al. (1998) also reviewed the standards being adopted in several other countries for roadway design. Germans use both design speed and 85th percentile operating speeds in designing rural roadways. They use design speed as a guiding factor to determine the horizontal and vertical features of an alignment and the 85th percentile operating speed to determine the superelevation rates and stopping sight distances. Swiss engineers use speed profile along an alignment to check for alignment consistency. British designers do not follow the concept of functional classification but they emphasize the effects of alignment and layout (cross-section and access control) constraints while selecting their design speed. Australians use 85^{th} percentile speed as the design speed for low-speed alignment (i.e., less than or equal to 52.5 mph) and traditional design speed procedures in designing their high-speed alignments (i.e., greater than or equal to 62.5 mph). Venezuela uses the Feedback Loop Procedure, which the driver speed behavior resulting from the designed alignment can be estimated and compared with the assumed design speed.

2.5.2 *Proposed Design Speed Selection Procedures*

Andueza (2000) proposed a speed selection approach as outlined below:

1) Select a design speed as a function of all factors.
2) Divide a road into analytical sections of at least 3 kilometers long and assign design speeds.
3) Construct a speed profile diagram using the set of prediction models for speeds on tangents and curves.
4) Adjust the elements of the geometric design based on these speed profiles to obtain a layout with a more uniform speed. This way, situations that are considered unsafe can be eliminated.
5) Design each element with a speed derived from the adjusted speed diagram.

Harwood et al. (2000) proposed a general design procedure based on a literature review. The steps of the procedure are:

1) Select a design speed first.
2) Develop a preliminary design based on the selected design speed.
3) Determine the projected operating speed and compare it with the design speed.
4) If the operating speed is higher than the design speed, the designer would select a higher design speed and go back to step 2, modify the geometric design, the traffic control plan, and other characteristics of the facility until consistency. If the

operating speed is less than or equal to design speed no adjustments are needed and the prepared preliminary design in Step 2 can be further developed.

A conceptual framework for improving the AASHTO's concept of design speed was presented by Donnell et al. (2002). At first, the desired operating speed could be determined based on the functional class, topography and land use pattern of the roadway. Then the design speed is calculated from the design and operating speed models. The design speed model uses a speed that is above or equal to the design speed recommended by AASHTO. The operating speed models use a speed that is based on the 85th percentile speed of that section. Using these models, the alignment consistency is checked by establishing ranges of acceptable differences. If they are consistent, the roadway will be constructed based on the recommended speed otherwise the desired operating speed will be recalculated and the process will be repeated until consistency is obtained. Once the roadway is opened for operation, speed limits will be set and operating speeds shall be observed. The collected data shall be used as reference for the determination of future design speeds.

2.6 Driver Expectancy, Workload, and Error

2.6.1 *Driver Expectancy*

Driver expectancy is defined as "driver's readiness to respond to situations, events, and information in predictable and successful ways" (Alexander, 1986). It influences all levels of the driving task that consists of control, guidance, and navigation. There are two forms of driver expectancies: priori and ad hoc expectancies. Priori expectancies are long-term expectancies that drivers bring to driving task based upon their previous experiences. Ad hoc expectancies are short-term expectations that structured during a particular trip on a particular site. Geometric inconsistency may violate a priori and/or ad hoc expectancy, influencing driving performance and driving situation. If drivers fail to recognize the violation that the geometric alignment is not consistent with their expectancies, the likelihood of a crash may increase.

2.6.2 *Driver Workload*

Workload has been defined by Senders (1970) as "a measure of the 'effort' expended by a human operator while performing a task, independently of the performance of the task itself." Highways should be designed to effectively use the workload capabilities of drivers. A successful highway design would make a driver's mental workload level high enough to keep the attention needed in driving performance, but would not exceed the driver's processing capacities. Abrupt increase in driver workload increases the probability of accident. Such increases could be caused by roadway features including (TSA, 1999; Krammes et al., 1993; Cafiso et al., 2003):

- Critical feature
- Limited sight distance
- Dissimilar feature
- Successively inconsistent feature (such as a sharp curve following a long tangent)

Wooldridge (1994) validated the relationship between driver mental workload and crash rates by applying the procedure of Messer et al. (1980, 1981), which is applicable to two or four-lane highways in flat or rolling terrain. A group of 19 rural two-lane highways in Texas were selected for the validation. Driver workloads associated with individual portions of the roadways were calculated using the procedure of Messer et al. (1980, 1981). It was concluded that high workload change rates were strongly associated with high crash rates.

Hulse et al. (1989) proposed a general model to quantify driving workload. In this model, sight distance, curve radius, distance of closest obstruction to road, and road width were taken into consideration as factors influencing driver workload. The model was defined as the equation shown below:

$$Q = 0.4A + 0.3B + 0.2C + 0.1D$$

Where:

Q = driving workload;

$A = 20 \times \log_2^{(500/SD)}$, if SD>500, then A=0; if SD<15.6, then A=100;

SD = sight distance (m)

$b = {(100 \times R_{\max})}/{R}$;

R = radius of curvature;

$R_{\max}$ = maximum value of the radius of curvature;

$C = -400 S_O + 100$, if S_O > 2.5, then C=0;

S_O = distance of closest obstruction to road (m);

$D = -36.5W + 267$, if W > 7.3, then D=0; if w<4.57, then D=100; and

W = road width for 2 lanes (m);

Green et al. (1994) examined the relationship between road geometry, workload ratings, and predictions from Hulse's driver mental workload model (Hulse et al., 1989). They found that the standard deviation of lateral position is negatively correlated with workload ratings. Sight distance leads to increase driver workload on rural two-lane highway horizontal curves.

Shafer (1996) employed the vision occlusion test method to quantify driver's mental workload on horizontal curves because of its sensitivity. A linear relationship between driver mental workload and the degree of curvature and deflection angle of horizontal curves was developed as:

$$WL = 0.193 + 0.016 \times DC$$

Where:

WL = workload; and

DC = degree of curvature(degree)

The author concluded that, the driver's mental workload increases as the degree of curvature increases, and the deflection angles seem to not significantly affect driver's mental workload.

2.6.3 *Driver Error*

Past research revealed that more than 90% of all crash causes are directly or indirectly due to driver error (Lamm et al., 2005). It has been known that the probability of driver error is high under mental overload condition as well as mental underload condition. The typical factors contributing to driver error are (Lamm et al., 1999, 2001, 2005; TAC, 1999; SANRAL, 2003; AASHTO, 2004; Cohen, 1994):

- Excessive task demands
- Unusual maneuvers
- Poor sight distance
- Expectancy violation
- Too high processing demand
- Too little processing demand
- Deficient, ambiguous, confusing, or missing information

2.7 Characteristics of Operating Speed on Horizontal Curves

Steyer (1998) examined the lateral placement of a vehicle passing a curve by video technique. Eight horizontal curves with high crash frequency were selected in Saxony State, Germany. The results indicated that drivers tend to decelerate mainly inside the horizontal curve where centripetal forces act.

However, the opposite finding was obtained in another study. Figueroa and Tarko (2005) developed two models to predict free-flow speeds on transition sections based on the collected free-flow speeds on rural two-lane horizontal curves in Indiana. The two models indicated that 65.5 percent of the deceleration transition and 71.6 percent of the acceleration transition occurred on the preceding and the following tangent segments to a curve. It means that drivers tend to decelerate or accelerate mainly on the tangent segments.

Ottesen and Krammes (2000) compared the 85th percentile speeds in inside and outside lanes on rural two-lane highways. It was found that the differences of the 85th percentile speeds in inside and outside lanes are not significant.

2.8 Design Speed, Operating Speed, and Speed limit

2.8.1 *Design Speed and Operating Speed*

Polus et al. (1998) conducted a survey where discrepancies between design speed and actual operating speed were observed. The study found that in general, the operating speeds were lower than the design speeds on high-speed roadways. However, the operating speeds were higher than the design speeds on low-speed roadways. A similar conclusion was drawn in another study where it was shown that the 85th percentile driver exceeded the design speeds on both horizontal as well as vertical curves (Fitzpatrick et al., 1995). This means that at these sections the operating speed is greater than the design speed.

Stamatiadis and Gong (2006) examined the relationships on highways in Kentucky[2]. It was concluded that the relationship between operating and design speeds varied according to the highway type considered. For two-lane highways, these two speeds were different and, in general, the operating speed was higher than the design speed. The average difference between operating speed and design speed reached 2.76 mph (operating speed minus design speed). The same trend was also noted for roads where the design speed was lower than the speed limit. However, the average difference between operating speed and design speed was significantly larger, 7.88 mph. For roads where the design speed was greater than the speed limit, the speeds were different but the design speed was greater than the speed limit. The average difference between operating speed and design speed was 8.72 mph. For four-lane highways, however there was no difference observed. It should be noted that the sample size of the four-lane sections is small.

In some studies, the relationship between design speed and operating speed has been examined by analyzing the relationships between design elements and operating speed. A more recent study reported that design elements such as radius, degree and length of

[2] The relationships were checked statistically.

curve, lane width, access density, hazard rating and grade had a relationship with operating speed (Fitzpatrick et al., 2003). The study also concluded that most of these design elements demonstrated minimal impact on the operating speed unless a tight horizontal or vertical curve exists indicating that the design speed has some relationship to operating speed.

Using the horizontal components of roadway, Ottesen and Krammes (2000) found a relationship between design and operating speeds. Their study revealed that tangent speeds on level roadways were higher than on rolling terrain. Also degree of curvature, length of curvature and deflection angle (degree of curvature times the length of curvature) have significant effect on curve speed. On the other hand, sight distance, approach tangent length, preceding degree of curvature, superelevation rate, lane width and pavement width were not statistically significant predictors.

2.8.2 *Speed Limit and Operating Speed*

Chowdhury and Warren (1991) collected speeds in 28 curves on two-lane highways. They observed that on most curves operating speeds were higher than the posted speeds. They also concluded that the posted advisory speeds did not have significant effect on drivers. However, Schurr et al. (2000) found that only mean speed at the midpoint of horizontal curves was influenced by posted speed limit.

In 1987, several states changed speed limits from 55 mph to 65 mph. In some states differential speed limits were imposed for restricting truck speeds. Garber and Gadiraju (1992) compared the impacts of "before" and "after", and statistically analyzed these impacts. The authors found that, for passenger cars the mean of operating speeds increased with the increase of speed limits. Speed dispersion for cars decreased with this increase.

Based on the data collected in Indiana, Khan and Kumares (2000) reported that the change in speed limits had a significant effect on the 85th percentile speed. In general, the change of speed limit had a greater impact on rural roadways than on urban streets. It was confirmed that 85th percentile speeds were higher than posted speed limits irrespective of roadway functional classification or geographic location.

Fitzpatrick et al. (2003) found that posted speed limit was related to operating speed. Generally, the 85th percentile speeds were higher than posted speed limits, and 50th percentile operating speed was close to the posted speed limit. They also found that 37 to 64 percent of the free flow vehicles were no higher than the posted speed limits on rural roads according to different road classification, while this occurred for only 23 to 52 percent in suburban or urban roadways.

In most studies, it was found that with the increase of speed limit, the 85th percentile speed increased. In some cases, speed dispersion also increased (TRR, 1998). These studies focused on interstate highways, and few studies have examined the effects of changing speed limits on lower-speed, non limited-access highways.

Dixon et al. (1999) studied the posted speed limit and free-flow speed for rural multilane highways in Georgia. They found posted speed limits of 55 and 65 mph directly influence free-flow speeds, and an increase in the posted speed limits resulted in increased operating speeds.

Lu et al. (2003) found that, on most multi-lane non limited-access arterial roadways in urban and suburban areas in Florida, the 85th percentile speeds are 5 to 10 miles higher than the posted speed limits. In urban arterial roads, vehicle operating speeds were rather less sensitive to the posted speed limit than in other types of roads. Lowering speed limit would not necessarily reduce operating speeds.

Stamatiadis and Gong (2006) found that the relationship between operating speed and posted speed limit showed a uniform pattern on rural two-lane and four-lane highways in Kentucky. In general, these two speed metrics were different and the posted speed limit was lower than the 85th operating speed.

2.9 Operating Speed Prediction Models

Operating speed is the speed at which drivers operate their vehicles under free-flow conditions (AASHTO, 2001). The 85th percentile of the distribution of observed speeds is usually used to be the measurement of operating speed (Fitzpatrick, 2003). Driver age, gender, attitude, perceived risks, weather, road and vehicle characteristics, speed zoning, and speed adaptation can influence operating speed. The most significant road characteristics contributed to the operating speeds include curvature, grade, length of grade, number of lane, surface condition, sight distance, lateral clearance, number of intersections, and built-up areas near the roadway (Warren, 1982).

The purpose of reviewing operating speed predicting models is to find out the relationships between operating speed and highway geometric elements. Researchers have developed numerous operating speed prediction models for rural two-lane highways and suburban/urban roadways, while none for rural four-lane highways. In order to compare the geometric components that influence or determine operating speeds on different types of roads, operating speed prediction models for other highways would be reviewed. This will also benefit and assist in the site selection of this study.

Numerous operating speed models have been developed to predict operating speeds for rural two-lane highways and suburban/urban roads in the past decades. These models are based on geometric features and roadway environments. Since the traffic flow on rural highways is uninterrupted while interrupted on non-free flow urban roadways, the existing operating speed models are reviewed separately based on the area where the roads are located.

2.9.1 *Operating Speed Models for Rural Highway Curves*

Operating speed models have been developed for rural two-lane highways in 38 studies since 1950. Table 2-1 presents a summary for the predictor variables used in the existing models along with the sample size in terms of number of sites used and the number of observations per site. More details on these models are presented in Appendix 1 (some sources are from Misaghi and Hassan, 2005).

Table 2-1: Existing Operating Speed Models for Rural Highway Curves

Author	Predictors	Data collection	Sample Size	Max R^2
Taragin (1954)	R	n/a	68(125)	0.86
Mclean (1978)	R, CCR	n/a	n/a	0.87
Mclean (1979)	R, V_T	n/a	120(n/a)	0.92
Kerman et al. (1982)	R, V_a	n/a	n/a	0.91
Glennon et al. (1983)	DC	n/a	56(n/a)	0.84
Guidelines of German (1984)	CCR, LW	n/a	n/a	0.79
Glennon et al. (1985)	R	n/a	n/a	0.84
Setra (1986)	CCR, LW	n/a	n/a	0.85
Lamm and Choueiri (1987)	CCR, R, LW, SW	stop watch	261(n/a)	0.84
Kanellaidis et al. (1990)	R, V_d	n/a	58(200)	0.93
lamm et al. (1990)	DC	radar gun	261(120~140)	0.79
Lamm (1993)	CCR	n/a	n/a	0.73
Islam and Seneviratne (1994)	DC	radar gun	8(125)	0.98
Krammes et al. (1994)	DC, DF, L_C	following vehicle	138(50)	0.92
Morrall and Talarico (1994)	DC	radar gun	9(n/a)	0.99
Ottesen and Krammes (1994)	CCR, R	n/a	n/a	0.80
Al-Masaeid et al. (1995)	DC, P_{con}, G, R_1, R_2, L_T, DF_1, DF_2	40m speed trap	93(n/a)	0.81
Choueiri et al. (1995)	CCR	n/a	n/a	0.81
Krammes et al. (1995)	DC, L_C, DF, L_T, V_T	radar gun	284(50~100)	0.90
Lamm et al. (1995)	CCR	n/a	n/a	0.81
Voigt (1996)	R	n/a	n/a	0.84

Table 2-1 (continued)

Author	**Predictors**	**Data collection**	**Sample Size**	**Max R^2**
McFadden and Elefteriadou (1997)	DC, L_C, V_T, DF	n/a	284(50~100)	0.98
Abdelwahab et al. (1998)	DC, DF	stop watch	46(35)	0.92
Andjus and Maletin (1998)	R	radar gun	9(70~80)	0.81
Cardoso et al. (1998)	V_T, R	n/a	50(n/a)	0.92
Passetti and Fambro (1999)	R	counter/classifier	51(100)	0.68
Andueza (2000)	R, R_a, DC, L_T	radar gun	39(30~64)	0.85
Fitzpatrick et al. (2000)	R, K, G	radar&lidar gun	176(100)	0.92
McFadden and Elefteriadou (2000)	V_{85T}, L_T, R	lidar gun	21(75)	0.71
Ottesen and Krammes (2000)	DC, L_C, DF, L_T, V_T	radar gun	216(50)	0.81
Donnell et al. (2001)	R, G_1, G_2, L_{T1}, L_{T2}	lidar gun	17(100)	0.61
Gibreel et al. (2001)	R, L_V, G_1, G_2, A, L_0, e, K, DF	radar gun	38(1h)	0.98
Jessen et al. (2001)	V_P, G_1, ADT	counter/classifier	70(275)	0.61
Liapis et al. (2001)	DC, E	magnetic counter	20(n/a)	0.75
Schurr et al. (2002)	DF, L_C, G, ADT, SL	detector	70(n/a)	0.46
Medina and Tarko (2004)	DC, SD, RES, e	laser gun	158(100)	n/a
Misaghi and Hassen (2005)	R, V_T, DFC, SW, Curve − dir, G, Drv_flag	counter/classifier	20 (24h)	0.89
Stamatiadis and Gong (2006)	R, DS, L_C, DL RSW	Radar gun	103(25~158)	0.54

Notes: Sample size is number of sites and number of observations per site respectively.
n/a= information was not provided.
A description of the predictors is in Appendix 2.

Of the models developed, most were based on speed prediction for passenger-car vehicles while few were for heavy or light trucks. Most of the studies used the 85th percentile speed to represent the operating speed. Linear and non-linear regression models were mainly developed. The methodologies used for developing models are primarily statistic methods. Among the statistic models used: simple linear regression, multiple linear regression, ordinary least squares (OLS) model for panel data, and two-step were included.

Most of the existing models are 2-dimensional models, which only considered horizontal curve and vertical curve. According to a study intended to develop 3D (cross section, horizontal curve and vertical curve) models for predicting operating speed, the maximum differences between the observed and predicted speed using 3D model and 2D model on some sites reached 35% (Gibreel et al, 2001). The 3D models have significant higher values of coefficient of determination due to cross section considerations in the model.

Most of the studies have developed regression models based on the data collected, but without any validation. Also almost all studies provided the measurement of fit of the models, like R^2 and R^2_{adj}, but did not assess the quality of their prediction.

Early studies used mostly curve radius (radius or degree of curvature) as the predictor but later studies used more predictors. The predictors mainly consisted of geometric features, while in some models, traffic and pavement information was also used. The variables that have been identified as significantly relating to operating speed include: radius of the curve, length of the curve, length of the preceding and successive tangents, grades, superelevation, average daily traffic volume, pavement condition, approach speed, and speed limit. The most frequently used predictors are: radius (R), length of curve (LC), length of tangent (LT), grade (G), surperelvation (e), and lane width (LW). In the previous 38 studies, 35 studies used radius as an explanatory variable for operating speed prediction while 6 studies used length of curve. Length of tangent and grade were also used as explanatory variables in 6 studies.

The data collection devices used to record vehicle speed mainly include: radar gun, lidar gun, following vehicle, stop watch, detector. In most cases, manually operated radar guns were used. The utilization of radar gun is usually accompanied by human error and cosine error, since at least two persons are needed in operation and reading angle. In some cases, the presence of the speed collectors might influence drivers' behavior.

In some studies, the number of observations per site is less than 100, and there are some sites with only 25 vehicle speeds. In most studies, few considered the quality of the samples with less than 100 vehicles observed. The accuracy of these models might be questionable.

Previous studies did not consider the effects of the characteristics of drivers and vehicles on operating speed. Due to difficulties in collecting driver information and vehicle characteristics in field, most studies potentially assumed that driver and vehicle characteristics do not influence operating speeds.

2.9.2 *Operating Speed Models for Suburban and Urban Roadways*

Several operating speed models have been developed for the roadways in suburban and urban area. Appendix 3 lists the existing models based on the summary made by Wang (2006). These models focus on the operating speed prediction for passenger-car vehicles. Some of the models were developed for horizontal curves. In such models, the primary predictors include: radius, grade, lane width, approach density, posted speed limit, and some environment indices. In addition, compared to the parameters used in the models for rural highways, some surprising predictors such as number of lanes were used in some of the models for urban roadways.

2.10 Design Consistency Evaluation

This section reviews the methodologies used to evaluate geometric design consistency as well as the evaluations of these methodologies. Several operating speed prediction models have been developed to evaluate design consistency in the past years. In this section, these models are not discussed because this review focuses on the general methodologies used to evaluate geometric design consistency.

So far there is no official definition of design consistency. The recommended definition for design consistency in the US is "the conformance of a highway's geometric and operational features with driver expectancy" (Wooldridge, 2003). Three studies in Canada (Al-Masaeid, 1995, Nicholson, 1998, Gibreel et al, 1999) offered definitions close to the definition recommended for the US.

Lunenfield and Alexander (1984) explained the main factors resulting in design inconsistency. The three factors are:

- Considerations such as cost and environmental impact often take precedence in the applications of design
- Design standards have changed progressively so that highway geometric features and alignment are inconsistent
- Continuous sections of a highway were constructed at different times.

2.10.1 *Methodologies for Evaluation of Design Consistency*

Fitzpatrick et al. (2000) reviewed the design speed–based method for evaluating design consistency in a study. The design speed-based method has been the most common approach in the United States to ensure geometric design consistency. The key of the method is the selection of the design speed. The premise of the method is that a design speed is used on the entire alignment of a roadway. Design speeds selected for individual curves in the alignment should be equal to or higher than the design speed selected for the entire roadway. US DOTs have been using diverse procedures to select design speed. The selection of design speed has been previously reviewed in the preceding section.

The operating speed-based method has been used widely in Europe, Australia, and Canada (Fitzpatick et al., 2000). The primary ways in which operating speeds are used to ensure geometric design consistency are:

- Use of speed profiles. A speed profile is a plot of operating speeds versus distance along the alignment of a roadway. Speed profile models are used to estimate operating speeds along the alignment. Design consistencies are identified in light of the differentials in operating speed between successive alignment features (Fitzpatick et al., 2000).
- Use of the differential in design speed and operating speed. The way is to compare the design speed selected for the segment and the operating speed measured in field (Krammes et al., 1995; Lamm et al., 1986).

Switzerland is one of the first countries to use speed-profile models for identifying geometric design consistency. Two conditions that any speed profile does not meet are considered inconsistent (Krammes et al., 1995; Lindeman et al., 1978):

- The maximum speed differential between a curve and the preceding tangent or curve with a large radius is greater than 5km/h
- The maximum speed differential in successive curves is greater than 10km/h

Germany uses the speed differential in design speed and operating speed to identify geometric consistency. It was determined that, the 85th percentile speed should not exceed the design speed on any given segment by more than 20km/h, and the maximum difference in the 85th percentile speed between successive segments should not exceed 10km/h (Lamm et al., 1986).

In USA, three principal methods for evaluating design consistency have been developed by Leisch and Leisch (1977), Lamm et al. (1988), and Krammes et al. (1995). Leisch and Leisch (1977) proposed the “15-km/h rule” as the criteria to detect design consistency. The rule states that, passenger car speeds should not vary more than 15km/h along an alignment and truck speeds should not be more than 15km/h lower than average passenger car speeds. Lamm et al. (1988) and Krammes et al. (1995) developed speed profile models to detect design consistency. However, speed profile models are not widely used in USA because these models are not incorporated into design policy (McLean, 1988; Leisch and Leisch, 1977; Lam et al., 1988).

Alignment indices are quantitative measures of the general character of an alignment of a roadway. This method assumes that geometric inconsistency will result when the general character of an alignment changes significantly (Hassan et al., 2001). This method has several advantages, such as simplification of application and quantitative comparison (Anderson et al., 1999).

The alignment indices that have been analyzed in previous research include (Fitzpatrick et al., 2000; Castro et al., 2005): curvature change rate (CCR), degree of curvature (DC), the ratio of curve length to roadway length (CL: RL), average radius (Avg R), average tangent (Avg T), maximum radius / minimum radius (MR/mR), vertical curvature change rate (VCCR), average rate of vertical curvature (V Avg K) average gradient (V Avg G), and composite alignment index (CCR Combo).

Numerous models were developed using the alignment indices. For instance, Lamm and Choueiri (1987), Morrall and Talarico (1994), McDadden and Eleferiadou (1997), Faghri and Harbeson (1999), and Ottesen and Krames (2000) have used CCR and DC to detect geometric design consistency.

After studying 260 curves in New York State, Lamm et al. (1988) advocated a possible design procedure to promote design consistency in light of European experiences. The procedure evaluates the change of degree of curve and change of the 85^{th} percentile speeds on two consecutive elements to identify design consistency for rural two-lane highways.

Messer (1980) presented a methodology for evaluating the geometric design consistency of rural non-freeways. The methodology is based on driver mental work-load and empirical evidence. Driver mental work-load may be calculated based on geometric features, operating speeds, and driver behavior parameters. The level of consistency of design was categorized in light of the work-load value. The work-load value is the criteria to judge design consistency.

Polus and Dagan (1987) developed some spectral models for evaluating design consistency based on time series spectral analysis of the highway alignment. Spectral analysis is often used to describe cyclical physical phenomena. In evaluating design consistency, it was used to determine whether an alignment consists of a repeating pattern, i.e. indication of consistency. This model was validated using 23 theoretical roads. It was found that the spectral model is valid for quantifying consistency in highway design.

2.10.2 *Evaluation of Some of the Methodologies*

Among the methodologies established for detecting design consistency, the speed-based methodologies and alignment index methodology are the most popular. Leisch and Leisch (1977) concluded that the design speed-based method does not guarantee to produce design consistency. This was confirmed in a later study conducted by Krammes and Glascock (1992). Other evaluations mostly focused on the operating speed-based methodology and alignment index methodology.

The primary measures used in operating speed-based methodology are the differences between design speed and operating speed and the differences between operating speeds on successive sections. The two measures utilize operating speed prediction models. Therefore, selecting operating speed prediction models might affect the precision of the evaluation. Richl and Sayed (2005) evaluated 12 operating speed prediction models using the data collected on two alignments in the mountainous terrain of British Columbia. It was found that the selection of the speed prediction model had a significant effect on evaluating design consistency.

Hirshe (1987) posed the hypothesis that the use of the 85th percentile speed for evaluating design consistency tends to underestimate speed reduction. McFadden and Elefteriadou (2000) validated his hypothesis using the data collected on 21 sites from two states. Three comparisons were conducted in the study. The three comparisons were: 1) speed reduction between the speeds measured on the middle of tangent and on the middle of curve; 2) speed reductions between the speeds measured on 9 locations along approach tangent and horizontal curve; 3) the maximum speed reduction among the speed reductions based on the speeds measured on the 9 locations. Based on the statistical analysis, it was concluded that the use of 85th percentile speeds might not be the most practical statistic for evaluating design consistency.

Since it has been argued that the simple subtraction of the 85^{th} percentile speeds on two successive elements would not give reasonable results (Hirsh, 1987; McFadden and Elefteriadou, 2000), an alternative measure, named "the 85^{th} speed differential", was proposed. This is calculated as the 85^{th} percentile value of speed differences for each vehicle. The two measures are definitely different. The first measure is the difference between the two 85^{th} percentile speeds on two successive elements, while the second measure is the 85^{th} percentile speed calculated from the speed differences of all vehicles on successive elements. Hassan et al. (2005) validated the two measures by relating operating speed consistency to safety. It was found that the two measures provided good results in terms of safety-explicit consistency evaluation criteria. Their sensitivity analysis of the two models indicated that the use of the 85^{th} speed differential would yield more reasonable results for a wider range of speed reductions.

Lamm et al. (1986) compared three methods for evaluating design consistency employed in United States (Leisch and Leisch method), Switzerland, and Germany. It was found that the three methods produced the same basic results, but the German CCR method is the most convenient for application.

Castro et al. (2005) validated the previous 10 indices based on a case study. They found that the curvature change rate (CCR), vertical curvature change rate (VCCR), and composite alignment index (CCR Combo) had highest correlation with accident rates. They suggested that horizontal and vertical alignment characteristics should be taken into consideration when evaluating alignment consistency.

2.11 Data Collection Technology

This section reviews the data collection methodologies, devices usually used for data collection, and error measurement. This review is helpful for selecting devices and methodologies employed in the operating speed data collection.

Hanscom (1987) validated six non automated speed data collection methodologies using both radar and manual timing methods. The six vehicle-selection strategies are: subjective, systematic, computer assisted random and platoon-weighted procedures. It was found that all results based on the strategies were statistically equivalent to real traffic speeds. The randomized (designated vehicle) strategy was found to be the best. It was also concluded that spot speed sampling accuracy for 85^{th} percentile speed required a minimum of 100 vehicles to meet the accuracy of 1.0 mph.

Gates et al. (2004) compared five common portable speed measurement systems: pneumatic tubes, piezoelectric sensors, tape switch sensors, radar guns, and lidar guns manufactured by Kustom Signals Inc. They performed an experiment using 50 vehicles each at speeds 35 mph and 55 mph. They found that all devices performed equally well when vehicles traveling at 35mph. When vehicles were traveling at 55 mph, lidar and radar guns were the most accurate and precise devices. Since the deviations from the true speed for an individual measurement were almost always relatively less than $\pm$1.5 mph, the authors recommended that portable speed measurement equipment could be selected to meet the characteristics of a given data collection situation.

Speed measurement devices were also tested by Antonucci et al. (1996). Each device set up on urban streets was evaluated over speeds ranging from 10 to 55 mph measuring a sample of 100 speeds. The Lateral Acceleration Sensor System developed by FHWA was used as reference. This referred system provides speed measurements accurate to 0.1 mph. The authors found that radar and lidar were the most accurate devices at speeds greater than 46 mph. The accuracy increased as speed increased. It was also concluded that the devices without equipment on the roadway surface had less effect on driver behavior than those devices that have equipment on the roadway.

Fisher (1980) pointed out the shortcomings of radar speed measurement. For stationary mode, there are two potential errors: one is the cosine effect which results to underestimate target's true speed. The other error is the lack of test for oscillator frequency. For moving mode, there also are two potential errors: one comes from the deflection of the target to the center of the radar beam, while the second error occurs when assessing large objects.

In the "Basic Training Program Radar Speed Measurement Trainee Instructional Manual" (NHTSA, 1982), the stationary angular effect (cosine effect) was detailed. When a target vehicle's traveling direction creates an angle with the antenna of the stationary Radar, the speed of the vehicle would be underestimated. The difference between the measured and true speeds depends on the angle; the larger the angle, the higher the difference. If the angle remains small, the cosine effect is not significant. If the angle reaches 10 degrees while the vehicle is traveling at 60mph, the difference is less than 1 mph, since the radars only display speeds in whole numbers, the speed actually displayed is rounded.

2.12 Summary of the Literature Review

Although previous studies have been conducted for other types of roadways, the methodologies used and the conclusions reached would benefit this study. Some of the findings provide an understanding to driver's behavior on roadway curves, which benefits the site selection of this study. The methodologies can be considered as references to select the more appropriate methodology for this study.

Previous crash studies have found that horizontal curves are highly related to crashes. A few studies indicate that horizontal curves experience a higher accident rate than tangents. Generally, the crash rate on horizontal curves increases as the degree of curve increases. The benefits to safety for using a transition section have been conflicting. In some studies, it was concluded that the transition section was beneficial. However, other studies found that the presence of the transition section would give drivers a false picture of the true curvature and can potentially lead to a crash.

Crash rates and severity have been found highly related with speed. The U-shape curve representing the relationship between speed and crashes has been identified and verified. According to this curve, crash rates were the lowest when the travel speeds are close to the mean speed of the traffic. As the deviation increases in excess of 15 mph, the likelihood of being involved in a crash also increases. When the speeds are less than 65mph, crash rates decrease as the speeds increase. Generally, the severity of a crash on rural roads increased with an increase in speeds.

A few factors have effects on vehicle speed. Numerous studies have found that road alignment and land use affect vehicle speed. It has been showed that speed could be affected by driver characteristics as well, such as age, gender, attitude, income, perceived risks, and so forth. Road environment is another influential factor to speed. Reduced visibility due to light rain, light snow, and fog could cause a 2 to 8 km/h drop in speed. Greater speed reductions were observed when weather conditions worsened, like heavy rain or snow. Ice or snow-covered pavement could cause approximately a 20 km/h reduction in speed. Generally, traffic control technologies are effective in influencing speeds. However, some of the methodologies played no significant role in the reduction of speed, such as public information and education. Of the factors influencing operating speeds, radius is the most important factor.

The controlling factor used in many countries in determining the radii of horizontal curves is design speed. AASHTO uses functional classification and road condition for determining the design speed values. State DOTs use the AASHTO Green Book procedure, legal speed limit, legal speed limit plus a value (5 or 10 mph), and anticipated volume, anticipated operating speed, development, costs and consistency to determine the values. Other countries use diverse procedures, some of which taking operating speed into consideration.

The current AASHTO procedure has been found unable to guarantee design consistency. This design policy controls only the minimum values for design speed and encourages the use of above minimum values, which leads to underestimate driver's desired speeds.

In practice, there is a tendency for selecting high speeds. To remedy potential problems, some conceptual procedures have been proposed, such as considering driver's desired speed in the design speed selection procedure.

Driver expectancy is formed by driving experience. It has a significant influence on driving task. Inconsistent geometry may violate driver expectancy, influencing driving performance. Driver workload has been found associated with some of roadway features. A successful highway design would make driver mental workload level high enough to keep the attention, but would not exceed driver's processing capacities. Too high workload or too low workload causes driver error. Research has revealed that more than 90% of all accident-causes were directly or indirectly due to driver error.

In general, design speeds are higher than operating speeds on high-speed roadways, while lower than operating speeds on low-speed roadways. In most studies, it was found that the 85th percentile operating speed increased with the increase of speed limit. Previous studies have revealed that operating speeds were generally higher than speed limits.

In the past 50 years, a lot of models have been developed for operating speed prediction, most of which are for rural two-lane highways. Most of the models were developed using statistic methodologies. The characteristics of the traffic flows on rural highways and suburban/ urban roadways are not the same. The predictors used in the models for rural and suburban/urban areas have apparent differences, although some are used for both areas. Most of the studies used manually operated radar guns to collect speed data. Sample size is questionable since the sample size in some of the studies is small.

Methodologies for detecting design consistency have been proposed. Among the methodologies, speed-based methodologies and alignment index methodology are widely used. The operating speed-based methodology is popular in other countries, since this methodology considers driver's speed choice. However, it is not widely used in USA due to not being incorporated into design policy.

The design-speed based methodology does not guarantee design consistency. The premise of the operating speed-based methodology is that operating speed prediction models exist. It was concluded that the selection of operating speed prediction model had a significant effect on evaluating design consistency. The alignment index methodology is easy to use. The measures used in the alignment index methodology have been evaluated having same results. However, these measures are only quantitative measures of the general character of the alignment of a roadway.

Six non automated speed data collection methodologies have been tested. All results based on the methodologies were statistically equivalent to real traffic speeds. Among the usually used portable speed measurement devices, radar and lidar guns are the most accurate devices. The shortcomings of radar gun have been pointed out, including the cosine effect which usually occurs when the deflection of the target to the center of the radar beam exists. The magnitude of the cosine effect depends on the angle where the larger the angle, the higher the error. If the angle is small and the object aimed moves at high speed, the error is much smaller.

CHAPTER 3: DATA COLLECTION

This chapter describes the site selection and data collection methodologies used. Sites were restricted in Kentucky. The data collected focused on two main components: roadway geometry and vehicle travel speeds. A database was developed based on the collected geometric and speed data.

3.1 Site Selection

3.1.1 *Selection Criteria*

Past studies (Warren, 1982; Fildes et al. 1987, 1990; Tignor and Warren 1990) found that significant road characteristics contributed to the operating speeds include: horizontal curvature, vertical curvature, grade, length of grade, number of lanes, surface condition, sight distance, lateral clearance, number of intersections, built-up areas near the roadway, access points, near by commercial development, and road width. The rationale of the site selection is to highlight the effect of highway elements by reducing the effects of other non-highway-element factors on operating speed. For instance, sites close to commercial development will not be used since in these commercial areas traffic flow is often interrupted.

The Kentucky Highway Performance Monitoring System (HPMS) database and the Measured Statewide Geographic Information System database (Statewide_m) were used as the primary data sources for identifying study sections. The 2005 HPMS and 2004 Statewide_m databases were used as these were the most current version of the two databases available at the time of the study.

The general criteria used to select sites are summarized in Table 3- 1. These criteria were used in different steps of the site selection. It should be noted that the site sample was not random and selection bias might exist because databases of all possible horizontal curves in Kentucky were not available at the time of the study.

Table 3- 1: Site Selection Criteria

Control	Criteria
Area Type	Rural
Number of Lanes	4
Functional Class	Non-Freeway or parkway
Facility Type	2-way
Terrain Type	No Restrictions
Radius	No Restrictions
Grade	-6.5% to +6.5%
Design Speed	No Restrictions
Speed Limit	>=45mph

The basic criteria of site selection were: rural four-lane highway; non-interstate & parkway highway; and not at grade intersection, since this study focuses on rural four-lane highways. To reduce the effect of an intersection or traffic signals & signs, roadway segments at grade intersections or in the vicinity of an intersection were excluded. Sites under reconstruction also were discarded after a site visit.

Terrain type was not restricted due to the topographic differences throughout Kentucky. From east to west, topography gradually varies from mountainous to level. Rural four-lane highways are distributed across the three general terrain types (level, rolling, mountainous). The radius of the curve was not available in either database, and therefore was not set as a criterion. Since there were no grades greater than 6.5%, the criterion of grade was set using this maximum grade. The speed limits for all rural four-lane highway segments collected in the databases were equal to or higher than 45 mph.

3.1.2 *Site Selection Steps*

In the first step for site selection the sections available in the HPMS database that adhere to the site selection criteria were extracted. The general criterion used in this step was rural four-lane non-freeways or parkways. The 2005 HPMS database was used initially, since the HPMS and Statewide_m databases were not exactly in agreement with each other and the available version of the Statewide_m database was 2004. There were 371 sections available in the HPMS database, which satisfied the initial criteria.

The second step identified the possible curves from the sections selected in the first step. The 371 sections consist of straight and/or curved sections. To identify the sections with curves all sections were imported to the ArcMap based on the Statewide_m database. The curves of each section selected were identified one by one in ArcMap environment. The general criterion used in this step was that a curve is not within or close to any intersection. Among the 283 potential curves, a total of 121 curves were randomly selected using this approach for on site verification.

The third step completed an on-site visit to verify the data accuracy. A total of 63 sites were finally determined appropriate for speed data collection after site visits. The following considerations were used:

- Not all roads have been included in the Kentucky GIS. The data of some local roads have not been entered into the database. Some curves appear not to be within or close to any intersection in GIS but were found to be within intersections after the site visit.
- The accuracy of data. For instance, two sites appear to be in rural area but they are actually located in a downtown.
- Grades are unavailable.
- Some curves possibly are under reconstruction.
- Traffic flows on some curves are influenced by upstream or downstream traffic flow or traffic control devices such as signals.

3.1.3 *Site Distribution*

The 63 sites were distributed widely across Kentucky (Figure 3- 1) and were selected from 35 of the 120 counties. No sites were selected from the southwest and northeast parts of the state mainly because the rural four-lane highways in these areas are interstates or parkways.

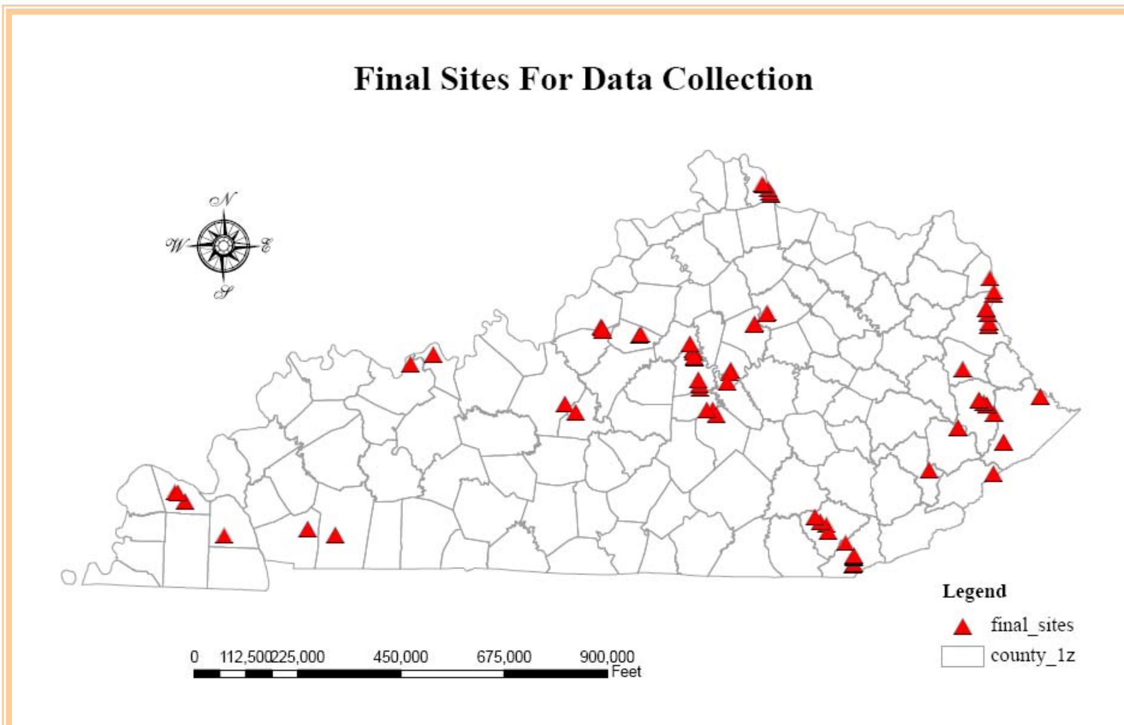

Figure 3- 1: Geographic Distribution of Sites

3.2 Geometric Data Collection

Sites have been selected but the mile points to be used in estimating the approaching tangent length are still unknown. Although mile points could be estimated by roughly measuring them in ArcMap, measure error could be an issue, since the mile points will be used to determine tangent lengths. Most of the previous studies obtained mile points by using maps, measuring length in the field or using computerized geometric data based on field measurements. One of the important weaknesses of measurement is accuracy. The measurement error would be enlarged because of the map scale. Field measurement error can not be avoided too, since it is difficult to accurately locate the elements of a curve.

To relatively accurately locate the curves selected, the Map Click software developed by Kentucky Transportation Center was used. This software is able to identify mile points with 3 significant figures. Figure 3- 2 shows a sample of mile point identification. Data needed in this software are county and road name. Curves need to be located in the Map Click software in accordance with their locations in ArcMap.

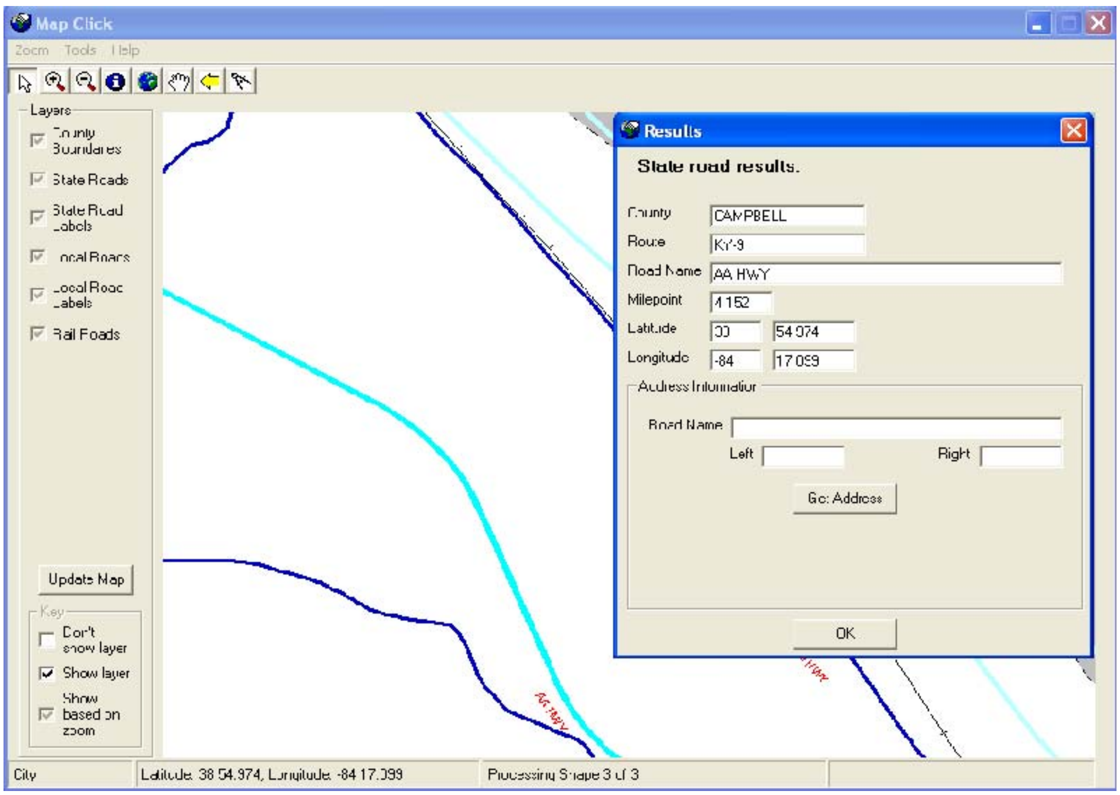

Figure 3- 2: A Sample of Mile Point Identification

Using the site location information (county, road name, section ID, and mile point), the geometric data of each location were extracted from the Highway Performance Monitor System (HPMS) 2005 version. A program written using Visual Basic for Applications (VBA) was used to automatically extract the sites from the 2005 HPMS database. The extracted geometric data that would be used for developing the database include: lane width, shoulder width, should type, median width, median type, type of terrain. Other data such as pavement type and Average Annual Daily Traffic (AADT) were also extracted from the 2005 HPMS database. Design speed was obtained from the 2003 HPMS database, since design speed data was not provided in the 2005 version. Speed limit was obtained from HPMS and was verified on site.

Preceding tangents followed by a curve have been found as a potential factor influencing speed in previous studies. Tangent lengths were calculated based on the data provided in the Kentucky Highway Information System (HIS) database. Grades of curves were extracted from the HIS too. It should be noted that the grades have been classified as categories therefore accurate values are unknown. Both extractions were automatically conducted by using a macro.

One of the most important speed predictors in the literature review was curve radius. In HPMS, a road has been separated into segments with the same geometric characteristics. Although the horizontal geometric data were also recorded in HPMS, there was no detailed data such as curve radius for each horizontal curve. In HIS, degree of curve has been provided. However, due to the questionable accuracy of the data, the curve radii provided were not used, and then the curve radii had to be estimated. In this study, Arc Geographic Information System (ArcGIS) and AutoCAD were used to measure the horizontal curve radii. The entire procedure followed for this is described below:

1) Extract location information from HPMS to develop a database;
2) Add the mile points obtained from the Map Click software to the database;
3) Import the database and the shape file of the measured statewide roads to ArcGIS;
4) Mark the sites selected in ArcGIS;
5) Export the marked sites and these roadway sections to AutoCAD;
6) Draw horizontal curves to simulate the real curves; and
7) Measure the curve radii and the lengths of the curves using AutoCAD tools.

In most cases, a curve is an arc in AutoCAD when exported from ArcGIS. But sometimes it is made up of several short lines or a series of points. In these situations, a curve should be drawn carefully so as to represent the real curve. Figure 3- 3 shows a sample of curve simulation and radius measurement in AutoCAD. The length of radius also can be obtained through AutoCAD. The green curve in Figure 3- 3 was drawn based on the lines exported from ArcGIS. The red point marker was used to conveniently locate the curve selected in ArcGIS. The white lines are the GIS center line representing the highway.

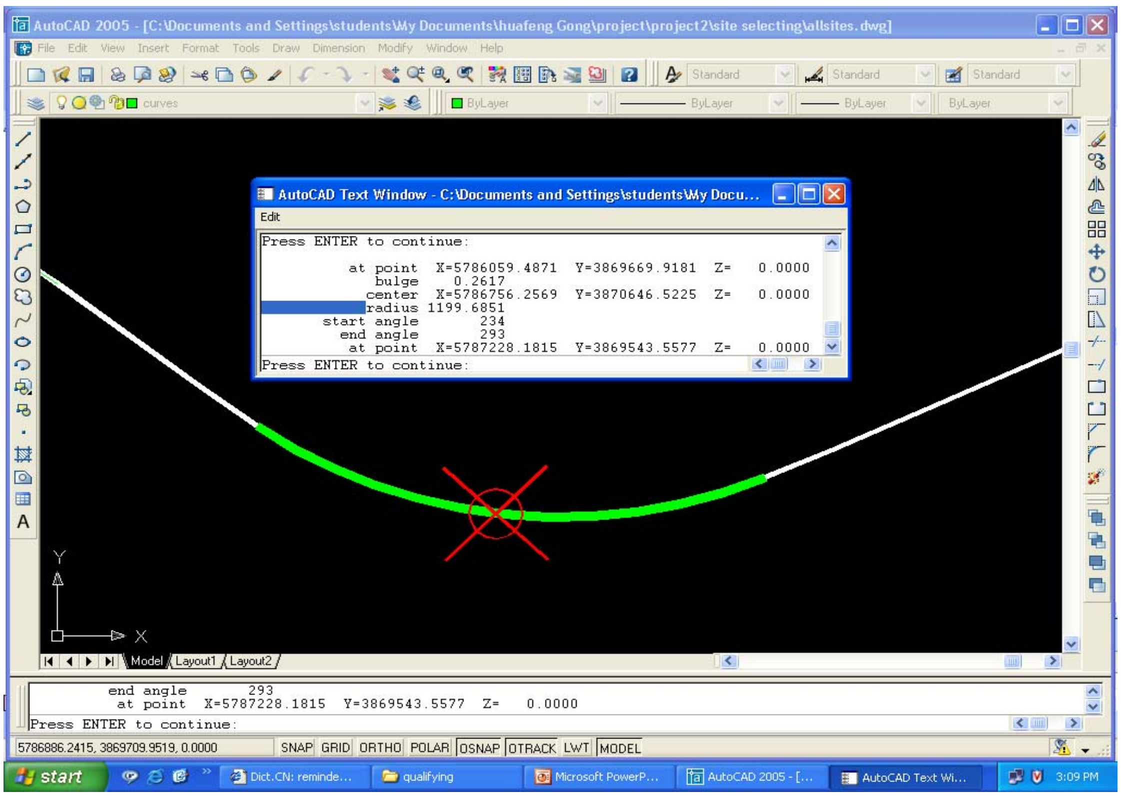

Figure 3- 3: Curve Simulation and Radius Measurement

Before going out for site verification, site maps were made based on the site information in ArcGIS. The maps provide the information such as county, road name, and vicinity of the sites. These maps also were used for directions to get there. Figure 3- 4 shows a sample of the maps for site verification.

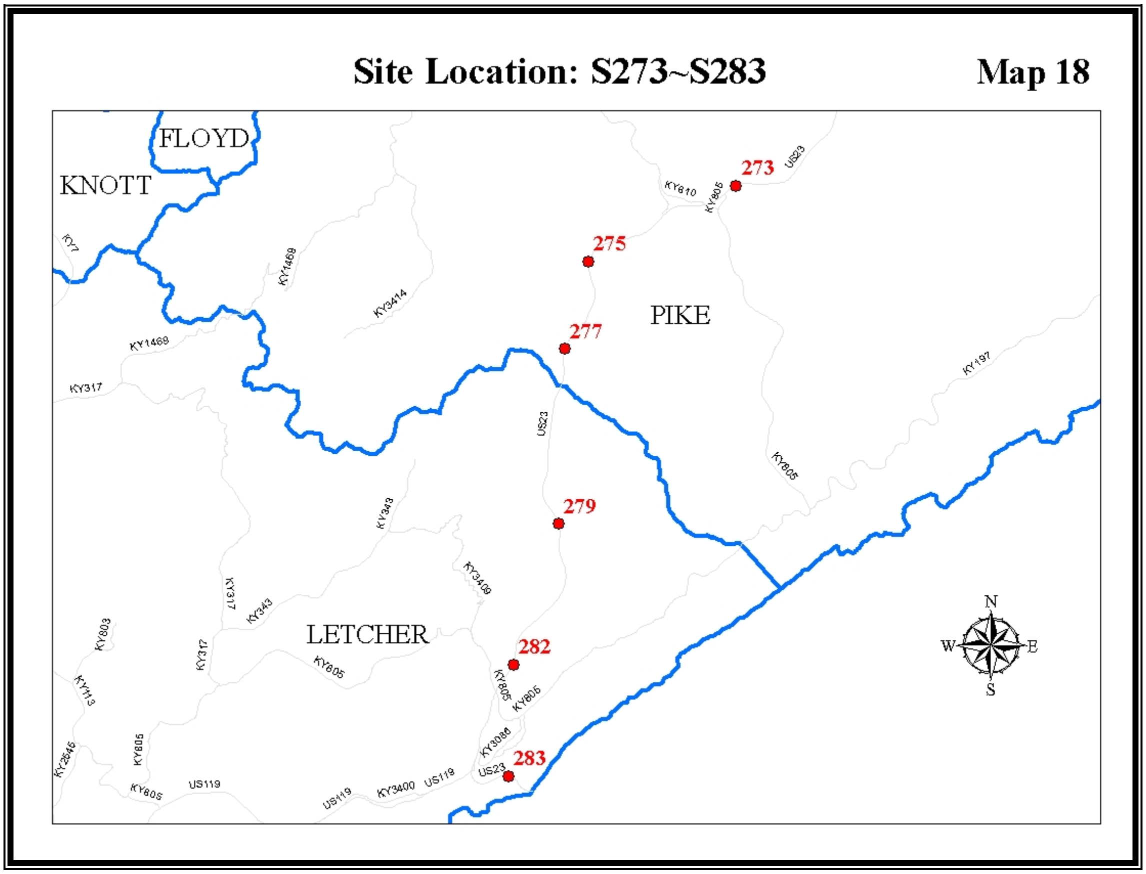

Figure 3- 4: A Sample of the Maps

The site visit allowed for a verification of the pavement type, median type, shoulder type, speed limit, preceding and following segments, and grade classification. The lane, unpaved or paved shoulder, median, and clear zone widths were measured. Errors of the HPMS data were corrected when found. Sites were recorded using photos.

Table 3- 2 summarizes the characteristics of the sites selected. It should be noted that most of median widths are less than 45 ft. Two sites have median width greater than 45 ft because the section where the two sites are located has a wide median for protecting trees.

Table 3- 2: Characteristics of the Sites Selected

Element	Value
Lane width	12 ft
Right shoulder width	2~14 ft
Left shoulder width	0~10 ft
Median width	0~107 ft
Shoulder type	Surfaced, stabilized, combined
Median type	Curbed, positive barrier, unprotected, none
Clear zone distance	2~40 ft
AADT (2005)	5,220~26,900
Pavement type	Intermediate mixed, high flexible, high rigid
Terrain type	Mountainous, rolling, level
Design speed	40~70 mph
Speed Limit	45~55 mph
Grade	-6.5% ~ +6.5%
Radius	538~7,704 ft
Length of Curve	775.82~5,780.83 ft
Preceding tangent length	0~21,181.52 ft
Following tangent length	0~13,182.65 ft

3.3 Speed Data Collection

The speed data were collected from May to December in 2006 during daylight, off-peak periods, and under good weather conditions. The speed collectors were required to record and verify all site information. Vehicle type was identified on site by observation. The time headway is required to be at least 5 seconds between consecutive vehicles to collect truly free flow speeds (HCM, 2000).

The large number of sites where data was to be collected required the use of automated devices for speed data collection. However, the currently available automated devices used for speed measurement are not able to accurately identify vehicle types. Moreover, the installation of these devices would required the presence of State Police or

Transportation Cabinet personnel to close the road in order for the devices to be installed and uninstalled. This was not possible and it was decided to use portable manual devices (radar guns) in this study to specifically collect data on passenger cars and avoid long delays due to scheduling conflicts.

The speed data were collected using radar guns, and were recorded at the beginning, middle and ending points of the curve and for each lane under free-flow conditions. Therefore there are 6 speed measurement spots in each horizontal curve. To locate the speed measurement spots in each horizontal curve, eTrexVista personal GPS navigators were used. Since there were no plans available, it was difficult to obtain the accurate coordinates of the beginning and ending of a curve and to locate them on site. To relatively accurately locate the three points of the curves, the GPS coordinates of middle points have been roughly obtained from ArcGIS using the Statewide_m database before the speed data collection. The following procedure was used to locate the three points of a curve on-site:

1) Drive the entire curve and roughly estimate the three locations;
2) Use the GPS coordinate to locate the middle point;
3) Locate the beginning and ending points based on the length of the curve; and
4) Use the mile points in the HIS to roughly verify the three locations.

It should be pointed out that this procedure still cannot guarantee to obtain accurate locations. However, this procedure can ensure that the three points are close to their true locations.

The data collectors were required to be located where they can see the measurement point while drivers could not see them to avoid influencing the driver's operating speeds. However, it is hard to satisfy these requirements in most cases. To find a method to decrease the effects of the presence of data collectors, two experimental approaches on a highway were used. At the first approach, a vehicle was pulled over on the right shoulder. It was found that vehicles slowed down and traveled at speeds less than the posted speed limit. Most of the drivers looked at the experimental vehicle when they passed by. At the second approach at the same location, the vehicle was also pulled over on the right shoulder while the trunk was open. It was observed that vehicles traveled at speeds higher than the posted speed limit. Most drivers just glanced at the stopped vehicle. One possible reason was that the drivers considered the vehicle in the first approach as a police car. Therefore, speed collectors were required to cover their vehicles on site to reduce the effects to maximum extent. Two methods were mainly used to cover the vehicles on site. One method was using a pickup truck. The other method was opening the trunk when a vehicle similar to police car was used.

When radar meters are used to record speeds, the cosine error always occurs if the deflection of the target to the center of the radar beam exists. Theoretically, a radar meter should be located at a line with a moving vehicle aimed. In practice, it is hard to guarantee no deflection, and hard to record the deflection for each vehicle observed. In this study speed collectors were required to be in an alignment with the center lines of the lanes and to collect speeds when vehicles are located at the appointed spots. Figure 3- 5 shows a schematic illustration of speed data collection.

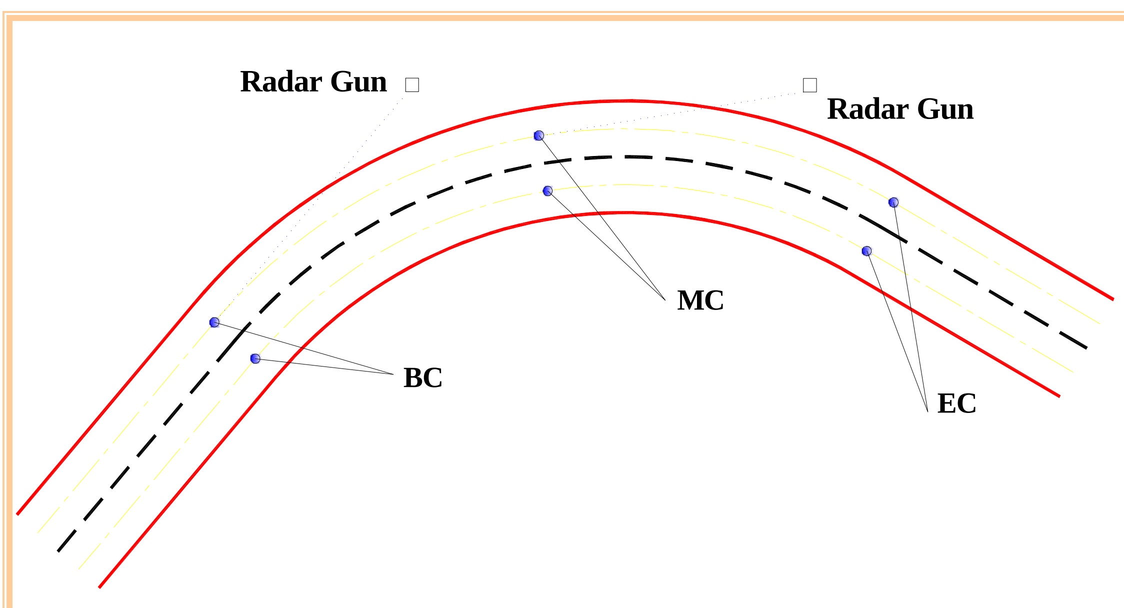

Figure 3- 5: Location of Radar Meters on Horizontal Curve

Initially, speed data were collected at the first 13 sites (both directions of travel) with measurements at six points. After analyzed the relationship of the speeds on both directions of travel and found that there was no difference, speeds were then collected at another 26 sites (one direction) with measurements at six points. An analysis performed at that point showed that the speeds for these sites at the three locations were statistically the same, and therefore it was determined to continue the collection process by measuring speeds only at the middle of the curve. Therefore, at the remaining 24 curves, speeds were measured at the two middle points on both the inside and outside lanes along a curve.

Based on prior speed collection experience, at least 100 observations were to be taken at each site. However, there are some segments with low AADT, and on some sites most vehicles travel on outside lane (close to right shoulder) so that few vehicles travel on inside lane. Therefore, fewer observations were typically taken at sites with low AADT, as well as on some inside lanes[3].

3.4 Database Development

To compare the operating speeds in each direction of travel, speed data in each direction of travel on some sites were collected. To achieve this, 13 of the total 63 sites were randomly selected. Therefore data was collected at 76 curves in total[4]. On each curve of the first 39 sites, speed data were collected at 6 spots (BC, MC, and EC on inside and outside lanes). At the remaining 24 sites, only 2 spots (MC on inside and outside lanes) at each site were used to collect speeds, since it was found that the speeds at the three points (BC, MC, and EC on a same lane) were statistically the same[5]. Descriptive statistics for each spot were calculated using Excel Macro. These statistics include mean speed, the 85^{th} percentile speed, and standard deviation.

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[3] The normality of these data was checked in Data Deduction in Chapter 4.

[4] 76 curves = 26 curves selected at the 13 sites (both directions) + 50 curves selected at the remaining 50 sites (one direction).

[5] There were 360 spots in total used for speed data collection. The 360 was calculated as:
360 spots = 6 spots x 2 directions x 13 sites + 6 spots x 26 sites + 2 spots x 24 sites

Using the collected geometric data and the descriptive statistics, a database was developed. The database consists of the following data, shown in Table 3- 3:

Table 3- 3: Data Included in the Database

Index	Left_Shoulder_Width	HC_Grade
Site	Shoulder_Type	First_Tangent_Grade
County	Median_Type	Second_Tangent_Grade
Road	Median_Width	BC_Grade
Mile_ Point	Clear_Zone_BC	EC_Grade
Date	Clear_Zone_MC	MC_Grade
Sample_Size	Clear_Zone_EC	Front_Curve
Mean_Speed	AADT	Radius
The 85th Speed	Pavement_Type	HC_Length
Standard_Dev.	Terrain_Type	First_Tangent_length
Lane_Width	Design_Speed	Second_Tangent_length
Right_Shoulder_Width	Speed_Limit	

CHAPTER 4: SPEED CHARACTERISTICS

4.1 Introduction

This chapter analyzes the characteristics of the 85th percentile speeds in each lane of horizontal curves on rural four-lane highways. Three paired comparisons are conducted:

1) Comparison between the 85th percentile speeds in each direction of travel;
2) Comparison between the 85th percentile speeds on inside and outside lanes in same direction of travel; and
3) Comparison between the 85th percentile speeds on beginning point (BC), middle point (MC), and ending point (EC) on a same lane;

Among the 63 sites, 13 were randomly selected to collect speeds for the comparison between the 85th percentile speeds in each direction of travel. The other two comparisons were conducted using all qualified sites. These two comparisons were based on the conclusion of the first comparison.

Prior to the data analysis, some extreme and/or unreasonable data should be identified and eliminated from the dataset. The data reduction is presented in this chapter. It should be pointed out that the sample sizes used for the three comparisons were not same. This was because the third comparison was based on the conclusions in the prior two comparisons and the second comparison was based on the conclusion in the first comparison.

4.2 Data Reduction

4.2.1 *Extreme or Unreasonable Data*

Extreme or unreasonable data usually refers to outlier in statistics. An outlier is an observation that lies outside the overall pattern of a distribution (Moore and McCabe 2006). The presence of an outlier indicates some possible problem. This can be a case which does not fit the model under study or an error in measurement. The most often used statistic graphic methods to spot outlier are box plot, scatter plot, and histogram. Box plots were used to spot outliers here.

When using the Statistical Package for the Social Sciences (SPSS) software to obtain the box plots of the 85th percentile speeds, it was found that there were only 2 outliers in each of the two categories (Figure 4- 1). These two outliers in each category come from the same sites with operating speed lower than the posted speed limits. The design speeds at these two sites are 70 mph and the radii are 1810 and 1975 ft, respectively. After checking the notes of the site information, it was found that the speeds were slightly influenced by downstream construction. So these two curves were eliminated from the database. The original number of the sites (63) decreases to 61. In addition, speeds on 13 sites were measured in both directions resulting in total of 74 sites. Therefore, 74 curves were qualified for data analysis.

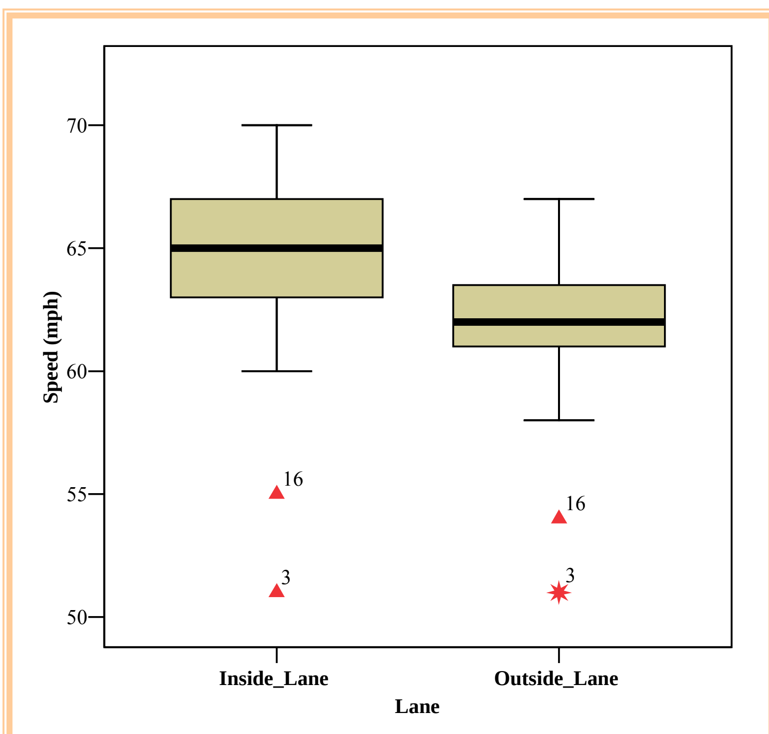

Figure 4- 1: The 85th Percentile Speed Box Plots

4.2.2 *Normality Examination*

A basic assumption for speeds is that the observations obtained are from a normal distribution. This assumption needs to be verified for each site. Moreover, this was more important for the sites where few spot speeds were obtained, before using the collected data in the analysis. Insufficient spot speed samples cannot represent the real population, and therefore they will likely produce meaningless results. Therefore 183 spots with less than 100 spot speeds were examined. The remaining 165 spots had more than 100 spot speeds and met the requirement of minimum sample size for speed study (HCM, 2000). These sites were not examined based on the assumption that the samples are normally distributed if the sample size is greater than 100.

The normality examination procedure includes the Kolmogorov-Smirnov test and probability plotting. The Kolmogorov-Smirnov test (D'Agostino and Stephens, 1986) is a non-parametric test for goodness-of-fit. It can be used to test whether the distribution of a sample matches a specific distribution, in this case the normal distribution. If the p-value of the Kolmogorov-Smirnov test is less than the significance level considered, the distribution of the sample is not normal at the significance level. If the p-value is greater than the significance level, a probability plot should be used to determine whether the distribution of a sample is normal or not. SPSS was used for the Kolmogorov-Smirnov test, and Matlab was used for probability plotting.

After using the Kolmogorov-Smirnov test and the normal probability plots, 11 of the 183 spots were identified as lacking normality. Therefore the 85th percentile speeds measured on 337 of the total 348 spots[6] were available for further data analysis.

4.3 Comparison of the Speeds in Two Directions

A total of 13 sites were randomly selected to collect speeds for the comparison between the 85th percentile speeds in each direction. Of the 11 spots which have been identified as lacking normality, 7 spots were among the selected 13 sites. These 7 spots were discarded

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[6] Since 2 sites were eliminated due to construction, the number of the total spots decreased from 360 to 348.

from the analysis. Therefore the 85th percentile speeds measured on the 149 spots of the 13 sites were available for the comparisons.

The two curves at a site generally have same geometric features, except for clear zone distance[7]. The speeds on the two inside lanes in each direction are compared, as well as the outside lanes. The pairs of the comparisons are six (Figure 4- 2). These spots are beginning points, middle points, and ending points, respectively.

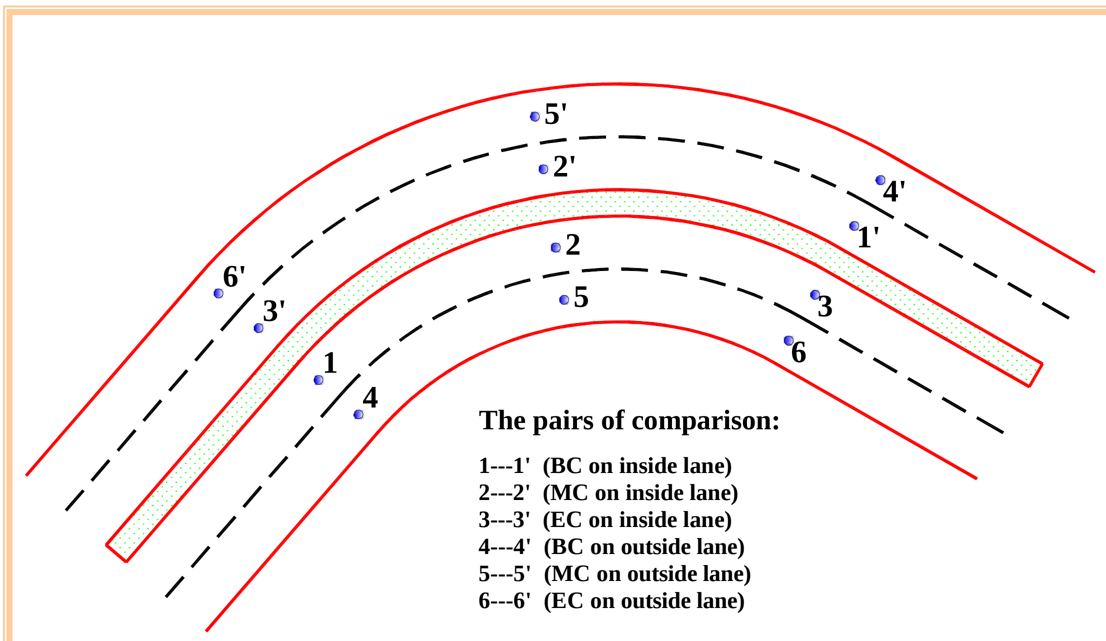

Figure 4- 2: Pairs of Speed Comparisons

Since the normality examination indicated that speeds collected at the 7 spots of the 13 sites were not qualified, these speeds could not be used for the comparisons, resulting in different sample sizes for each comparison pair (Table 4- 1).

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[7] Although the two curves generally have same geometric features, the two curves were considered as two different curves in the comparisons.

Table 4- 1: Speed Differences of Comparison Pairs (mph)

Site	Spot1	Spot2	Spot3	Spot4	Spot5	Spot6
1	0.20	-2.30	-1.00	-5.00	-6.00	-2.00
2	-3.50	-2.35	3.50	-1.00	-3.00	-2.00
3	3.45	0.50	-1.50	4.70	2.00	0.05
4	-3.00	1.50	0.00	0.00	-1.00	-2.00
5	1.00	-4.00	-5.00	-0.90	-2.00	-1.00
6	3.00	-2.40	-2.55	0.00	-4.00	-3.00
7	3.00	3.00	1.00	3.00	1.55	2.00
8	-1.65	-0.45	1.65	-3.00	-0.25	3.00
9	**-2.00**	0.05	2.00	0.00	2.00	2.95
10	**1.00**	**0.00**	2.00	**-2.15**	-1.00	2.00
11	**0.00**	1.00	-1.00	-4.00	**-3.70**	-0.10
12	-5.50	-2.00	-3.70	-2.25	-0.05	-4.00
13	-1.95	-0.90	-4.00	-1.00	-0.85	-0.15

Notes: 1. Bold figures are not normally distributed.

2. The speeds collected at the beginning points on inside lanes at site 11 in both travel directions are not normally distributed.

Table 4- 1 shows the speed differences of the comparison pairs. The paired t-test was used to examine the relationships of the pairs. One of the important assumptions of paired t-test is that the distribution of data is normal. After checking the normality with Kolmogorov-Smirnov test, the null hypotheses that the distributions were normal could not be rejected (p-value > 0.05). The normal probability plots also indicated that the data were normally distributed. Table 4- 2 shows the results of the normality check.

Table 4- 2: Normality Check of the Differences

	Spot1	Spot2	Spot3	Spot4	Spot5	Spot6
Number of Sites	10	12	13	12	12	13
Mean	-0.495	-0.696	-0.662	-0.788	-1.050	-0.327
Std. Deviation	3.100	2.006	2.641	2.7221	2.430	2.279
Most Absolute Extreme Differences	0.170	0.159	0.120	0.220	0.175	0.154
Kolmogorov-Smirnov Z	0.538	0.550	0.432	0.760	0.606	0.555
Asymp. Sig. (2-tailed)	0.934	0.923	0.992	0.610	0.857	0.917

Note: different sample size due to lack of normality of the original data

The paired t-tests showed that the 85th percentile speeds in each pair were statistically equal when the significance level was set at 95%. In other words, the 85th percentile speeds on the inside lanes and outside lanes in each direction were statistically the same for each comparison point. It indicates that the 85th percentile speeds in the two directions have same characteristics. One possible reason is that the two curves at each of the selected 13 sites generally have the same geometric features. The statistical analysis results are showed in Table 4- 3.

Table 4- 3: Results of the Paired t-Tests

Pair	Description	Mean	σ^2	t	df	p
Pair 1	1---1’	-0.495	3.100	-0.505	9	0.626
Pair 2	2---2’	-0.696	2.006	-1.201	11	0.255
Pair 3	3---3’	-0.662	2.641	-0.903	12	0.384
Pair 4	4---4’	-0.788	2.722	-1.002	11	0.338
Pair 5	5---5’	-1.050	2.430	-1.497	11	0.163
Pair 6	6---6’	-0.327	2.279	-0.517	12	0.614

4.4 Comparison of the Speeds on Inside and Outside Lanes

In the preceding analysis, it was concluded that the 85th percentile speeds in two directions are statistically the same. To compare the speeds on inside and outside lanes, the speeds collected on the 26 curves at the selected 13 sites could be used. A total of 50 curves[8] were used for this comparison. Of the 300 spots at the selected curves, 11 spots have been identified as lacking normality. Therefore, the speeds collected at 289 spots were qualified for the comparison. In this comparison, three sub-comparisons were conducted:

1) Comparison between the 85th percentile speeds at BC;
2) Comparison between the 85th percentile speeds at MC; and
3) Comparison between the 85th percentile speeds at EC;

The same normality test also should be conducted before using the paired t-test to test relationships. After using Kolmogorov-Smirnov test to check the normality, the null hypotheses that their distributions were normal could not be rejected (Table 4- 4). The normal probability plots of the differences indicated that the data were normally distributed as well.

Table 4- 4: Normality Check of the Differences

	Diff_BC	Diff_MC	Diff_EC
Number of Sites	42	48	49
Mean	2.092	2.642	2.671
Std. Deviation	1.752	1.603	1.981
Most Absolute Extreme Differences	0.103	0.156	0.137
Kolmogorov-Smirnov Z	0.666	1.078	0.961
Asymp. Sig. (2-tailed)	0.767	0.196	0.314

Note: different sample size due to lack of normality of the original data

[8] See section 3.3 for data collection procedure.

By using the paired t-test, the hypotheses that the 85th percentile speeds on inside and outside lanes are equal were rejected with p-value=0.000 for each pair (Table 4- 5), when the significance level was set at 95%. This indicates that the 85th percentile operating speeds at each of the three points on inside and outside lanes are not same. It can be further concluded that the operating speeds on inside and outside lanes are different.

Table 4- 5: Results of the Paired t-Tests

Pair	**Description**	**Mean**	σ^2	t	df	p
Pair 1	1---4	2.092	1.752	7.738	41	0.000
Pair 2	2---5	2.642	1.603	11.417	47	0.000
Pair 3	3---6	2.671	1.981	9.439	48	0.000

Figure 4- 3 graphically depicts the differences of the average operating speeds between the inside and outside lanes. This figure shows that, in general, the average operating speeds at the three points on inside lane are higher than on outside lane and, their differences are approximately 3 mph.

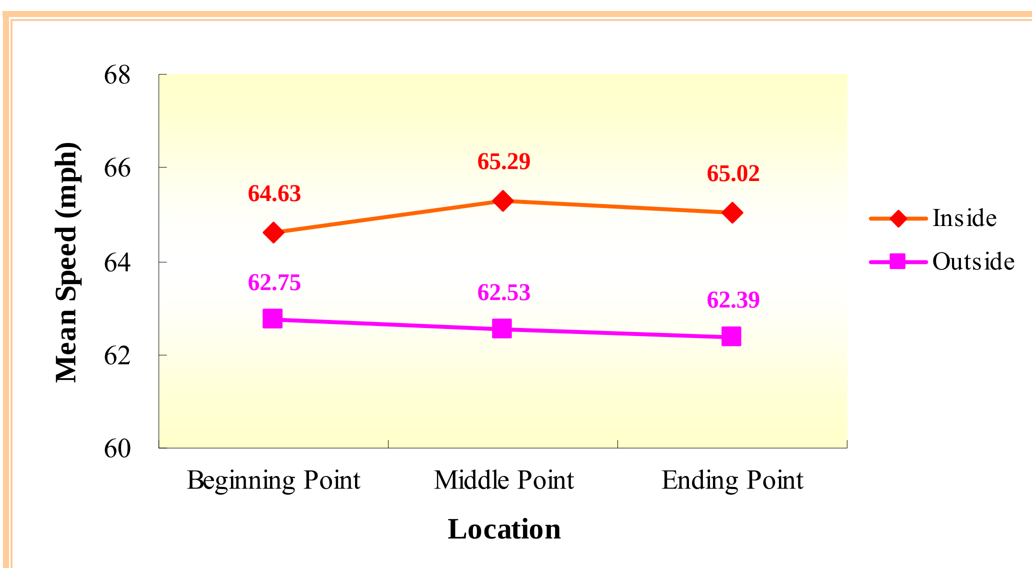

Figure 4- 3: Speed Comparisons between Inside and Outside Lanes

4.5 Comparison of the Speeds at BC, MC, and EC

It has been shown from the preceding two comparisons that the speeds in each direction have no differences but the speeds on inside and outside lanes are different. Of interest is also whether speeds at the three points (BC, MC, and EC) along a curve on either lane (inside or outside) are equal. To obtain the characteristics of the 85th percentile speeds along a curve, two sub-comparisons based on the speeds collected on different lanes were conducted:

1) Comparison on inside lane; and
2) Comparison on outside lane.

4.5.1 *Comparison on Inside Lane*

The 85th percentile speeds at the three points were compared. Again 50 curves were available for the comparison. Of the 150 spots, 8 spots were identified as lacking normality. Therefore the speeds collected at 142 spots were qualified to conduct the comparison.

The normality examinations showed that the distributions of the differences are normal, which indicate that the paired t-test is suitable for the comparisons. Table 4- 6 shows the results of the normality check.

Table 4- 6: Normality Check of the Differences

	BC_MC	**BC_EC**	**MC_EC**
Number of Sites	44	43	48
Mean	-0.533	-0.192	0.189
Std. Deviation	2.370	2.332	2.311
Most Absolute Extreme Differences	0.114	0.156	0.092
Kolmogorov-Smirnov Z	0.755	1.025	0.641
Asymp. Sig. (2-tailed)	0.619	0.244	0.806

The paired t-tests showed that all p-values were greater than 0.05 when setting the significance level at 95% (Table 4- 7). It means that there is no statistical difference in the speeds of each pair. Therefore, it could be concluded that on inside lane the operating speeds at the three points in a curve are statistically the same.

Table 4- 7: Results of the Paired t-Tests

Pair	**Description**	**Mean**	σ^2	t	df	p
Pair 1	BC - MC	-0.53295	2.370	-1.492	43	0.143
Pair 2	BC - EC	-0.19186	2.332	-0.539	42	0.592
Pair 3	MC - EC	0.18854	2.311	0.565	47	0.575

4.5.2 *Comparison on Outside Lane*

The same procedure as described above was applied for this comparison. The 50 curves were used for the comparison. Of the 150 spots at the selected curves, 3 spots were identified as lacking normality. Therefore, the speeds collected at 147 spots were qualified to conduct the comparison.

Same normality tests were conducted before choosing statistic procedures to test the relationships. Using the nonparametric method Kolmogorov-Smirnov test, it was observed that all p-values were greater than 0.05 when setting the significance level at 95%. The null hypothesis tests indicated that the distributions were normal (Table 4- 8).

Table 4- 8: Normality Check of the Differences

	BC_MC	BC_EC	MC_EC
Number of Sites	47	48	49
Mean	0.099	0.285	0.241
Std. Deviation	1.755	2.109	2.007
Most Absolute Extreme Differences	0.134	0.137	0.130
Kolmogorov-Smirnov Z	0.916	0.950	0.912
Asymp. Sig. (2-tailed)	0.371	0.327	0.376

The paired t-test was employed to test the relationships. The results of the paired t-tests showed that in each pair the difference was not significant when setting the significance level at 95% (Table 4- 9). It could be concluded that on outside lane the operating speeds at the three points in a curve are not statistically different.

Table 4- 9: Results of the Paired t-Tests

Pair	Description	Mean	σ^2	t	df	p
Pair 1	BC - MC	0.099	1.755	0.386	46	0.701
Pair 2	BC - EC	0.285	2.109	0.938	47	0.353
Pair 3	MC - EC	0.241	2.007	0.840	48	0.405

4.6 Summary

This chapter analyzed the characteristics of operating speeds on horizontal curve. A couple of hypotheses have been evaluated to test the relationships. All hypotheses were tested using the paired t-test, since this method was suitable for the tests. According to the analyses conducted in this chapter, it was concluded that:

- The operating speeds in each direction are similar;
- The operating speeds on inside and outside lanes were significantly different; and

- On each of the two lanes in same direction, the operating speeds at the beginning point, middle point, and ending point were statistically the same.

CHAPTER 5: RELATIONSHIPS RELATED TO OPERATING SPEED, DESIGN SPEED AND SPEED LIMIT

5.1 Normality Examination

The preceding chapter concluded that the operating speeds at the three measurement points on the inside or outside lane in a curve have no significant differences. Therefore, the speeds observed at the three points on the inside and outside lanes of a curve were aggregated to calculate a new 85^{th} percentile speed. This speed was considered as the representative of the operating speed on either lane of the curve.

After aggregating the speed samples, the number of observations for each measurement location of a curve increased. The new sample sizes vary from 91 to 312. There were 4 curves with speed sample size less than 100, and these speed samples should be tested for normality using the Kolmogorov-Smirnov test. The tests showed that all speed samples were normally distributed (Table 5- 1). It also indicated that all sample locations were appropriate for the data analysis.

Table 5- 1: p-values of the Normality Examination

Site	103_O	113_I	106_I	110_I
Observations	91	95	96	97
Mean	61.550	62.653	61.333	62.701
Std. Deviation	4.072	4.448	4.391	4.767
Kolmogorov-Smirnov Z	0.933	0.678	0.770	1.024
Asymp. Sig. (2-tailed)	0.348	0.747	0.593	0.246

5.2 Relationships between Operating Speed and Design Speed

Since the operating speeds on inside and outside lanes were significantly different, their relationships related to design speed are assumed to be also different. Therefore, the relationships for inside lane and outside lane were examined separately.

5.2.1 *Inside Lane*

Of the 74 sample curves, there were 23 curves (31.08%) with a higher operating speed than the design speed. The maximum difference (design speed minus operating speed) reached 28 mph. However, the average difference between design speed and operating speed was only 1.38 mph (design speed greater than operating speed).

Before examining the relationship between operating and design speeds, the normality of the differences between these speeds should be tested in order to determine which statistic method is suitable. Using the Kolmogorov-Smirnov test, it was determined that the hypothesis of no difference between operating speed and design speed could not be rejected because the p-value was 0.061 when setting the significance level at 95%. It could be therefore concluded that the differences between operating and design speeds were normally distributed. The normal probability plot (Figure 5- 1) also showed that most points were distributed along the red line, which indicates that the distribution of the differences is normal. The results indicate that the paired t-test is suitable for the comparison.

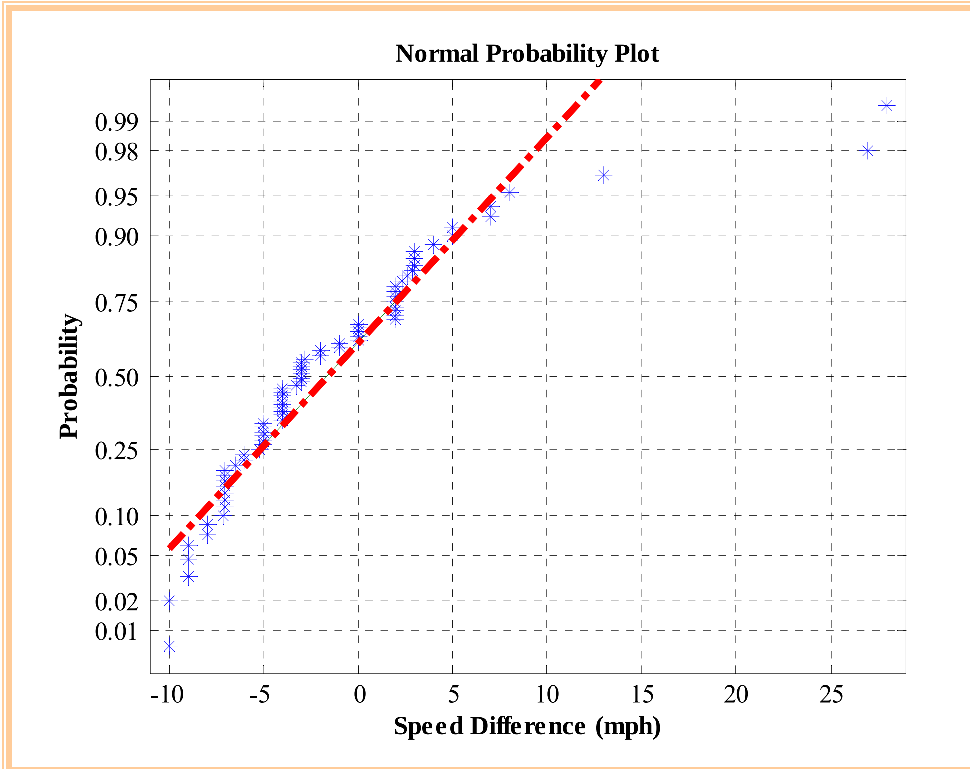

Figure 5- 1: Probability Plot of the Difference between Operating and Design Speeds on Inside Lane

The paired t-test showed that the design speed was equal to the 85th percentile operating speed with p-value=0.083, when the significance level was set at 95%. This finding was in agreement with the previous finding on four-lane highways (Stamatiadis and Gong, 2006).

5.2.2 *Outside Lane*

Among the 74 sample curves, there were only 12 sample curves (16.22%) with a higher operating speed than the design speed. The maximum difference (design speed minus operating speed) reached 27 mph. The average difference between design speed and operating speed was 4.11 mph (design speed greater than operating speed).

Using the Kolmogorov-Smirnov test, it was determined that the hypothesis of no difference between operating and design speeds should be rejected because the p-value (p-value=0.029) was less than 0.05 when setting the significance level at 95%. It indicated that the differences between the operating and design speeds were not normally distributed. The normal probability plot (Figure 5- 2) also verified that the data were not normally distributed. The crucial assumption of normality for the paired t-test was violated and therefore it was not suitable for this comparison. Another statistic method—such as a nonparametric test should be used for the analysis.

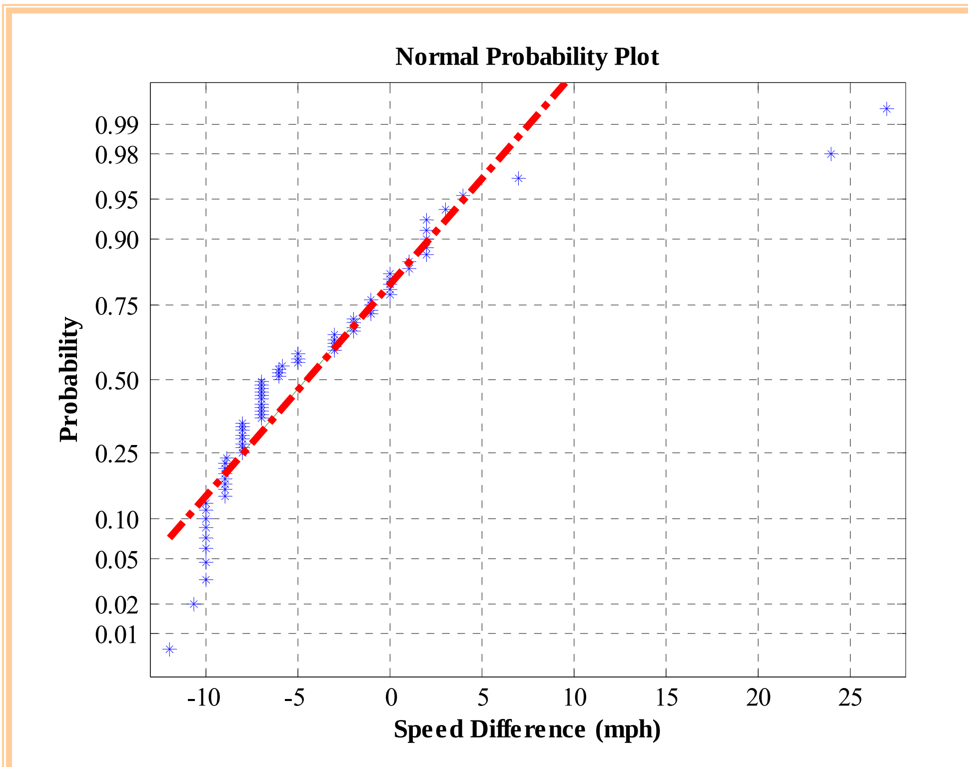

Figure 5- 2: Probability Plot of the Difference between Operating and Design Speeds on Outside Lane

Non-Parametric tests have much less stringent assumptions concerning the distributions of the variables and the variances of comparison groups. Therefore, they are often used in place of their parametric counterparts when certain assumptions about the underlying population are not met. There are several nonparametric procedures, such as Wilcoxon signed rank test, Wilcoxon rank sum test, Spearman rank correlation coefficient, and Kruskal-Wallis Test (Hollander and Wolfe, 1999). These tests tend to rely on the rank of the individual observations rather than their absolute numeric values. In this analysis, the Wilcoxon signed rank test was used, since it is a nonparametric alternative to the paired t-test.

The Wilcoxon signed rank test (Wilcoxon, 1945) is used to test whether the median of a continuous and symmetric population is 0. This test assumes that there is certain information in the magnitudes of the differences between paired observations, as well as in their signs. The calculation procedure is simple. First, the differences are calculated for the paired observations, and then they are ranked from the smallest to largest without regard to their signs. Second, the signs of the original observations are attached to their corresponding ranks. Finally, the one sample z statistic (mean / standard error of the mean) is calculated from the signed ranks. For small samples, the statistic is compared to likely results if each rank was equally likely to have a "+" or "-" sign affixed. For large samples, the z statistic is compared to percentiles of the standard normal distribution.

Using the Wilcoxon signed rank test, the hypothesis of no difference between the speeds was rejected due to the small p-value (p-value =0.000) when setting the significance level at 95%. It indicated that the operating and design speeds were not equal. The negative ranks were 12 while the positive ranks were 57 when using design speed minus operating speed. These figures showed that the operating speed was lower than the design speed.

5.2.3 *Summary*

In the two comparisons, the parametric (paired t-test) and nonparametric (Wilcoxon signed rank test) tests were used due to the different distributions and requirements of the data. The comparisons showed that for the inside lane the operating speed was statistically equal to the design speed. On the other hand the data for the outside lane showed that the operating speed was significantly lower than the design speed.

5.3 Relationships between Operating Speed and Speed Limit

As the previously noted, the relationships of operating speed with the posted speed limit are also assumed to be different, since the operating speeds on inside and outside lanes were significantly different. Therefore, the relationships for inside and outside lanes were examined separately too.

5.3.1 *Inside Lane*

All of the operating speeds collected in the 74 sample curves were higher than the posted speed limits. The maximum difference (operating speed minus posted speed limit) was 15 mph and the minimum difference was 5 mph. The average difference between the operating speed and the posted speed limit reached 10.17 mph (operating speed greater than posted speed limit).

Using the nonparametric method Kolmogorov-Smirnov test, the hypothesis that there was no difference between operating speed and posted speed limit was rejected because of the small p-value (p-value=0.047) when setting the significance level at 95%. It indicated that the differences between the operating speeds and posted speed limits were not normally distributed. The normal probability plot (Figure 5- 3) showed that the data were not normally distributed as well. Therefore, the nonparametric method should be used for the analysis.

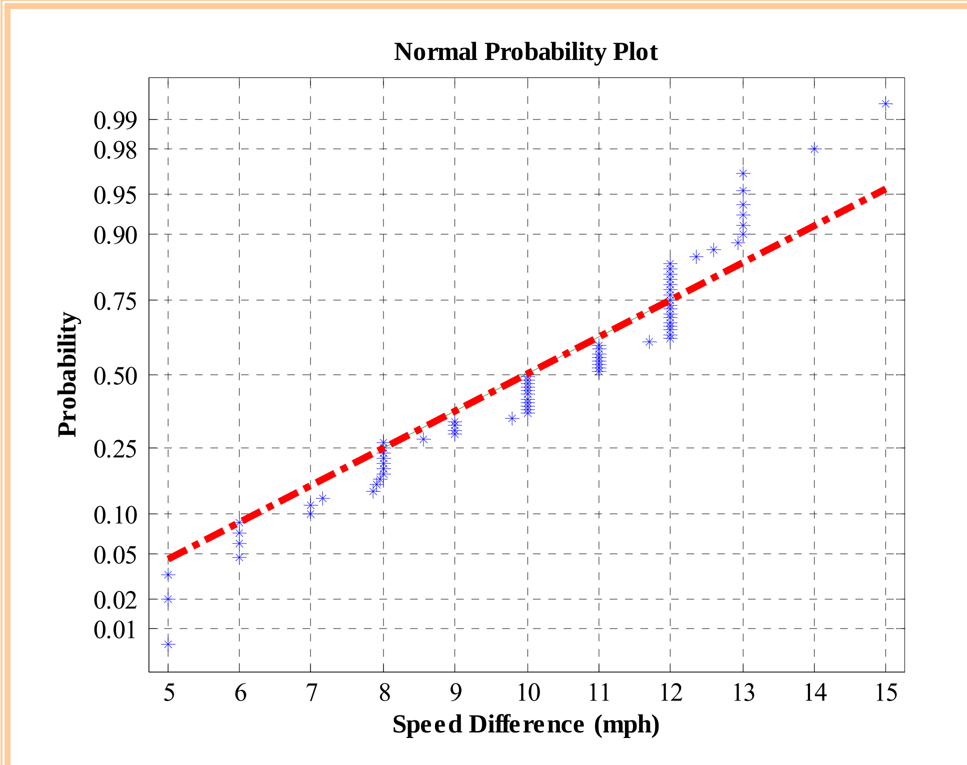

Figure 5- 3: Probability Plot of the Difference between Operating Speed and Speed Limit on Inside Lane

The Wilcoxon signed rank test was used as previously described. The test showed that the hypothesis was rejected due to the small p-value (p-value=0.000) when setting the significance level at 95%. It indicated that the operating speeds were not statistically equal to the posted speed limits. All 74-site operating speeds were higher than the posted speed limits. It could be further concluded that for the inside lane the operating speed was higher than the posted speed limit.

5.3.2 *Outside Lane*

All operating speeds collected on the outside lanes in the 74 curves were higher than the posted speed limits. The differences (operating speed minus speed limit) ranged from 3 mph to12 mph. The average difference between operating speed and speed limit reached 7.44 mph (operating speed greater than speed limit), which was smaller than the difference for the inside lane.

Using the Kolmogorov-Smirnov test, the hypothesis of no difference could not be rejected because of the p-value (p-value=0.153) when setting the significance level at 95%. It indicated that the differences between the operating speeds and posted speed limits were normally distributed. The normal probability plot (Figure 5- 4) also showed that the distribution was normal because the points representing the differences between the operating speeds and the posted speed limits were along the line.

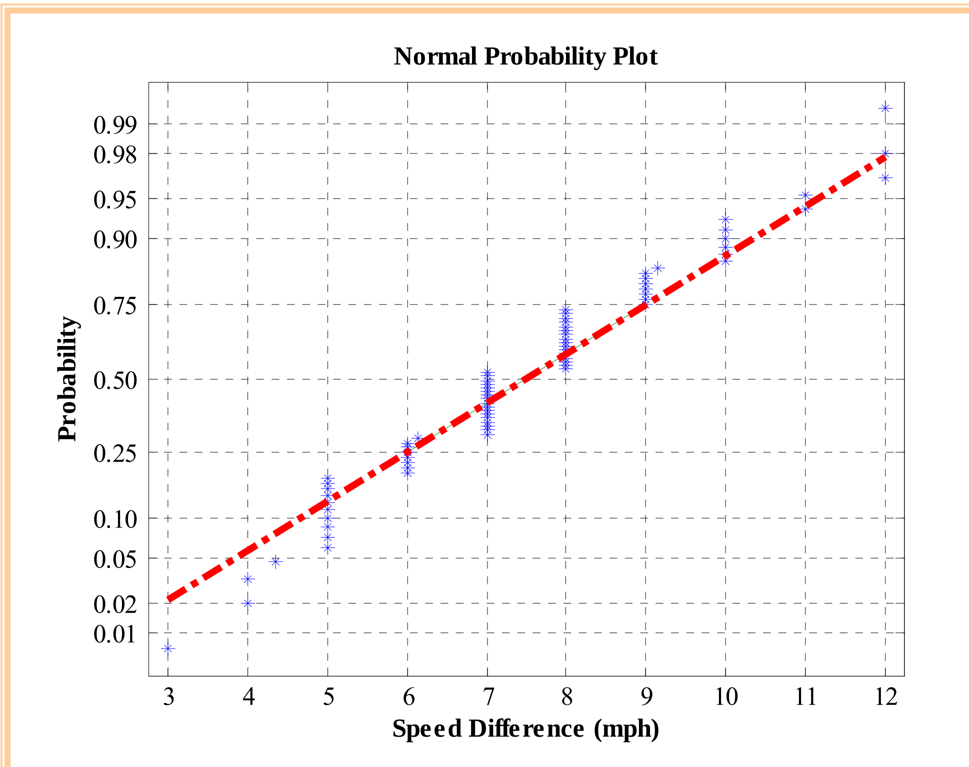

Figure 5- 4: Probability Plot of the Difference between Operating Speed and Speed Limit on Outside Lane

The paired t-test showed that the hypothesis should be rejected with the low p-value (p-value=0.000) when the significance level was set as 95%. It means that differences between the operating speeds and the posted speed limits existed on outside lanes. Therefore, it could be concluded that for the outside lane the operating speeds were also higher than the posted speed limits.

5.3.3 *Summary*

In the two comparisons, the parametric (paired t-test) and nonparametric (Wilcoxon signed rank test) tests were used due to the different distributions and requirements of the data. The paired t-test and Wilcoxon signed rank test showed that for both inside and outside lanes the operating speed was statistically higher than the posted speed limit.

CHAPTER 6: DESIGN SPEED AND GEOMETRIC ELEMENTS

This chapter examines and identifies the potential trends between design speed and geometric elements. A number of design elements were examined in relation to the design speeds used. These elements included the radius and length of the curve, shoulder type and width, median type and width (when present), terrain type, approaching tangent length, AADT, and roadway width.

The trends that are present are examined using analysis of variance (ANOVA) procedure. The significance level used in all of the statistical analyses is set at 95%. All extreme or unreasonable data were identified and eliminated before the data analysis.

6.1 Horizontal Curve

6.1.1 *Curve Radius*

Traditionally, design speeds are selected to determine the minimum radii of horizontal curves. The minimum radii in relation to design speeds are provided in design manuals (AASHTO 2001). The general rule is that greater design speeds allow for larger curve radii. The data used in the study supported this assumption (Figure 6- 1). On the highways sampled in this study, it was found that greater radii of the horizontal curves were selected for highways with greater design speed.

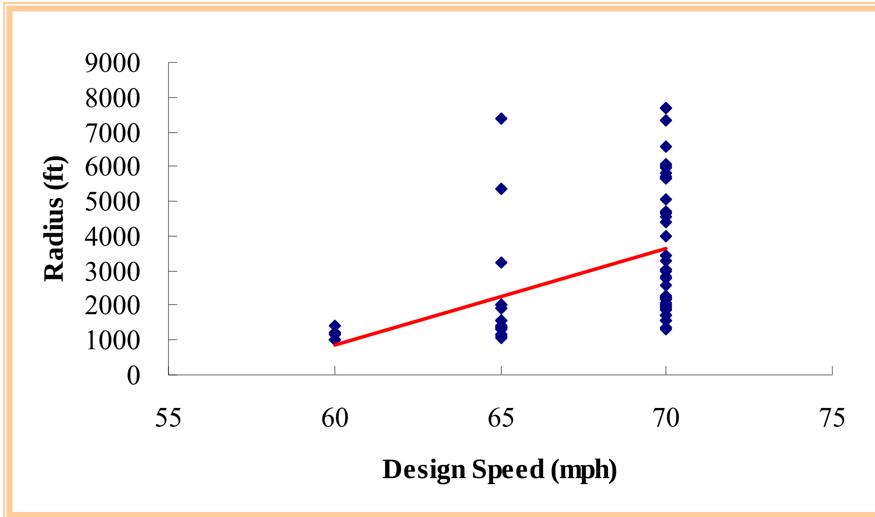

Figure 6- 1: Design Speed and Radius

The significance of the trend was examined by analyzing the correlation between design speed and the radius. The slope of the red line in Figure 6- 1 represents this trend. The null hypothesis was that there was no significant trend between the two variables (in other words, the slope rate is zero). Three extreme design speeds were identified and excluded. The SAS outputs of the analysis showed that the null hypothesis should be rejected at the 95% significance level since the p-value (<0.0001) was much less than 0.05, indicating that the trend was significant.

6.1.2 *Curve Length*

The trend between design speed and length of horizontal curve was also examined (Figure 6- 2). Shorter horizontal curves had lower design speeds. However, the statistical analysis showed that the hypothesis that the trend was not significant could not be rejected (p-value=0.2573), indicating that the trend was not significant. The trends for these two curve design elements indicate that the choice of the radius and, subsequently, the length of the curve are dependent of the design speed selected. These trends are similar to the trends observed on rural two-lane highways in Kentucky (Stamatiadis and Gong, 2006).

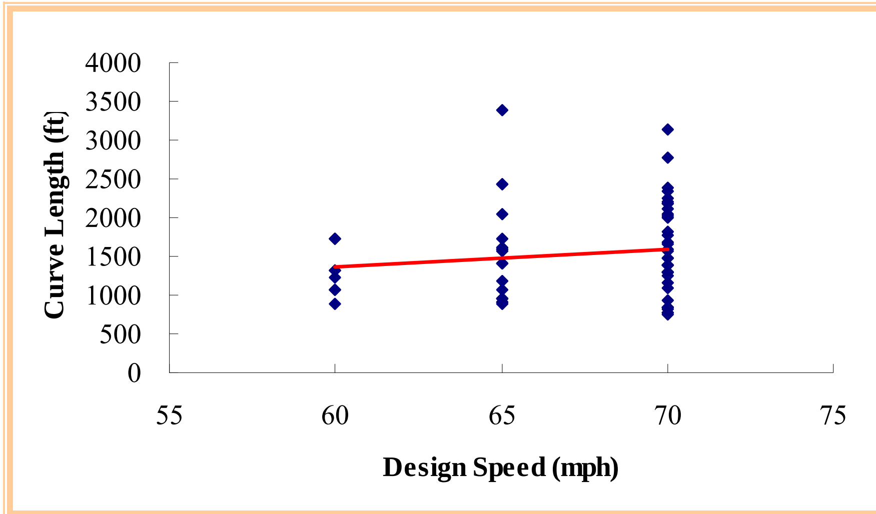

Figure 6- 2: Design Speed and Curve Length

6.2 Shoulder

6.2.1 *Shoulder Type*

On the highways sampled, all horizontal curves have right and left shoulders. Since the type of the left shoulders is always paved, in the analyses the shoulder type refers to the type of the right shoulder.

Shoulder types were classified into six categories in the HPMS Field Manual (HPMS, 2005). For the highways investigated here there were only three types: surfaced shoulder, stabilized shoulder, and combination shoulder. The sample sizes of the later two categories were only 1 and 6, respectively, and these two categories were combined into one category. Therefore there were only two shoulder types: Type 1-- surfaced shoulder; and Type 2-- stabilized or combination shoulder.

The analysis of variance showed that the design speeds selected for highways with different shoulder types were significantly different (p-value=0.041). The design speeds selected for highways with Type 2 (non surfaced) shoulder were significantly greater than these highways with Type 1 (surfaced) shoulder.

6.2.2 *Shoulder Width*

The trends between the design speeds and the right and left shoulder widths were analyzed. The data (Figure 6- 3 and Figure 6- 4) showed that narrower shoulders were used with greater design speeds. The trend between the design speeds and the left shoulder widths was more apparent than the trend in the relation to the right shoulder widths. The statistical analyses showed that the trend between the design speeds and the right shoulder widths was not significant (p-value=0.6059) as it can be observed from Figure 6- 3 too. Instead, the trend between the design speeds and the left shoulder widths was found significant (p-value=0.0135). One possible reason is the limited right-of-way. The trends for these two shoulder design elements indicated that the choice of the shoulder width is independent of the design speed selected.

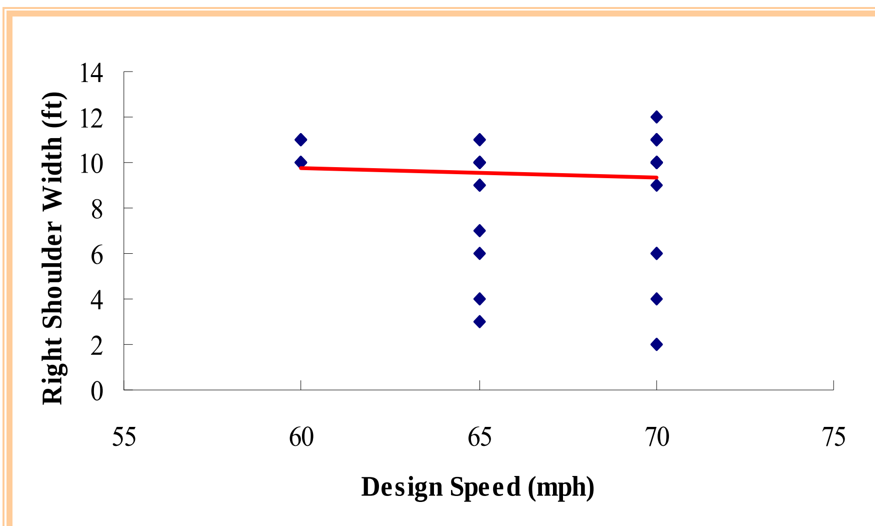

Figure 6- 3: Design Speed and Right Shoulder Width

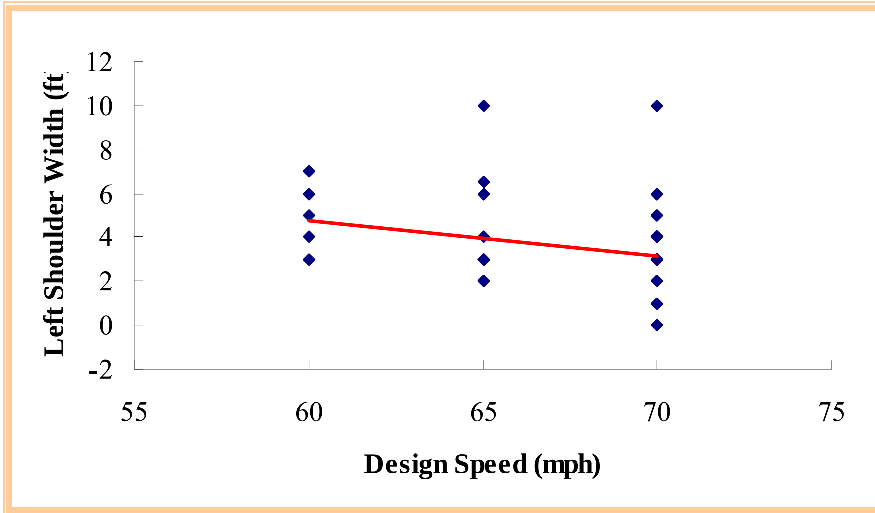

Figure 6- 4: Design Speed and Left Shoulder Width

6.3 Median

6.3.1 *Median Type*

Median types were categorized into four categories in the HPMS Field Manual (HPMS, 2005). For the highways investigated here there were only three types: positive barrier, unprotected, and none median. Since there was only one site in the category "none median", the last two categories were combined into one category. Therefore there were only two median types.

The results of ANOVA showed that the design speeds selected for highways with different median types were significantly different (p-value=0.007). The design speeds selected for highways with positive barrier median were significantly lower than those on highways with unprotected median.

6.3.2 *Median Width*

Three extreme data were identified and excluded from the analysis. The remaining data showed that wider medians resulted in greater design speeds (Figure 6- 5). The statistical analysis indicated that this trend was significant (p-value=0.0274).

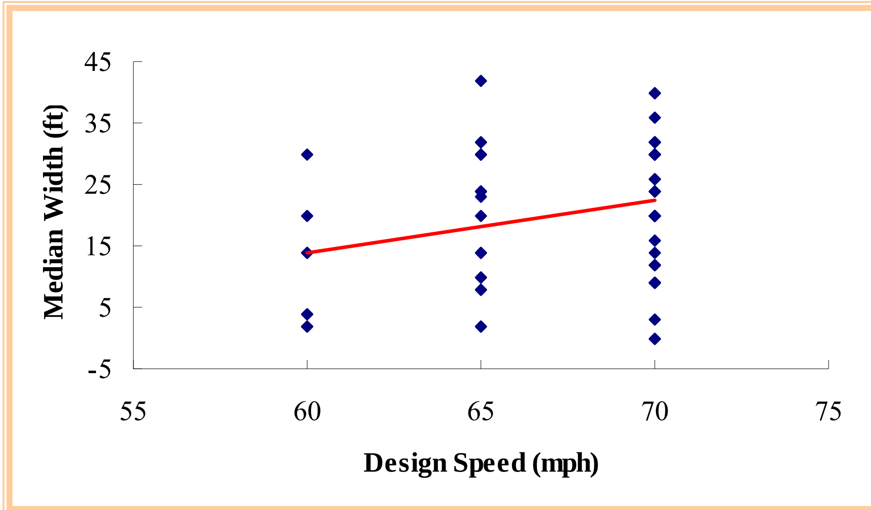

Figure 6- 5: Design Speed and Median Width

6.4 AADT

For rural two-lane highways in Kentucky, it was found that higher volumes resulted in greater design speeds (Stamatiadis and Gong, 2006). However, on the four-lane highways used here, an opposite trend was observed (Figure 6- 6). Higher volumes resulted in slightly lower design speeds. This trend was not clearly apparent. The statistical analysis also indicated that the trend was not significant at 95% significance level (p-value=0.2632). One possible reason is that the volumes (AADT in 2005) exceeded the volumes projected when the highways were designed.

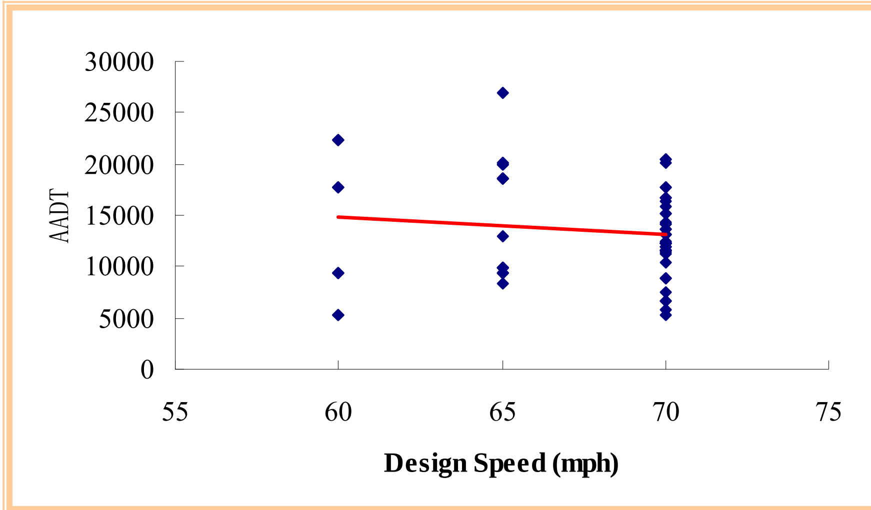

Figure 6- 6: Design Speed and AADT

6.5 Approaching Tangent Length

Minimum tangent lengths are recommended in the Green Book, since these tangents are used to accommodate superelevation runoffs when transition curves are not used and the roadway tangents directly adjoin the main circular curves. The general relationship between design speed and the minimum tangent length is that longer tangents allow for greater design speeds. The sampled data supported this approach. Longer tangents were used with greater design speeds (Figure 6- 7). The statistical analysis indicated that the trend was not significant when setting the significant level at 95% (p-value=0.0799).

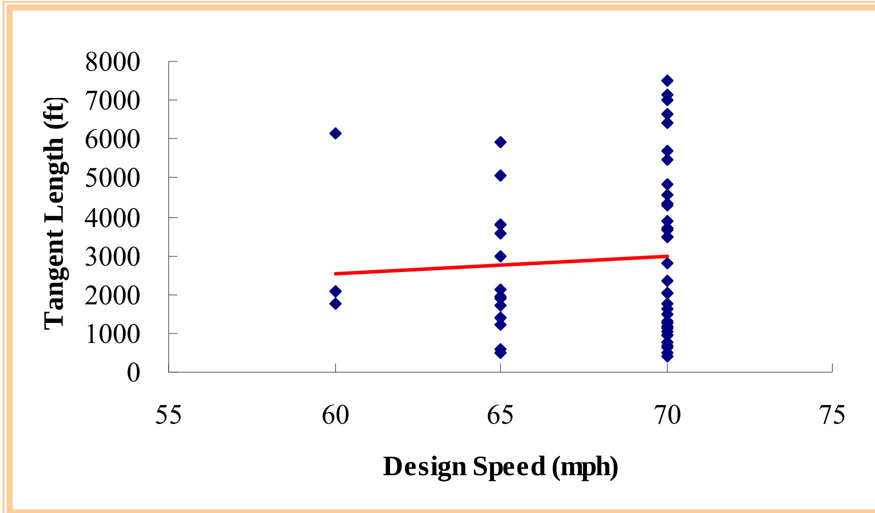

Figure 6- 7: Design Speed and Approaching Tangent Length

6.6 Roadway Width

The design speed data in relation to roadway width showed some trend (Figure 6- 8). Generally, the roadway widths increased as the design speeds increased. However, the trend was not clearly apparent. The statistical analysis also showed that the trend was not significant at the 95% significance level (p-value=0.1066).

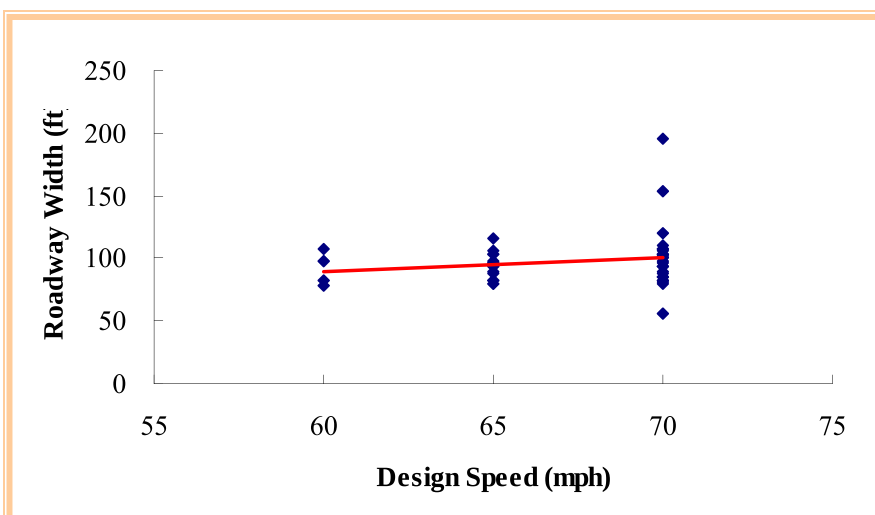

Figure 6- 8: Design Speed and Approaching Tangent Length

6.7 Terrain

For highways with same functional class, the design speeds are selected depending on terrain. The sampled horizontal curves are located across Kentucky. These sites can be grouped into three types of terrain: level, rolling, and mountainous. The results of ANOVA showed that the design speeds by the three terrain types were significantly different (p-value<0.0001). The design speeds were the highest for highways in level terrain while the lowest for highways in mountainous terrain. The trend is in agreement with highway design manuals.

6.8 Summary

The various relationships and trends between design speed and geometric elements are summarized in Table 6- 1 and Table 6- 2. The data showed that there are some relationships between design speed and the various geometric elements examined. Most of them seem to follow the general assumption that greater design speeds lead to larger values for the elements selected. However, some surprising and unexplainable opposite trends were also observed. These trends indicate that the choice of design speed does not impact the value chosen for the element. It could be assumed that these values are affected more by other parameters, such as terrain, location, and roadway context. All of these surprising and unexplainable opposite trends were found insignificant in statistical analyses.

Table 6- 1: Summary of Design Speed and Road Elements (Numerical)

Item	Trend	Significant
Curve Radius	+	y
Curve Length	+	n
Right Shoulder Width	-	n
Left Shoulder Width	-	n
Median Width	+	y
AADT	-	n
Approaching Tangent Length	+	n
Roadway Width	+	n

Notes: + operating speed changes in the same direction as element changes;
- operating speed changes in opposite direction as element changes;
y significant in statistical test;
n not significant in statistical test.

Table 6- 2: Summary of Design Speed and Road Elements (Categorical)

Item	Significant
Shoulder Type	y
Median Type	y
Terrain Type	y

Notes: y significant in statistical test.

CHAPTER 7: OPERATING SPEED AND ROAD ELEMENTS

Previous studies have investigated the impacts of road elements on operating speed on other types of roads. It has been found that the most significant road characteristics impacting operating speeds include curvature, grade, length of grade, number of lane, surface condition, sight distance, lateral clearance, number of intersections, and built-up areas near the roadway (Warren, 1982). This chapter investigates the impacts of road elements on operating speeds at horizontal curves on rural four-lane highways.

This chapter separately depicts the impacts of road elements on operating speeds for inside and outside lanes, since the operating speeds on both lanes are different. Before analyzing the impacts of each roadway element, outliers were identified and excluded. The ANOVA procedure was used for determining the impacts. It should be noted that in all statistical tests the significance level was set at 95%. Also, the impacts from the combination of two or more elements were not evaluated here.

7.1 Impacts for Inside and Outside Lanes

Ten road elements discussed here include: shoulder, median, roadway width, clear zone, pavement, approaching tangent and curve, grade, and horizontal curve. Since the posted speed limits were homogenous (all were set as 55 mph), the impacts of the posted speed limit are not discussed.

7.1.1 *Shoulder*

7.1.1.1 Shoulder type

The classifications of the shoulder type are same as the classifications in Chapter 6. Two shoulder types were used in the study. When identifying the operating speed outliers by shoulder type, it was found that there was only 1 outlier for inside lane (Figure 7- 1), and no outlier has been found for outside lane.

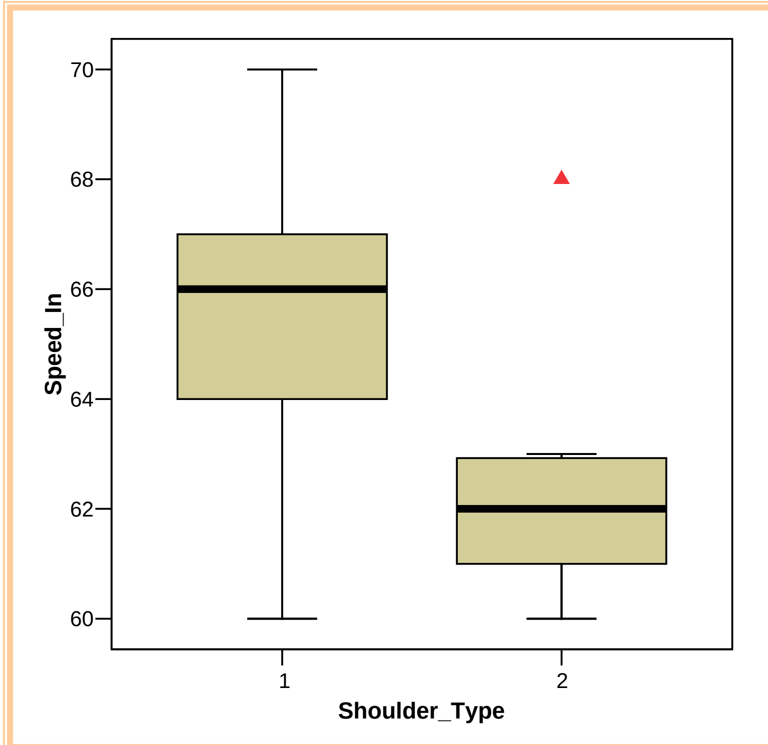

Figure 7- 1: Outliers by Shoulder Type (Inside Lane)

The one-way ANOVA analysis showed that for both inside and outside lanes the operating speeds classified by shoulder type were significantly different (p-value<0.0001 for inside lane and 0.0011 for outside lane). It indicated that the shoulder type had an impact on operating speed. The operating speeds on the roads with surfaced shoulder were apparently higher than on other types. This trend could be simply observed from Figure 7- 1[9] by comparing the locations of the boxes.

In chapter 6, it was found that the design speeds for highways with shoulder Type 1 were lower than these for highways with shoulder Type 2. The trends between the operating speeds and the shoulder types were opposite to the trends between the design speeds and the shoulder types.

[9] The figure for outside lane is not presented because it is similar to this.

7.1.1.2 Shoulder width

The widths of the right shoulders varied from 2 to 12 ft. Through statistical analysis, it was found that the impacts of right shoulder width for both inside and outside lanes were not significant (p-value=0.0861 for inside lane and 0.2766 for outside lane). However, some trends were observed on these highways. The operating speeds increased with the increase of right shoulder width (Figure 7- 2 and Figure 7- 3).

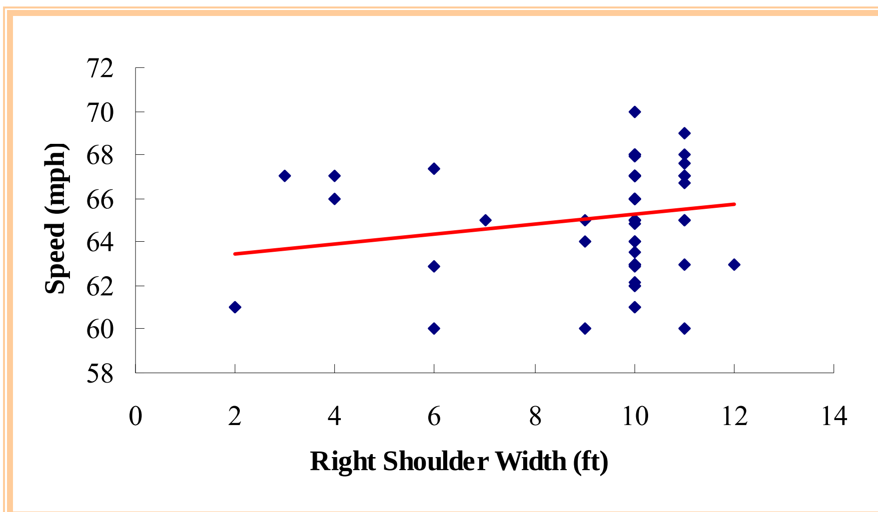

Figure 7- 2: Operating Speed and Right Shoulder Width (Inside lane)

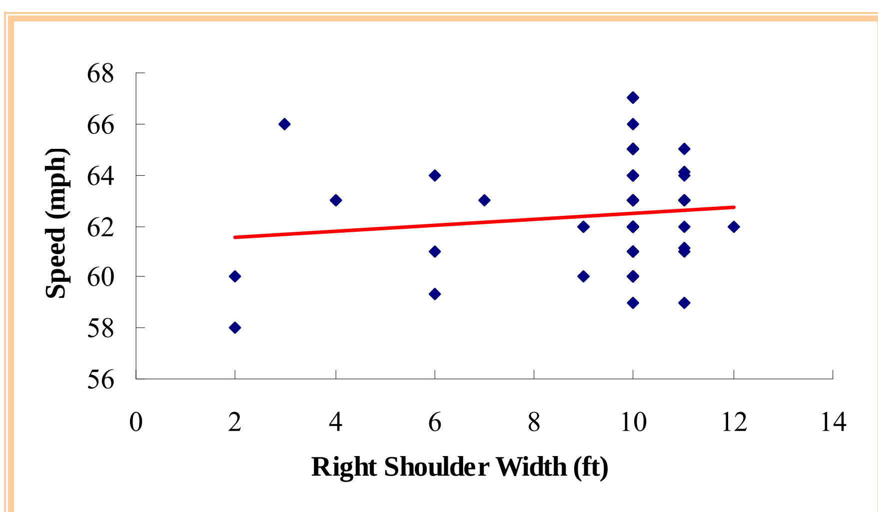

Figure 7- 3: Operating Speed and Right Shoulder Width (Outside Lane)

Left shoulder widths ranged from 0 to 10 ft. Two outliers were identified. After excluding the two outliers, it was found that the impacts for both inside and outside lanes were not significant and the same trends as right shoulder width were observed. The operating speeds increased as the left shoulder widths increased.

7.1.2 *Median*

7.1.2.1 Median type

The classifications of the median types are same as the classifications in Chapter 6. Two median types were classified in the study. Type 1 is positive barrier and Type 2 is unprotected or none median. Two outliers have been identified only for outside lane (Figure 7- 4).

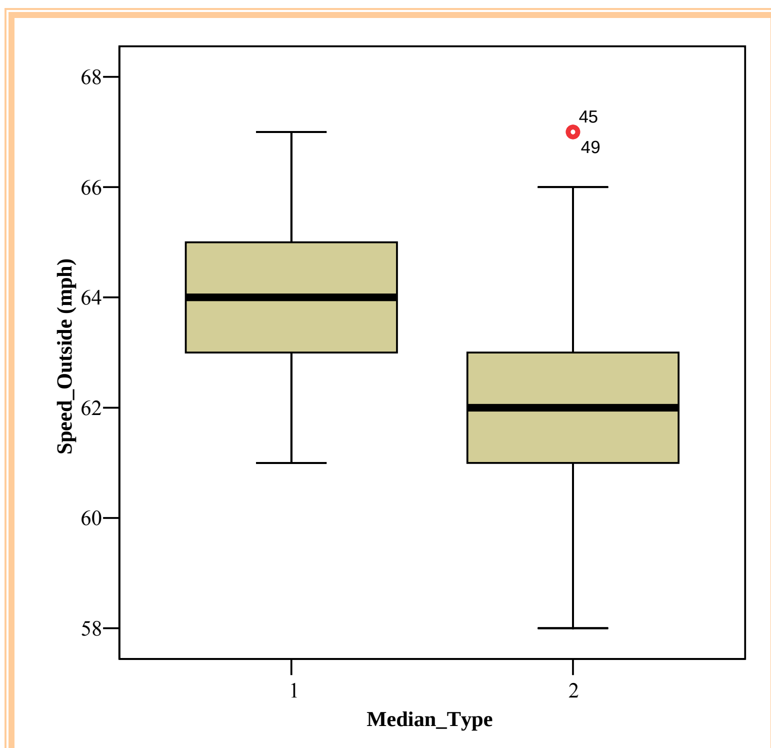

Figure 7- 4: Outliers by Median Type (Outside Lane)

The one-way ANOVA analysis showed that for both inside and outside lanes the operating speeds classified by median type were significantly different (p-value=0.001

for inside lane and 0.0005 for outside lane). It indicated that median type had significant impacts on operating speed. The operating speeds on roads with positive barrier were higher than these with other median types. Figure 7- 4 shows the different impacts on operating speed by comparing the two means.

7.1.2.2 Median width

Of the sample curves, three outliers were identified (Figure 7- 5). These roads had extremely wider medians than other roads investigated. Except for these three outliers, the median widths ranged from 0 to 42 ft. The mean of the median widths was 19.7 ft.

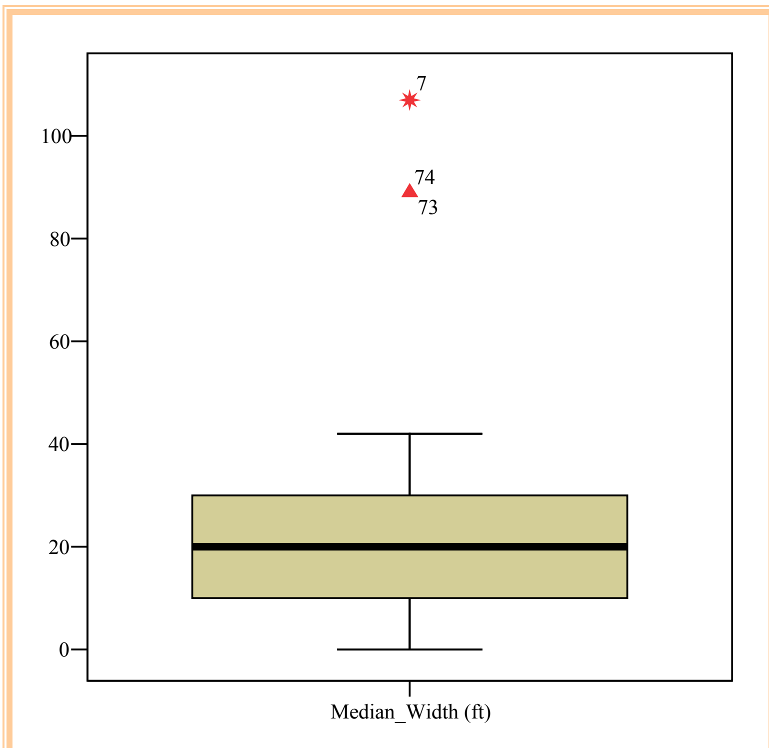

Figure 7- 5: Outliers of the Median Widths

The ANOVA results showed that for both inside and outside lanes the impacts of median width on operating speed were not significant at the 95% significant level (p-value=0.1030 for inside lane; 0.1815 for outside lane). However, there was a slightly surprising trend observed. The operating speeds on both inside and outside lanes decreased when the median widths increased. When comparing the change rates, it could be found that the impacts of median width on both lanes were significantly different. If increasing median width 1 foot, the operating speed on inside lane will decrease by 0.040 mph while a decrease of 0.029 mph was need for the outside lane. Both change rates were not statistically significant.

7.1.3 *Roadway Width*

Three outliers were also identified for the roadway widths (Figure 7- 6), which were resulted from the three outliers of the median widths. These outliers were excluded from the analysis. The roadway width range was from 56 to 120 ft. The mean of the roadway width was 93.6 ft.

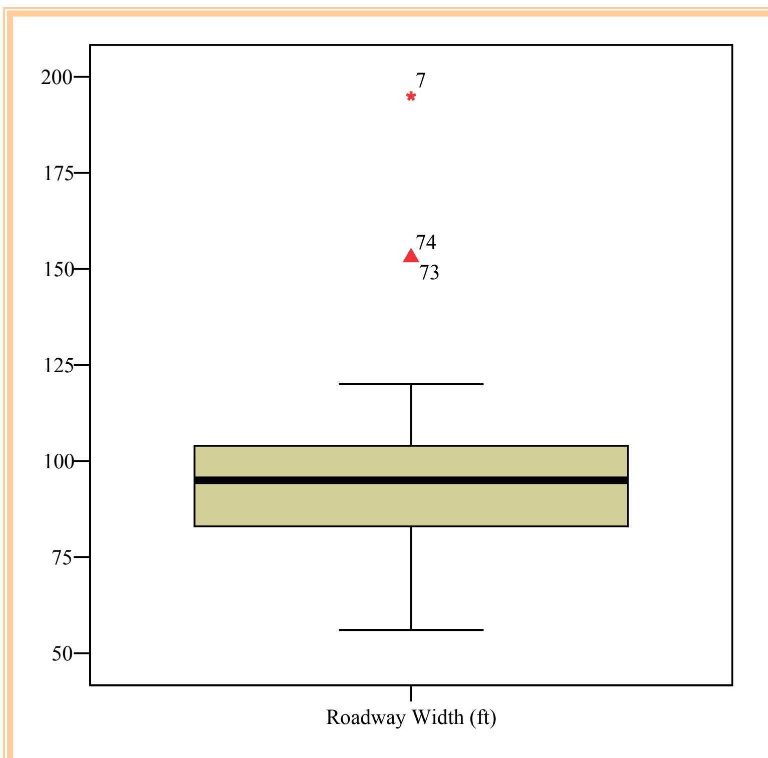

Figure 7- 6: Outliers of the Roadway Widths

The ANOVA results showed that the impacts of roadway width on both inside and outside lanes were not significant when setting the significance level at 95% (p-value=0.5237 for inside lane; 0.5709 for outside lane). This was the same trend as the one observed in median width. The operating speeds on both lanes decreased slightly when the roadway widths increased. The change rates were 0.01 mph on both inside and outside lanes and both change rates were not statistically significant. Therefore, it could be concluded that roadway width had no significant impacts on operating speed.

7.1.4 *Clear Zone*

The clear zone is defined as the roadside border area measured from the edge of the traveled lane that is available for the safe use by errant vehicles (AASHTO, 2001). This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. In the study, the clear zones corresponding to the inside and outside lanes were separately examined.

Since the right shoulders were wider than the left shoulders, the clear zones were categorized based on different criteria. The categories were defined as Table 7- 1. The right clear zone corresponds to outside lane while the left clear zone is used for the inside lane. The widths of the right clear zones at the beginning, middle, and ending points along a curve are not always uniform, and therefore the impacts of the clear zones at the three points (beginning, middle, and ending points) were considered separately. The widths of left clear zones at the three points were uniform.

Table 7- 1: Clear Zone Categories

Category	Right Clear Zone	Left Clear Zone
1	0~10 ft	0~5 ft
2	10~20 ft	5~10 ft
3	>20 ft	>10 ft

7.1.4.1 Left clear zone

One outlier was detected only for the outside lane when analyzing the left clear zone impacts on operating speed for both inside and outside lanes. Since the clear zone width was classified into three categories, there would be three comparisons. To avoid the experimentwise Type 1 error risk, the Post Hoc tests (multiple comparisons) should be used in this analysis. Dunnett's T3 test -- one of the Post Hoc tests was appropriate for the data because of the unequal group sizes and unequal variances of the data[10].

It is expected that operating speed increases with the increase of clear zone width. In general the trend shown in Figure 7- 7 was in agreement with such an expectation. The trends of the impacts on both inside and outside lanes were similar. The operating speeds increased as the clear zone widths increased.

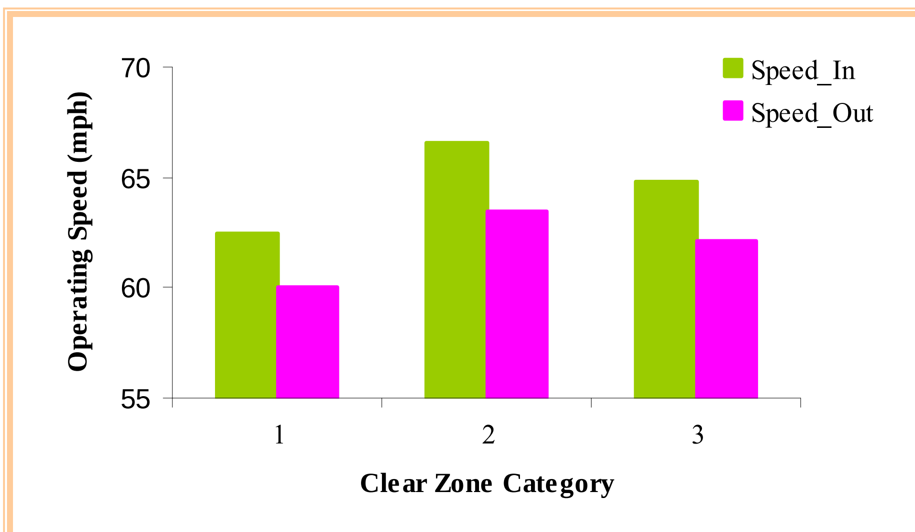

Figure 7- 7: Mean Speed and Left Clear Zone

The statistical analysis showed that left clear zone had impacts on operating speeds for both inside and outside lanes. The Dunnett's T3 tests showed that highways with category 2 clear zone had operating speeds significantly higher than these of sites with the other two clear zone categories, moreover, the operating speeds on these highways

[10] http://staff.harrisonburg.k12.va.us/~gcorder/test_post_hocs.html, visited on 30 April, 2007

with category 1 and 3 clear zones were statistically equal, which possibly was due to other factors such as grade and horizontal curve radius. It should be noted that the sample size in category 1 was only 4. Table 7- 2 and Table 7- 3 show the results of Dunnett's T3 tests for inside and outside lanes, respectively.

Table 7- 2: Dunnett's T3 Test for Inside Lane

(I) Clear Zone Category	N	Mean (mph)	Variance	(J) Clear Zone Category	Mean Difference (I-J) (mph)	Std. Error	p-value
1	4	62.50	3.667	2	-4.055 *	1.026	0.043
				3	-2.299	1.014	0.209
2	21	66.55	2.841	1	4.055 *	1.026	0.043
				3	1.756 *	0.497	0.003
3	49	64.80	5.454	1	2.299	1.014	0.209
				2	-1.756 *	0.497	0.003

Note: *--the mean difference is significant at the 0.05 level.

Table 7- 3: Dunnett's T3 Test for Outside Lane

(I) Clear Zone Category	N	Mean (mph)	Variance	(J) Clear Zone Category	Mean Difference (I-J) (mph)	Std. Error	p-value
1	4	60.00	2.667	2	-3.490*	0.911	0.037
				3	-2.091	0.852	0.181
2	21	63.49	3.435	1	3.490 *	0.911	0.037
				3	1.400 *	0.472	0.016
3	48	62.09	2.843	1	2.091	0.852	0.181
				2	-1.400 *	0.472	0.016

Note: *--the mean difference is significant at the 0.05 level.

7.1.4.2 Right clear zone

The impacts of the right clear zones at the beginning, middle, and ending points were examined separately. Trends opposite to these of left clear zone were observed. The operating speeds decreased as the right clear zone widths increased in general. However, theses differences were not statistically significant.

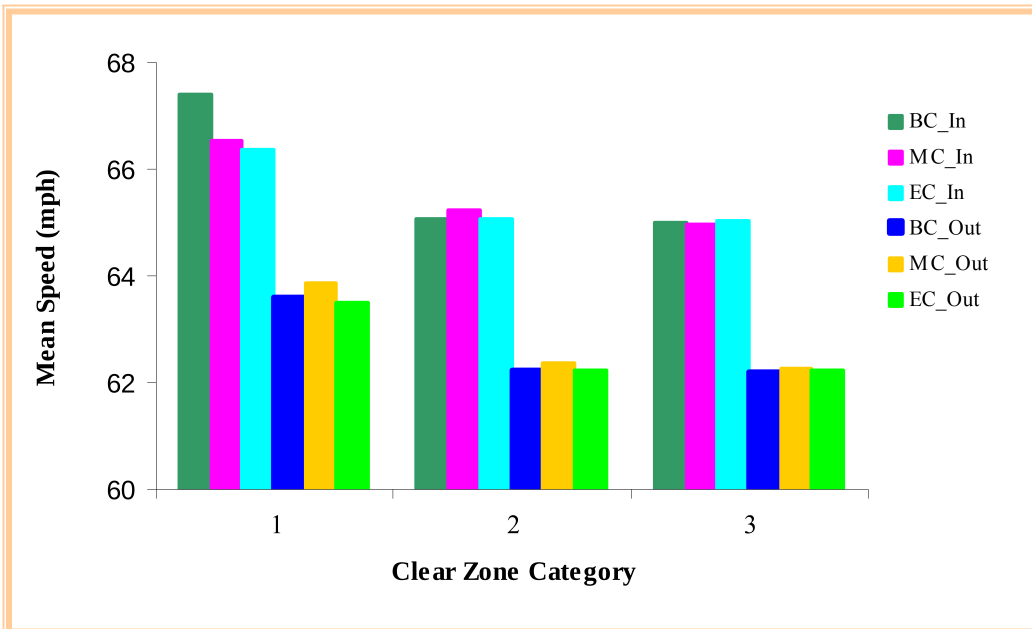

Figure 7- 8: Mean Speed and Right Clear Zone

At each of the three points, the sample sizes of the three clear zone categories were not same and the variances were not equal. Therefore the Dunnett's T3 test was appropriate for the analysis too. The results of the Dunnett's T3 tests showed that in general there was no significant difference in operating speed among the three clear zone categories when setting the significance level at 95%. It could be concluded that the width of right clear zone had no significant impacts on operating speed.

7.1.5 *Pavement type*

Pavement type was classified as two categories: bituminous and Portland cement concrete pavements. The statistical tests indicated that, for inside lane the impacts of pavement types were significant (p-value=0.0162); instead, for outside lane the impacts were not significant (p-value=0.1122). For inside lane, higher operating speeds were observed on these highways with bituminous pavement while lower operating speeds on these highways with Portland cement concrete pavement.

7.1.6 *Approaching Segment*

7.1.6.1 Tangent length

The impacts of approaching tangents on operating speed were examined. Two outliers were identified and excluded. The lengths of the tangents ranged from 390.51 to 7,512.58 ft[11]. The average length was 2,609.66 ft.

No apparent trends were observed from the scatter plot of operating speed and tangent length (Figure 7- 9). The ANOVA results showed that for both inside and outside lanes there was no significant changes in operating speed when tangent lengths changed (p-value=0.7297 for inside lane; 0.2582 for outside lane). Therefore it could be concluded that tangent length had no significant impacts on the operating speed on horizontal curve.

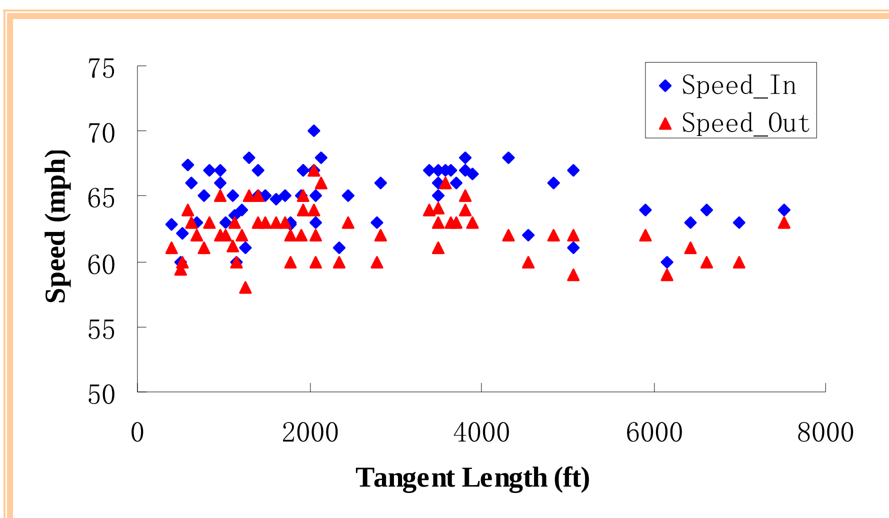

Figure 7- 9: Operating Speed and Tangent Length

[11] The length of an approaching horizontal curve was not considered.

7.1.6.2 Tangent grade

In the Kentucky HIS database, highway grades were categorized without "±". The tangent grade referred to the average grade of all sections with different grade within a tangent, if there were several grades within the tangent. The grades were classified into three categories since few tangents with grade greater than 4.5%. The first two categories were in agreement with the classifications in Kentucky HIS.

Table 7- 4: Tangent Grade Categories

Category	Grade
1	0~0.5%
2	0.5%~2.5%
3	>=2.5%

The ANOVA results showed that, when grades were less than 0.5%, there were no significant impacts on operating speeds for both inside and outside lanes; when grades ranged from 0.5% to 2.5%, the impacts for both lanes were significant; however, when grades were greater than 2.5%, the impacts for outside lane were significant while insignificant for inside lane. One possible reason was lack of the direction "±". In general, when grades were greater than 0.5% operating speeds decreased as grades increased.

Table 7- 5: ANOVA Test for Tangent Grade

		p-value	
Category	**Grade**	**Inside Lane**	**Outside Lane**
1	0~0.5%	0.3065	0.9343
2	0.5%~2.5%	0.0098	0.0076
3	>=2.5%	0.1676	0.0342

7.1.6.3 Curve Presence

Preceding statistical analysis showed that for both inside and outside lanes approaching tangent length did not significantly affect operating speed while tangent grade did significantly affect operating speed. When the approaching section was a horizontal curve, it was found that the impacts for both inside and outside lanes were different. For outside lane, the impacts of the curves were not significant at the 95% significance level (p-value=0.1180). Instead, for inside lane, the impacts of the curve were significant (p-value=0.005). The operating speeds on inside lane in the curves with the presence of an approaching curve were higher than these with the presence of an approaching tangent. There are possible two reasons. One reason is that in this study no sharp curves or curves with limited sight distances were examined. The second possible reason is that these consecutive horizontal curves are well designed curves that allow for a much smoother flowing horizontal change.

7.1.7 ***Horizontal Curve***

7.1.7.1 Curve radius

Previous studies have found that curve radius was a significant factor that affects operating speed on rural two-lane highways. Operating speed was linear to the reciprocal of the curve radius. In this study, the curve radii ranged from 540 to 7,705 ft. The median of the curve radii was 2,000 ft. The scatter plots for inside and outside lanes showed that operating speeds increased as the radii decreased (Figure 7- 10 and Figure 7- 11). However, when using ANOVA analysis, it was found that these trends were not significant at 95% confident level for both inside and outside lanes. It indicated that the trend existed but it was not significant.

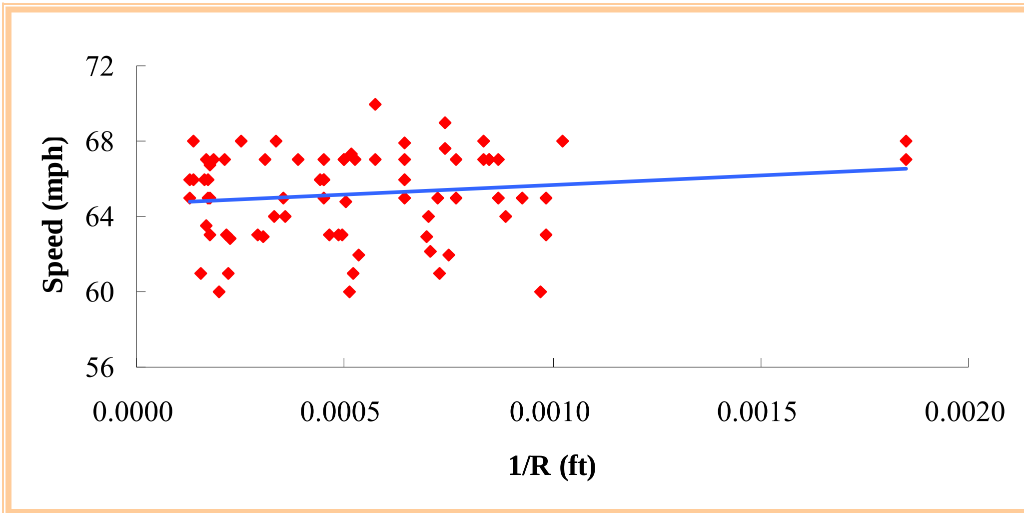

Figure 7- 10: Operating Speed and Radius (Inside Lane)

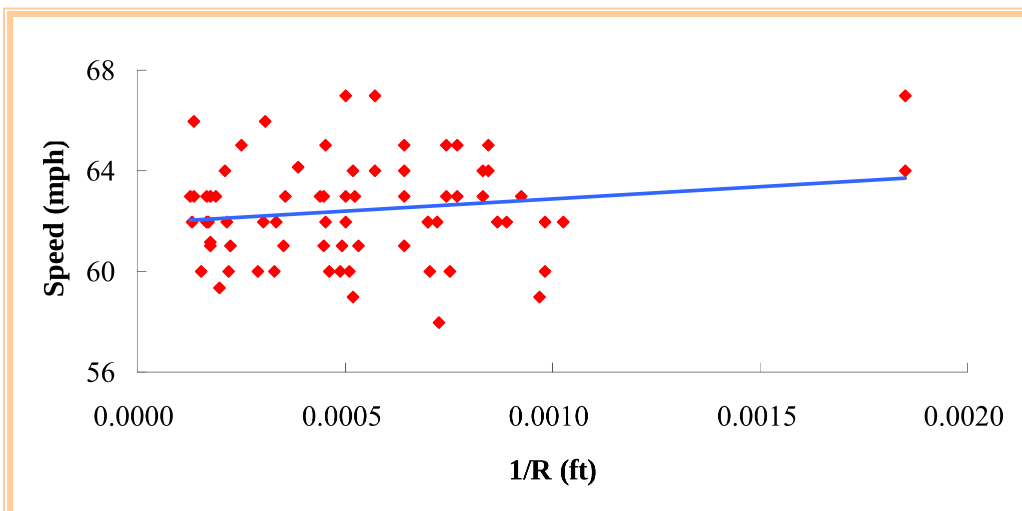

Figure 7- 11: Operating Speed and Radius (Outside Lane)

7.1.7.2 Curve length

Five outliers were identified and eliminated from the analysis. The curve lengths ranged from 755.82 to 3,391.24ft. The median of the curve lengths was 1,396.22 ft. Both scatter plots (Figure 7- 12 and Figure 7- 13) showed an apparent trend that operating speeds increased with the increase of curve length.

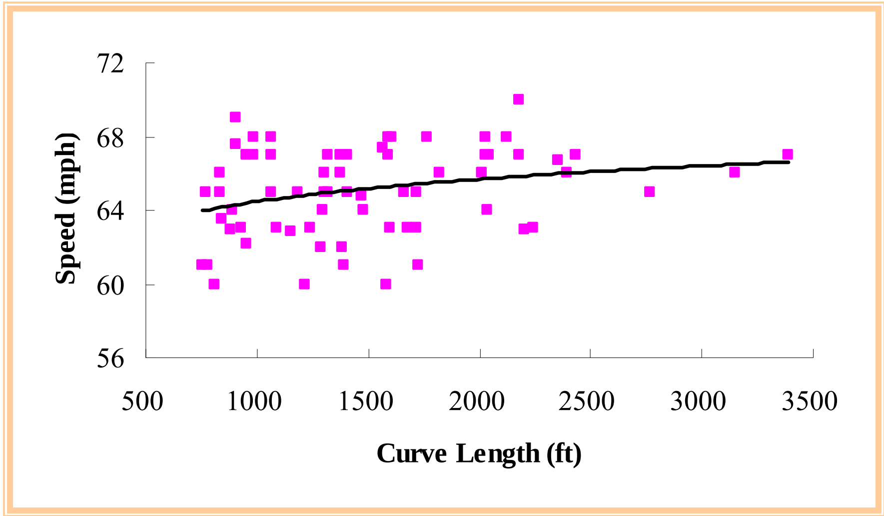

Figure 7- 12: Operating Speed and Curve Length (Inside Lane)

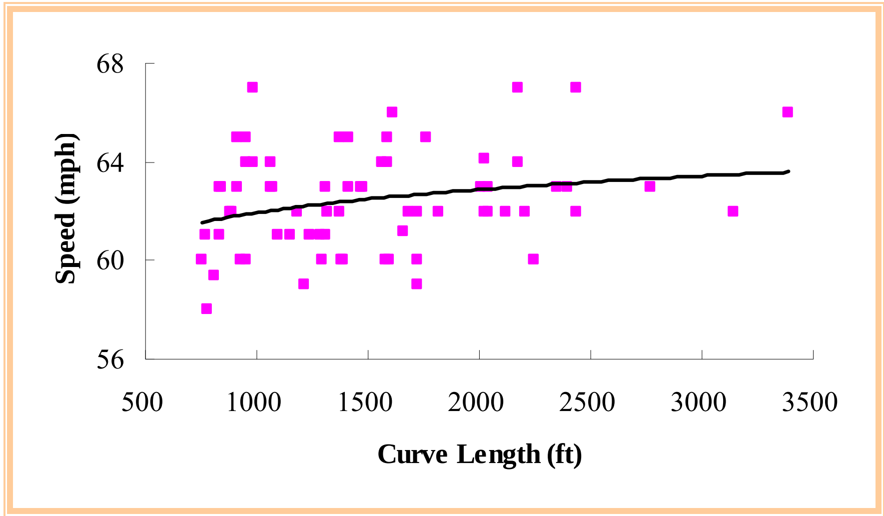

Figure 7- 13: Operating Speed and Curve Length (Outside Lane)

The ANOVA tests showed that both increase trends were significant because the p-values (0.0246 for inside lane and 0.0346 for outside lane) were less than 0.05 when setting the significance level at 95%.

7.1.7.3 Curve grade

Since the grades varied within some curves and some of the curves were long, each of the horizontal curves was evenly divided into three sections. In a horizontal curve, grades close to the beginning, middle, and ending points were used as the grades of the three sections, respectively. The purpose of dividing a curve in this manner was to study the impacts of vertical grades on operating speed in horizontal curves at specific points of the curve.

The grades were also extracted from Kentucky HIS database. The classification method was same as in the preceding section (tangent grade). The ANOVA results showed that, for all the grades when grades were less than 0.5%, no significant impacts on operating speed was detected; when grades were greater than 2.5%, there was no significant impacts observed neither, which possibly resulted from the inaccuracy of the data; when grades ranged from 0.4% to 2.5%, certain impacts were observed. Generally the operating speeds decreased as the grades increased (Figure 7- 14).

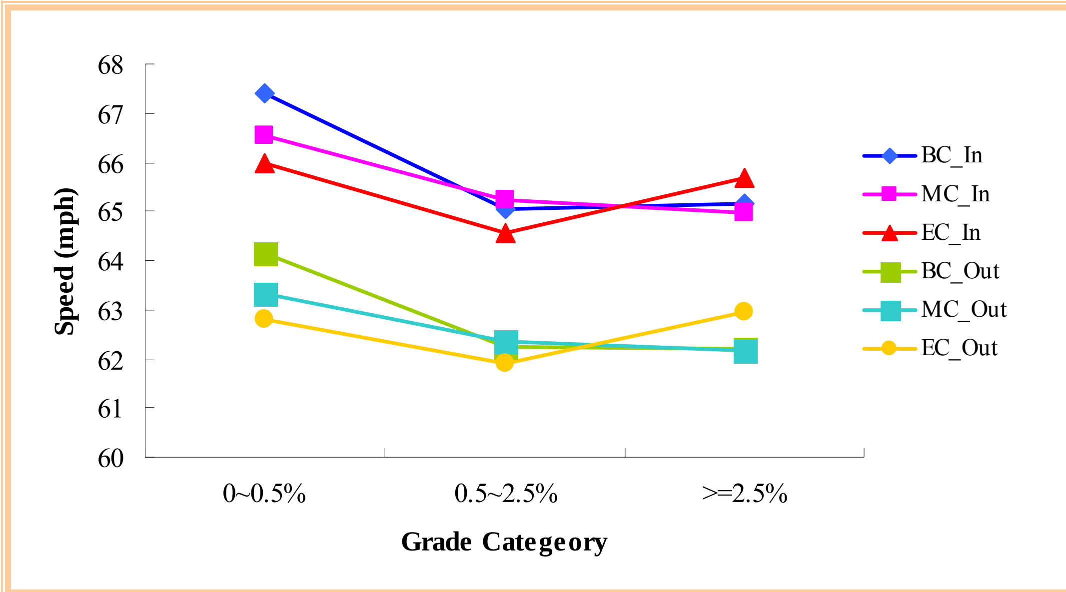

Figure 7- 14: Mean Speed and Curve Grade

Previous statistical analyses indicated that the operating speeds at the three points were equal. In comparison to the trends related with curve grades (Figure 7- 14), it could be found that the trends in Figure 7- 14 were not in agreement with the previous conclusions in past research.

7.2 Summary

The various relationships and trends between operating speed and road elements are summarized in Table 7- 6 and Table 7- **7** . The data showed that greater values generally resulted in higher operating speeds. There are some surprising trends such as the trend between operating speed and right clear zone. These trends however are not apparent or statistically significant.

Table 7- 6: Summary of Operating Speed and Road Elements (Numerical)

	Trend		
Element	**Inside Lane**	**Outside Lane**	**Significant**
Right Shoulder Width	+	+	n
Left Shoulder Width	+	+	n
Median Width	o	o	
Roadway Width	o	o	
Left Clear Zone	+	+	y
Right Clear Zone	-	-	n
Approaching Tangent Length	o	o	
Approaching Tangent Grade	-	-	y
Curve Radius	-	-	n
Curve Length	+	+	y
Curve Grade	-	-	y/n

Notes: + operating speed changes in the same direction as element changes;
- operating speed changes in opposite direction as element changes;
o no apparent trend between operating speed and element;
y significant in statistical test;
n not significant in statistical test.

Table 7- 7: Summary of Operating Speed and Road Elements (Categorical)

	Significant	
Element	**Inside Lane**	**Outside Lane**
Shoulder Type	y	y
Median Type	y	y
Pavement Type	y	y
Approaching Curve	y	n

Notes: y significant in statistical test; and
n not significant in statistical test.

CHAPTER 8: MODEL DEVELOPMENT

8.1 The Model Development Procedure

Some studies on rural two-lane highways used simple linear regression for developing operating speed-prediction models. In this study, both simple linear and multiple regression methods were used. The purpose was to obtain the best model by comparing the simple linear regression models and the multiple linear regression models. The model development procedure is shown as Figure 8- 1.

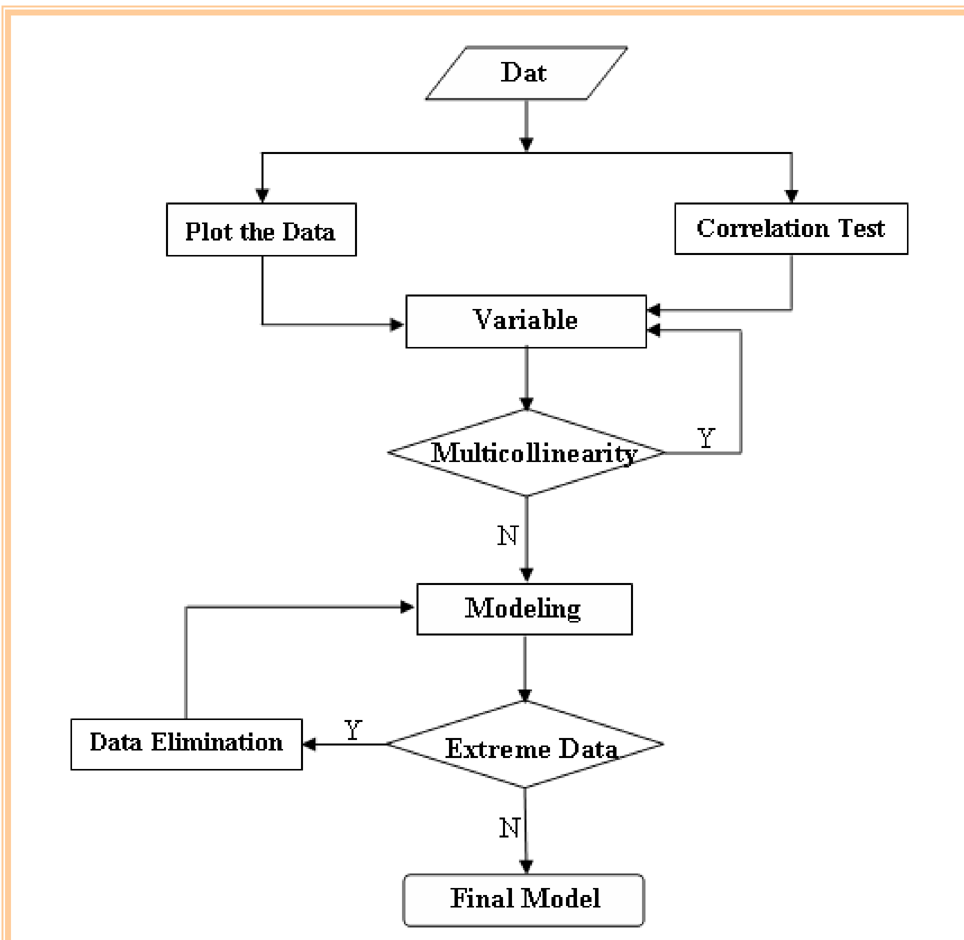

Figure 8- 1: Model Development Procedure

Scatter plots were used to identify possible relationships between the independent variables and the 85th percentile speed. Using the available variables, possible regression models were then developed. The statistic C_p, the coefficient of determination R^2, and the adjusted coefficient of determination R^2_{adj} would be used to select candidate variables. At the same time, multicollinearity among the candidate variables based on the regression models should be examined for reducing potential bias. The variance inflation factor (VIF) would be used to test multicollinearity. The models with high R^2_{adj} (using R^2 in simple linear models) and appropriate C_p then could be chosen.

In the data reduction step, it was difficult to identify extreme data like leverage data through scatter plot. Extreme data would be checked on basis of statistical modes and traffic engineering judgment. Cook's distance (Cook's D), studentized residuals (RSTUDENT), and the hat matrix (Hat Diag H) would be used to detect such extreme data. If extreme data exists, then the extreme data would be eliminated and the models should be redeveloped. To fit curves to data, the Box-Cox procedure would be used to identify whether it is necessary to transform variables to exponential or logarithmic curves. The final models would then be obtained following these procedures.

The coefficient of determination R^2 describes how much the independent variables associated with a model can explain the dependant variable. High values of R^2 indicate good regression models. However, R^2 does not account for the number of variables in a multiple regression model. As the number of variables increase, so does R^2. Therefore it is difficult to compare multiple regression models with different numbers of variables by simply using R^2. The adjusted coefficient of determination R^2_{adj} is a better criterion compared to R^2 in a multiple regression model because it also considers the numbers of variables. Higher values of R^2_{adj} usually indicate better fit regression models.

The C_p criterion measures the total mean square error of the fitted values of the regression. The total mean square error includes two components: one from random error, and another from bias. When no bias exists in an estimated regression model, the desired value of C_p is close to the number of coefficients to be estimated. It is recommended that regression models with small C_p value that is close to the number of coefficients are the best models. If the value of C_p is much larger than the number of coefficients, a larger bias is present. Models generated a C_p value larger than 10 usually indicate that important variables are lost when only one explanatory variable exists. A model with a high R^2_{adj} value and C_p value close to the number of coefficients would well explain the variability of the dependent variable, and therefore could be considered a “reasonable” model.

8.2 Data Splitting

The R^2 or R^2_{adj} of a regression is a measure of the fit of the regression to the sample data. They are not considered adequate measures of the regression model’s ability to assess the quality of prediction (Dielman, 2001). For assessing the prediction quality, the data splitting method is used in this study. In this method, the data set is partitioned into two groups. One group of N_1 samples is used to develop models for predicting. The second group of N_2 samples is used to assess predictive ability of the models. The mean square forecast error and the mean absolute forecast error are two commonly computed measures for model validation. Usually, when the sample size is huge, the data set is partitioned evenly. When it is small, some statisticians suggested using more samples to develop models and the remaining to validate (Lattin et al. 2003).

Using a macro program, 50 of the 74 sample curves were randomly selected to develop the models in this study. The remaining 24 sample curves were used to validate the models. It should be noted that 26 curves came from the 13 sites which were selected for examining the speed relationships between the two directions of travel. The two curves at each of the 13 sites were considered separately, since some of the roadway features were not exactly the same, for example, the right shoulder width, shoulder type, clear zone, presence of the approaching curve, grade of the approaching tangent, and so forth.

8.3 Model Development

Preceding statistical analyses showed that the operating speeds on inside and outside lanes were significantly different and on each of the two lanes, the speeds at the beginning, middle, and ending points were statistically equal. Therefore, two separate models were developed for each lane, i.e. inside and outside lane. In this study, the estimated 85th percentile speeds refer to the operating speeds at the middle points of the curves.

Each model was developed based upon the model development procedure presented in Figure 8- 1. The best variables capable of predicting operating speed were selected among all possible variables. These variables included the AADT, shoulder type, right shoulder width, left shoulder width, pavement type, median type, median width, left clear zone, right clear zone, approaching tangent length, approaching tangent grade, presence of approaching curve, radius of curve, length of curve, and road width. A model was developed for each variable alone as well as combinations of variables. Each model was evaluated and its ability to predict operating speeds was determined. The most appropriate model was then selected as the "best" prediction.

8.3.1 *Inside Lane*

The scatter plots for all variables considered were examined to determine potential relationships between operating speed and geometric features. All variables showed a graphical relationship. Some of the scatter plots indicated that some variables needed to be transformed since the linear relationship was not clearly apparent. Of the interest was the curve itself and its two curve elements (curve radius and length). Their scatter plots are shown in Figure 8- 2 and Figure 8- 3. Since the number of the scatter plots is big, other scatter plots are not presented here.

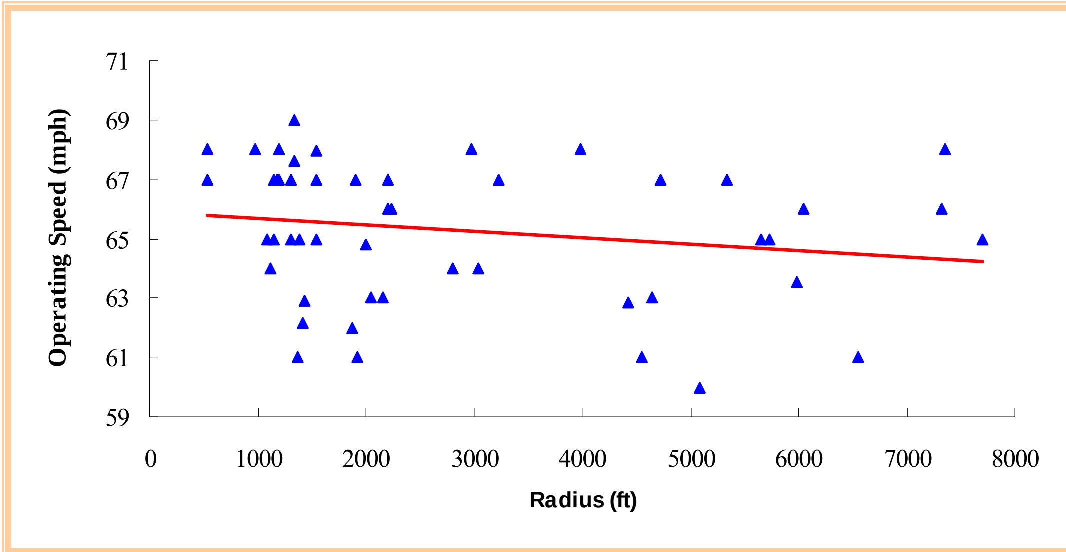

Figure 8- 2: Operating Speed and Radius

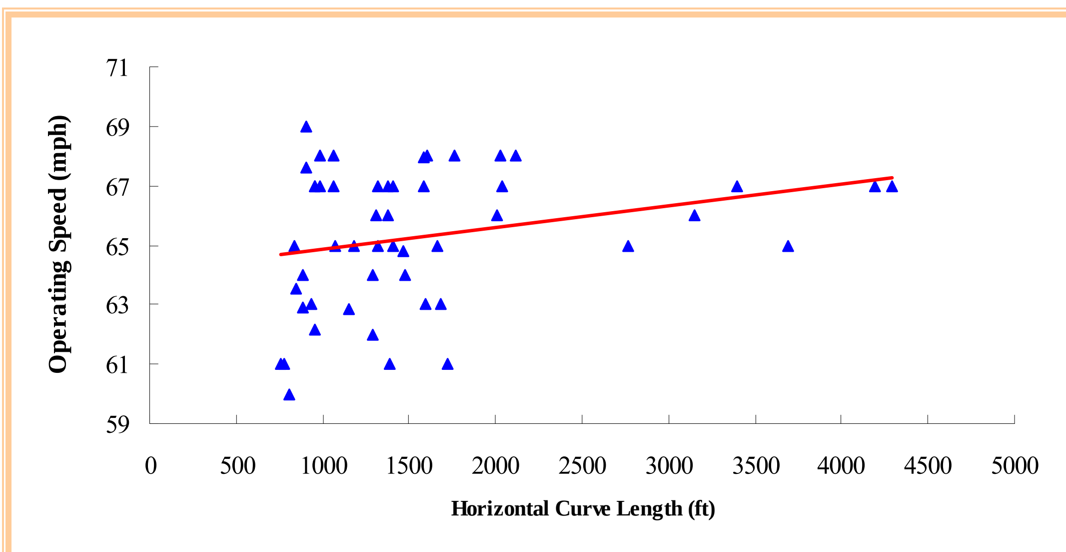

Figure 8- 3: Operating Speed and Curve Length

Previous studies on rural two-lane highways found that a greater horizontal curve radius resulted in a greater operating speed. The sampled data did not clearly support this finding. Moreover, the correlation test showed that the trend was not significant (p-value=0.1798) at the 95% significance level. When examining the scatter plot of the curve length, it was found that longer horizontal curves resulted in greater operating speeds. The two opposite trends indicated that operating speed at horizontal curves on rural four-lane highways was not determined only by radius. The operating speed should be determined by multiple factors. This hinted that the multiple regression models might be more appropriate for modeling.

The next step aimed in identifying the statistically supported relationships of these variables. Individual correlation tests were conducted to identify possible linear relationships between the variables and the 85th percentile operating speed. The scatter plots and the correlation tests also provide hints regarding whether the variables should be transformed to fit the data. At the 95% significance level, a few variables showed statistical significance and are shown in Table 8- 1. It should be noted that the final model does not have to include all the variables or the variables only from this table, since some variables may interact. Also, the trends of some variables were not reasonable although the p-values were less than 0.05. For example, the trends of the median width and roadway width have a low p-value but their trends are not reasonable, as noted in the previous section.

Table 8- 1: Significant Factors for Prediction

Variable	**p-value**
Right Shoulder Width	0.0231
Shoulder Type	0.0011
Median Type	0.0008
Median Width	0.0013
Pavement Type	0.0177
Design Speed	0.0094
Approaching Curve	0.0313
Inverse of Radius	0.0328
Curve Length (Ln)	0.0322
Roadway Width	0.0237
Left Clear Zone Width	0.0008

The stepwise procedure showed that there were six variables satisfying the significance level for entry into model. The statistic metrics C_p and R^2 were calculated for all possible models. This analysis indicated that the models using as predictors the shoulder type, median type, pavement type, grade of the approaching section, radius (inverse), and curve length (transformed as natural logarithm) have low C_p and high R^2 values. After checking the p-value of possible variables in all models, it was found that the best variables were the shoulder type, median type, pavement type, grade of the approaching section, and curve length (transformed as natural logarithm) (model has a C_p of 8.4044 and R^2 0.6836).

It is necessary to examine the multicollinearity among the variables since the best predictors included several variables. A high degree of multicollinearity among the explanatory variables usually results in disproportionately large standard deviations of the regression coefficients and unstable regression coefficient estimates. After computing the variance inflation factors (VIFs), it was found that there was no multicollinearity among the variables since all VIFs were less than 10 and the average VIF was not considerably larger than 1.

When detecting extreme data, it was found that there were 2 sites with absolute RStudent-value greater than 2, which indicates that possibly unusual speeds have been observed. After examining the data of the two sites, it was found that it was unnecessary to exclude the two sites since all speeds observed at the sites are normally distributed. Therefore, all 50 sample curves selected for the model development were used to develop operating speed-prediction models. The final model for inside lane is:

$$V_{85} = 50.937 - 1.567ST - 2.795 \times MT - 4.000 \times PT + 2.150 \times AG + 2.221 \times Ln(LC)$$

Where:

V_{85} = the 85^{th} percentile speed (mph)

ST = shoulder type index (if the type is surfaced, ST = 0, else, ST = 1)

MT = median type index (if the type is positive barrier, MT = 0, else, MT = 1)

PT = pavement type index (if the type is bituminous, PT = 0, else, PT = 1)

AG = approaching section grade index (if the absolute grade < 0.5%, AG = 1, else, AG = 0)

LC = length of curve (ft)

$R^2 = 0.6836$

$R^2_{adj} = 0.6477$

Mean error: 1.38 (mph)

The SAS outputs are shown in Appendix 4. It should be noted that, the grade of the approaching section is the grade of the approaching section directly connected with the curve, regardless if the section is a curve or tangent.

Since the model was developed based on the 50 curves selected randomly, there are several limitations of the model that should be noted here:

- This model is only applicable for sections with a horizontal curve. The range of radius for this model is between 538 and 7,704 ft.
- The range of lengths of horizontal curves is from 775 to 5,780 ft.
- The range of AADT for this model is 5,220-26,900. The use of this model for roadway sections outside of these ranges is not recommended without any additional validation.
- The range for design speeds was between 40 to 70 mph. As noted above, the use of this model for sections beyond these ranges should be conducted cautiously.

8.3.2 *Outside Lane*

All variables showed an apparent linear relationship. The scatter plots of the curve radius and length showed similar trends as those observed for the inside lane (Figure 8- 4 and Figure 8- 5). The operating speeds decreased as the curve radii increased. Instead, the operating speeds increased as the curve lengths increased. The correlation tests indicated that both trends were not significant. These two scatter plots also indicated that the operating speeds were not determined by a single factor.

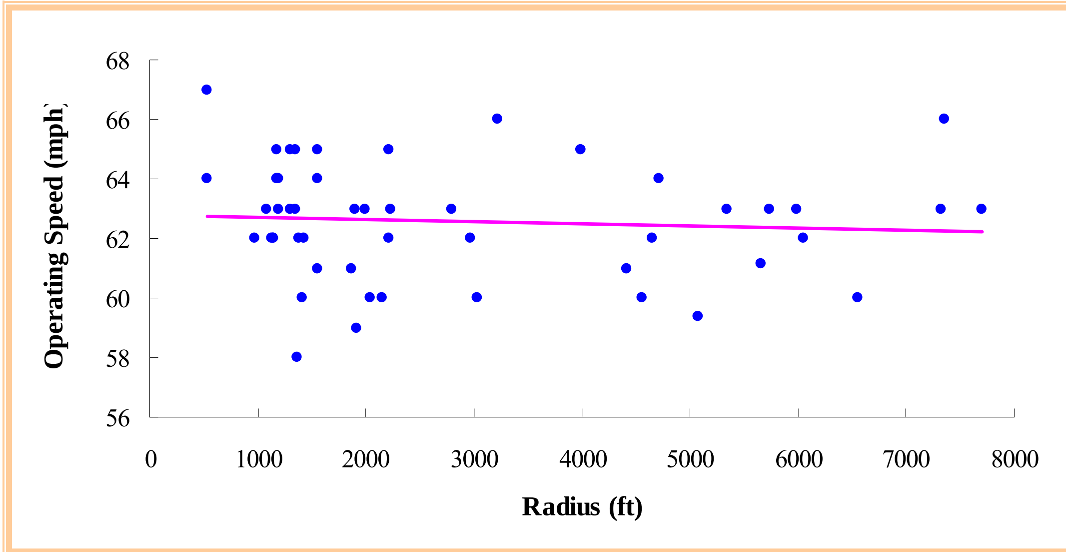

Figure 8- 4: Operating Speed and Radius

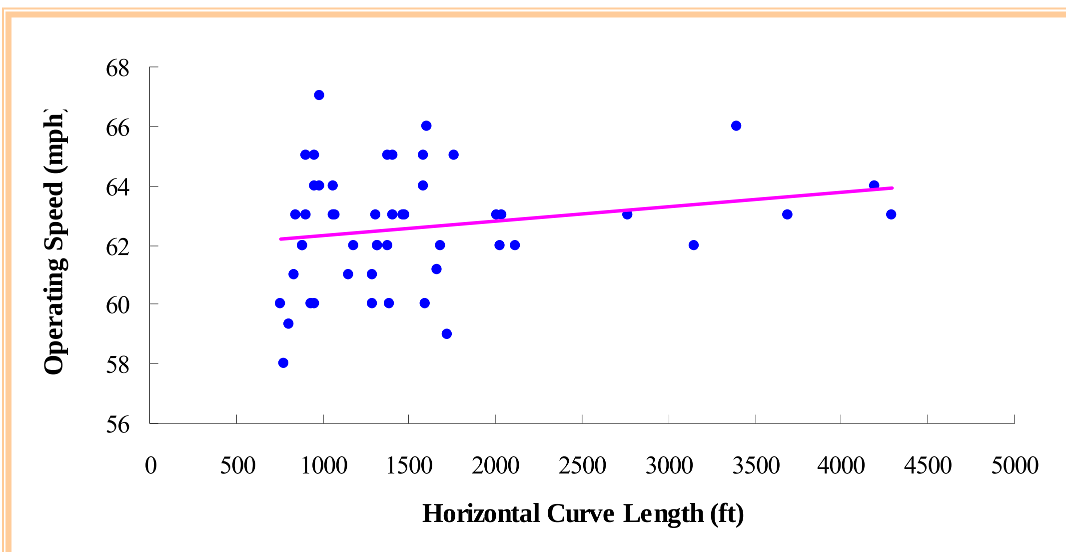

Figure 8- 5: Operating Speed and Curve Length

The individual correlation tests conducted to identify possible linear relationships between the variables and the 85th percentile speed showed that at the 95% significance level, nine variables had low p-values (Table 8- 2). These variables are different from the variables identified for the inside lane model.

Table 8- 2: Significant Factors for Prediction

Variable	**p-value**
Shoulder Type	0.0011
Median Type	0.0049
Median Width	0.0243
Design Speed	0.0063
Tangent Grade	0.0272
Left Clear Zone Width	0.0012
Right Clear Zone Width (Beginning)	0.0161
Right Clear Zone Width (Middle)	0.0284
Right Clear Zone Width (Ending)	0.0161

The above table showed that the significant factors did not include any of the curve elements. After transforming the curve radius and length, the two variables were still not significant. In the preceding chapters, it noted that these variables might interact. One alternative method to create a compound effect is crossing the two variables. The term "crossing" refers to multiplying two regressors. Therefore in this model development procedure, the two variables curve radius and length were crossed.

Using the Stepwise systematic variable search technique, which evaluates all possible variable combinations in a model, it was suggested that eleven variables met the significance level for entry into the model. After calculating the statistic C_p and R^2 for all possible models, it was determined that the model made up of six variables[12] has low C_p and high R^2 values, which indicated that this model has low bias. After checking the p-value of possible variables in all models, it was found that the best variables are the six variables (model has a C_p of 7 and R^2 0.5015). Among the variables, no multicollinearity was detected.

[12] One of the six variables was created by crossing the curve radius and length.

When detecting extreme data, it was found that there were four sites with absolute RStudent-value greater than 2, which were similar to the sites detected in preceding model development procedure. All 50 sample curves were suitable to develop operating speed-prediction models.

Using the speed data from the 50 sample curves, the best model was obtained. These eight variables could mostly explain the operating speed at 50.15%. The final model for outside lane is:

$$V_{85} = 60.779 + 1.804 \times ST - 2.521 \times MT - 1.071 \times AG - 1.519 \times FC + 0.000472 \times R + 2.408 \times \frac{LC}{R}$$

Where:

V_{85} = the 85^{th} percentile speed (mph)

ST = shoulder type index (if the type is surfaced, ST = 1, else ST = 0)

MT = median type index (if the type is positive barrier, MT = 0, else, MT = 1)

AG = approaching section grade index (if the absolute grade >= 0.5%, AG = 1, else, AG = 0)

FC = front curve index (if the approaching section is a curve, FC =1, else, FC =0)

R = curve radius (ft)

LC = length of curve (ft)

R^2 = 0.5015

R^2_{adj} = 0.4320

Mean error: 1.47 (mph)

The SAS outputs are shown in Appendix 5. The grade of the approaching section is defined in the same manner as that in the model for the inside lane. The limitations of the use of the model are also the same as these in the model for the inside lane.

Of interest is the impact of curve itself on operating speed. In the model developed for inside lane, the impact of curve itself on operating speed is clearly intuitive, since only the curve length is included in the model. The increase of curve length resulted in increase in operating speed. However, in the model for the outside lane, the impact of curve itself is not intuitive because both horizontal curve radius and length are in the model. For assessing the impact on operating speed, a 3-D figure (Figure 8- 6) is helpful. This figure shows the operating speed changes due to the changes of curve radius and deflection angles (it is equal to the value of curve length divided by radius). It clearly shows that in general, operating speed increased as curve radius and/or deflection angles increased.

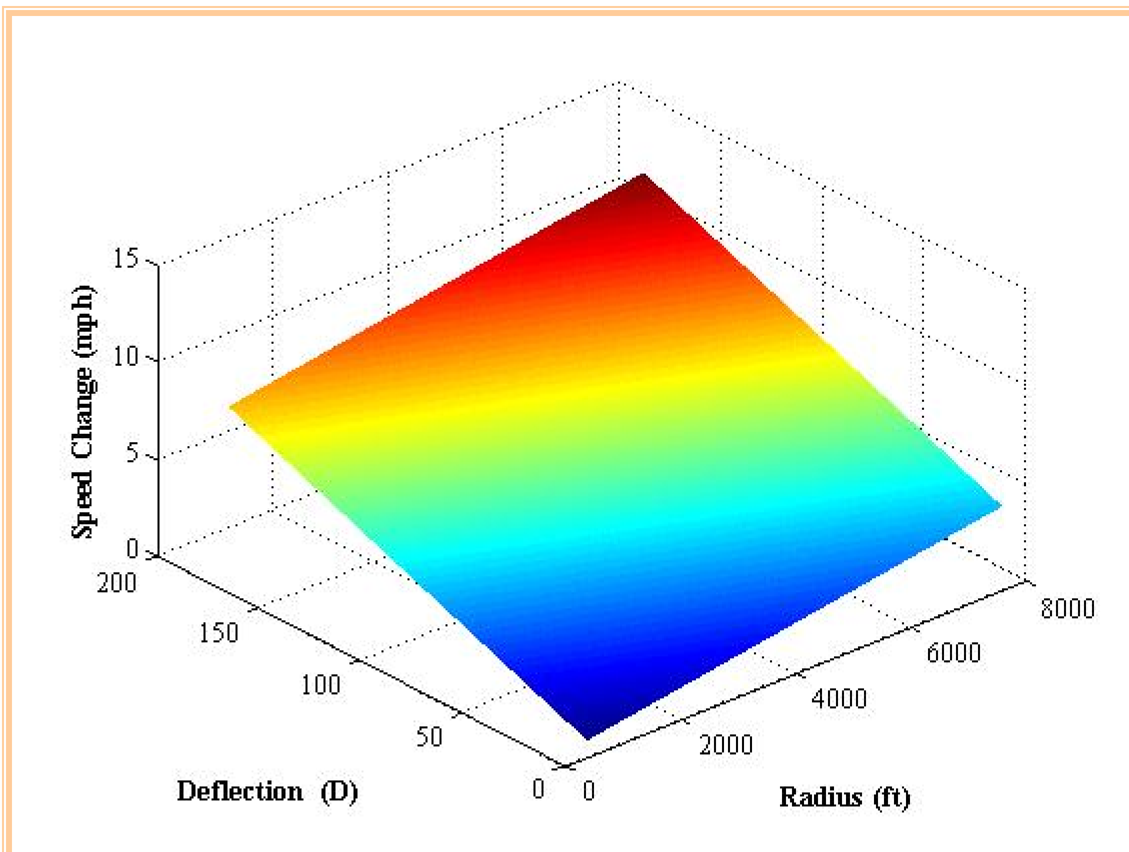

Figure 8- 6: Impacts of Curve Radius and Deflection Angle (Outside Lane)

8.4 Model Assumption Diagnostic

The multiple linear regression model is presented as:

$$y_i = \beta_0 + \beta_1 x_{1i} + \beta_2 x_{2i} + ... + \beta_k x_{ki} + e_i$$

Where:

i = the i th observation

y_i = the dependent variable

β_k = coefficient

x_k = the explanatory variable

e_i = random error or disturbance

The assumptions of the multiple linear regression model are (Dielman, 2001):

1) The relationship between the dependent and the explanatory variable is linear;
2) The random errors are normally distributed and the mean is zero;
3) The random errors have constant variance;
4) The random errors are independent;
5) The explanatory variables used in the models are not highly interrelated.

Except for the assumptions 2 and 3, the other assumptions have been examined during the model development. This section examines assumptions 2 and 3 for each of the two models developed.

8.4.1 *Inside Lane Model*

The random errors are the differences between the true values of the dependent variable and the corresponding values on the regression line. Many of the methods of assessing the validity of the assumptions depend on the use of the residuals (the differences between the true and fitted values for the points in the sample). The Kolmogorov-Smirnov Test showed that the hypothesis that the distribution of the residuals was normal could not be rejected (p-value=0.792). Moreover, the results of the Kolmogorov-Smirnov test showed that the mean of the residuals were equal to zero (Table 8- 3). The normal probability plot of the residuals showed that the residuals were around the line (Figure 8-

7), which indicated that the data were normally distributed. These results indicated that assumption 2 was not violated.

Table 8- 3: One-Sample Kolmogorov-Smirnov Test

	Residual
N	50
Mean	0.000
Std. Deviation	1.310
Kolmogorov-Smirnov Z	0.650
Asymp. Sig. (2-tailed)	0.792

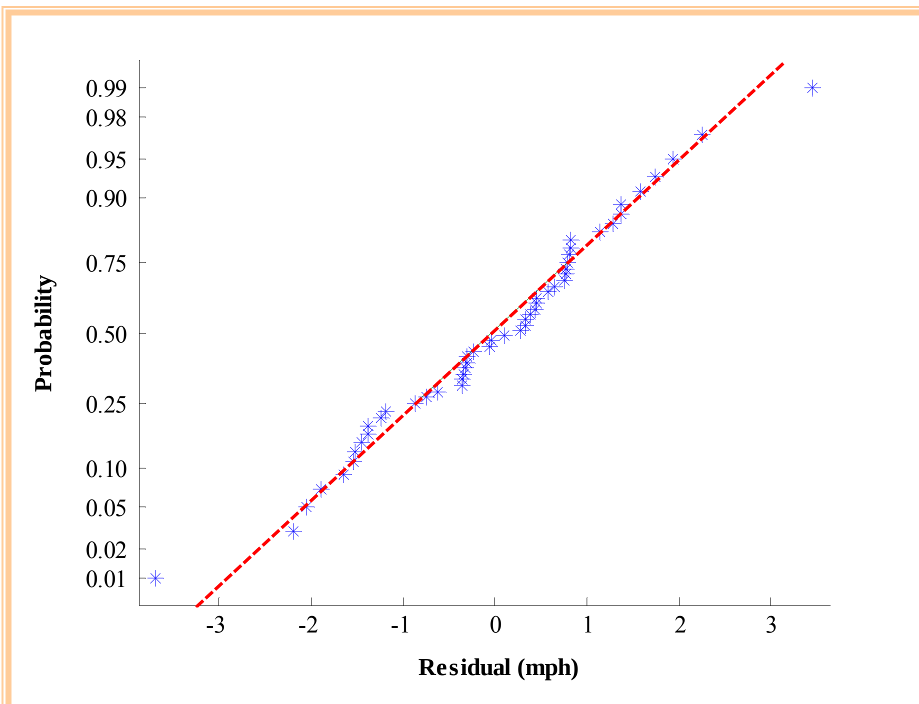

Figure 8- 7: Normal Plot of the Residuals (Inside Lane)

The residual plot versus an explanatory variable is usually used to assess the assumption that the variance around the regression line is constant. The residual should be scattered randomly about the zero line. Figure 8- 8 showed that these residuals were randomly scattered around the zero line. It indicted that assumption 3 was satisfied.

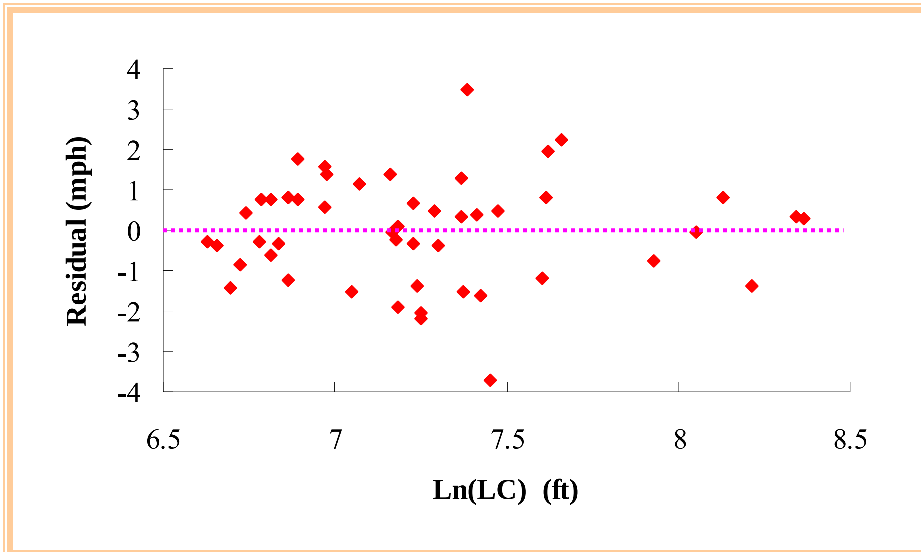

Figure 8- 8: Residual and the Explanatory Variable (Inside Lane)

These diagnostics conducted for verifying the assumptions of the model indicated that the multiple linear regression model was appropriate for the data.

8.4.2 *Outside Lane Model*

For outside lane, the results of the Kolmogorov-Smirnov test showed that the hypothesis that the residual data were normally distributed could not be rejected since the p-value (0.969) was considerably greater than 0.05 when the significance level was set at 95% (Table 8- 4). The normal probability plot also showed that the distribution of the residual data were normal (Figure 8- 9). The mean of the residuals was equal to zero. Therefore, the data supported assumption 2.

Table 8- 4: One-Sample Kolmogorov-Smirnov Test

	Residual
N	50
Mean	0.000
Std. Deviation	1.381
Kolmogorov-Smirnov Z	0.492
Asymp. Sig. (2-tailed)	0.969

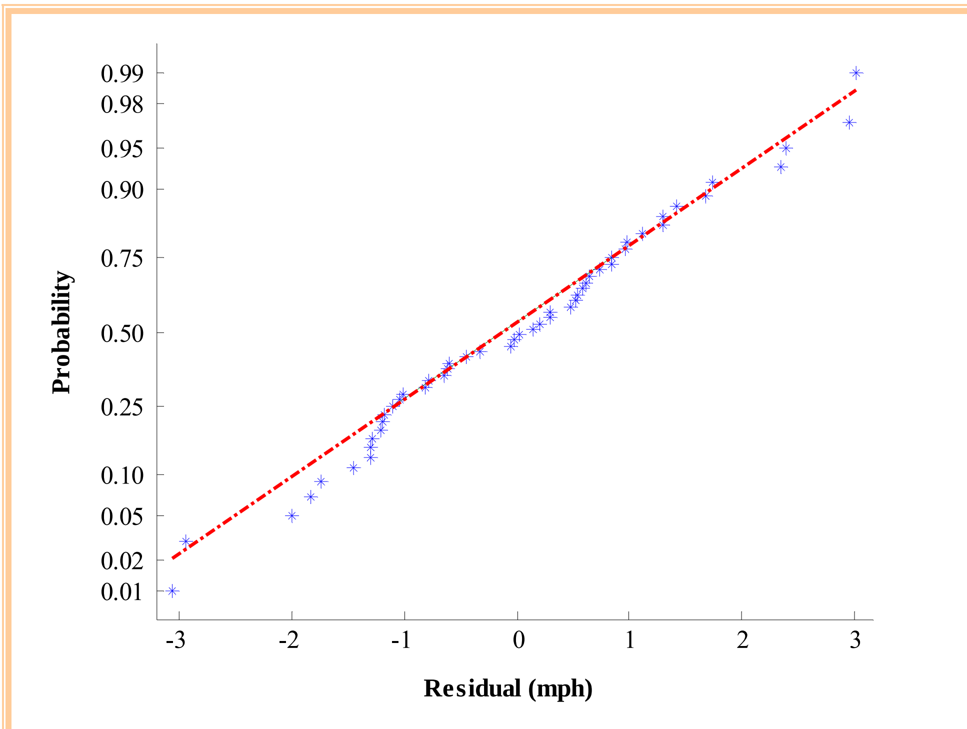

Figure 8- 9: Normal Plot of the Residuals (Outside Lane)

The residual versus the explanatory variable showed that the residuals were randomly scattered around the zero line (Figure 8- 10). It indicated that assumption 3 was met.

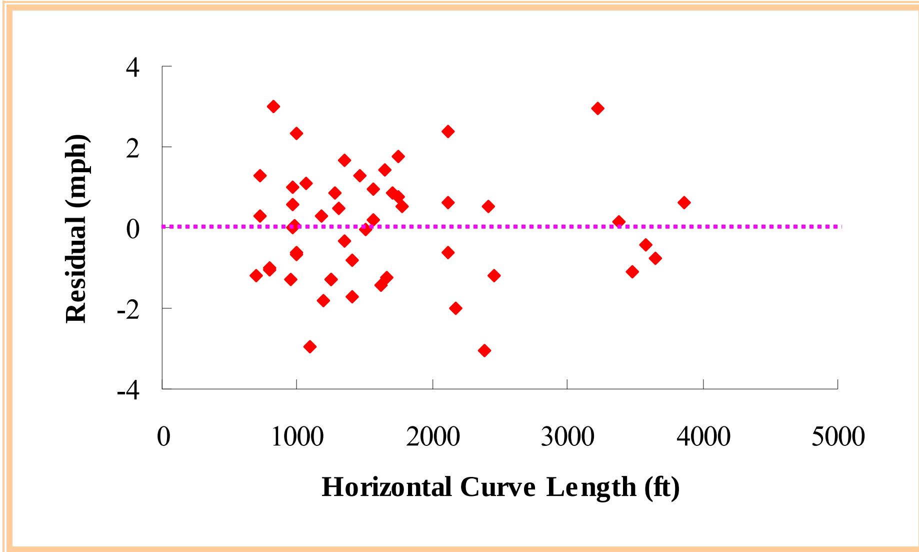

Figure 8- 10: Residual and the Explanatory Variable (Outside Lane)

These assumption diagnostics indicated that the assumptions of the multiple linear regression model were met. The multiple linear regression model was suitable for developing the model.

CHAPTER 9: VALIDATION OF THE MODELS

This chapter presents the validation results for the operating speed prediction models developed in chapter 8. The objective of the validation effort is to evaluate the accuracy with the speeds predicted by using the models developed. The two models for inside and outside lanes are validated separately.

The two models were developed using 50 sample curves randomly selected from the 74 sample curves of the database. The data for validation (24 sample curves) are the remaining after the random selection.

The following analyses are conducted for the model validation. The statistics used to describe the discrepancy between the measured and predicted speeds are also presented here.

- Calculate the mean of the difference (Diff), the mean absolute difference (AD), the mean squared error (MSR), and the mean absolute percent difference (MAPD). The mean absolute percent difference was defined as:

  Mean absolute percent difference = mean of $\left|\dfrac{measured - predicted}{predicted}\right|$x100%
- Draw Box-plots of the statistics to illustrate their distributions
- Assess the differences statistically

9.1 Inside Lane

The differences between the measured and predicted speeds (measured minus predicted) ranged from -4.50 to 4.79 mph (Figure 9- 1)[13]. The mean of the differences was only 0.20 mph. The mean of the absolute differences was 1.88 mph. The mean squared error reached 4.89 mph^2 . The mean absolute percent difference was only 2.89%, which

[13] The forecast errors were sorted by ascending order.

indicated that the prediction error rate was very low. These statistics indicated that the prediction error was low.

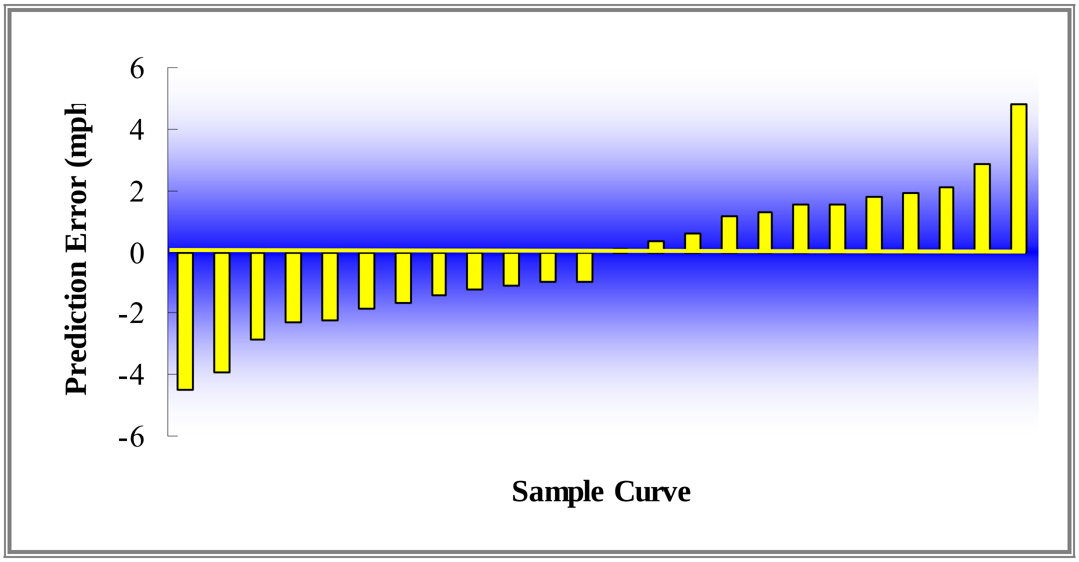

Figure 9- 1: Prediction Error (Inside Lane)

The box-plots of the statistics showed that there were three outliers detected in the squared differences. Examining the data of the sample curve, it was found that the observed speed at these sample curves were the greatest speeds among the speeds for validation. If excluding the outliers, the mean squared error decreased to 2.80 mph, but there were no significant changes in the other three statistics.

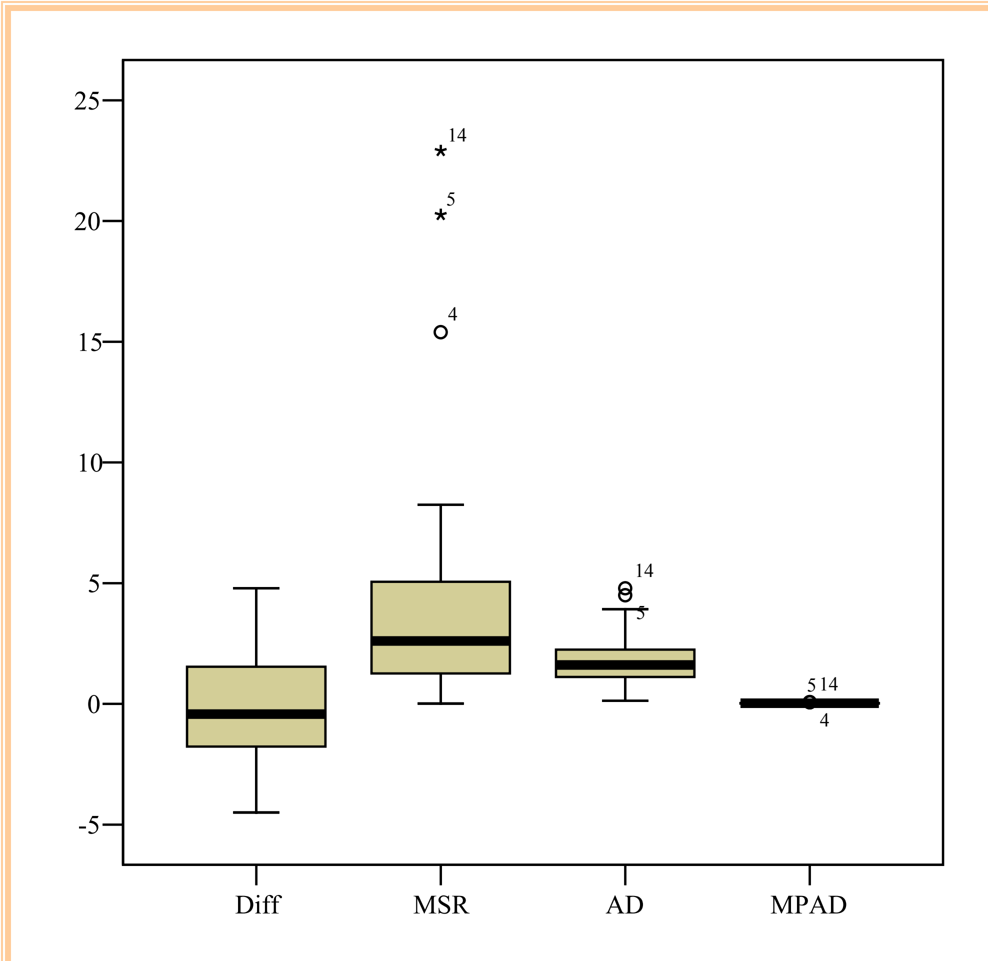

Figure 9- 2: Box-plots of the Statistics (Inside Lane)

The paired t-test was used to statistically examine the differences. However the assumptions of the paired t-test should be first examined. The most important assumption is that the data are normally distributed. Using the Kolmogorov-Smirnov test, it was found that the null hypothesis that the distribution of the data was normal could not be rejected at the 95% confidence level since the great p-value (0.793). The normal probability plot of the data also showed that the data were around the line, which indicates that the data are normally distributed (Figure 9- 3). The assumption diagnostic indicated that the paired t-test was suitable for the data.

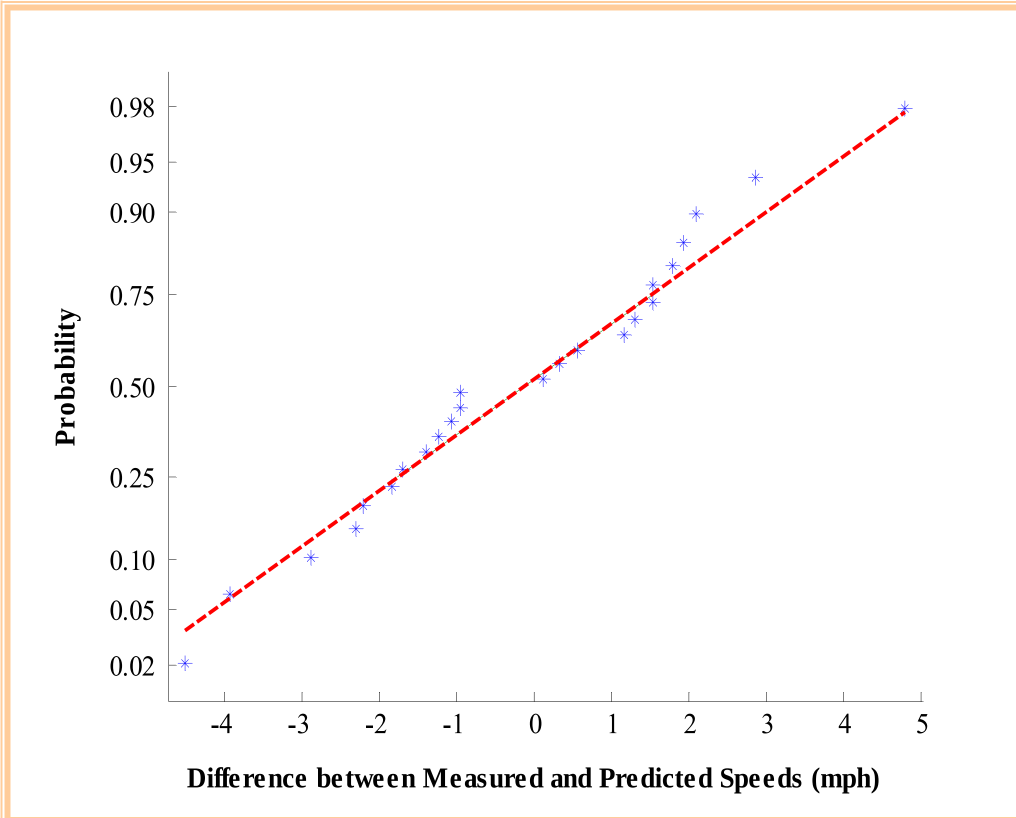

Figure 9- 3: Normal Probability Plot of the Difference (Inside Lane)

The results of the paired t-test showed that, at the 95% confidence level, the null hypothesis, that there was no difference between the means of the measured and predicted operating speeds, could not be rejected (p-value=0.663). It could be further concluded that there was no statistical difference between the measured and predicted operating speeds.

9.2 Outside Lane

The range of the speed differences (measured minus predicted) on outside lane was from -3.54 to 4.58 mph (Figure 9- 4). The mean of the differences was only -0.68 mph while the mean of the absolute differences was 1.80 mph. The mean squared error was 4.61 mph^2. The mean absolute percent difference was 2.85%. These statistics are very close to the statistics calculated for inside lane and indicated that the prediction error was low.

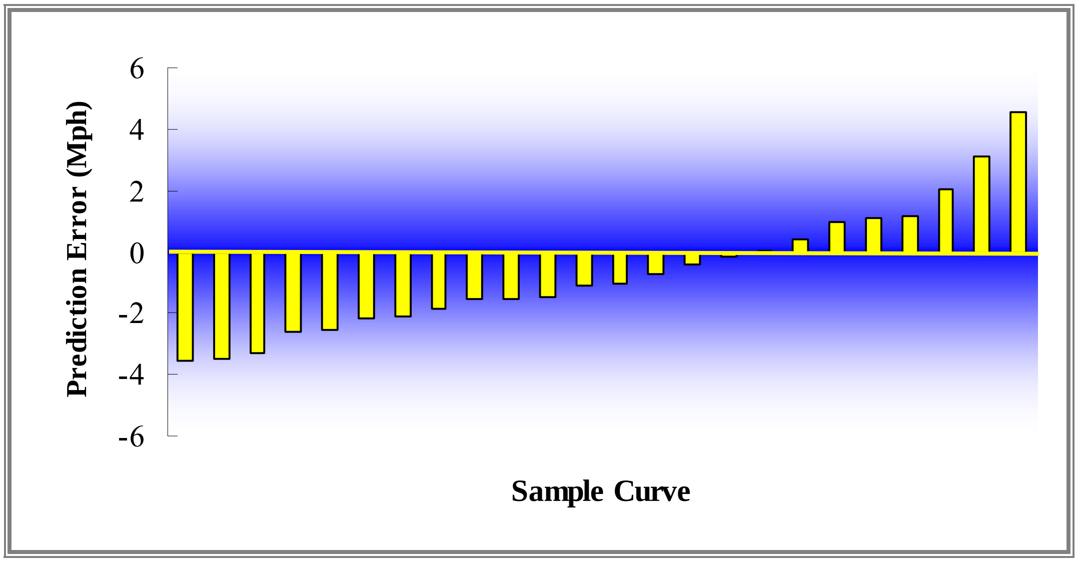

Figure 9- 4: Prediction Error (Outside Lane)

The box-plots showed that there were few unusual data (Figure 9- 5). Examining the distribution of the measured operating speeds, it was found that there was one outlier among the 24 speeds, which resulted in the unusual data in the box-plots.

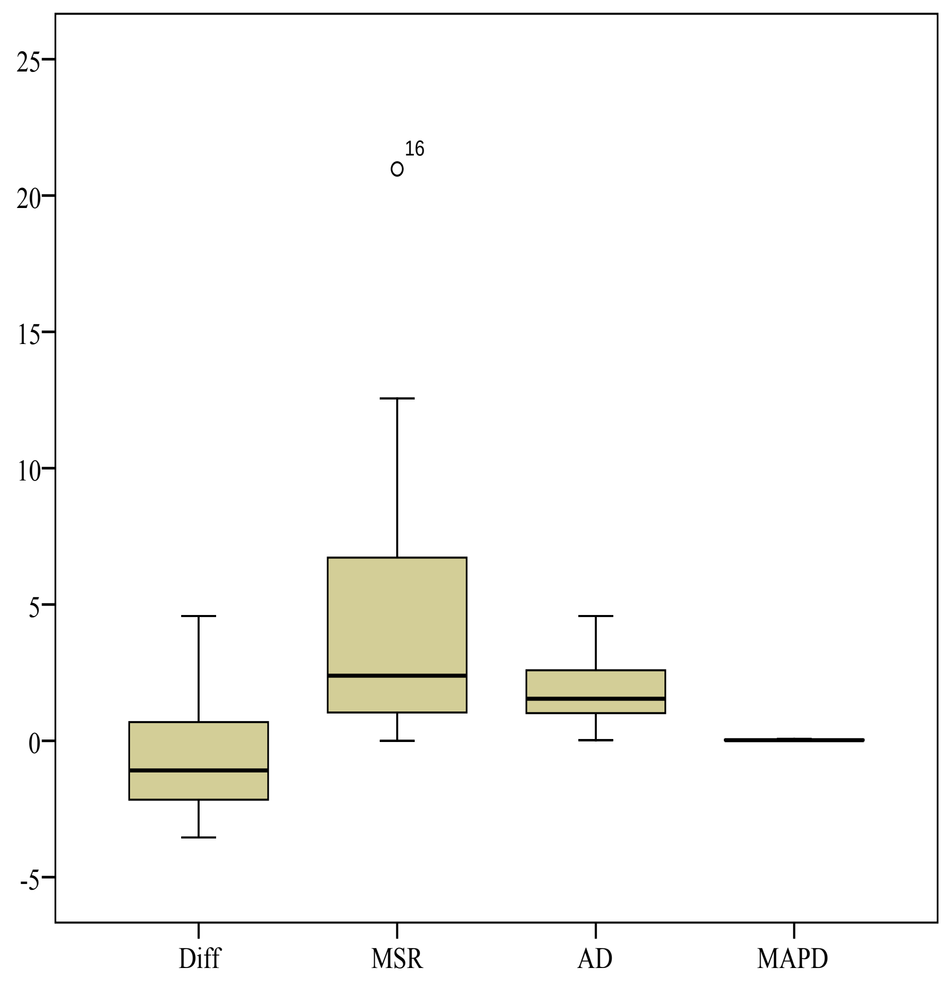

Figure 9- 5: Box-plots of the Statistics (Outside Lane)

Using the Kolmogorov-Smirnov test, it was found that the null hypothesis that the distribution of the data was normal could not be rejected at the 95% confidence level since the great p-value (0.923). The normal probability plot of the data also indicated that the data were normally distributed (Figure 9- 6). The assumption assessment indicated that the paired t-test was suitable for the data.

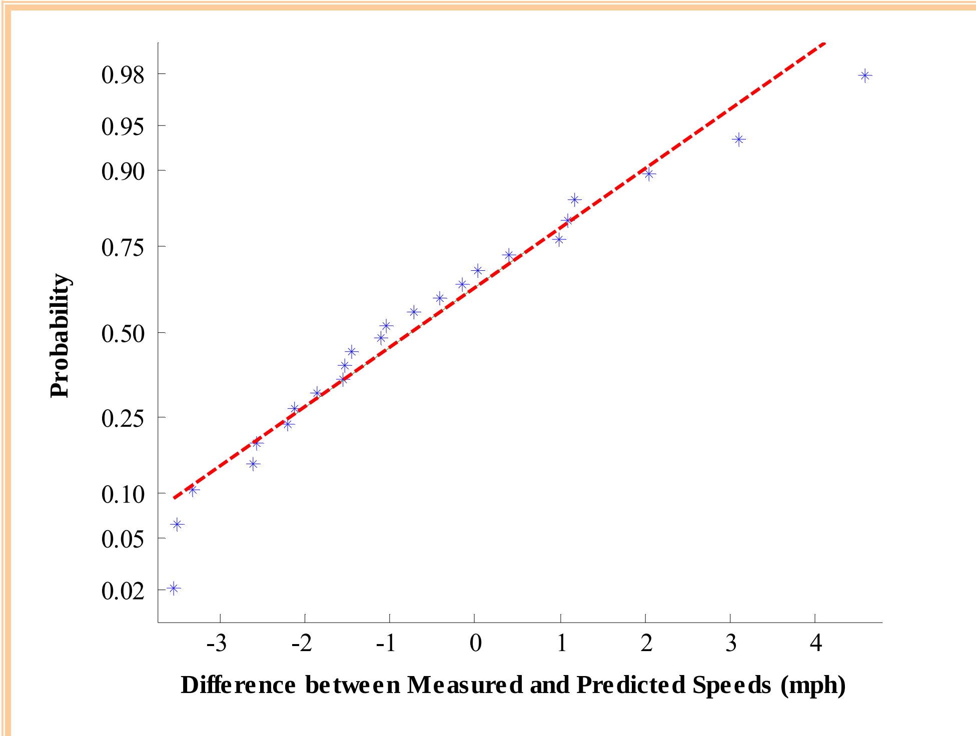

Figure 9- 6: Normal Probability Plot of the Difference (Outside Lane)

The results of the paired t-test showed that the null hypothesis that there was no difference between the means of the two speeds could not be rejected at the 95% confidence level (p-value=0.122). It could be further concluded that there was statistically no difference between the measured and predicted operating speeds.

9.3 Summary

The validation of the two models for inside and outside lanes was performed by comparing the predicted operating speeds to the field-observed operating speeds. Three statistical analyses were conducted for the model validation. The two models perform well in that their mean absolute percent errors are 2.89% for inside lane and 2.85% for outside lane. The hypothesis tests showed that there was no statistical difference between the predicted and field-observed operating speeds.

CHAPTER 10: FINDINGS AND RECOMMENDATIONS

10.1 Findings

This dissertation is the first comprehensive study on predicting operating speed models focusing on horizontal curves of rural four-lane non-freeway highways (non-interstates or parkways). Most of the previous studies focused on rural two-lane highways due to high fatality rates, and large number of miles of such roads. This study focused examination on rural four-lane non-freeway highways.

The sites selected are widely distributed across Kentucky. The site selection was conducted using the GIS technology. This technology is able to bilaterally convert between data and graphic, such as Excel database to ArcGIS shape file. Site information can be operated as numeric data, and site surroundings can be visualized. By contrast, most previous studies selected study sites based on highway databases, maps, or on-site visits. The GIS technology facilitates and improves efficiency and reduces cost for site selection.

Geometric data, especially for curve radius and length, are a difficult problem when original design or construction documents are not available. In most previous studies, curve radius and length were mostly measured from maps or calculated based on GPS data. The accuracy of measurement or calculation was questioned. This study simulated the horizontal curves in AutoCAD by fitting the GPS points which were exported from ArcGIS.

This study examined the operating speed characteristics in horizontal curves of rural four-lane non-freeway highways. A couple of hypotheses have been conducted to test the various relationships. Based on the statistical analyses, the operating speed characteristics were concluded as the following:

- The operating speeds in each direction of travel had no statistical differences, i.e. were the same
- The operating speeds on inside and outside lanes were significantly different
- On each of the two lanes in same direction, the operating speeds at the beginning point, middle point, and ending point were statistically the same

The relationships between operating speed and the two other speeds (design speed and posted speed limit) were examined too. The relationships between operating speed and design speed for inside and outside lanes were different. For the inside lane, the operating speed was statistically equal to the design speed. However, for the outside lane, the operating speed was significantly lower than the design speed. The relationships between operating speed and posted speed limit for both inside and outside lanes were similar. It was found that the operating speeds were statistically higher than the posted speed limits. Previous studies that examined the relationships among the three speeds did not consider the operating speed difference between inside and outside lane, but generally drew conclusions based on combined speed data collected on both lanes. The results of this study indicate that considering the operating speed difference on both lanes will be much more appropriate when studying the relationships among the three speeds.

The various trends between design speed and geometric elements were identified. The analyses showed that there are some relationships between design speed and various geometric elements. For most of these elements present, the general assumption that greater design speeds lead to larger values for the elements seems to hold. However, some surprising and unexplainable opposite trends were also observed, such as the trends of right shoulder widths. These trends indicated that the choice of design speed does not impact the value chosen for this design element. It could be assumed that these values are affected more by other factors, such as terrain, location, right-of-way, and roadway context.

The relationships between operating speed and values of geometric elements were more uniform. For all values examined, generally, larger values of the elements resulted in greater operating speeds. These trends may indicate that, in general, drivers select their operating speeds based on the various geometric elements they face. Moreover, this also implies that the use of specific values for these elements could affect driver's operating speeds and thus this is a bidirectional relationship. There also were some insignificant trends observed. It seemed that approaching tangent length, median width, and roadway width did not significantly affect a driver's speed choice at four-lane rural highways.

Two multiple linear regression models were developed for operating speed prediction since the operating speeds on inside and outside lanes were different. The two models focused on horizontal curves of rural four-lane highways. For the inside lane, the significant factors are: shoulder type, median type, pavement type, approaching section grade, and horizontal curve length. For the outside lane, the significant factors are more, including: shoulder type, median type, curve radius, curve length, presence of approaching curve, and approaching section grade. Comparing the significant factors in the two models, it could be concluded that there were some common factors including shoulder type, median type, approaching section grade, and curve length. These factors indicate that the curve itself mainly influences a driver's speed choice. Some points about the models should be noted here:

- Not all of the geometric elements examined were included in the two models. This does not mean that the elements excluded from the models did not significantly affect operating speed. The included elements only meant that these elements were significant to operating speed and their combination could mostly explain the operating speed. Other significant elements were listed in chapter 8. This helps designers to understand the impacts of geometric elements when choosing a value for any of the design elements.

- When separately examining the impact of each element, it was found that some elements did not significantly affect operating speed. However, when collectively considering the geometric elements, some elements were found significant. This indicated that there were certain interactions among the geometric elements. It further showed that multiple linear regression model was much more appropriate for rural four-lane highways.

- In both models, curve length was found significant. It indicated that curve radius did not solely determine operating speed on rural four-lane highways. In contrast, some models for rural two-lane highways only used curve radius as the explanatory variable.

- In the two models, the approaching section grade was used as an explanatory variable. The weighted average grade of a curve was not found significant to operating speed. The possible reason was that the grades were categorized in general categories and so the directions of the grades were unknown.

- The validation for the models indicated that the models were appropriate for application and the explanatory variables were reliable. However, the limitations of use of these models should be pointed out. The two models are only applicable for sections with a horizontal curve. The range of radius for these models is between 538 and 7,704 ft, and the range of lengths of horizontal curves is from 775 to 5,780 ft. The range of AADT for these models is 5,220-26,900 pcpd. The range for design speeds was between 40 to 70 mph. The use of these models for roadway sections outside of these ranges is not recommended without any additional validation.

10.2 Recommendations

The ultimate objective of this study is to develop recommendations for design and traffic control practices and future research based on the findings. The analyses conducted indicated that there were some relationships between operating speed and geometric elements. These trends are indicative of the influence of specific values of the geometric elements on the drivers' operating speeds. Similar relationships were examined and identified between these geometric features and design speed.

10.2.1 *Recommendations for Practice*

Based upon the findings discussed in the preceding sections the following points are recommended as good design practices:

- The selected design speed should be chosen based on the desired 85^{th} percentile operating speed. This will reduce any disparity between these two speeds. The current design speed selection procedures widely used do not consider operating speed.

- The selected design speed should be chosen based on the 85^{th} percentile operating speed on inside lane. It was found that design speed was statistically equal to operating speed on inside lane while higher than that on outside lane. For safety, the selected design speed should be chosen based on the 85^{th} percentile operating speed on inside lane.

- The models developed for predicting the 85^{th} percentile speed in horizontal curves of rural four-lane highways are recommended to determine this speed. Once a design is developed, its operating speed could be estimated using the two models to examine whether the geometric features can provide the desired operating speed. If not, geometric features should be adjusted so that the desired operating speed can be achieved.

- Current design practices tend to result in a design speed selected in order to address the most restrictive geometric elements of the alignment like horizontal curves, while ignoring the impacts of other elements, such as shoulder width, grade, and clear zone. On the highways examined here, other elements also showed that they have an impact on operating speed, such as shoulder type, pavement type, and shoulder width. Therefore, ignoring these elements and their influence on operating speeds may lead to greater disparity between operating speed and design speed.

- The impacts of the types of shoulder, median, and pavement on operating speed should be considered in design practice. These impacts are usually not fully evaluated by designers. In the design process, these factors are often determined based on construction cost and thus their impacts on operating speed were ignored.

The setting of speed limits is usually based on an operating speed study. Previous studies did not consider the operating speed difference between inside and outside lanes. The analyses conducted in this study showed that the operating speed difference between inside and outside lanes did exist. For traffic control practices, the difference should be taken into consideration when conducting operating speed studies for setting speed limit.

10.2.2 *Recommendations for Future Research*

This dissertation is the first comprehensive study focusing on speed in horizontal curves of rural four-lane non-freeway highways (non-interstates or parkways). Speed characteristics on tangents of four-lane non-freeway highways are unknown. Further research is needed to study the roadway features that may affect the speed characteristics on tangents as well as to develop models for predicting operating speed on tangents based on geometric elements.

In this study, spot speeds were measured on three points along a curve. For studying the speed change along a curve, a speed profile is necessary. Furthermore, speed change between the approaching segment (tangent or curve) and the curve is unknown. In future studies, more spots along a curve or consecutive segments (a tangent and a curve or two consecutive curves) should be used to measure speeds.

Efforts to create and evaluate design consistency on rural two-lane highways have been conducted. Procedures and models for examining and evaluating design consistency for these highways have been developed. However, no similar efforts have been conducted for rural four-lane highways. Therefore, models to create and evaluate design consistency are needed for the four-lane rural non-freeway highways. These models could be developed based on geometric features and roadway environments.

The sites used in this study were selected in Kentucky. Only 13 sites were obtained to study the speed difference in both directions. On inside lane, the number of the speed observations varied, some of which were less than 100. More site samples and speed observations may be appropriate for evaluating the speed difference, and further validating the proposed models.

Some of the data used were extracted from some databases such as HPMS and HIS. The accuracy of the data is still questionable. In both databases, the values of grades are not provided accurately. Grades were categorized and their signs were missing. Future data collection should provide specific numeric values of grades. Moreover, research to accurately estimate and evaluate the elements of a curve is needed.

The method to manually collect speed data is widely used. Previous studies focusing on measurement error were only concerned the errors resulted from equipment. No research has been conducted to study the impact resulted from the presence of operators.

Studies to identify hazardous spots on rural two-lane highways have been conducted previously. These studies related crashes with geometric features. Models for predicting crashes have been developed for rural two-lane highways. These studies help designers to examine the values of geometric elements and thus to reduce potential crashes. For rural four-lane non-freeway highways, such efforts are needed in the future.

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APPENDIX 1: EXISTING MODELS FOR RURAL HIGHWAY CURVES

Author	Operating Speed Model	R^2
Taragin (1954)	V_{85} =88.87− 2,554.76/R	0.860
Mclean (1978)	V_{85} =101.2−0.0675CCR =101.2− 2,730/R	0.870
Mclean (1979)	V_{85} =53.80+0.464V_T − 3,260 /R + 85,000/ R^2	0.920
Kerman et al. (1982)	$V_{85} = V_a - V_a^3 / 398R$	0.910
Glennon et al. (1983)	V_{85} = 150.08 – 4.14DC	0.840
Guidelines for German (1984)	$V_{85} = 60 + 3.970 \times e^{-0.00358CCR}$ [LW=3.5 m]	0.790
Glennon et al. (1985)	V_{85} =103.96−(4,524.94/R)	0.840
Setra (1986)	V_{85} ={102.1+346/[(57,300/CCR) $^{-1.5}$]}	N/A
Lamm and Choueiri (1987)	V_{85} =88.72−0.084CCR [LW=3.0 m]	0.846
	V_{85} =89.55−(2,862.69/R) [LW=3.0 m]	0.753
	V_{85} =92.69−0.080CCR [LW=3.3 m]	0.731
	V_{85} =93.83−(2,955.40/R) [LW=3.3 m]	0.746
	V_{85} =95.77−0.076CCR [LW=3.6 m]	0.836
	V_{85} =96.15−(2,803.70/R) [LW=3.6 m]	0.824
	V_{85} =94.39−(3,188.57/R)=93.85−0.045CCR	0.787
	V_{85} =55.84−(2,809.32/R)+0.634LW+0.053SW+0.0004AADT	0.842
Kanellaidis et al. (1990)	V_{85} =109.09−3,837.55/R	0.647

Appendix 1 (continued)

Author	Operating Speed Model	R^2
Kanellaidis et al. (1990)	V_{85} =32.20+0.839 V_d +2,226.9/R−5,33.6 / $\sqrt{R}$	0.925
	V_{85} =129.88−6,23.1/ $\sqrt{R}$	0.777
lamm et al. (1990)	V_{85} = 93.85 – 1.82DC	0.790
Lamm (1993)	V_{85} =10^6 /8,270+7.20CCR	0.730
Islam and Seneviratne (1994)	V_{85} =95.41−1.48DC−0.012 DC^2 (point of curve)	0.990
	V_{85} =103.30−2.41DC−0.029 DC^2 (middle of curve)	0.980
	V_{85} =96.11−1.07DC (point of tangency)	0.980
Krammes et al. (1994)	V_{85} = 102.4 – 1.57DC + 0.012LC – 0.10DF	0.820
Morrall and Talarico (1994)	$V_{85} = e^{4.561-0.00586DC}$	0.631
Ottesen and Krammes (1994)	V_{85} =103.04−0.0477CCR=103.70−3,403/R	0.800
Al-Masaeid et al. (1995)	V_{85PC} =3.64+1.78DC	0.510
	ΔV_{85PC} =2.00DC	0.690
	ΔV_{85LT} =4.32+1.44DC	0.420
	ΔV_{85HT} =3.30+1.58DC	0.620
	ΔV_{85ALL} =1.84+1.39DC+4.39 P_{con} +0.07 G^2	0.770
	ΔV_{85ALL} =5,081/ R_2 −5,081/ R_1 (continuous curves)	0.810
	ΔV_{85ALL} =108.30−3,498/ L_T −0.71($DF_1 \times DF_2$) / ($DF_1 + DF_2$) (common tangents)	0.720
Choueiri et al. (1995)	V_{85} =91.03−0.050CCR	0.810

Appendix 1 (continued)

Author	Operating Speed Model	R^2
Krammes et al. (1995)	V_{85} =103.66−1.95DC	0.800
	V_{85} =102.45−1.57DC+0.0037 L_C −0.10DF	0.820
	V_{85} =41.62−1.29DC+0.0049 L_C −0.12DF+0.95 V_T	0.900
Lamm et al. (1995)	V_{85} =10^6 / 10,150.1+7.676CCR	0.810
Voigt (1996)	V_{85} =99.61−2,951.37/R	0.840
McFadden and Elefteriadou (1997)	V_{85} = 104.61 – 1.90DC	0.740
	V_{85} = 103.13 – 1.58DC + 0.0037 L_C – 0.09DF	0.760
	V_{85} = 54.59 – 1.50DC + 0.0006 L_C – 0.12DF + 0.81 V_a	0.810
Abdelwahab et al. (1998)	ΔV_{85} =0.9433DC+0.0847DF	0.920
Andjus and Maletin (1998)	V_{85} = 16.92 ln(R) – 14.49	0.980
Cardoso et al. (1998)	V_{85} = 49.220 × 292736/ R^2 + 0.454 V_a (France)	0.800
	V_{85} = 51.765*337.78/ $\sqrt{R}$ + 0.6049 V_a (Finland)	0.710
	V_{85} = 41.363*294.000/ $\sqrt{R}$ + 0.699 V_a (Greece)	0.920
	V_{85} = 25.010*271.500/ $\sqrt{R}$ + 0.877 V_a (Portugal)	0.900
Passetti and Fambro (1999)	V_{85} =103.90−3,020.50/R	0.680
Andueza (2000)	V_{85} =98.25−2,795/R−894/ R_a +7.486DC+9.308 L_T (horizontal curve)	0.840
	V_{85} =100.69−3,032/R+27.819 L_T (tangent)	0.850
Fitzpatrick et al. (2000)	V_{85} =106.30−3,595.29/R (HC: $0 \le G < 4$,or HC+sag VC)	0.920

Appendix 1 (continued)

Author	**Operating Speed Model**	R^2
Fitzpatrick et al. (2000)	V_{85} =96.46−2,744.49/R $(4 \leq G < 9)$	0.560
	V_{85} =100.87−2,720.78/R $(-9 \leq G < 0)$	0.590
	V_{85} =101.90−3,283.01/R (HC+LSD crest VC)	0.780
	V_{85} =111.07−175.98/K (LSD crest VC)	0.540
	V_{85} =100.19−126.07/K (HT+sag VC)	0.680
McFadden and Elefteriadou (2000)	$V_{85MSR} = -14.90 + 0.144 V_{85T} - 954.55/R + 0.0153 L_T$	0.712
	$V_{85MSR} = -0.812 + 998.19/R + 0.017 L_T$	0.603
Ottesen and Krammes (2000)	$V_{85} = 102.44 - 1.57\ \mathrm{DC} - 0.012 L_C - 0.01\ \mathrm{DC} \times L_C$	0.810
Donnell et al. (2001)	$V_{85} = 56.1 + 0.117\mathrm{R} - 1.15 G_1 + 0.006 L_{T1} - 0.000097 L_{T1} \times R$	0.613
	$V_{85} = 78.4 + 0.0140\mathrm{R} - 1.40 G_2 - 0.00724 L_{T2}$	0.562
	$V_{85} = 75.1 + 0.0176\mathrm{R} - 1.48 G_2 - 0.00836 L_{T2}$	0.600
	$V_{85} = 74.5 + 0.0176\mathrm{R} - 1.69 G_2 - 0.00810 L_{T2}$	0.611
	$V_{85} = 83.1 - 2.08 G_2 - 0.00934 L_{T2}$	0.577
Gibreel et al. (2001)	$V_{85S1} = 91.81 + 0.010R + 0.468\sqrt{L_V} - 0.006 G_1^{\ 3} - 0.878 \ln(A) - 0.826 \ln(L_0)$ (AT, sag)	0.980
	$V_{85S2} = 47.96 + 7.216 \ln(\mathrm{R}) + 1.534 \ln(L_V) - 0.258 G_1 - 0.653\ \mathrm{A} + 0.02\ e^E - 0.008 L_0$ (BC, sag)	0.980
	$V_{85S3} = 76.42 + 0.023\ \mathrm{R} + 0.00023 K^2 - 0.008\ e^A + 0.062 e^E - 0.00012 L_0^{\ 2}$ (MC, sag)	0.940
	$V_{85S4} = 82.78 + 0.011\ \mathrm{R} + 2.068 \ln(\mathrm{K}) - 0.361 G_2 + 0.036 e^E - 0.00011 L_0^{\ 2}$ (EC, sag)	0.950

Appendix 1 (continued)

Author	**Operating Speed Model**	R^2
Gibreel et al. (2001)	V_{85S5} =109.45−1.257 G_2 −1.586 ln(L_0) (DT, sag)	0.790
	V_{85C1} =82.29+0.003R−0.05DC+3.441ln(L_V)−0.533 G_1 +0.017 e^E −0.000097 $L_0{}^2$ (AT, crest)	0.940
	V_{85C2} =33.69+0.002 R+10.418 ln(L_V)−0.544 G_1 +[8.699/ ln(1+A)]+0.032 e^E −0.011 L_0 (BC, crest)	0.970
	V_{85C3} =26.44+0.25 $\sqrt{R}$ +10.381 ln(L_V)−0.423 G_1 +[6.462/ ln(1+A)]+0.051 e^E −0.028 L_0 (MC, crest)	0.980
	V_{85C4} =74.97+0.292 $\sqrt{R}$ +3.105 ln(K)−0.85 G_2 +0.026 e^E −0.00017 $L_0{}^2$ (EC, crest)	0.900
	V_{85C5} =105.32−0.418 G_2 −0.123 $\sqrt{L_0}$ (DT, crest)	0.830
Jessen et al. (2001)	V_{85} =86.80+0.297 V_P −0.614 G_1 −0.00239ADT (LSD)	0.540
	V_{85} =72.10+0.432 V_P −0.00212ADT (NLSD)	0.420
Liapis et al. (2001)	V_{85} = -0.360839DC – 3.683548E + 75.161	0.750
	V_{85} = -0.472675DC – 3.795879E + 85.186	0.730
Schurr et al. (2002)	V_{85} = 103.3 – 0.1253DA + 0.0238L – 1.038G1	0.460
Medina and Tarko (2004)	V_{per} =47.664+0.003SD-2.639RES-2.541DC+7.954SE-0.624 SE^2 +4.158 Z_p +0.236 Z_p *DC -0.199 Z_p *SE	n/a
Misaghi and Hassen (2005)	$\Delta V_{85} = -83.63 + 0.93 V_T + e^{(-8.93+3507.10/R)}$	0.640
	$\Delta V_{85} = -198.74 + 21.42\sqrt{V_T} + 0.11\text{DFC} - 4.55\text{SW} - 5.36(\text{curve} - \text{dir}) + 1.30\text{G} + 4.22(\text{drv\_flag})$	0.889

Appendix 1 (continued)

Author	**Operating Speed Model**	R^2
Stamatiadis and Gong (2006)	V_{85} =26.903+ 0.495 DS +0.003 LC-0.437 DL -1633.641/R (all 2-lane sites)	0.537
	V_{85} =56.914 – 3883.586/R (DS<SL)	0.440
	V_{85} =39.295+0.203 DS+1.024*RSW-2949.627/R (DS>SL)	0.395

Notes: n/a= information was not provided;
A description of the predictors is in Appendix 2.

APPENDIX 2: NOTATION OF THE PREDICTORS

A = algebraic difference of vertical grades (%)
ADT = average daily traffic (vehicles/day)
CCR = curvature change rate (degree/km)
Curve−dir = curve direction (right-turn: curve-dir=1, else, curve-dir=0)
DC = degree of curvature (degrees)
DF = deflection angle (degrees)
DF_1 = deflection angle for curves 1 of compound curve, (degrees)
DF_2 = deflection angle for curves 2 of compound curve, (degrees)
DFC = deflection angle of circular curve (degrees)
DL = design speed –posted speed limit (mph)
Drv-flag = driveway flag (intersection on curve: drv-flag=1; otherwise: drv-flag=0
DS = design speed (mph)
e; E = superelevation rate (%)
G = vertical grade (%)
G_1 = first grade in direction of travel (%)
G_2 = second grade in direction of travel (%)
Int-flag = intersection flag (intersection on curve: int-flag=1; otherwise: int-flag=0)
K = length of vertical curve for 1% change in grade (m)
L_C = length of horizontal circular curve (m or ft)
L_T = length of tangent (m)
L_{T1} = length of preceding tangent (m)
L_{T2} = length of succeeding tangent (m)
L_V = length of vertical curve (m)
L_0 = distance between horizontal and vertical points of intersection (m)
LW = lane width (m)
P_{con} = pavement condition (PSR>=3: P_{con} =0; otherwise: P_{con} =0)
R = radius of the curve (m or ft)
R_a = radius of previous curve (m)
RES = equal to 1 if segment has 10 or more residential driveways per mile; 0 otherwise
R_1 = radius of curve 1 of the compound curve (m)
R_2 = radius of curve 2 of the compound curve (m)
RSW = right shoulder width (ft)
SD=sight distance
SE = maximum superelevation rate, percent
SL=speed limit
Sp-flag = spiral flag (curve with spiral: sp-flag=1; otherwise: sp-flag=0)
SW = shoulder width (m)
V_a = curve approach speed (km/h)
V_d = desired speed (km/h)

V_P = post speed limit (km/h)
V_{Per} = any percentile speed (km/h)
V_T = approach tangent speed (km/h)
V_{85} = 85th percentile speed (km/h)
V_{85T} = 85th percentile speed on approach tangent speed (km/h)
V_{85MC} = 85th percentile speed at middle of curve (km/h)
V_{85pC} = 85th percentile speed for passenger –car class vehicles (km/h)
Zp = standardized normal variable corresponding to a selected percentile, (km/h)
ΔV_{85} = 85th percentile speed differential calculated as difference between V85 on two elements
ΔV_{85ALL} = 85th percentile speed differential calculated as difference between V85 on two elements (for passenger –car class vehicles)
ΔV_{85HT} = 85th percentile speed differential calculated as difference between V85 on two elements (for heavy-truck class vehicles)
ΔV_{85LT} = 85th percentile speed differential calculated as difference between V85 on two elements (for light-truck class vehicles)
ΔV_{85PC} = 85th percentile speed differential calculated as difference between V85 on two elements (for passenger –car class vehicles)
$\Delta 85_V$ = 85th percentile speed differential calculated as 85^{th} percentile value of speed differentials of individual drivers

APPENDIX 3: EXISTING MODELS FOR URBAN CONDITIONS

Author	Operating Speed Model	R2
Fitzpatrick et al. (1997)	$V_{85} = 56.34 + 0.808\,R^{0.5} + 9.34/AD$ (horizontal curves)	0.72
	$V_{85} = 39.51 + 0.556\,(IDS)$ (vertical curves)	0.56
Fitzpatrick et al. (2001)	$V_{85} = 42.916 + 0.523PSL - 0.15DA + 4.402AD$ (horizontal curves)	0.71
	$V_{85} = 29.180 + 0.701PSL$ (straight sections)	0.53
	$V_{85} = 44.538 + 9.238MED + 13.029\,L_1 + 17.813\,L_2 + 19.439\,L_3$ (horizontal curves, without speed limits)	0.52
	$V_{85} = 18.688 + 15.050WD$ (straight sections, without speed limits)	0.25
Bonneson (1999)	$V_{85} = 63.5R(-B + (B^2 + 4C/127R)^2$ $c = E/100 + 0.256 + (B - 0.0022)V_a$ $B = 0.0133 - 0.0074\,I_{TR}$	0.96
Tarris et al. (1996)	$V_{85} = 53.5 - 0.265D$ (aggregated speed data)	0.82
	$V_{85} = 53.8 - 0.272D$ (individual speed data)	0.63
	$V_{85} = 52.18 - 0.231D$ (panel analysis)	0.80
Poe et al. (2000)	$V_{85} = 49.59 + 0.5D - 0.35G + 0.74W - 0.74HR$ (150 ft before the beginning of curve)	0.99
	$V_{85} = 51.13 - 0.1D - 0.24G - 0.01W - 0.57HR$ (beginning of curve)	0.98
	$V_{85} = 48.82 - 0.14D - 0.75G - 0.12W - 0.12HR$ (middle of curve)	0.90
	$V_{85} = 43.41 - 0.11D - 0.12G + 1.07W + 0.3HR$ (ending of curve)	0.90
Fitzpatrick et al. (2003)	$V_{85} = 8.666 + 0.963PSL$ (arterial)	0.86

Appendix 3 (continued)

Author	Operating Speed Model	R2
Fitzpatrick et al. (2003)	V_{85} = 21.131 + 0.639PSL (collector)	0.41
	V_{85} = 36.453 + 0.517PSL (local)	0.14
Wang (2006)	V_{85} = 57.558+4.899×lane.num + 1.193×lane.width – 0.059×driveway +2.557×median.indicator – 1.308×direction – 0.074×roadside.d – 7.805×parking.indicator– 3.187×sidewalk.indicator (horizontal curve)	n/a
	V_{95} = 58.097 + 4.477×lane.num + 1.359×lane.width – 0.083×driveway +2.5×median.indicator – 1.396×direction – 0.074×roadside.d –8.058×parking.indicator – 3.054×sidewalk.indicator (horizontal curve)	n/a
	V_{85} = 50.503 + (10.386×lane.num) – (0.079×roadside.d) – (0.129×driveway) – (0.211×intersection) + (4.816×curb.indicator) – (6.824×sidewalk.indicator) – (5.104×parking.indicator)+(5.299×land.use1)+(5.237×land.use2) (tangent)	n/a
	V_{95} = 49.828+(10.673×lane.num)–(0.075×roadside.d)–(0.122×driveway)– (0.198×intersection) + (5.319×curb.indicator) – (7.078×sidewalk.indicator) – (4.583×parking.indicator)+(5.611×land.use1) + (5.406×land.use2) (tangent)	n/a

Notes: Sources are from Wang (2006);
n/a = information was not provided;
A description of the predictors is in next page.

Description of the predictors used in Appendix 3:

AD = approach density (approaches per km)
curb.indicator (f there is no curb then 0, else 1)
D = degree of curve (degree)
driveway = density of driveways (number of driveways per km)
E = superelevation rate
HR = hazard rating (0 to 4)
IDS = inferred design speed (km/h)
intersection = density of T-intersections (number of T-intersection per km)
I_{TR} = indicator variable (if $V_a > V_{85}$ then 1.0, else 0.0)
L_1 = if school then 1, otherwise 0
L_2 = if residential then 1, otherwise 0
L_3 = if commercial then 1, otherwise 0
land.use (if land use is commercial then land.use1 = 0 and land.use2 =0; if land use is residential then land.use1 = 1 and land.use2 = 0; else land.use2 = 1 and land.use1 = 0)
lane.num = number of lanes
MED = if raised or TWLTL then 1, otherwise 0
parking.indicator (if there is no on-street parking then 0, else 1)
PSL = posted speed limit (km/h)
R = horizontal curve radius (m)
roadside.d = density of roadside objects (utility poles and trees) divided by their average offsets from roadside (number of objects per km/offset (m))
sidewalk.indicator (if there is no sidewalk then 0, else 1)
V_{85} = the 85th percentile speed (km/h)
V_a = the 85th percentile speed on approach tangent (km/h)
W = lane width (m)

APPENDIX 4: SAS OUTPUTS FOR MODEL DEVELOPMENT (INSIDE LANE)

1. Variable Selection (Stepwise)

```
                              The REG Procedure
                                Model: MODEL1
                       Dependent Variable: speed speed

                          Stepwise Selection: Step 6

       No other variable met the 0.1500 significance level for entry into the model.
```


Summary of Stepwise Selection

Step	Variable Entered	Variable Removed	Label	Number Vars In	Partial R-Square	Model R-Square	C(p)	F Value
1	Median_Type_2		Median_Type_2	1	0.2102	0.2102	69.9841	12.77
2	HC_Length_LOG		HC_Length_LOG	2	0.2354	0.4455	37.4231	19.95
3	FT_Grade_1		FT_Grade_1	3	0.0759	0.5214	28.2846	7.29
4	Pavement_Type_2		Pavement_Type_2	4	0.1235	0.6448	12.1539	15.64
5	SHLD_Type_2		SHLD_Type_2	5	0.0388	0.6836	8.4571	5.40
6	Radius_R		Radius_R	6	0.0232	0.7068	7.0489	3.40

Summary of Stepwise Selection

Step	Pr > F
1	0.0008
2	<.0001
3	0.0097

4	0.0003
5	0.0249
6	0.0719

2. Model Selection

The REG Procedure
Model: MODEL1
Dependent Variable: speed

C(p) Selection Method

Number of Observations Read	50
Number of Observations Used	50

Number in Model	C(p)	R-Square	Variables in Model
6	7.0000	0.7068	SHLD_Type_2 Median_Type_2 Pavement_Type_2 FT_Grade_1 HC_Length_LOG Radius_R
5	8.4044	0.6836	SHLD_Type_2 Median_Type_2 Pavement_Type_2 FT_Grade_1 HC_Length_LOG
5	8.8919	0.6803	Median_Type_2 Pavement_Type_2 FT_Grade_1 HC_Length_LOG Radius_R
4	12.0947	0.6448	Median_Type_2 Pavement_Type_2 FT_Grade_1 HC_Length_LOG
5	19.1567	0.6103	SHLD_Type_2 Median_Type_2 FT_Grade_1 HC_Length_LOG Radius_R
4	20.9143	0.5847	SHLD_Type_2 Median_Type_2 FT_Grade_1 HC_Length_LOG
4	23.8517	0.5647	Median_Type_2 FT_Grade_1 HC_Length_LOG Radius_R
5	24.0144	0.5772	SHLD_Type_2 Pavement_Type_2 FT_Grade_1 HC_Length_LOG Radius_R
5	26.9612	0.5571	SHLD_Type_2 Median_Type_2 Pavement_Type_2 HC_Length_LOG Radius_R
4	27.0317	0.5430	SHLD_Type_2 Median_Type_2 Pavement_Type_2 FT_Grade_1
4	27.0431	0.5429	Median_Type_2 Pavement_Type_2 HC_Length_LOG Radius_R
3	28.2048	0.5214	Median_Type_2 FT_Grade_1 HC_Length_LOG
4	29.0119	0.5295	SHLD_Type_2 Median_Type_2 Pavement_Type_2 HC_Length_LOG
5	29.0279	0.5430	SHLD_Type_2 Median_Type_2 Pavement_Type_2 FT_Grade_1 Radius_R
4	29.8837	0.5236	Pavement_Type_2 FT_Grade_1 HC_Length_LOG Radius_R
3	30.5169	0.5056	Median_Type_2 Pavement_Type_2 HC_Length_LOG
4	31.1562	0.5149	SHLD_Type_2 Median_Type_2 HC_Length_LOG Radius_R
3	33.0704	0.4882	Median_Type_2 HC_Length_LOG Radius_R
3	33.3580	0.4862	SHLD_Type_2 Median_Type_2 HC_Length_LOG
2	37.3307	0.4455	Median_Type_2 HC_Length_LOG
4	37.6747	0.4704	SHLD_Type_2 FT_Grade_1 HC_Length_LOG Radius_R

4	39.7643	0.4562	SHLD_Type_2 Pavement_Type_2 FT_Grade_1 Radius_R
3	39.8189	0.4422	SHLD_Type_2 Median_Type_2 FT_Grade_1
4	40.4733	0.4514	SHLD_Type_2 Pavement_Type_2 FT_Grade_1 HC_Length_LOG
4	41.7996	0.4423	SHLD_Type_2 Median_Type_2 FT_Grade_1 Radius_R
3	41.9598	0.4276	Median_Type_2 Pavement_Type_2 FT_Grade_1
4	42.9062	0.4348	SHLD_Type_2 Pavement_Type_2 HC_Length_LOG Radius_R
4	43.8328	0.4285	Median_Type_2 Pavement_Type_2 FT_Grade_1 Radius_R
3	44.0790	0.4131	SHLD_Type_2 Pavement_Type_2 FT_Grade_1
3	46.0339	0.3998	Pavement_Type_2 HC_Length_LOG Radius_R
3	47.9951	0.3864	FT_Grade_1 HC_Length_LOG Radius_R
3	48.3181	0.3842	SHLD_Type_2 HC_Length_LOG Radius_R
3	49.3787	0.3770	SHLD_Type_2 Median_Type_2 Pavement_Type_2
4	51.3545	0.3772	SHLD_Type_2 Median_Type_2 Pavement_Type_2 Radius_R
2	53.6386	0.3343	SHLD_Type_2 Median_Type_2
3	53.7269	0.3474	SHLD_Type_2 FT_Grade_1 Radius_R
2	54.7971	0.3264	HC_Length_LOG Radius_R
3	55.6013	0.3346	SHLD_Type_2 Median_Type_2 Radius_R
3	55.9266	0.3324	SHLD_Type_2 FT_Grade_1 HC_Length_LOG
3	57.9178	0.3188	Pavement_Type_2 FT_Grade_1 HC_Length_LOG
3	58.8271	0.3126	Pavement_Type_2 FT_Grade_1 Radius_R
2	59.1662	0.2967	SHLD_Type_2 FT_Grade_1
3	60.4192	0.3017	SHLD_Type_2 Pavement_Type_2 HC_Length_LOG
2	60.5075	0.2875	Median_Type_2 Pavement_Type_2
3	60.8736	0.2986	SHLD_Type_2 Pavement_Type_2 Radius_R
2	61.4931	0.2808	Median_Type_2 FT_Grade_1
3	62.3405	0.2886	Median_Type_2 Pavement_Type_2 Radius_R
3	63.2333	0.2826	Median_Type_2 FT_Grade_1 Radius_R
2	65.1504	0.2559	SHLD_Type_2 Pavement_Type_2
2	66.1036	0.2494	SHLD_Type_2 Radius_R
2	66.9524	0.2436	SHLD_Type_2 HC_Length_LOG
2	68.1665	0.2353	Pavement_Type_2 FT_Grade_1
1	69.8524	0.2102	Median_Type_2
1	71.1760	0.2011	SHLD_Type_2
2	71.5891	0.2120	Median_Type_2 Radius_R
2	73.4382	0.1993	Pavement_Type_2 HC_Length_LOG
2	75.5630	0.1849	Pavement_Type_2 Radius_R
2	80.8873	0.1486	FT_Grade_1 Radius_R
2	82.1357	0.1401	FT_Grade_1 HC_Length_LOG

1	84.2958	0.1117	Pavement_Type_2
1	87.1816	0.0920	HC_Length_LOG
1	87.2671	0.0914	Radius_R
1	93.3971	0.0496	FT_Grade_1

3. Model Selected

The REG Procedure
Model: MODEL1
Dependent Variable: speed speed

Number of Observations Read	50
Number of Observations Used	50

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	5	181.81280	36.36256	19.02	<.0001
Error	44	84.13840	1.91224		
Corrected Total	49	265.95120			

Root MSE	1.38284	R-Square	0.6836
Dependent Mean	65.27600	Adj R-Sq	0.6477
Coeff Var	2.11845		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Intercept	Intercept	1	50.93727	3.53830	14.40	<.0001
SHLD_Type_2	SHLD_Type_2	1	-1.56723	0.67471	-2.32	0.0249
Median_Type_2	Median_Type_2	1	-2.79548	0.49185	-5.68	<.0001
Pavement_Type_2	Pavement_Type_2	1	-4.00101	1.07868	-3.71	0.0006
FT_Grade_1	FT_Grade_1	1	2.15028	0.46443	4.63	<.0001
HC_Length_LOG	HC_Length_LOG	1	2.22137	0.50229	4.42	<.0001

4. VIF Examination

The REG Procedure
Model: MODEL1
Dependent Variable: speed speed

Number of Observations Read	50
Number of Observations Used	50

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	5	181.81280	36.36256	19.02	<.0001
Error	44	84.13840	1.91224		
Corrected Total	49	265.95120			

Root MSE	1.38284	R-Square	0.6836
Dependent Mean	65.27600	Adj R-Sq	0.6477
Coeff Var	2.11845		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Variance Inflation
Intercept	Intercept	1	50.93727	3.53830	14.40	<.0001	0
SHLD_Type_2	SHLD_Type_2	1	-1.56723	0.67471	-2.32	0.0249	1.25697
Median_Type_2	Median_Type_2	1	-2.79548	0.49185	-5.68	<.0001	1.27521
Pavement_Type_2	Pavement_Type_2	1	-4.00101	1.07868	-3.71	0.0006	1.16826
FT_Grade_1	FT_Grade_1	1	2.15028	0.46443	4.63	<.0001	1.13700
HC_Length_LOG	HC_Length_LOG	1	2.22137	0.50229	4.42	<.0001	1.27039

5. Box-Cox Examination

The TRANSREG Procedure

Transformation Information
for BoxCox(speed)

Lambda	R-Square	Log Like
-3.00	0.69	-17.9724
-2.75	0.69	-17.8217
-2.50	0.69	-17.6765 *
-2.25	0.69	-17.5367 *
-2.00	0.69	-17.4024 *
-1.75	0.69	-17.2735 *
-1.50	0.69	-17.1500 *
-1.25	0.69	-17.0319 *
-1.00	0.69	-16.9191 *
-0.75	0.69	-16.8117 *
-0.50	0.69	-16.7096 *
-0.25	0.69	-16.6127 *
0.00	0.69	-16.5212 *
0.25	0.69	-16.4348 *
0.50	0.68	-16.3537 *
0.75	0.68	-16.2777 *
1.00 +	0.68	-16.2068 *
1.25	0.68	-16.1411 *
1.50	0.68	-16.0804 *
1.75	0.68	-16.0248 *
2.00	0.68	-15.9742 *
2.25	0.68	-15.9285 *
2.50	0.68	-15.8878 *
2.75	0.68	-15.8520 *
3.00	0.68	-15.8210 <

< - Best Lambda
* - Confidence Interval
+ - Convenient Lambda

The TRANSREG Procedure

TRANSREG Univariate Algorithm Iteration History for BoxCox(speed)

Iteration Number	Average Change	Maximum Change	R-Square	Criterion Change	Note
1	0.00000	0.00000	0.67803		Converged

Algorithm converged.

APPENDIX 5: SAS OUTPUTS FOR MODEL DEVELOPMENT (OUTSIDE LANE)

1. Variable Selection (Stepwise)

The REG Procedure
Model: MODEL1
Dependent Variable: speed speed

Summary of Stepwise Selection

Step	Variable Entered	Variable Removed	Label	Number Vars In	Partial R-Square	Model R-Square	C(p)	F Value
1	SHLD_Type_1		SHLD_Type_1	1	0.2000	0.2000	26.7799	12.00
2	Clear_BR_2		Clear_BR_2	2	0.0880	0.2880	20.7748	5.81
3	Median_Type_2		Median_Type_2	3	0.0775	0.3655	15.7248	5.62
4	FT_Grade_2		FT_Grade_2	4	0.0507	0.4162	13.1118	3.91
5	HC_Length_LOG		HC_Length_LOG	5	0.0405	0.4567	11.4266	3.28
6	Pavement_Type_2		Pavement_Type_2	6	0.0242	0.4808	11.2279	2.00
7	AADT		AADT	7	0.0262	0.5071	10.8428	2.23
8	LSW		LSW	8	0.0282	0.5353	10.2737	2.49
9		HC_Length_LOG	HC_Length_LOG	7	0.0132	0.5221	9.4710	1.16
10	Front_curve		Front_curve	8	0.0351	0.5572	8.2787	3.25
11	Radius		Radius	9	0.0246	0.5818	8.0395	2.35
12	lcr		lcr	10	0.0352	0.6170	6.8395	3.58
13	HC_Length_LOG		HC_Length_A_LOG	11	0.0280	0.6450	6.2955	2.99

Summary of Stepwise Selection

Step	Pr > F
1	0.0011
2	0.0199
3	0.0220
4	0.0542
5	0.0769
6	0.1643
7	0.1425
8	0.1221
9	0.2875
10	0.0788
11	0.1328
12	0.0658
13	0.0917

Note: lcr is created by using curve length divided by curve radius.

2. Model Selection

```
                    The REG Procedure
                      Model: MODEL1
                Dependent Variable: speed

                  C(p) Selection Method

         Number of Observations Read          50
         Number of Observations Used          50
```

Number in Model	C(p)	R-Square	Variables in Model
6	7.0000	0.5015	SHLD_Type_1 Media_Type_2 FT_Grade_2 Front_curve Radius lcr
5	9.6837	0.4472	SHLD_Type_1 Media_Type_2 FT_Grade_2 Radius lcr
5	9.9881	0.4437	SHLD_Type_1 Media_Type_2 Front_curve Radius lcr
10	11.3872	0.6320	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr Clear_BR_2 HC_Length_LOG
9	11.6622	0.6108	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr HC_Length_LOG
11	12.0000	0.6450	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT Clear_BR_2 HC_Length_LOG
9	12.6458	0.6016	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr Clear_BR_2
10	12.7832	0.6190	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT HC_Length_LOG
10	12.9933	0.6170	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT Clear_BR_2
8	13.0473	0.5792	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr
9	13.9646	0.5893	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT
9	14.7586	0.5818	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius AADT Clear_BR_2
8	15.3932	0.5572	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve AADT Clear_BR_2
8	15.8945	0.5526	Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr Clear_BR_2
7	15.9832	0.5330	Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr
8	16.0418	0.5512	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius lcr Clear_BR_2
9	16.1607	0.5687	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius lcr Clear_BR_2 HC_Length_LOG
8	16.1993	0.5497	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius Clear_BR_2

8	16.2281	0.5494	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius AADT Clear_BR_2
8	16.2508	0.5492	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius lcr Clear_BR_2
9	16.3497	0.5670	Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT Clear_BR_2
7	16.3639	0.5295	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius lcr
8	16.3808	0.5480	Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr HC_Length_LOG
8	16.3810	0.5480	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius lcr HC_Length_LOG
9	16.4088	0.5664	Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr Clear_BR_2 HC_Length_LOG
7	16.7149	0.5262	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius lcr
9	16.7514	0.5632	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve AADT Clear_BR_2 HC_Length_LOG
10	16.7552	0.5819	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius AADT Clear_BR_2 HC_Length_LOG
9	16.9027	0.5618	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius lcr AADT Clear_BR_2
8	16.9527	0.5427	Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT
7	16.9670	0.5238	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius lcr Clear_BR_2
10	17.0449	0.5792	Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT Clear_BR_2 HC_Length_LOG
7	17.1495	0.5221	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW AADT Clear_BR_2
9	17.1883	0.5591	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius lcr AADT Clear_BR_2
9	17.1983	0.5591	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT Clear_BR_2
9	17.2834	0.5583	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve lcr AADT Clear_BR_2
6	17.3563	0.5015	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius lcr
7	17.4422	0.5194	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Clear_BR_2
9	17.4908	0.5563	Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT HC_Length_LOG
10	17.5146	0.5748	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT Clear_BR_2 HC_Length_LOG
8	17.7406	0.5353	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW AADT Clear_BR_2 HC_Length_LOG
8	17.8533	0.5343	SHLD_Type_1 Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr HC_Length_LOG
8	17.8920	0.5339	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT
8	17.9434	0.5334	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius lcr Clear_BR_2
9	18.0256	0.5513	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius AADT Clear_BR_2 HC_Length_LOG
9	18.0482	0.5511	SHLD_Type_1 Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr Clear_BR_2 HC_Length_LOG
9	18.0511	0.5511	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius lcr AADT HC_Length_LOG
8	18.0978	0.5320	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius lcr AADT
9	18.1029	0.5506	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Radius Clear_BR_2 HC_Length_LOG
9	18.1559	0.5501	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius lcr Clear_BR_2 HC_Length_LOG
7	18.1694	0.5126	SHLD_Type_1 Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr
8	18.1912	0.5311	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve Clear_BR_2 HC_Length_LOG
8	18.2439	0.5306	SHLD_Type_1 Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr Clear_BR_2
8	18.2988	0.5301	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius lcr AADT Clear_BR_2
7	18.3833	0.5106	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius Clear_BR_2

8	18.4369	0.5288	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius AADT Clear_BR_2
8	18.6050	0.5272	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius lcr HC_Length_LOG
8	18.6581	0.5267	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius lcr AADT
10	18.6999	0.5637	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve lcr AADT Clear_BR_2 HC_Length_LOG
9	18.7084	0.5449	SHLD_Type_1 Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr AADT Clear_BR_2
10	18.7280	0.5634	SHLD_Type_1 Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr AADT Clear_BR_2 HC_Length_LOG
8	18.7359	0.5260	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve AADT Clear_BR_2 HC_Length_LOG
7	18.7635	0.5071	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 AADT Clear_BR_2 HC_Length_LOG
8	18.8539	0.5249	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius lcr Clear_BR_2 HC_Length_LOG
10	18.8560	0.5623	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius lcr AADT Clear_BR_2 HC_Length_LOG
7	18.8885	0.5059	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius lcr
7	18.9081	0.5057	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius Clear_BR_2
9	18.9803	0.5424	SHLD_Type_1 Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr AADT HC_Length_LOG
6	18.9889	0.4863	SHLD_Type_1 Median_Type_2 LSW Front_curve Radius lcr
7	19.0163	0.5047	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius lcr AADT
7	19.0436	0.5044	SHLD_Type_1 Median_Type_2 LSW Front_curve Radius lcr Clear_BR_2
8	19.0799	0.5228	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve lcr Clear_BR_2
8	19.0822	0.5228	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW lcr AADT Clear_BR_2
7	19.1036	0.5039	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Clear_BR_2 HC_Length_LOG
7	19.1056	0.5039	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Radius AADT Clear_BR_2
8	19.1290	0.5223	SHLD_Type_1 Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr AADT
10	19.1484	0.5595	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius lcr AADT Clear_BR_2 HC_Length_LOG
7	19.2273	0.5027	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius lcr HC_Length_LOG
7	19.2518	0.5025	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve AADT Clear_BR_2
7	19.4219	0.5009	SHLD_Type_1 Median_Type_2 LSW Front_curve Radius lcr HC_Length_LOG
9	19.5488	0.5371	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius AADT Clear_BR_2 HC_Length_LOG
6	19.5699	0.4808	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Clear_BR_2 HC_Length_LOG
8	19.5729	0.5182	SHLD_Type_1 Median_Type_2 LSW Front_curve Radius lcr Clear_BR_2 HC_Length_LOG
8	19.5928	0.5180	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius AADT Clear_BR_2
7	19.5971	0.4993	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Clear_BR_2 HC_Length_LOG
9	19.6290	0.5363	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW lcr AADT Clear_BR_2 HC_Length_LOG
8	19.6786	0.5172	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Radius AADT Clear_BR_2 HC_Length_LOG
8	19.7672	0.5164	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius Clear_BR_2 HC_Length_LOG
8	19.8100	0.5160	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius Clear_BR_2 HC_Length_LOG
7	19.8479	0.4969	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Front_curve Radius Clear_BR_2
6	19.8544	0.4782	Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr
8	19.8691	0.5154	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Radius lcr AADT Clear_BR_2

9	19.9223	0.5336	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Radius lcr Clear_BR_2 HC_Length_LOG
6	19.9725	0.4771	SHLD_Type_1 Median_Type_2 FT_Grade_2 AADT Clear_BR_2 HC_Length_LOG
9	20.0004	0.5329	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Front_curve lcr Clear_BR_2 HC_Length_LOG
6	20.0291	0.4766	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius Clear_BR_2
9	20.0342	0.5326	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Radius lcr AADT HC_Length_LOG
7	20.0353	0.4952	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Radius lcr Clear_BR_2
6	20.0495	0.4764	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 LSW Clear_BR_2
6	20.1019	0.4759	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Clear_BR_2 HC_Length_LOG
8	20.1034	0.5132	SHLD_Type_1 Median_Type_2 LSW Front_curve Radius lcr AADT Clear_BR_2
7	20.1362	0.4942	Median_Type_2 Pavement_Type_2 LSW Front_curve Radius lcr Clear_BR_2
7	20.1364	0.4942	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius AADT Clear_BR_2
5	20.1570	0.4567	SHLD_Type_1 Median_Type_2 FT_Grade_2 Clear_BR_2 HC_Length_LOG
6	20.1682	0.4753	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 AADT Clear_BR_2
6	20.1947	0.4750	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Radius Clear_BR_2
7	20.2060	0.4936	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Radius AADT Clear_BR_2
7	20.2165	0.4935	SHLD_Type_1 Median_Type_2 FT_Grade_2 LSW Radius lcr Clear_BR_2
9	20.2346	0.5307	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve Radius lcr AADT Clear_BR_2 HC_Length_LOG
6	20.2421	0.4746	SHLD_Type_1 Median_Type_2 Pavement_Type_2 FT_Grade_2 Front_curve Clear_BR_2
7	20.3011	0.4927	SHLD_Type_1 Median_Type_2 FT_Grade_2 Front_curve AADT Clear_BR_2 HC_Length_LOG
6	20.3136	0.4739	SHLD_Type_1 Median_Type_2 FT_Grade_2 Radius lcr Clear_BR_2

3. Model Selected

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The REG Procedure
Model: MODEL1
Dependent Variable: speed speed

Number of Observations Read	50
Number of Observations Used	50

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	6	94.03618	15.67270	7.21	<.0001
Error	43	93.46382	2.17358		
Corrected Total	49	187.50000			

Root MSE	1.47431	R-Square	0.5015
Dependent Mean	62.57000	Adj R-Sq	0.4320
Coeff Var	2.35625		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Intercept	Intercept	1	60.77945	1.10561	54.97	<.0001
SHLD_Type_1	SHLD_Type_1	1	1.80443	0.70720	2.55	0.0144
Media_Type_2	Media_Type_2	1	-2.52085	0.66958	-3.76	0.0005
FT_Grade_2	FT_Grade_2	1	-1.07067	0.47939	-2.23	0.0308
Front_curve	Front_curve	1	-1.51933	0.70203	-2.16	0.0360
Radius	Radius	1	0.00047184	0.00014698	3.21	0.0025

lcr	lcr	1	2.40775	0.75041	3.21	0.0025

Note: the variable “lcr” represents the created new variable using the two variables curve length and radius.

4. VIF Examination

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The REG Procedure
Model: MODEL1
Dependent Variable: speed speed

Number of Observations Read	50
Number of Observations Used	50

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	6	94.03618	15.67270	7.21	<.0001
Error	43	93.46382	2.17358		
Corrected Total	49	187.50000			

Root MSE	1.47431	R-Square	0.5015
Dependent Mean	62.57000	Adj R-Sq	0.4320
Coeff Var	2.35625		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Variance Inflation
Intercept	Intercept	1	60.77945	1.10561	54.97	<.0001	0
SHLD_Type_1	SHLD_Type_1	1	1.80443	0.70720	2.55	0.0144	1.21491
Media_Type_2	Media_Type_2	1	-2.52085	0.66958	-3.76	0.0005	2.07919
FT_Grade_2	FT_Grade_2	1	-1.07067	0.47939	-2.23	0.0308	1.06577

Front_curve	Front_curve	1	-1.51933	0.70203	-2.16	0.0360	1.94548
Radius	Radius	1	0.00047184	0.00014698	3.21	0.0025	2.03161
lcr	lcr	1	2.40775	0.75041	3.21	0.0025	2.14306

5. Box-Cox Examination

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The TRANSREG Procedure

Transformation Information
for BoxCox(speed)

Lambda	R-Square	Log Like
-3.00	0.51	-19.4685 *
-2.75	0.51	-19.4275 *
-2.50	0.51	-19.3914 *
-2.25	0.51	-19.3604 *
-2.00	0.51	-19.3344 *
-1.75	0.51	-19.3133 *
-1.50	0.51	-19.2973 *
-1.25	0.51	-19.2862 *
-1.00	0.51	-19.2801 *
-0.75	0.51	-19.2790 <
-0.50	0.51	-19.2828 *
-0.25	0.51	-19.2916 *
0.00	0.51	-19.3053 *
0.25	0.50	-19.3240 *
0.50	0.50	-19.3475 *
0.75	0.50	-19.3760 *
1.00 +	0.50	-19.4094 *
1.25	0.50	-19.4476 *
1.50	0.50	-19.4907 *
1.75	0.50	-19.5386 *
2.00	0.50	-19.5914 *
2.25	0.50	-19.6489 *
2.50	0.50	-19.7113 *
2.75	0.49	-19.7784 *
3.00	0.49	-19.8503 *

```
< - Best Lambda
* - Confidence Interval
+ - Convenient Lambda


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                The TRANSREG Procedure

    TRANSREG Univariate Algorithm Iteration History for BoxCox(speed)

Iteration    Average    Maximum                Criterion
   Number     Change     Change    R-Square       Change    Note
_________________________________________________________________________

        1    0.00000    0.00000     0.50764                 Converged

Algorithm converged.
```

VITA

Huafeng Gong was born on November 28, 1972, in Chongqing, China. In 1996, he graduated from Huazhong University of Science and Technology, where he received the degree of Bachelor of Science in Civil Engineering. After graduation, he worked for the Chengdu Municipal Engineering Design&Research Institute as a project engineer between 1996 and 2002. In 2003, he was enrolled into the graduate school at University of Kentucky and started pursuing his master degree in the Department of Civil Engineering. In 2004, he obtained his Master of Science degree from the Department of Civil Engineering in University of Kentucky. He continued the doctoral program right after the graduation. He worked as a research assistant in the Department of Civil Engineering to date.

Huafeng Gong

10/01/2007