OPERATING SPEED MODELS FOR LOW SPEED URBAN ENVIRONMENTS BASED ON IN-VEHICLE GPS DATA

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OPERATING SPEED MODELS FOR LOW SPEED URBAN ENVIRONMENTS BASED ON IN-VEHICLE GPS DATA

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To:

My mother Mingying Li and my father Wenliang Wang

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ABSTRACT

Low speed urban streets are designed to provide both access and mobility, and accommodate multiple road users, such as bicyclists and pedestrians. Generally, low operating speeds are desired on these facilities to achieve the intended functions and improve overall safety. However, speeds on these facilities often exceed the intended design speeds.

Current design speed approach for low speed urban streets often results in operating speeds higher than design speeds and may therefore be inappropriate for urban street environments. The design speed approach incorporates a significant safety factor to account for worst case scenarios, such as wet pavements and older drivers. As a result, the selected design speed may be lower than the speed a driver is likely to expect. Therefore, it is not surprising that during good weather conditions, general drivers feel comfortable traveling at speeds higher than the roadway's design speeds.

Numerous studies have indicated that the design speed concept, as implemented in the roadway design process in the United States, could not guarantee a consistent alignment that promotes uniform operating speeds less than design speeds. To overcome the shortfalls of the design speed approach, several previous studies have proposed a new performance-based design procedure with the incorporation of operating speeds. Under this procedure, the geometric parameters of the roadways are selected based on their influences on the desired operating speeds. This approach provides design consistency checks of existing highways as well as proposed preliminary alignment designs with a feedback loop. However, the operating speed approach

requires clear linkage between the relationships of operating speeds and various road characteristics. Although numerous studies have developed operating speed models, most of the previous research concentrated on rural two-lane highways. In contrast, highway designers and planners have very little quantifiable information regarding the influence of low-speed urban street environments on drivers' speeds.

The dataset used in this dissertation is generated by over 200 vehicles equipped with the Global Positioning System (GPS) receivers. The vehicle sample set is a random sample of personal vehicles in the Atlanta metro area. The vehicle location and speed information was recorded at one-second interval and periodically transferred to a data server via secure wireless access. The collected GPS-based vehicle activity data were projected onto a Geographic Information System (GIS) digital road map based on the latitude and longitude information so that the researchers know precisely where, when, and how fast the drivers were driving. By analyzing the detailed vehicle activity data, the author studied the relationship between the drivers' speed and the road environment. This dissertation determined that roadside objects including trees and utility poles, access density including driveway and intersection densities, number of lanes, lane width, on-street parking and sidewalk presence had significant influences on drivers' operating speeds.

This dissertation develops and calibrates operating speed models for low-speed urban streets based on roadway environments, including alignment, cross-section characteristics, roadside features, and adjacent land uses. The research results can help highway designers and planners to design and evaluate proposed low-speed urban roadway designs and improvements.

CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Low speed urban streets include urban local streets, collectors, minor arterials, and principle arterials with posted speed limits less than or equal to 64 km/h (40 mph). Low-speed urban streets are designed to provide both access and mobility, and accommodate multiple road users, such as bicyclists and pedestrians. Lower operating speeds are generally desired on these facilities to achieve the intended function and improve overall safety. However, speeds on these facilities often exceed the intended speeds of the roadways. These excessive speeds may cause potential safety problems as speed is directly related to the probability and severity of crashes, especially pedestrian involved crashes.

Researchers (McLean, 1979; Garber, 1989; Krammes, 1994; Fitzpatrick, 2003) found the current design speed approach for low speed urban streets often resulted in operating speeds higher than the design speeds and was therefore inappropriate for urban street environments. One possible reason is that the design speed approach incorporates a significant factor of safety, such as old drivers, poor weather and light conditions. As a result, the selected design speed may be lower than the speed a driver is likely to expect. Therefore, it is not surprising that under good weather conditions, general drivers feel comfortable traveling at speeds higher the roadway’s design speeds.

1.1.1 The Design Speed Concept

The most fundamental criterion in highway and street design in the Unites States is the design speed concept. The 2004 AASHTO's *A Policy on Geometric Design of Highways and Streets* (2004) defines the design speed as "a selected speed used to determine the various geometric design features of the roadway." For a given design speed, the 2004 AASHTO guideline presents the design values for geometric elements such as stopping sight distance, minimum curve radius, and the length of vertical curve.

The 2004 AASHTO's *A Policy on Geometric Design of Highways and Streets* presents the design speed concept to provide a roadway with the consistency in design features that encourage most drivers to operate uniformly at their desired speeds. The design consistency refers to the following two concepts:

- For an individual alignment element, the roadway design should encourage most drivers to operate consistently with the intended function of the facility. That is, the operating speeds should be less than the design speeds,
- For successive alignment elements, the roadway design should encourage most drivers to operate uniformly along the alignments. That is, the change of operating speeds between successive alignments should be less than some acceptable values.

The current design process begins with the selection of a design speed, which is determined by functional classification, speed limit, traffic volume, the characters of the terrain and adjacent land use, and environmental factors. Fitzpatrick et al. (2003) found that the most important factors in selecting a design speed value were functional classification and speed limit. Once the design speed is selected, the AASHTO design policy presents minimum design values for geometric elements to incorporate safety factors. Designers can choose geometric characteristic above minimum values based on the terrain and economic constraints.

1.1.2 Limitations of the Design Speed Concept

The practice of the design speed concept in the United States demonstrates that current design approaches do not always result in a consistent roadway design. Several studies (McLean, 1979; Garber, 1989; Krammes, 1994; Fitzpatrick, 2003) have found the disparity between operating speeds and design speeds. To explain the disparity, many researchers have analyzed the limitations in the selection and application process of the design speed. In several studies, researchers have proposed a new performance-based design approach with the incorporation of operating speed to overcome the limitations of the design speed concept.

1.1.2.1 Disparity between Operating Speeds and Design Speeds

The 2004 AASHTO's *A Policy on Geometric Design of Highways and Streets* defines the operating speed as the speed at which drivers are observed operating their vehicles under free flow conditions. The 85^{th} percentile speed is the most frequently used measure of the operating speed associated with a particular location, time of day, or geometric feature.

Garber and Gadiraju (1989) found that operating speeds were greater than design speeds when the design speed was less than 80 km/h (50 mph). McLean (1979) observed that horizontal curves with design speeds less than 90 km/h (55 mph) had 85th percentile speeds consistently higher than the design speeds, and horizontal curves with design speeds greater than 90 km/h had 85th percentile speeds consistently lower than the design speeds. Similarly, Krammes et al. (1994) found that the 85th percentile speeds were consistently higher than the design speeds on horizontal curves with design speeds less than 80 km/h (50 mph) and consistently lower than the design speeds on horizontal curves with design speeds greater than 100 km/h (65 mph). This study also found that the 85th percentile speeds averaged about 20 km/h (13 mph) higher than the design speed on horizontal curves with design speed between 40 km/h (25 mph) and 64 km/h (40 mph).

In another study, Fitzpatrick et al. (2003) found that a significantly larger percentage of vehicles exceeded the speed limits on suburban/urban roadways than on rural roadways. On roads with speed limits of 40 km/h (25 mph), 56 km/h (35 mph), and 64 km/h (40 mph), only 28, 22, and 32 percent, respectively, of the free-flow vehicles were traveling at or below the posted speed limits.

These studies demonstrate that the design speed approach does not always result in operating speed consistent with the intended speeds and functions of the roadways.

1.1.2.2 Limitations in the Selection of Design Speeds

In order to explain the disparities between operating and design speeds, researchers have examined the selection process of design speeds. The proposed functional classification and proposed speed limit were found to be the most important factors in the selection of design speed. Fitzpatrick et al. (2003) indicates that functional classification and speed limit were the first and second important factors used in the selection of design speed. Although speed studies are the accepted engineering method for setting speed limits, social and political pressures sometimes result in speed limits lower than the 85^{th} percentile speeds. Therefore, the selected design speed may be lower than the speed a driver is likely to expect. McLean (1988) pointed out that design speeds were no longer the speeds adopted by the faster driving group, but rather a value for the design and correlation of roadway elements.

Since drivers navigating the roadways neither know nor observe design speeds, they tend to drive at speeds that they consider comfortable and safe based on their perceptions of the roadway geometry regardless of the speed limit. Therefore, the overall speed of roadways may not be in agreement with the roadway's intended function.

1.1.2.3 Limitations of Design Speed Application Process

Several studies (Krammes, 2000; Fambro, 2000) have been conducted to explain the disparities between operating speeds and design speeds. They found several inherent fundamental flaws in the AASHTO design policy for applying the design speed concept.

- Design speed only applies to horizontal and vertical curves and not to the tangents between these curves. Design speed does not provide any guidance to determine the maximum tangent length. Therefore, designers can not control the maximum operating speeds on tangents since longer tangents encourage higher operating speeds, drivers may have to reduce their speeds significantly when they approach a sharp curve after driving a long, straight road segment.

- Design speed does not address the maximum operating speed issue, but simply assures that minimum design criteria are achieved. The AASHTO's *A Policy on Geometric Design of Highways and Streets* recommends higher minimum values whenever terrain and economy permit. Thus, different road features may have different minimum design values, which may violate drivers' expectancies of the roadway. For example, the non-controlling element (tangent) may be designed based on design speeds much higher than that of the controlling element (curve). When drivers approach the controlling element, the operating speeds may exceed the design speeds. In addition, minimum design standards incorporate many safety factors suitable for elder drivers and wet pavement condition. Therefore, it is not surprising to find that general drivers drive safely at speeds higher than the minimum design speeds under normal conditions.

- The current design speed approach lacks a feedback loop to compare the operating speed resulting from the designed roadway with the assumed design speed.

1.1.3 Performance-Based Design Approach with the Incorporation of Operating Speed

To overcome the shortfalls of the design speed approach, several studies (Krammes, 1997; Harwood, 2000; Fitzpatrick 2001) have suggested the incorporation of operating speed model with a feedback loop into the design speed concept. Under this approach, the geometric elements of roadways are selected based on their influences on the desired operating speeds. Generally, this method predicts the operating speed along the alignments, checks the design consistency, and if necessary, adjusts the design features until the predicted operating speeds are consistent with the design speeds. This approach, as shown in Figure 1-1, consists of the following steps:

- Chose an initial trial design speed
- Design the initial roadway element based on the selected design speed
- Predict the operating speed using the operating speed model
- Check the difference between the estimated operating speed and the design speed, and the difference between the operating speeds on successive geometric elements
- Modify the roadway design features to reduce these differences to acceptable levels if necessary.

The advantage of this iterative approach is that designers can check consistency between the design speed and the operating speed on individual alignment as well as between the predicted operating speeds on successive alignment elements for existing or proposed roadway design. Designers can iteratively assess the design prior to building a new roadway. Evaluating designs in this way would be more cost and time effective than having to alter roadways features after construction in order to bring the operating speed consistent with the designer’s goal.

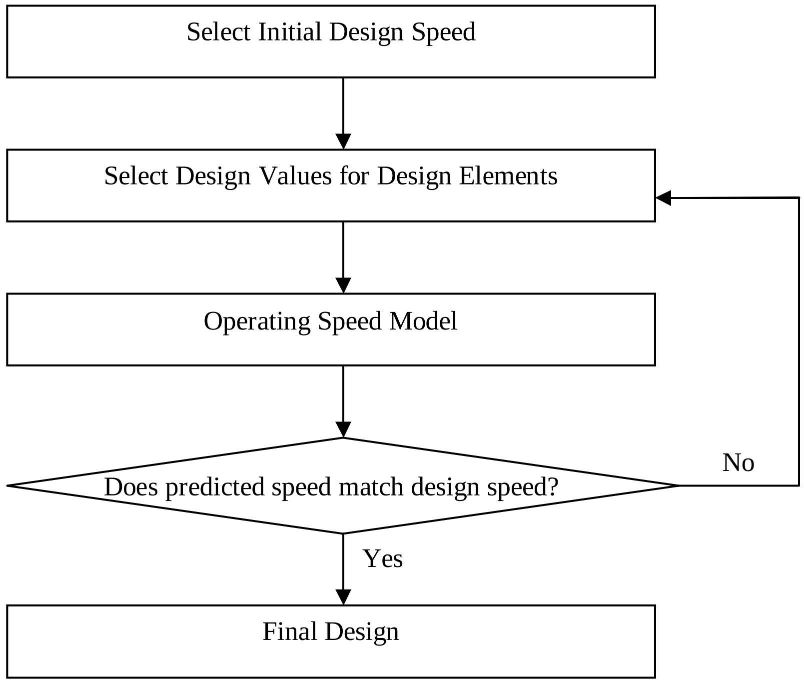

Figure 1-1 Operating Speed Design Approach

This approach requires operating speed models for different road environments. While numerous previous studies have developed operating speed models, most of them have concentrated on high-speed, rural highways. As a result, highway designers and planners have very little information about the influence of the low-speed street environment on operating speeds.

1.2 Dissertation Objectives

The objective of this dissertation is to study drivers' operating speed on low-speed urban roadways with the GPS-based vehicle activity data. The primary objectives of this study are the following:

- Develop a methodology for operating speed studies with GPS-based vehicle activity data,
- Study drivers' deceleration and acceleration behaviors on low-speed urban streets,
- Develop operating speed models to estimate operating speeds on low-speed urban roadways based on roadway environments, including alignment, cross-section characteristics, roadside features, and adjacent land uses.

1.3 Dissertation Contributions

This dissertation advances the state of art in modeling drivers' operating speeds on low-speed urban streets with GPS-based vehicle activity data.

1. Develop a methodology to study operating speeds on urban streets with GPS-based vehicle activity data. GPS has been widely used in transportation fields, such as vehicle tracing and navigation systems, road geometry measurements, trip reporting and travel time studies. This study is the first large scale comprehensive speed study on low-speed urban streets with in-vehicle GPS technologies. The GPS data, including speed and location, are collected by in-vehicle GPS equipments without any human interventions. This dissertation develops methods for GPS trips summarize, site selection, data reduction and analysis with in-vehicle GPS data. This study also demonstrates how to use GPS data to estimate horizontal curve radius using a curve fitting regression, which is a safe and effective alternative method of field measurement.

2. Study drivers' acceleration and deceleration behaviors on low-speed urban streets. The most significant character of low-speed urban streets is the closely spaced intersections with traffic control devices. Drivers have to make frequent stops. Drivers' acceleration and deceleration characteristics are a very important part in studying speed profiles on low-speed urban streets. Most previous acceleration/deceleration studies were based on outdated data rather than recent observations. Hence, the conclusions may not be reflective of today's drivers. Furthermore, due to the limitations of data collection methods, most previous studies could not provide accurate estimations of drivers' acceleration and deceleration behaviors, such as acceleration or deceleration time and distance. This is because different drivers may start to accelerate or decelerate at different time and location. In this study, the second-by-second speed profile data from in-vehicle GPS equipment can provide more detailed information about drivers' acceleration and deceleration behavior, such as acceleration time and distance, deceleration time and distance, and average acceleration and deceleration rates.

3. This dissertation is the first comprehensive attempt to develop operating speed models based on continuous speed on low speed urban streets. Previous studies have developed numerous operating speed models. However, most of these models were based on spot speeds, which means the researchers collected the speed data at specific locations of a roadway, mostly at the middle point of tangents and horizontal curves. Generally, the highest speeds along a tangent are considered to be the drivers' desired speeds. For horizontal curves, if its preceding tangent is long enough so that drivers reach their desired speed on the tangent, the lowest speeds along the horizontal curve are considered to be the desired speeds on horizontal curves. Therefore, most previous operating speed models are based on the assumptions that drivers reach their highest

speeds in the middle of tangents and reach their lowest speeds in the middle of horizontal curves. These assumptions may not be true. With second-by-second continuous speed data collected in this study, this dissertation can provide more detailed speed profile information and verify these assumptions.

4. This study is the first comprehensive attempt to develop operating speed models with the consideration of driver and vehicle effects. Most previous studies could not collect drivers' information because of the limitation of the data collection methods since it is difficult to obtain drivers' information by field observations. In this study, each speed record has its associated driver/vehicle information so that the researchers can include driver/vehicle effects in the operating speed modeling. This study collected drivers' information, such as age, gender, and vehicle type, through surveys. Although the driver/vehicle' characteristics are not included in the operating speed models as predictors, they are modeled as random effects in the linear mixed effects model while the road environment features are modeled as fixed effects. With traditional cross-sectional speed studies, all unexplained speed variation can be only considered as within-subject variation. Therefore, the researchers have no way to know if the variations across drivers (between-subject variance) is significant compared to within-subject variation. The mixed effects model used in this dissertation separates the unexplained speed variation into within-subject variation and between-subject variation, and calculates the proportion of speed variation that caused by individual driver and vehicle effects.

5. Develop preliminary speed profile models based on the roadway environmental features of low-speed urban streets including roadside objects, access densities, cross-section features,

alignment characteristics, and adjacent land uses. The results can help highway designers and planners to design and evaluate proposed low-speed urban roadway designs and improvements. These models could assist in estimating driver's selection of appropriate operating speeds on proposed roadways and enable designers and planners to assess the appropriateness of their designs.

CHAPTER 2

LITERATURE REVIEW

This dissertation reviews factors that may influence a driver's speed choice and existing speed models (and modeling techniques) of previous studies. The factors influencing drivers' speed choice include geometric alignment features, cross-section characteristics, roadside objects, adjacent land uses, traffic control devices, traffic volume, traffic calming measures, driver and vehicle characteristics. The existing speed models are divided into rural and urban conditions. Within the rural environment, researchers typically separately evaluate speed for roads with horizontal geometric controls (e.g. curves versus tangents) and roads with vertical controls; however, several models exist that evaluate a corridor including the combined influences of horizontal and vertical controls.

2.1 Factors Influencing Speed Choice

The *Highway Capacity Manual* (HCM) (2000) indicates that the speed of vehicles on urban streets is influenced by street environment, interaction among vehicles, and traffic control. The HCM further defines the street environment as the geometric characteristics of the facility, the character of roadside activity, and adjacent land use. The interaction among vehicles is due to traffic density, the proportion of trucks and buses, and turning movements. Traffic control refers to induced delays to the traffic stream such as the addition of signals and signs.

Numerous studies have identified a similar but separate category for the factors influencing vehicle speeds. These factors can generally be categorized as physical road characteristics, environmental influences, vehicle characteristics, and driver characteristics.

2.1.1 Physical Road Characteristics

Oppenlander (1966) reviewed several studies to identify variables that influence vehicle speed. He found that the roadway characteristics with the most significant influence on observed operating speed include horizontal curvature, functional classification, length of grade, gradient, number of lanes and surface type. Sight distance, lateral clearance and frequency of intersections were also determined to influence vehicle speeds. His list of factors is consistent with those identified in similar studies.

2.1.1.1 Functional Classification/Road Type

A Policy on Geometric Design of Highways and Streets by the American Association of State Highway and Transportation Officials (AASHTO) (2004) suggests urban and rural functional systems should be classified separately due to fundamentally different characteristics. A hierarchy of functional classification generally includes principal arterials, minor arterials, collectors, and local roads and streets.

The HCM (2000) indicates the urban environment street classes should be as further separated as follows:

- High Speed -- urban street with low driveway/access-point density, separate left-turn lanes, and no parking. Roadside development is low density and the speed limit for high speed streets is typically 72 to 88 km/h (45 to 55 mph).
- Suburban -- street with low driveway/access-point density, separate left-turn lanes, and no parking. Roadside development is low to medium density, and speed limits range from 64 to 72 km/h (40 to 45 mph).
- Intermediate -- urban street with a moderate driveway/access-point density, may have some separate or continuous left-turn lanes, and parking is permitted for portions of the road. Roadside development is higher than suburban streets and speed limits range from 48 to 64 km/h (30 to 40 mph).
- Urban -- streets with a high driveway/access-point density, parking may be permitted, there are few separate left-turn lanes, and possible pedestrian presence. Roadside development is dense with commercial uses and speed limits are 40 to 56 km/h (25 to 35 mph).

In the past, most urban speed analysis focused on speed conditions at interrupted locations such as signalized intersections. A few evaluated corridor speed characteristics. A study by Ericsson (2000), for example, compared driving patterns between and within different street configurations, traffic conditions, and types of drivers. There were four street types involved in this study; main street in a residential area, local feeder road in a residential area, radial arterial towards the city center, and street in the city center. The researchers found that average speed was significantly different for all investigated street types. The radial arterial towards the city center experienced the highest average speed while street in the city center had the lowest speeds.

Driving patterns varied greatly among the different street types. The findings of this experiment indicate that the greatest influence on an individual’s driving pattern was type of street followed by the driver type.

Gattis et al. (1999) analyzed the relationship between urban street width and vehicle speed for six two-lane urban streets in Fayetteville, Arkansas. The findings suggested that street width might played a small role in vehicle speed, but other factors such as street function might be more significant determinants of the average and 85th percentile speeds. In fact, they tentatively suggested that elevated speeds appeared to be associated with uninterrupted travel distance opportunities rather than road type and width.

2.1.1.2 Geometric Characteristics

Physical road and roadside characteristics directly impact the operating speed a driver selects. In general, past research has included the following eight "geometric" categories that strongly influence operating speed:

- Horizontal curvature,
- Vertical grade (and length of grade),
- Available sight distance,
- Number of lanes,
- Surface type and condition,
- Number of access points (intersections/driveways),

- Lateral clearance, and
- Land use type and density.

Kanellaidis (1995) surveyed drivers to determine the factors influencing their choice of speed on interurban road curves. A total of 207 Greek drivers were asked to rate 14 elements of the road environment as to how important the factors influence their speed choice on the interurban road curves. Sight distance was the most significant factor while free roadside space and speed limit signage influences were perceived to be minimal. Analysis of the survey data indicated that speed choice on curves can be described by four road-environment factors: separation of opposing traffic, cross-section characteristics, alignment, and signage.

Ottesen (2000) et al. studied the operating speeds on 138 horizontal curves and 78 approach tangents for 29 rural highways in 5 states. The researchers concluded that in addition to degree of curvature (radius), the length of curvature and deflection angle also significantly influenced vehicle speeds on curves. Kanellaidis et al. (1990) investigated the relationship between operating speed on curves and various geometric design parameters, including radius of curvature, desired speed, superelevation rate, lane width, shoulder width, and length of curve. They determined that the operating speed was strongly related to the horizontal curvature and the driver's desired speed.

Warren (1982) suggested the most significant roadway characteristics to be curvature, grade, length of grade, number of lanes, surface condition, sight distance, lateral clearance, number of intersections, and built-up areas near the roadway. Tignor and Warren (1990) similarly reported that the number of access points and nearby commercial development have the greatest influence on vehicle speeds.

Rowan (1962) studied the operating speeds within the urban environment in 1962. He observed a substantial speed reduction when sight distance was below 300 to 360 m (984 to 1180 ft) at a curbed urban cross section. Though the adjacent land use appeared to influence speed reduction, lateral restrictions influenced speed reduction more significantly than development density.

Cooper et al. (1980) found that average vehicle speeds increased by 2 km/h (1.2 mph) after resurfacing major roads in the United Kingdom; no change in traffic speed occurred in locations where surface unevenness remained the same after resurfacing. Parker (1997) found no change in speeds on two rural highways and a 5 km/h (3 mph) increase on two urban streets that were resurfaced and subsequently subjected to an increased speed limit.

The European Transport Safety Council (1995) reported that width, gradient, alignment and layout, and the consistency of these variables were the determinants of speed choice on a particular stretch of road. Road characteristics determine what is physically possible for a vehicle, but they also influence "...what seems appropriate to a driver." In this regard, the interaction of all roadway geometric variables appears to play a more significant role upon driver selected speed than that of one individual feature.

Tenkink (1991) performed an experiment where subjects in a driving simulator drove on a winding road. Each "driver" was asked to identify the highest possible safe speed. In one experiment, the researchers evaluated the subject's response to lead vehicle speed. "It concluded that uncertainty about the ability to respond adequately to lead vehicles, rather than uncertainty about roadway preview, dominates speed choice at these sight distances." Previous studies demonstrated reduced speeds at sight distances below 200 m (656 ft).

AASHTO encourages the use of operating speed under free-flow conditions for designing urban roadside features. The guideline indicates that more severe crashes can occur during high-speed conditions, and the nature of the urban environment deems it is likely that during high traffic volume conditions the operating speed will be lower due to the interaction of vehicles. The guideline also encourages designers to perform individual site studies before establishing restrictions regarding roadside environment design since the clear roadside concept is rarely attainable in a dense urban setting.

2.1.1.3 Traffic Volume

The influence of increasing traffic volume levels on operating speed is intuitive. Simply put, the more vehicles there are in a traffic stream, the less likely a driver can freely select his or her optimal speed. Similarly, the interaction of vehicles (e.g. slow vehicle turning into a driveway) directly influences the speed of vehicles in the vicinity. As a result, free-flow speed is commonly assumed to best represent driver's preferred operating speed. Free-flow speed on an urban street is the speed that a vehicle travels under low-volume conditions. The HCM (2000) further suggests the free-flow speed should be measured mid-block and as far as possible from the nearest signalized or stop-sign-controlled intersection.

Research studies where observed speeds, rather than just free-flow speeds, were collected support the influence of traffic volume on overall speed. Polus et al. (1984) evaluated the effect of traffic and geometric measures on highway vehicle speeds. The study determined that average curvature, average hilliness, and traffic volume each had a moderate negative correlation with average running speed. Driver's selected speeds were higher under low traffic volume conditions. Under heavy traffic flow, speeds were lower due to the influence of other vehicles in the traffic stream. This influence of prevailing traffic conditions was also observed by Ericsson (2000).

2.1.1.4 Traffic Control Devices

"The purpose of traffic control devices, as well as the principles for their use, is to promote highway safety and efficiency by providing for the orderly movement of all road users on streets and highways throughout the nation." (MUTCD, 2000)

Traffic control devices are implemented to regulate, direct, or advise drivers. The MUTCD (2000) emphasized that vehicle speed should be carefully considered when implementing various traffic control strategies. The regulatory posted speed limit is the traffic control device most frequently used as an indicator of operating speed. Several studies determined that posted speed limit is not an effective traffic control device for regulation of vehicle speed. Mustyn and Sheppard (1980) indicate more than 75-percent of drivers claim they drive at a speed that traffic and road conditions permitted, regardless of the posted speed limit. Although the drivers interviewed for the study tended to consider speeding to be one of the primary causes of crashes, they did not consider driving 16 km/h (10 mph) over the limit to be dangerous. Most of those interviewed did consider driving 32 km/h (20 mph) over the limit to be a serious offense.

Garber and Gadiraju (1989) studied speed variance for 36 roadway locations including intersections, arterials, and rural collectors. Their results suggested that drivers increased speed as geometric characteristics improved regardless of posted speed limit. A similar study by Leish and Leish (1977) pointed to the fact that drivers selected their speeds according to the highway ahead and may exceed both the speed limit and the design speed.

Parker (1997) evaluated the influence of rising and lowering posted speed limits on driver behavior for urban and rural unlimited access roadways for 98 sites in 22 states. He found that changing speed limits had no significant influence on driver speeds. He concluded that drivers determined speed according to their perception of the road. This perception is not changed due to the posted speed limit.

Other studies, however, have inconclusive observations about the level of influence of posted speed limits on driver behavior. Fitzpatrick et al. (2001) investigated geometric, roadside, and traffic control device variables and their influence on driver behavior for major suburban four-lane arterials. They observed that the only significant variable influencing speed on tangent sections of road was the posted speed limit. In addition to posted speed limit, deflection angle and access density influenced speed on curve sections.

Zwahlen (1987) found that advisory speed signs on curves are not generally heeded by drivers and may even produce opposite effect for which they are intended.

Other traffic control devices have little impact on driver selected speeds. Várhelyi (1998) studied drivers' speed behavior at zebra pedestrian crossings. He suggested that the willingness of drivers to give priority to pedestrians at the zebra crossing was low, and that drivers did not obey the law concerning speed behavior at the zebra crossings.

2.1.1.5 Traffic Calming Techniques

"There's more to life than increasing its speed."

Mahatma Gandhi

The above quotation by Gandhi embraces the concept of traffic calming. Traffic calming is the implementation of unique traffic control strategies that reduce traffic and lower vehicle speeds in residential and local service regions. Traffic calming strategies may range from physical modifications (chokers, speed humps, etc.) to increased enforcement, modified road use (on-street parking, bicycle lanes, etc.), and time-based exclusions. Several researchers have evaluated feasible traffic calming strategies and their impact on operating speed.

Ewing (1999) explained that speed impacts of traffic calming measures depend primarily on geometrics and device spacing. He identified numerous speed studies where before-after evaluation of calming devices resulted in speed reductions. Representative examples of traffic calming strategies that resulted in reduced speeds as summarized in his report include speed humps, raised intersections, traffic circles, narrowings, and diagonal divertors.

Amour (1986) determined that the presence of an enforcement symbol (e.g. a police car) might reduce the vehicle speeds on an urban road. He also demonstrated it was possible to produce a memory effect of police presence in an urban situation, but drivers returned to their normal driving behavior very soon after passing a police vehicle.

Roadway restrictions are effective traffic calming strategies. Many residential streets are considerably wider than necessary for prevailing traffic conditions. Officials in Anne Arundel County, Maryland, painted parking lane lines without centerline striping on residential streets. This method visually narrowed the street and reduced vehicle speed by 4.8 to 6.4 km/h (3 to 4 mph) (Water, 1994). It is important to note, however, the opponents of this strategy suggest the visually narrowed street directs vehicles into the path of approaching traffic and introduces safety hazards.

Comte et al. (2000) used a driving simulator to investigate the effectiveness of speed-reducing measures ranging from intrusive devices (speed limiter or in-car advice) to informational devices such as variable message signs or transverse bars. All speed-reducing measures evaluated proved to be effective, with speed limiters proving to be the most influential.

Barbosa et al. (2000) investigated the influence of varying combinations of traffic calming measures on vehicle speeds by evaluating differences in speed profiles. Five roads in the City of

York (UK) were selected for this case study. The study focused on traffic calming measures including speed humps, speed cushions, and chicanes implemented in sequence. The researchers concluded that calming measures of the same design tended to produce similar influences on speeds, and the effectiveness of the measures in reducing speed decreased under higher entry speed conditions.

Stop signs are the most publicly requested regulatory measures to slow traffic on streets. Many studies indicate, however, this strategy has a weak or negligible effect on overall traffic speeds. (Basically, drivers who do slow their speed at the intersection generally pick up speed quickly in mid-block locations to compensate for the "lost time.") Before-after speed studies conducted in the City of Troy indicated that stop signs were not effective in controlling speeds and compliance with these stop signs was extremely poor (Beaubien, 1989).

2.1.2 Physical Environment Characteristics

Lighting conditions (e.g. daylight, dawn, dark, etc.) and environmental influences such as heavy rain or snow may influence operating speed. Very few studies address specifically light or weather constraints, and most of the past studies focused on rural road locations.

The AASHTO *Roadside Design Guide* (1996) indicates that operating speeds on urban and suburban roads have greater variation by time of day than rural roads. During the lower volume and higher speed period of 7 p.m. to 7 a.m. (generally corresponding to nighttime conditions) there are a greater percentage of crashes due to the higher speeds and greater speed variances.

Liang et al. (1998) evaluated the effect of visibility and other environmental factors on driver speed. They determined that drivers reduced their speeds during poor environmental condition such as heavy rainfall or high winds. This reduction was accompanied by a higher variation in speeds.

Lamm et al. (1986) compared vehicle speeds during dry and wet conditions on two-lane rural highways in New York. This research team concluded that operating speeds on dry pavements were not statistically different for operating speeds on wet pavements.

2.1.3 Vehicle Characteristics

Very little research exists on the speed characteristics of individual vehicle types in a general traffic stream. A common segregation of vehicles is the categories of passenger cars, heavy vehicles, buses, and recreational vehicles. For emission analysis, vehicle fleet characteristics are further defined based on number of axles and age of the vehicle. For speed analysis, due to the random nature of the data collection, the most common means of evaluating vehicle characteristics is to simply separate heavy vehicles from all other vehicles and study their behavior independently. The existing speed model section of this chapter summarizes several methods for estimating operating speeds. It is interesting to note that the predominant approach to speed modeling is to limit the study to passenger cars only. In the rural environment, only one researcher summarized elected to model truck behavior and that was at the exclusion of the passenger cars. In this environment a variety of vehicle fleet characteristics were included in the models. The isolation of specific speed influences beyond the broad categories of truck versus car does not appear in the available literature.

2.1.4 Driver Characteristics

Many previous studies concentrated on the relationship between drivers' speed selection and road/vehicle characteristics without considering other important factors such as personal characteristics and drivers' perception of the roadway environment.

Scallen and Carmody (1999) investigated the effects of roadway design on human behavior in Tofte, Minnesota. They found that white pavement treatments produced more moderate speeds and large speed changes, and landscape architecture treatments on the medians and roadside also produced desirable effects in driver's selection of speeds.

A speed management Transportation Research Board report (1998) stated:
"In many speed zones, it is common practice to establish the speed limit near the 85th percentile speed, that is, the speed at or below which 85 percent of drivers travel in free-flow conditions at representative locations on the highway or roadway section. This approach assumes that most rivers are capable of judging the speed at which they can safely travel."

This speed approach is not recommended for urban roads, however, because of the mix of road users, high traffic volume, and level of roadside activity. Perception of safe speed is influenced by judgment of vehicle capability, anticipation of roadway conditions (further influenced by familiarity with the route), fatigue or similar factors, and judgment of speed on crash probability and severity. Most drivers do not perceive the act of driving as life-threatening, they believe themselves to be good drivers, and they often misjudge vehicle speed. People use the following information in determining driving speed:

"…characteristics of the road; the amount of traffic on the road; weather conditions and time of day; the speed limit and its enforcement; the length and purpose of the trip; the vehicle's operating characteristics, such as handling and stopping as well as fuel consumption and emissions; and driver-related factors, such as the propensity to take risks and the pleasure associated with driving fast."

Perceptual countermeasures can be used to influence drivers' perception of safe speed. These countermeasures include patterned road surfaces, center and edge-line treatment, lane-width reduction, curvature enhancements, and delineators (guideposts).

Kang (1998) analyzed Korean drivers' speed selection behavior by taking into account such factors as personal, vehicle, attitudinal and trip characteristics. He concluded that male drivers with higher income tended to drive faster, experienced drivers drove at a higher speed than others, and trip distance and frequent use of the road were also important factors for speed selection behavior.

Poe et al. (1996) studied the relationship of operating speed and roadway design speeds for low-speed urban streets. In this study, both driver and vehicle characteristics were evaluated. They observed that gender, number of passengers, and passenger vehicle types were not significant. The analysis indicated that senior drivers traveled about 2 km/h (1.2 mph) slower than young drivers. They also investigated how the perspective view of horizontal curves might influence the relationship between perceived speed, operating speed, and geometric design speed. Their results indicated that the visual perspective view of a horizontal curve might be an important factor in the selection of an appropriate speed on horizontal curves. This suggests that a three-dimensional approach to horizontal curve design for low-speed alignments would assist in design consistency.

Hassan et al. (2000) suggested that combined horizontal and vertical alignment could cause a distorted perception of the horizontal curvature and could affect the drivers' choice of operating speed on horizontal curves. They determined that horizontal curvature looked consistently sharper when overlapped with a crest vertical curve and consistently flatter when overlapped with a sag vertical curve. Gibreel et al. (2001) also found that overlapping vertical alignment could influence the drivers choice of speed on horizontal curves. They found that drivers adopt higher operating speeds on horizontal curves combined with sag vertical curves compared to the speeds on horizontal curves combined with crest vertical curves.

Based on data from Swedish drivers on roads with speed limits of 88 km/h (55 mph), researchers investigated drivers' attitudes towards speeding and influences from other road users on the drivers' speed choice. Haglund (2000) suggested that drivers might influence the driving patterns of others and that they might adjust their own speed in accordance with their estimate of the behavior of other drivers. Elslande et al. (1997) found that in most situations, experienced individuals can use knowledge of a task to enhance performance. However, it is possible for experienced individuals to become overconfident, and particularly in a driving task, to encounter more risky situations. Drivers use consistent behavior in an environment, even if their vision is impaired by some object. The automaticity driving prevents them from executing a complete visual search of the environment. Also, drivers sometimes fail to update information. They ignore cues which present information indicating a change to their expectancies. These problems can be characterized as perceptive negligence, interpretational errors, or temporary breakdown of observation.

Alison Smiley (1999) found that a driver's main cue for speed comes from peripheral vision. When peripheral vision is eliminated, drivers use only the central field of view to determine speed. If peripheral stimuli are close by, then drivers feel that they are going faster than if they encounter a wide-open situation. Dr. Smiley pointed out that speed was most influenced by geometric demand (i.e. sight distance, sharpness of curves, grades, etc.).

Bartmann et al. (1991) also examined the effects of driving speed and route characteristics in the visual field. As speed increases, the visual field, from which the driver gathers information, decreases. Thus, peripheral vision gets greatly reduced at higher speeds, taking away a number of relevant driving cues. Six subjects wore eye movement helmets and were asked to drive on three different road types at varying speeds. On the urban street they were asked to drive at 50 km/h (31 mph) and 30 km/h (19 mph). Relevant eye fixations fell in the following categories: mirror, traffic control devices, traffic, and road related. The researchers concluded that urban street at higher speed corresponded to greater relevant object fixation. Driving speed influences perceptual behavior depending on road type.

In 1997, the National Highway Traffic Safety Administration (NHTSA) (1998) commissioned a national survey of the driving public. The survey was conducted by telephone by the national survey research organization, Schulman, Ronca and Bucuvalas, Inc. A total of 6,000 interviews were completed with a participation rate of 73.5 percent. Six basic speed-related questions were presented to the subjects:

(1) Drivers were asked how important a series of factors were in selecting their speeds.

- The most important factor was the weather condition. 86 percent of drivers felt weather was extremely important.
- The second most important factor is posted speed limits. 54 percent of the respondents rated this factor as extremely important and 35 percent of the respondents rated it as moderately important
- The third most important factor is previous experience on the road, which was rated as extremely or moderately important by 84 percent of drivers.
- The next three factors are traffic density, the likelihood of being stopped by police, and the speed of other traffic.

(2) Drivers felt the maximum safe speed for residential streets, whether in urban or rural settings, was 40 to 56 km/h (25 to 35 mph). The maximum safe speed for non-interstate urban roads was 72 to 88 km/h (45 to 55 mph).

(3) Drivers were asked why they consider speeds greater than the maximum speed to be unsafe on residential streets.

- The most important factor are the presence people, especially children, schools and playgrounds in close proximity to the roads
- The second most common reason is drivers' reaction times and the ability of the vehicle to stop quickly.
- The third reason is traffic patterns, especially heavy traffic and merging, cited by about one in six drivers.
- Other factors include safety and road conditions, weather conditions, and presence of other vehicles.

(4) Drivers who reported that they drove faster now than they used to one year ago said they were driving faster due to the increased speed limits, cited by more than half of the respondents. The second most common reason was the increased experience of the driver and improved traffic flow conditions.

(5) Drivers who reported they drove slower than they used to also were asked to explain the reasons. Two of five drivers identified driver-related issues, especially the maturity of the driver. One of three drivers indicated that safety concerns were the reason for driving slower. About half of these concerns were related to more cautious driving behavior. Manny of the slower drivers reported that they reduced speeds due to vehicle-related factors, such as having children or other family members in the car. Finally, the heightened police enforcement is also a reason for driving slower.

(6) Those drivers who reported that other drivers were driving more aggressively in their area than during the previous year were asked the possible reasons. About 23 percent drivers said that drivers were more aggressive now because they were hurried. About 22 percent drivers indicated that the increased aggressiveness of driving was due to the increased traffic volume and congestion. Several respondents indicated that higher speed limits were a contributing factor for increases in aggressive driving in their areas and less police enforcement was also a factor.

2.2 Reviews of Existing Operating Speed Models

Existing operating speed models primarily focus on rural environments where drivers encounter uninterrupted traffic flow conditions and minimal variability. Limited research to date exists for urban environment speed estimation. Operating speed in urban areas may be influenced by a vast array of land use development issues, numerous road geometric features, and varying driver or vehicle characteristics not consistent with the rural environment. As a result, rural speed models and their "critical influences" on operating speed are initially reviewed in this summary to help identify factors transferable from rural speed models to a future urban speed model.

The 85th percentile speed is the general statistic used to describe operating speeds when assessing the influence of the driver's environment on speed selection. The 85th percentile speed is the speed at or below which 85-percent of the vehicles in the traffic stream travel. This speed measure is the most common factor used to set speed limits on existing roads in the United States and is internationally accepted as a reasonable representation of operating speed. However, conditions under which the 85th percentile speeds are measured strongly influence perceived significant variables. For example, if a researcher elects to assess the influence of roadside trees on operating speed and only collects speed data during peak hour conditions, it is likely the prevailing traffic will exert a strong influence on the observed 85th percentile speed and minimize the influence of extraneous roadside features. It is reasonable to consider the 85th percentile speed for only free-flowing vehicles. Again the peak hour influence may confound the tree influence. Drivers may be in a hurry to return home or retrieve their children from school. As a result, the time of day may influence the driver's behavior. It is necessary, therefore, to identify a comprehensive model that captures variables beyond physical road features and to study operating speeds for a variety of road, driver, and environment configurations.

2.2.1 Operating Speed Models for Rural Highways

Existing speed models are divided into rural and urban conditions. Within the rural environment, researchers typically separately evaluate speed for roads with horizontal geometric controls (e.g. curves versus tangents) from roads with vertical controls; however, several models exist that evaluate a corridor that includes the combined influences of horizontal and vertical influences collectively.

2.2.1.1 Operating Speed Models for Rural Horizontal Geometric Controls

Many researchers have developed similar models for the estimation of the 85^{th} percentile speeds on rural roads at horizontal curves, for a variety of speed limits, vertical grades, and vehicle types (primarily passenger cars or heavy-duty vehicles). Most of previous studies identified the primary independent variable influencing operating speeds was the radius of the curve (or a surrogate measure such as degree of the curve or inverse of the radius).

McLean (1979) studied operating speeds on 120 two-lane rural alignments with low, intermediate and high-speed. He observed that the 85th percentile curve speeds were dominantly influenced by both the driver's desired speed and the curve radius. The model is represented by the following equations:

$$V85 = 53.8 + 0.464V_F - 3.26(1/R)*10^3 + 8.5(1/R)^2*10^4 \qquad R^2 = 0.92 \qquad (2\text{-}1)$$

Where

$V85$ = Estimate of 85th percentile curve speed (km/h),

V_F = Desired speed of the 85th percentile (km/h),

R = Curve radius (m), and

R^2 = Coefficient of determination.

Glennon et al (1983) studied operating speeds of passenger vehicles on 56 alignments in four states. The relationship between operating speeds and degree of curve was quantified by the following model:

$$V85 = 150.08 - 4.14DC \qquad R^2 = 0.84 \qquad (2\text{-}2)$$

Where:

$V85 = 85^{th}$ percentile speed (km/h), and

DC = Degree of curve (degree/30m),

Lamm et al. (1986) compared one American and two European methods for evaluating speed consistency on horizontal alignments. He found that the curvature-change-rate was the most convenient for predicting changes in operating speed profile along a rural roadway. Later, Lamm et al. (1987, 1988, 1990) investigated operating speeds on 261 two-lane, rural highway section in New York state and suggested the lane width, shoulder width, and traffic volume explained approximately 5.5 percent of the variation in operating speeds over a simple speed model that only considers curve radius:

$$V85 = 93.85 - 1.82DC \qquad R^2 = 0.787 \qquad (2\text{-}3)$$

Where,

$V85 = 85^{th}$ percentile speed (km/h),

DC = Degree of curve (degree/100 ft). Range: 0^o to 27^o, and

R^2 = Coefficient of determination.

Kanellaidis et al. (1990) investigated the passenger vehicle speeds on horizontal alignment of two-lane rural highways in Greece and developed a simple model to predict the 85^{th} percentile speed on the basis of degree of curvature solely:

$$V85 = 129.88 - 623.1/(1/R)^{0.5} \qquad R^2 = 0.78 \qquad (2\text{-}4)$$

Where:

V85 = 85the percentile speed on the horizontal curve (km/h), and

R = curve radius (m).

Krammes et al. (1995) studied operating speeds on 138 horizontal curves and 78 of their approach tangents in five states (Texas, New York, Oregon, Pennsylvania, and Washington), and developed thee operating speed models to evaluate consistency of horizontal alignment designs for two-lane rural roadways.

$$V85 = 103.66 - 1.95D \qquad R^2 = 0.80 \qquad (2\text{-}5)$$

$$V85 = 102.4 - 1.57D + 0.0037L - 0.10\Delta \qquad R^2 = 0.82 \qquad (2\text{-}6)$$

$$V85 = 41.62 - 1.29D + 0.0049L - 0.12\Delta + 0.95V_t \qquad R^2 = 0.90 \qquad (2\text{-}7)$$

Where,

V85 = 85th percentile speed on a curve (km/h),

D = degree of curvature (degrees),

L = length of curve (m),

Δ = deflection angle (degrees), and

V_t = 85th percentile speed on approach tangent (km/h)

McFadden and Elefteriadou (1997) used bootstrapping to formulate and validate speed profile models using the same dataset collected by Krammes et al. (1995). The new “bootstrap” models were not significant different from those developed by Krammes et al. (1995).

$$V85 = 104.61 - 1.90D \qquad R^2 = 0.74 \qquad (2\text{-}8)$$

$$V85 = 103.13 - 1.58D + 0.0037L - 0.09\Delta \qquad R^2 = 0.76 \qquad (2\text{-}9)$$

$$V85 = 54.59 - 1.50D + 0.0006L - 0.12\Delta + 0.81V_t \qquad R^2 = 0.86 \qquad (2\text{-}10)$$

Where,

V85 = 85th percentile speed on a curve (km/h),

D = degree of curvature (degrees),

L = length of curve (m),

Δ = deflection angle (degrees), and

V_t = 85th percentile speed on approach tangent (km/h)

In 2000, McFadden and Elefteriadou (2000) suggested a new parameter for analyzing design consistency, the 85th percentile maximum reduction in speeds (V85MSR). They calculated the V85MSR by using the 85th percentile speed at the midpoint of approach tangent and the 85th percentile speed at the midpoint of the horizontal curve and determining the maximum speed reduction.

$$V85MSR = -14.90 + 0.144*V_t + (0.0153*LAPT + 954.55/R \qquad R^2 = 0.71 \qquad (2\text{-}11)$$

$$V85MSR = -0.812 + 998.19/R + 0.017*LAPT \qquad R^2 = 0.60 \qquad (2\text{-}12)$$

Where,

V85MSR = 85th percentile speed reduction into curve (km/h),

V_t = 85th percentile speed at 200 meter prior to point of curvature (km/h),

R = horizontal curve radius (m), and

LAPT = length of approach tangent (m).

Islam et al. (1997) investigated operating speeds of passenger vehicles on two-lane rural highways at eight sites in Highway 89 in Northeastern Utah. They collected data at three points for each site: the beginning of curve (PC), middle of curve (MC) and end of the curve (PT) and determined a statistical relationship between the 85th percentile speed and degree of curve:

$$V_{85}^{1} = 95.41 - 1.48*DC - 0.012*DC^2 \qquad R^2 = 0.99 \qquad (2\text{-}13)$$

$$V_{85}^{2} = 103.03 - 2.41*DC - 0.029*DC^2 \qquad R^2 = 0.98 \qquad (2\text{-}14)$$

$$V_{85}^{3} = 96.11 - 1.07*DC \qquad R^2 = 0.90 \qquad (2\text{-}15)$$

Where:

V_{85}^{1} = 85th percentile speed (km/h) at the beginning of curve (PC),

V_{85}^{2} = 85th percentile speed (km/h) at the middle of curve (MC),

V_{85}^{3} = 85th percentile speed (km/h) at the end of curve (PT), and

DC = degree of curvature (degrees per 30m)

Cardoso et al. (1998) studied operating speeds on 50 curves in four countries. They found that the only significant variables were curve radius and the 85th percentile speed on the preceding tangent. The same study was conducted on 80 tangents. The significant variables included tangent length, bendiness, lane width, average grade, and preceding curve radius.

$$V_{85}{}^{1} = 49.220\ \frac{292736}{R^2} + 0.454Va \qquad R^2 = 0.80 \qquad (2\text{-}16)$$

$$V_{85}{}^{2} = 51.765\ \frac{337.780}{\sqrt{R}} + 0.6049Va \qquad R^2 = 0.71 \qquad (2\text{-}17)$$

$$V_{85}{}^{3} = 41.363\ \frac{294.000}{\sqrt{R}} + 0.699Va \qquad R^2 = 0.92 \qquad (2\text{-}18)$$

$$V_{85}{}^{4} = 25.010\ \frac{271.500}{\sqrt{R}} + 0.877Va \qquad R^2 = 0.90 \qquad (2\text{-}19)$$

$$V_{85}{}^{5} = 97.737 + 0.007436\ L - 45.707\ Bend \qquad R^2 = 0.65 \qquad (2\text{-}20)$$

$$V_{85}{}^{6} = -17.17 + 0.02657\ L + 33.711LW - 21.936 \qquad R^2 = 0.77 \qquad (2\text{-}21)$$

$$V_{85}{}^{7} = 134.069 - 3.799Hill - 126.59\ Bend \qquad R^2 = 0.92 \qquad (2\text{-}22)$$

$$V_{85}{}^{8} = -29.95 - 34.835LW - 0.0347PRad - 43.124\ Bend \qquad R^2 = 0.82 \qquad (2\text{-}23)$$

Where,

$V_{85}{}^{1}$= 85th percentile speed on France horizontal curves (km/h),

$V_{85}{}^{2}$= 85th percentile speed on Finland horizontal curves (km/h),

$V_{85}{}^{3}$= 85th percentile speed on Greece horizontal curves (km/h),

$V_{85}{}^{4}$= 85th percentile speed on Portugal horizontal curves (km/h),

$V_{85}{}^{5}$= 85th percentile speed on France tangents (km/h),

$V_{85}{}^{6}$= 85th percentile speed on Finland tangents (km/h),

$V_{85}{}^{7}$= 85th percentile speed on Greece tangents (km/h),

$V_{85}{}^{8}$= 85th percentile speed on Portugal tangents (km/h)

Va = 85th percentile speed on approach tangent (km/h),

R = horizontal curve radius (m),

L = length of curve (m),

Bend = bendiness (degree/km),

LW = land width (m),

Hill = hilliness (percent), and

PRad = radius of the preceding curve (m)

Andjus (1998) studied the passenger vehicle speeds on 9 horizontal curves on two-lane rural roads and developed the following models:

$$V85 = 16.92 \ln R - 14.49, \quad R^2 = 0.975 \tag{2-24}$$

$$V50 = 14.75 \ln R - 11.69, \quad R^2 = 0.969 \tag{2-25}$$

Where,

V85 = free-flow 85th percentile passenger car speed (km/h),

V50 = free-flow 50th percentile passenger car speed (km/h), and

R = radius of the horizontal curve (m).

Passetti et al. (1999) collected operating speeds and geometric data at 12 spiral transition curves and 39 circular curves on rural two-lane highways that had similar geometric characteristics in six states and found that spiral transitions did not significantly affect the 85th percentile speed of drivers on horizontal curves. A model was developed to estimate the 85th percentile speed on a horizontal curve:

$$V85 = 103.9 – 3030.5(1/R) \qquad R^2 = 0.68 \qquad (2\text{-}26)$$

Where,

V85 = 85the percentile speed on the horizontal curve (km/h), and

1/R = inverse of curve radius (1/m).

Andueza (2000) studied operating speeds of passenger cars on horizontal curves and tangents on rural two-lane roads of the Venezuelan Andean Highway and developed a rural speed model that included radii for consecutive curves, tangent length before the curve, and a minimum sight distance for the horizontal curve.

$$V_{85}^{c} = 98.25 – 2795/R2 – 894/R1 + 7.486D + 9308L1 \qquad R^2 = 0.84 \qquad (2\text{-}27)$$

$$V_{85}^{t} = 100.69 – 3032/R1 + 27819L1 \qquad R^2 = 0.79 \qquad (2\text{-}28)$$

Where:

V_{85}^{c} = estimated 85th percentile speed on the curve (km/h),

V_{85}^{t} = estimated 85th percentile speed on the tangent (km/h),

R2 = radius of the following curve (m),

R1 = radius of the previous curve (m),

D = S/250,

S = minimum sight distance for the curve (m), and

L1 = tangent length before the curve (m).

Fitzpatrick et al. (2000) investigated vehicle speeds at 176 two-lane rural highway sites in six states (Minnesota, New York, Oregon, Pennsylvania, Texas, and Washington) and developed several models were developed to predict operating speed of passenger cars for different condition, such as on horizontal and vertical curves, and on tangent sections. In this study, the combination of horizontal and vertical alignment has been systematically studied for the first time.

Horizontal curve on grade between –9% and –4%:

$$V85 = 102.10 - 3077.13/R \qquad R^2 = 0.58 \qquad (2\text{-}29)$$

Horizontal curve on grade between –4% and 0%:

$$V85 = 105.98 - 3709.90/R \qquad R^2 = 0.76 \qquad (2\text{-}30)$$

Horizontal curve on grade between 0% and 4%:

$$V85 = 104.82 - 3574.51/R \qquad R^2 = 0.76 \qquad (2\text{-}31)$$

Horizontal curve on grade between 4% and 9%:

$$V85 = 96.61 - 2752.19/R \qquad R^2 = 0.53 \qquad (2\text{-}32)$$

Horizontal curve combined with sag vertical curve:

$$V85 = 105.32 - 3438.19/R \qquad R^2 = 0.92 \qquad (2\text{-}33)$$

Horizontal curve combined with unlimited sight distance crest vertical curve:

Use lowest speed of the speed predicted from equation 2-19 or 2-30 (for the upgrade) and equation 2-31 or 2-32 (for the downgrade).

Horizontal curve combined with limited sight distance crest vertical curve:

$$V85 = 103.24 - 3576.51/R \qquad R^2 = 0.74 \qquad (2\text{-}34)$$

Sag vertical curve on horizontal tangent:

V85 = assumed desired speed (2-35)

Vertical crest curve with non limited sight distance on horizontal tangent:

V85 = assumed desired speed (2-36)

Vertical crest curve with limited sight distance on horizontal tangent:

$$V85 = 105.08 - 149.69/K \qquad R^2 = 0.80 \qquad (2\text{-}37)$$

Where:

V85 = 85th percentile speed of passenger cars (km/h),

R = Radius of curvature (m), and

K = Rate of vertical curvature.

Ottesen et al. (2000) developed a speed profile model with speed and geometry data collected at 138 horizontal curves on and 78 approach tangents 29 rural two-lane highways in 5 states. The speed profile model added the horizontal curve length and the approach speed tangent to the model (in addition to the radius) as the following:

$$V85 = 102.44 – 1.57DC + 0.012L – 0.01DC*L \qquad R^2 = 0.81 \qquad (2\text{-}38)$$

Where,

V85 = Estimated 85th percentile speed on the curve (km/hr),

DC = Degree of curve (degree/100 ft), and

L = Length of curve (m).

Polus et al. (2000) invesitated passenger vehicle speeds on 162 tangent sections of two-lane rural highways and developed four speed models for tangents located between horizontal curves. They categorized the horizontal geometry as one of four conditions:

- Group 1 -- sharp curve radii and short connecting tangent,
- Group 2 -- sharp curve radii and moderate length tangent,
- Group 3 -- moderate curve radii and moderate length tangent, and
- Group 4 -- flat curve radii with long tangent.

The corresponding models are the following:

$$V_{85}^{1} = 101.11 - 3420/GMs \qquad R^2 = 0.55 \qquad (2\text{-}39)$$

$$V_{85}^{2} = 105.00 - 28.107/e^{(0.00108*GML)} \qquad R^2 = 0.74 \qquad (2\text{-}40)$$

$$V_{85}^{3} = 97.73 + 0.00067*GM \qquad R^2 = 0.2 \qquad (2\text{-}41)$$

$$V_{85}^{4} = 105.00 - 22.953/\ e^{(0.00012*GML)} \qquad R^2 = 0.84 \qquad (2\text{-}42)$$

Where:

V_{85}^{1} = 85th percentile speed for group 1 (km/h),

V_{85}^{2} = 85th percentile speed for group 2 (km/h),

V_{85}^{3} = 85th percentile speed for group 3 (km/h),

V_{85}^{4} = 85th percentile speed for group 4 (km/h),

TL = tangent length (m),

R1 = previous curve radii (m),

R2 = following curve radii (m),

GMs = (R1 + R2)/2 (m),

GML = $(TL^* (R1^*R2)^{0.5})/100$ (m^2), and

The research team determined, for group 1 operating speed, only the radii of the curves proved significant; however, for group 2, the length of tangent was also significant. Due to limited data sets available, their speed models for groups 3 and 4 were inconclusive. Preliminary models appeared to depend on factors similar to those in group 2, but the researchers cautioned that characteristics such as cross-section, vertical longitudinal slope, and change of vertical curve rate (if vertical curvature is present) also might influence operating speeds.

In the study by Gibreel et al. (2001), the operating speed models for two-lane rural highways accounted for the effect of the three-dimensional nature of highways. Two types of 3D combinations were considered: a horizontal curve combined with a sag vertical curve and a horizontal curve combined with a crest vertical curve. Operating speed data were collected at five points on each site to establish the effect of the 3D alignment combination on the trend of operating speed of the traveling vehicles.

- Point 1 was set out at about 60-80 m on the approach tangent before the beginning of the spiral curve.
- Point 2 was the end of spiral curve and the beginning of horizontal curve in the direction of travel (SC).
- Point 3 was the midpoint of horizontal curve (MC).
- Point 4 was the end of horizontal curve and the beginning of spiral curve in the direction of travel (CS).
- Point 5 was set out at about 60–80 m on the departure tangent after the end of the spiral curve.

$$V_{85}^{1} = 91.81 + 0.010R + 0.468\sqrt{Lv} - 0.006G_1^3 - 0.878\ln(A) - 0.826\ln(L_0) \qquad R^2 = 0.98 \qquad (2\text{-}43)$$

$$V_{85}^{2} = 47.96 + 7.217\ln(R) + 1.534\ln(\sqrt{Lv}) - 0.258G_1 - 0.653A - 0.008L_0 + 0.020\exp(E) \qquad R^2 = 0.98 \qquad (2\text{-}44)$$

$$V_{85}^3 = 76.42 + 0.023R + 2.300 \times 10^{-4} K^2 - 0.008 \exp(A) - 1.230 \times 10^{-4} L_0^2 + 0.062 \exp(E)$$

$$R^2 = 0.94 \qquad (2\text{-}45)$$

$$V_{85}^4 = 82.78 + 0.011R + 2.067 \ln(K) - 0.361 G_2 - 1.091 \times 10^{-4} L_0^2 + 0.036 \exp(E)$$

$$R^2 = 0.95 \qquad (2\text{-}46)$$

$$V_{85}^5 = 109.45 - 1.257 G_2 - 1.586 \ln(L_0), \qquad R^2 = 0.79 \qquad (2\text{-}47)$$

Where,

V_{85}^1 to V_{85}^5 = predicted 85th percentile operating speed at point 1 to point 5 (km/h),

R = radius of horizontal curve (m),

E = superelevation rate (percent),

A = algebraic difference in grades (percent),

K = rate of vertical curvature (m),

G_1 and G_2 = first and second grades in the direction of travel in percent,

L_0 = horizontal distance between point of vertical intersection and point of horizontal intersection (m),

Most operating speed models have generally focused on passenger cars with little consideration for other vehicle such as trucks. However, it may be important to consider the truck operating speed in cases where trucks represent a large percentage of the traffic stream. Donnell et al. (2001) studied truck speeds in two-lane rural highways in Pennsylvania and developed rural heavy-duty vehicle curve speed models that included the length and grade of approaching and departing tangents, radius, and curve length.

V_{85}^{1} = 51.5 + 0.137R - 0.779 GAPT + 0.0127 L1 - 0.000119 (L1 * R) R^2 = 0.62 (2-48)

V_{85}^{2} = 54.9 + 0.123 R - 1.07 GAPT + 0.0078 L1 - 0.000103 (L1 * R) R^2 = 0.63 (2-49)

V_{85}^{3} = 56.1 + 0.117 R - 1.15 GAPT + 0.0060 L1 - 0.000097 (L1 * R) R^2 = 0.61 (2-50)

V_{85}^{4} = 78.7 + 0.0347 R - 1.30 GAPT + 0.0226 L1 R^2 = 0.55 (2-51)

V_{85}^{5} = 78.4 + 0.0140 R - 1.40 GDEP - 0.00724 L2 R^2 = 0.56 (2-52)

V_{85}^{6} = 75.8 + 0.0176 R - 1.41 GDEP - 0.0086 L2 R^2 = 0.60 (2-53)

V_{85}^{7} = 75.1 + 0.0176 R - 1.48 GDEP - 0.00836 L2 R^2 = 0.60 (2-54)

V_{85}^{8} = 74.7 + 0.0176 R - 1.59 GDEP - 0.00814 L2 R^2 = 0.61 (2-55)

V_{85}^{9} = 74.5 + 0.0176 R - 1.69 GDEP - 0.00810 L2 R^2 = 0.61 (2-56)

V_{85}^{10} = 82.8 - 2.00 GDEP - 0.00925 L2 R^2 = 0. 56 (2-57)

V_{85}^{11} = 83.1 - 2.08 G2 - 0.00934 L2 R^2 = 0. 57 (2-58)

V_{85}^{12} = 83.6 - 2.29 G2 - 0.00919 L2 R^2 = 0. 60 (2-59)

V_{85}^{13} = 84.1 - 2.34 G2 - 0.00944 L2 R^2 = 0. 60 (2-60)

Where,

V_{85}^{2} = 85th percentile speed (km/h) at 200 meters prior to horizontal curve,

V_{85}^{3} = 85th percentile speed (km/h) at 150 meters prior to horizontal curve,

V_{85}^{4} = 85th percentile speed (km/h) at 100 meters prior to horizontal curve,

V_{85}^{5} = 85th percentile speed (km/h) at 50 meters prior to horizontal curve,

V_{85}^{6} = 85th percentile speed (km/h) at beginning of horizontal curve (PC),

V_{85}^{7} = 85th percentile speed (km/h) at QP,

V_{85}^{8} = 85th percentile speed (km/h) at middle of horizontal curve (MC),

V_{85}^{9} = 85th percentile speed (km/h) at 3QP,

V_{85}^{10} = 85th percentile speed (km/h) at end of horizontal curve (PT),

V_{85}^{11} = 85th percentile speed (km/h) at 50 meter beyond horizontal curve (PT50),

V_{85}^{12} = 85th percentile speed (km/h) at 100 meter beyond horizontal curve (PT100),

V_{85}^{13} = 85th percentile speed (km/h) at 150 meter beyond horizontal curve (PT150),

V_{85}^{1} = 85th percentile speed (km/h) at 200 meter beyond horizontal curve (PT200),

L1 = length of approach tangent (m),

GAPT = grade of approach tangent,

R = curve radius (m),

LCRV = length of curvature (m),

L2 = length of departure tangent (m), and

GDEP = grade of departure tangent (m).

Many researchers determined that a vehicle's speed changed as it traversed a sharp horizontal curve and the vehicle did not maintain a constant speed. Similarly, the influence of boundary horizontal curves extends to short tangent sections between the curves. Liapis et al. (2001) analyzed the speed behavior of passenger cars at 20 on- and off-ramps in rural Greece, and concluded the 85th percentile speed was dependent on the superelevation rate (directly correlated with curve radius) and the curvature change rate. They identified this curvature rate of change by adding the angular change in the horizontal alignment and then dividing by the length of the highway section studied.

$$V_{85}^{1} = -0.360839DC - 3.683548E + 75.161 \qquad R^2 = 0.75 \qquad (2\text{-}61)$$

$$V_{85}^{2} = -0.472675DC - 3.795879E + 85.186 \qquad R^2 = 0.73 \qquad (2\text{-}62)$$

Where,

V_{85}^{1} = off-ramp 85th percentile speed (km/h),

V_{85}^{2} = off-ramp 85th percentile speed (km/h),

DC = degree of curvature (degrees per 30 m), and

E = superelevation rate.

Schurr et al. (2002) investigated operating speeds on horizontal curves on two-lane rural highways in Nebraska and developed prediction equations for mean, 85th, and 95th percentile speeds at curve midpoint locations and approach tangents location that was 183 m (600 ft) in advance of the PC of the curve. It was assumed that drivers' operating speeds on tangent would not be affected by the horizontal curves at this distance.

$$V_{mean}^{c} = 67.4 - 0.1126\Delta + 0.02243L + 0.276V_p \qquad R^2 = 0.55 \qquad (2\text{-}63)$$

$$V_{85}^{c} = 103.3 - 0.1253\Delta + 0.0238L - 1.039G_1 \qquad R^2 = 0.55 \qquad (2\text{-}64)$$

$$V_{95}^{c} = 113.9 - 0.122\Delta + 0.0178L - 0.00184T_{ADT} \qquad R^2 = 0.55 \qquad (2\text{-}65)$$

$$V_{mean}^{t} = 51.7 + 0.508V_p \qquad R^2 = 0.55 \qquad (2\text{-}66)$$

$$V_{85}^{t} = 70.2 + 0.434\,V_p - 0.001307T_{ADT} \qquad R^2 = 0.55 \qquad (2\text{-}67)$$

$$V_{95}^{t} = 84.4 + 0.508V_p - 0.001399T_{ADT} \qquad R^2 = 0.55 \qquad (2\text{-}68)$$

Where,

V_{mean}^{c} = average speed of free-flow passenger cars at curve midpoint (km/h),

V_{85}^{c} = 85th percentile speed of free-flow passenger cars at curve midpoint (km/h),

V_{95}^{c} = 95th percentile speed of free-flow passenger cars at curve midpoint (km/h),

V_{mean}^{t} = average speed of free-flow passenger cars at approach tangent (km/h),

V_{85}^{t} = 85th percentile speed of free-flow passenger cars at approach tangent (km/h),

V_{95}^{t} = 95th percentile speed of free-flow passenger cars at approach tangent (km/h),

Δ = deflection angle (decimal degrees),

L = arc length of curve (m),

V_p = posted speed limit (km/h),

G_1 = approach grade (percent), and

T_{ADT} = average daily traffic (vehicle per day)

Appendix A summarizes the representative rural operating speed models developed by previous studies.

2.2.1.2 Operating Speed Models for Rural Vertical Geometric Controls

Roadway parabolic vertical curves can be either crest curves or sag curves. Generally, sag curves do not physically constrict a driver's line of sight; whereas, an abrupt crest vertical curve may impede the driver's sight distance.

Fambro et al. (1999) investigated operating speeds on 42 vertical crest curves of two-lane rural roads in three states and developed the following model:

$$V85 = 72.74 + 0.47Vd \qquad (2\text{-}69)$$

Where,

V85 = 85th percentile speed (km/h), and

Vd = inferred design speed (km/h).

Jesson et al. (2001) studied the passenger vehicle speeds on 70 crest curves on horizontal tangent sections of two-lane rural highways in Nebraska and developed operating speed models for crest vertical curves and approach tangents.

$$V_{mean}^{c} = 67.6 + 0.39V_p - 0.714G_1 - 0.00171\,T_{ADT} \qquad R^2 = 0.57 \qquad (2\text{-}70)$$

$$V_{85}^{c} = 86.8 + 0.297\,V_p - 0.614G_1 - 0.00239\,T_{ADT} \qquad R^2 = 0.54 \qquad (2\text{-}71)$$

$$V_{95}^{c} = 99.4 + 0.225\,V_p - 0.639G_1 - 0.0024T_{ADT} \qquad R^2 = 0.57 \qquad (2\text{-}72)$$

$$V_{mean}^{t} = 55.0 + 0.5V_p - 0.00148\,T_{ADT} \qquad R^2 = 0.44 \qquad (2\text{-}73)$$

$$V_{85}^{t} = 72.1 + 0.432V_p - 0.00212T_{ADT} \qquad R^2 = 0.42 \qquad (2\text{-}74)$$

$$V_{95}^{t} = 82.7 + 0.379V_p - 0.002T_{ADT} \qquad R^2 = 0.40 \qquad (2\text{-}75)$$

Where,

V_{mean}^{c} = average speed at crest curve (km/h),

V_{85}^{c} = 85th percentile speed at crest curve (km/h),

V_{95}^{c} = 95th percentile speed at crest curve (km/h),

V_{mean}^{t} = average speed at approach tangent (km/h),

V_{85}^{t} = 85th percentile speed at approach tangent (km/h),

V_{95}^{t} = 95th percentile speed at approach tangent (km/h),

V_p = posted speed limit (km/h),

G_1 = approach grade (percent), and

T_{ADT} = average daily traffic (vehicle per day)

Similarly, Fitzpatrick et al. (2000) evaluated crest vertical curves at horizontal tangent locations. They determined operating speed was essentially drivers' assumed desired speed for unlimited sight distance locations, while the vertical curve rate of change proved to be the only significant variable for the 85th percentile speed at limited sight distance crest curve locations. This research team further evaluated the speed for sag vertical curves at horizontal tangent locations and again concluded the operating speed represented a driver's selected speed at these locations. The developed models are represented by equations 2-35, 2-36, and 2-37.

2.2.2 Operating Speed Models for Urban Roadways

Urban street environment is characterized by a variety of influences that may conceivably influence the operating speed of a facility. As a result, horizontal curvature alone is unlikely to define the anticipated speed for an urban street as it does for many of the speed models for the rural environment. Numerous roadside features and access points create a complex driving environment. Poe et al. (1996) determined that access and land use characteristics had a direct influence on operating speed. For example, higher access density contributes to lower operating speeds due to the increased interaction with vehicles from driveways, intersections, median areas, and parking.

Poe et al. (1996) studied operating speeds on 27 urban collect streets in central Pennsylvania, and found that the geometric roadway elements, access, land-use characteristics, and traffic engineering elements influenced vehicle speeds on low speed urban street. The researchers collected free flow speed data at designated locations along a corridor. In addition, they determined basic road geometry. Field observation teams, positioned next to the road, attempted to document information about each vehicle and driver. It is important to note, that this study is the only field study in the United States identified where researchers attempted to include driver and vehicle influences (other than presence of heavy trucks) into a speed model.

With the same dataset, Tarris et al. (1996) compared different statistical approaches to model the speed choices of drivers at midpoint of horizontal curves on low-speed urban streets, including, linear regression with aggregated speed data, linear regression with individual driver speed data, and panel analysis. The following models were developed.

With aggregated speed data (mean speed):

$$V = 53.5 - 0.265D \qquad R^2 = 0.82 \qquad (2\text{-}76)$$

With individual driver speed:

$$V = 53.8 - 0.272D \qquad R^2 = 0.63 \qquad (2\text{-}77)$$

Panel analysis

$$V = 52.18 - 0.231D \qquad R^2 = 0.80 \qquad (2\text{-}78)$$

Where

V = 85th percentile speed (km/h), and

D = degree of curve (degree).

They suggested that using aggregated speed reduces the total variability and created an apparent improvement in explaining variation in operating speeds. However, the resulting high coefficient of determination is really a result of the individual driver speeds being represented by one aggregated statistic on each site. The influence of the geometric elements may be overstated or understated.

With the same dataset, Poe and Mason (2000) used a mixed-model statistical approach to analyze the influence of geometric, roadside, driver, and traffic control features on drivers' operating speeds. They considered the following variables during model development:

- geometric measures (e.g., curve radius, grade, sight distance),
- cross-section (e.g., lane width, road configuration),

- roadside (e.g., access density, land use, roadside lateral obstructions),
- traffic control devices (e.g., speed limit, pavement marking), and
- driver / vehicle (e.g., gender, age, number of passengers, vehicle type).

The following model was developed:

$$V_{85}^{1} = 49.59 + 0.5*DC - 0.35*G + 0.74*W - 0.74*HR \qquad R^2 = 0.99 \qquad (2\text{-}79)$$

$$V_{85}^{2} = 51.13 - 0.1*DC - 0.24*G - 0.01*W - 0.57*HR \qquad R^2 = 0.98 \qquad (2\text{-}80)$$

$$V_{85}^{3} = 48.82 - 0.14*DC - 0.75*G - 0.12*W - 0.12*HR \qquad R^2 = 0.90 \qquad (2\text{-}81)$$

$$V_{85}^{3} = 43.41 - 0.11*DC - 0.12*G + 1.07*W + 0.3*HR \qquad R^2 = 0.90 \qquad (2\text{-}82)$$

Where:

V_{85}^{1} = 85th percentile speed (km/h) at 150 ft before the beginning of curve (PC160),

V_{85}^{2} = 85th percentile speed (km/h) at the beginning of curve (PC),

V_{85}^{3} = 85th percentile speed (km/h) at the middle of curve (MC),

V_{85}^{4} = 85th percentile speed (km/h) at the end of curve (PT),

DC = degree of curvature (degrees per 30m),

G = grade (%),

W = lane width (m), and

HR = hazard rating (0 to 4).

Fitzpatrick et al. (1997) evaluated operating speeds for curve sections on suburban roadways. The roads in this study were four-lane divided sections with moderate approach density and signal spacing. The research team used approach density as a surrogate for roadside development. Only data from free-flow passenger cars, pickup trucks, and vans were included in this study. One variable used in the evaluation was an inferred design speed that generally represents road design constraints (e.g. available sight distance for crest vertical curvature conditions). For horizontal curve locations, the speed models resulted in a curvilinear regression equation with two significant independent variables -- horizontal curve radius and acess density. For crest vertical curve locations, the inferred design speed proved to be the only significant variable for predicting operating speed. It is important to note, all crest curve locations included in the study were characterized by limited sight distance, so the resulting speed model may not be applicable to unrestricted sight distance vertical conditions.

$$V_{85}^{1} = 56.34 + 0.808R^{0.5} + 9.34/AD \qquad R^2 = 0.72 \qquad (2\text{-}83)$$

$$V_{85}^{2} = 39.51 + 0.556\,(IDS) \qquad R^2 = 0.56 \qquad (2\text{-}84)$$

Where

V_{85}^{1} = 85th percentile speed on horizontal curves (km/h),

V_{85}^{2} = 85th percentile speed on vertical curves (km/h),

R = curve radius (m), and

AD = approach density (approaches per km).

Another study conducted by Fitzpatrick et al. (2001) evaluated the influence of geometric, roadside, and traffic control device on drivers' speed on four-lane suburban arterials. The authors found that posted speed limits were the most significant variable for both curve and straight sections, and deflection angle and access density class were significant variables for curve sections. They also performed similar analyses without including the speed limits, and found that median presence and roadside development were significant for curve sections while only lane width was significant for straight sections.

With posted speed limits:

$$V_{85}^{c} = 42.916 + 0.523PSL - 0.15DA + 4.402AD \qquad R^2 = 0.71 \qquad (2\text{-}85)$$

$$V_{85}^{t} = 29.180 + 0.701PSL \qquad R^2 = 0.53 \qquad (2\text{-}86)$$

Without posted speed limits:

$$V_{85}^{c} = 44.538 + 9.238MED + 13.029L1 + 17.813L2 + 19.439L3 \qquad R^2 = 0.52 \qquad (2\text{-}87)$$

$$V_{85}^{t} = 18.688 + 15.050WD \qquad R^2 = 0.25 \qquad (2\text{-}88)$$

Where

V_{85}^{c} = 85th percentile speed on horizontal curves (km/h),

V_{85}^{t} = 85th percentile speed on tangents (km/h),

PSL = posted speed limit (km/h),

AD = access density, if below 12 pts/km, then 1. Otherwise 0,

MED = if raised or TWLTL then 1, otherwise 0,

L1 = if school then 1, otherwise 0,

L2 = if residential then 1, otherwise 0,

$L3$ = if commercial then 1, otherwise 0, and

WD = lane width (m)

Bonneson (1999) studied vehicle speeds on horizontal curves at 55 sites in eight states. These sites included urban roadways, rural roadways, and turning roadways. He developed a curve speed model to identify the relationship between curve speed, approach speed, radius, and superelevation. He also developed a side friction model to explain the relationship between approach speed, speed reduction, and side friction demand at horizontal curves. Minimum radii and design superelevation rates were key variables in the development of the side friction model. The curve speed model included curve speed, approach speed, radius, and superelevation rate. It is important to note collinearity exists between the radius and the superelevation rate, so application of model using both variables may lead a bias toward the curve geometry.

$$V85 = 63.5R\left(-B + \sqrt{B^2 + \frac{4c}{127R}}\right) \leq Va \qquad R^2 = 0.96 \qquad (2\text{-}89)$$

with

$C = E/100 + 0.256 + (B – 0.0022)Va$

$B = 0.0133 – 0.0074I_{TR}$

Where,

V85 = 85th percentile curve speed (km/h),

Va = 85th percentile approach peed (km/h),

R = curve radius (m),

E = superelevation, and

I_{TR} = indicator variable (= 1.0 if Va > Vc; 0.0 otherwise).

The inequality in Equation 2-89 serves to ensure that the curve speed predicted by the equation does not exceed the approach speed. If the predicted speed is larger or equal to the approach speed, curve geometry is not likely to affect drivers' speed choice.

Appendix B summarizes the existing operating speed models on urban roadways.

2.2.3 Summary of Existing Operating Speed Models

Existing speed models range from simple linear regression models with speed as the dependent variable and horizontal curve radius as independent variable, up to complex curvilinear regression equations. The majority of the existing speed models attempt to quantify operating speed based primarily on physical conditions such as road geometric design and, in the urban environment, roadside development and traffic control devices. By using the 85^{th} percentile speed as a representative measure for operating speed, researchers are attempting to identify the operating speed threshold under which 85 percent of the drivers in the traffic stream select to travel at. Generally, these models represent roads under dry pavement and daylight conditions.

The following are the summary of the research results of previous related studies:

- The 85th percentile speed is the general statistic used to describe operating speeds in the existing models.

- Existing operating speed models primarily focus on two-lane rural highways. Radius of horizontal curve is the most significant variable. Other significant variables identified in previous studies include length of curve, grade, lane width, shoulder width, traffic volume, superelevation, approach tangent speed, and posted speed limit.

- Limited research to date exists for urban environment speed estimation. The reason is, for urban environment, numerous roadside features and access points create a complex driving environment. There are much more factors influencing driver's speed choices. As a result, horizontal curve radius is not significant in the urban environment as in the rural environment. Other identified features that affect speed on suburban/urban streets include lane width, roadside objects, access density, roadside development, median presence, stopping sight distance, grade, pedestrian/bicyclist activity, on-street parking, type of curb, and posted speed limit.

- Existing operating speed models focus on horizontal and vertical curves rather on tangents. Very few operating speed model exits for tangents. The possible reason is that the estimation of speeds at horizontal curves may be easier than the prediction of speeds at tangent sections because of the correlation of speeds to a few defined and limited variables, such as curve radius and superelevation rate. On tangent sections, there are no geometric constraints on drivers' speeds as the horizontal curves. Driver selected speeds are dependent on a wide variety of roadway characteristics including the tangent length, cross-section elements, vertical alignment, general terrain, sight distance, and driver's attitude.

- Previous studies assume that deceleration and acceleration rates are constant and all acceleration and deceleration take place prior to, or after the horizontal curves.

- Most existing operating speed models are point speed models, which refer to the estimation of the operating speeds at a point using the local characteristics, geometric alignment, roadside variables, land use characteristics, and traffic control variables within a specific distance (such as 30 meters) of that point. Only point speed models are available for urban environment. Those point speed models are based on assumptions, such as:
    - Drivers reach their lowest speed at the midpoint of curves,
    - Drivers reach their highest speed at the midpoint of a tangent section.

- Drivers select vehicle speeds based on the road environment through which they have just passed, and the road environment that they can see ahead of them. A point speed study may not adequately represent driver behavior upstream or downstream of the study location and capture the overall influence of road environment on drivers' speed choice.

- Previous studies did not consider the effects of driver and vehicle characteristics. Most previous studies could not collect drivers' information because of the limitation of the data collection methods since it is difficult to obtain drivers information by field observations and the traditional data collection method is not able to capture multiple trips from the same drivers.

CHAPTER 3

DATA COLLECITON

The data collection method in this dissertation is different from most previous studies. Most of previous studies selected the study sites first, and then measured the speed data on the selected sites. This study monitors the selected drivers' vehicle activity 24 hours a day for up to one year using in-vehicle GPS. Researcher then chose the study sites based on the collected speed data.

Three types of data are collected in this study, vehicle speeds, road environment features, and driver/vehicle characteristics. The speed data contain vehicle location and speed at one second interval. The driver and vehicle information includes driver's age, gender, and vehicle types. The road environment data include roadway characteristics, cross section features, roadside objects, and adjacent land uses.

3.1 In-Vehicle Global Positioning System

3.1.1 Introduction to Global Positioning System

GPS is a satellite-based navigation system consisting of 24 satellites orbiting the earth at an altitude of approximately 11,000 miles. GPS was initially developed for military services by the Unite States Department of Defense (DOD). However, over the past several years, GPS has been widely used in non-military areas. In transportation engineering, GPS has been widely used in studies of travel time, route choice, car following, and drivers' speed behaviors.

GPS consists of three components: the space segment, the control segment, and the user segment. The space segment consists of 24 satellites that emit high-frequency radio waves. The control segment consists of five ground stations located around the world, which monitor the GPS satellites and upload information from the ground. The user segment is the GPS receivers, which detect, decode, and processe GPS satellite signals.

GPS determines a location by calculating the distances between the receiver and 3 or more satellites. GPS measures distance by measuring the travel time of radio waves travel from the satellite to the receiver. Assuming the positions of the satellites are known, the location of the receiver can be calculated by determining the distance from each of the satellites to the receiver.

3.1.2 Data Collection Equipment

The in-vehicle data collection equipment used in this study consists of CPU, power system, cellular transceiver, GPS, and other sensors. The data collection equipment turns on and off automatically with the vehicle ignition. Recorded data are automatically transferred to a data server at Georgia Tech over wireless connection every week. Figure 3-1 shows the GPS data collection system.

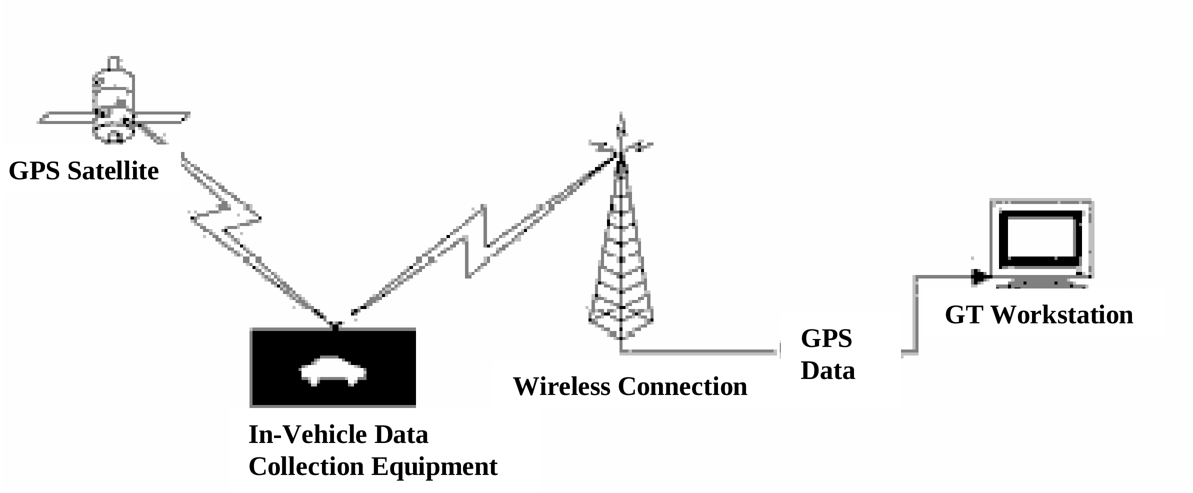

Figure 3-1 GPS Data Collection System Map

3.2 Speed Data from In-Vehicle Data Collection Equipment

This dissertation used two GPS datasets generated by vehicles equipped with in-vehicle data collection equipments. In the first dataset, 145 vehicles were randomly selected in the Atlanta urban area. The GPS data were collected between January, 2002 and May, 2004. About 25 millions one-second interval GPS data records were collected.

Another dataset is from an on-going vehicle instrument project. Since most of the received GPS data is still under processing, only two weeks of data from 455 randomly selected drivers are available for this dissertation, which includes about 15 millions one-second interval GPS data records. Table 3-1 presents the driver and vehicle profile of the two datasets. The GPS receivers in both projects provide speed accuracy less than 1.6 km (1 mph) for 95 percent of the time.

Table 3-1 Study Driver and Vehicle Profile

	Dataset 1	**Dataset 2**
Number of drivers	145	455
Received GPS data records	25,096,786	15,974,520
Female	61%	55%
Male	39%	45%
Age less than 18	5%	4%
Age between 18 and 45	44%	43%
Age between 45 and 60	35%	35%
Age larger than 60	17%	18%
Passenger car	58%	62%
Minivan	17%	20%
SUV	17%	7%
Pickup	8%	11%

Table 3-2 presents a sample of the speed data. The location and speed data were recorded at one-second interval. For example, the last record in Table 3-2 indicates that this vehicle was traveling at 34.64 km/h (21.5 mph) at latitude of 33. 778 and longitude of -84.400 at 16:32:00 on Aug. 6th in 2001.

Table 3-2 Example Speed Data from the In-Vehicle GPS Data Collection Equipment

Date	Time	Latitude	Longitude	Speed (km/h)
20020806	163148	33.778550	-84.401427	0.00
20020806	163149	33.778549	-84.401425	3.04
20020806	163150	33.778537	-84.401414	7.55
20020806	163151	33.778518	-84.401397	10.34
20020806	163152	33.778499	-84.401380	8.98
20020806	163153	33.778483	-84.401362	8.91
20020806	163154	33.778462	-84.401325	15.82
20020806	163155	33.778444	-84.401278	18.22
20020806	163156	33.778440	-84.401217	21.44
20020806	163157	33.778451	-84.401133	27.60
20020806	163158	33.778466	-84.401046	27.84
20020806	163159	33.778475	-84.400959	31.71
20020806	163200	33.778487	-84.400859	34.64

The collected GPS data records were overlaid with a GIS digital road network map based on the latitude and longitude information so that the researchers know where, when, and how fast the drivers were driving. Figure 3-2 shows a trip example overlaid onto a GIS road network.

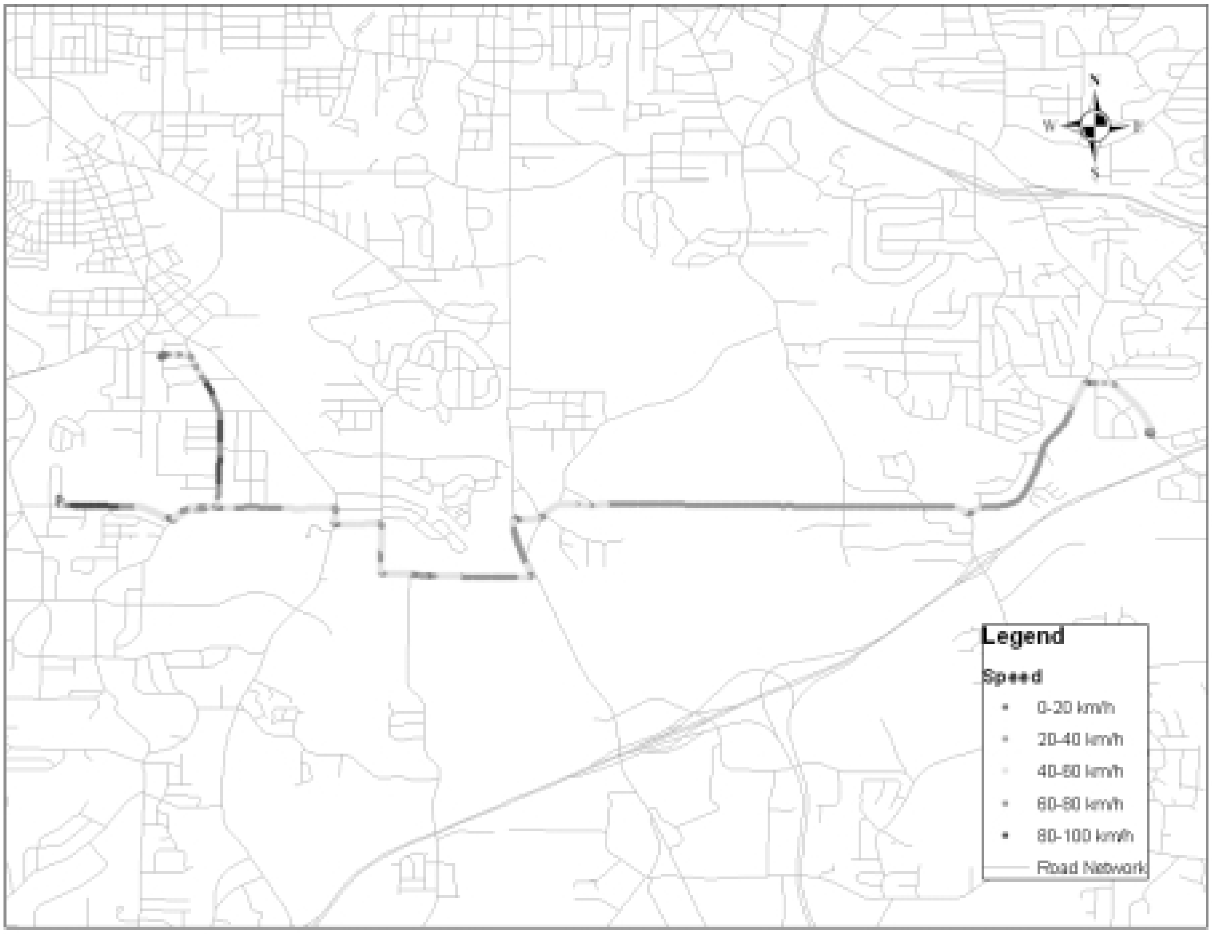

Figure 3-2 Sample Trip Overlaid with GIS Road Network

The overlaid GPS data points have associated road segment identification number (link ID in Figure 3-3), which are corresponding to the route identification number in the Georgia Department of Transportation (GDOT) Road Characteristics file (RC file), so that the researchers can identify the related road characteristics for each GPS data point.

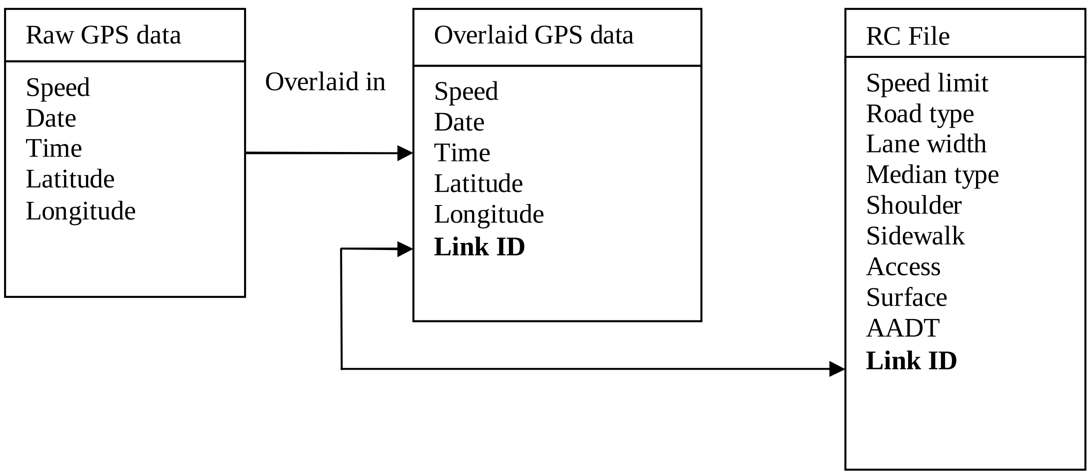

Figure 3-3 Relationship between GPS Data and Road Characteristics

3.3 GIS Road Network Database

The dissertation uses the base map provided by Georgia Department of Transportation (GDOT). The road network consists of routes identified by a RCLINK number. The RCLINK is a 10-digit GDOT route identification number that provides relational link between route features and the road characteristics database (RC File). Each route consists of several road segments identified by a MILEPOINT number, which is the mile measurement along a route and recorded to 1/100th

of a mile. The road segments are delimited by intersections, ramps and other physical discontinuities. An example road network is shown in Figure 3-4.

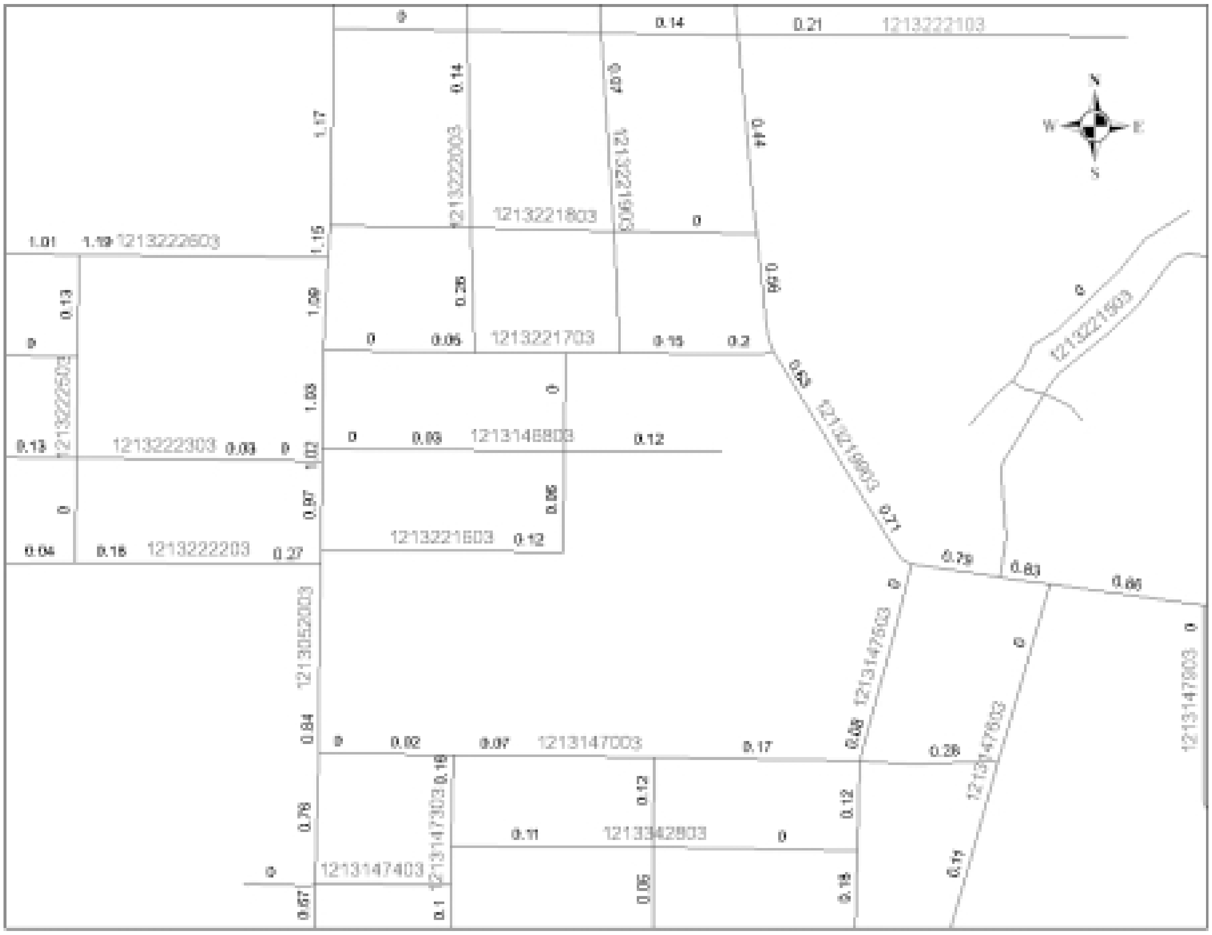

Figure 3-4 Example Digital Road Network

This dissertation uses a link to represent a road segment. Each link is identified by a unique link ID composed of RCLINK and MILEPOINT numbers.

The road network database was extracted from the Georgia Department of Transportation (GDOT) RC File. This 13 county database has a total of 220,634 records, which cover all road networks of the 13 counties in the metro Atlanta area counties. Each record has 61 attributes that describe the road characteristics such as road type, number of lanes, lane width, median type, and speed limit. Each record is identified by a unique combination of RCLINK and MILEPONT number and responds to one unique link in the road base map.

3.4 Study Drivers' Characteristics

The author compared the study drivers' age and gender distribution with the U.S. census data of licensed drivers in 2003. The characteristics of selected drivers are reasonably representative of the general population in the United States. The authors also compared the vehicle type distributions. The sample set has a larger percentage of minivans and smaller percentage of pickups than the general population, as shown in Table 3-3.

TABLE 3-3 Study Driver and Vehicle Characteristics

	Sample Population	U.S. Census Data
Gender		
Female	56.5%	50.1% (1)
Male	43.5%	49.9% (1)
Age Distribution		
Age less than 18	4%	4.7% (1)
Age between 18 and 45	43.9%	47.6% (1)
Age between 45 and 60	33.6%	27.1% (1)
Age larger than 60	18.6%	20.6% (1)
Vehicle Type		
Passenger Car	59.6%	56.8% (2)
Minivan	17.7%	9.1% (2)
SUV	13.2%	11.9% (2)
Pickup	9.5%	18.3% (2)

(1) Source: Age and Gender Distribution of U.S. Licensed Drivers, 2003, U.S. Department of Transportation, Federal Highway Administration, *Highway Statistics 2003.*
(2) Source: The 2001 National Household Travel Survey, vehicle file, U.S. Department of Transportation

CHAPTER 4

VEHICLE ACCELERATION AND DECELERATION CHARACTERISTICS

The most significant characteristic of urban streets is the presence of closely spaced intersections with traffic control devices. Drivers have to make frequent stops. Understanding vehicle acceleration and deceleration characteristics is a very important part of analyzing speed profiles on urban streets. Drivers' acceleration and deceleration distance also provides guidance in the determination of the minimum distance between two intersections with traffic signals or stop signs to ensure that the selected streets are long enough so that drivers are able to reach their desired speeds.

Most previous acceleration/deceleration studies were based on outdated data rather than recent observations. The research conclusions from previous studies may therefore not be reflective of current drivers. Furthermore, due to the limitations of data collection methods, most previous studies could not provide accurate estimations of drivers' acceleration and deceleration behaviors, such as acceleration or deceleration time and distance, since different drivers may start to accelerate or decelerate at different time and location. In this study, the second-by-second speed profile data from in-vehicle GPS equipment can provide more accurate acceleration time and distance, deceleration time and distance, and average acceleration and deceleration rates.

4.1 Vehicle Acceleration Characteristics

This dissertation investigates the acceleration behaviors of current passenger vehicles starting from rest at all-way stop-controlled intersections. This study defines the part of the trip when vehicles accelerate from rest to the point where the speed stops increasing as an acceleration profile. The following are several cases of accelerating from rest.

- Stopping at an intersection with all-way stop sign
- Stopping at an intersection with one-way stop sign
- Stopping at an intersection with a traffic control device such as a traffic signal

A typical normal acceleration behavior for a single vehicle is more likely to occur at a all-way stop location than at the other two stop conditions. At one-way stop and alternative traffic controlled intersections, vehicle acceleration behaviors may be influenced by other vehicles in the traffic stream. In fact, at an intersection with a one-way stop sign, drivers may accelerate more aggressively than normal since they want to clear the intersection as quickly as possible to avoid any conflicts from traffic on the major road. In contrast, at an intersection with a traffic control device such as a traffic signal, the driving behavior of non-leading vehicles would be affected by the leading vehicle. Since the data is from GPS equipment in an individually equipped vehicle, the author does not know if the study vehicle is the leading vehicle stopped at an intersection or a non-leading vehicle influenced by other vehicle in traffic stream. Therefore, this study only includes acceleration observations at all-way stop locations with the underlying assumption that each participating driver stopped at each stop sign.

In order to increase the likelihood of observing the typical acceleration behavior, the collected trips are filtered to remove trips with final speed less than corresponding speed limit since some drivers may have to stop accelerating prematurely due to the influence of leading vehicles in the traffic stream. If the final speed is larger than this threshold, it will be more likely that drivers were accelerating without the influence of other vehicles.

The following are the acceleration profile selection criteria:

- The stop position is at an all-way stop-controlled intersection.
- The initial speed is zero.
- Acceleration is assumed to end when the speed increase between two successive one-second speed data points is less than 0.16 km/h (0.1 mph).
- Final speed is larger than speed limits.

Based on the selection criteria, this dissertation identified a total of 415 acceleration trips on urban streets in the Atlanta metro area, including urban local streets, collectors, and arterials.

Since the speed data were collected at one-second interval, an acceleration rate for one second can be calculated from the speed difference. The acceleration rate at the n^{th} second is estimated by the average acceleration rates of previous and successive seconds. Assuming an acceleration profile with speeds of v_0, v_1, …, v_n, this study uses the following methods to calculate the acceleration rates at a specific second.

$$a_0 = (v_1 - v_0)/3.6 \qquad (4\text{-}1)$$

$$a_i = (v_{i+1} - v_{i-1})/(2\times3.6) \qquad 0 < i < n \qquad (4\text{-}2)$$

$$a_n = (v_n - v_{n-1})/3.6 \qquad (4\text{-}3)$$

where

a_i = estimated acceleration rate at the i^{th} second (m/s^2), and

v_i = speed at the i^{th} second (km/hr)

4.1.1 Acceleration Statistics with Different Final Speeds

This dissertation investigated the relationship between acceleration behaviors and final speeds. The author divided the collected acceleration trips into five groups based on final speeds in 10 km/h increments. Figure 4-1 and 4-2 show the distribution of acceleration time and distance in each group. In this study, the acceleration distance is defined as the distance travels from the start point to the point where speeds stop increasing. The corresponding time is acceleration time. Figure 4-1 and 4-2 indicate that drivers' acceleration time and distance are related with their desired final speeds. As expected, drivers normally decelerate over longer time and distance with higher final speeds.

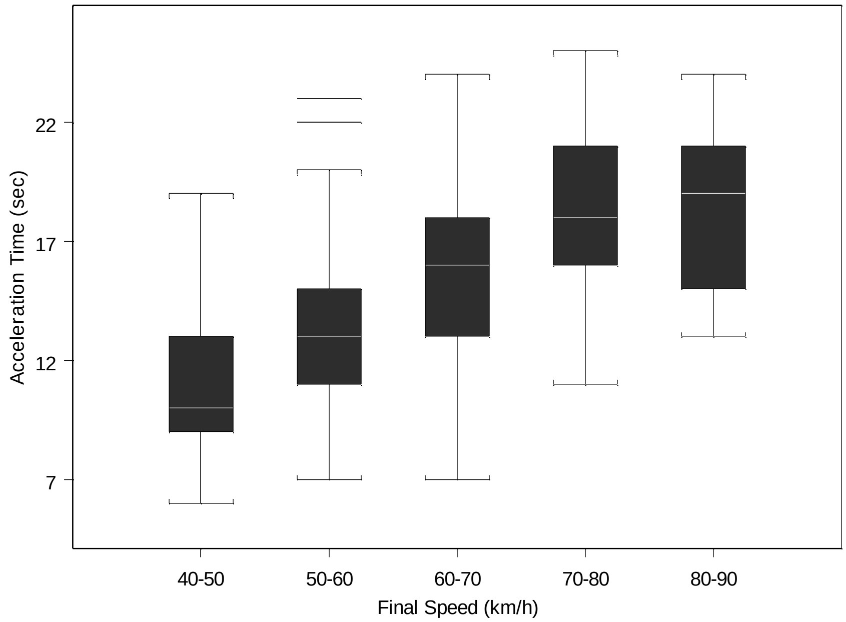

Figure 4-1 Average Acceleration Time with Different Final Speeds

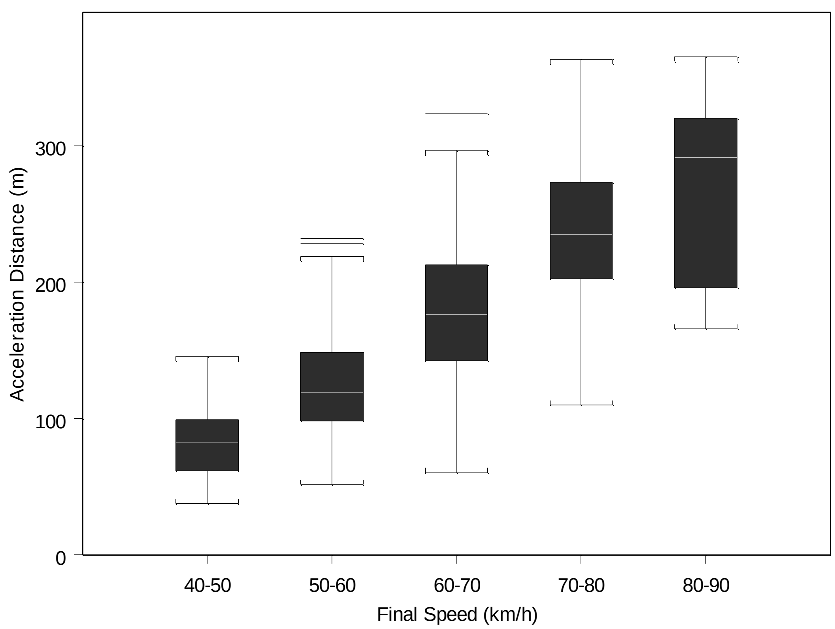

Figure 4-2 Average Acceleration Distance with Different Final Speeds

For each group, the author first calculated the average acceleration rate for each trip, which is defined as the mean of second-by-second acceleration rates during an acceleration trip. Then, the author calculated the group-level means for average acceleration rates, acceleration time and distance for each final speed group, which is shown in Table 4-1. One thing worth to note is that the average acceleration time with final speeds between 80 and 90 km/h (50 and 56 mph) is less than that with final speeds between 70 and 80 km/h (44 and 50 mph). The primary reason is that the sample size for the 80 to 90 km/h (50 to 56 mph) group is much smaller compared with other groups. Therefore, the results for this group may not be realistic reflection of drivers' acceleration behaviors. These results provide a good estimate of drivers' acceleration behaviors when their final speeds are known.

Table 4-1 Average Accelerate Rate, Time and Distance by Final Speeds

Final Speed (km/h)	**40-50**	**50-60**	**60-70**	**70-80**	**80-90**
Number of trips	104	166	103	31	11
Average accelerate rates (m/s^2)	1.24	1.21	1.22	1.17	1.32
Average acceleration time (sec)	10.8	13.2	15.6	18.5	18.2
Average acceleration distance (m)	82.8	124.4	175.0	238.9	267.1
85% acceleration distance (m)	111.9	166.1	229.4	308.5	355.6

4.1.2 Acceleration Statistics with Different Speed Limits

Since drivers normally would accelerate to higher final speeds on roads with higher speed limits, this dissertation also investigated drivers' acceleration behaviors on roads with different speed limits. The author divided the acceleration trips into different groups by the associated speed limits. The results are very close to those observed for the varying final speeds. Higher speed limits are normally associated with longer acceleration distance and time. This observation is intuitive since drivers normally drive at higher final speeds on roads with higher speed limits. Table 4-2 presents the average acceleration rates, time, and distance on roads with different speed limits.

Table 4-2 Average Accelerate Rate, Time and Distance by Speed Limits

Speed Limits (km/h)	**40**	**48**	**56**	**64**	**72**
Number of trips	132	77	153	37	16
Average accelerate rates (m/s^2)	1.25	1.19	1.22	1.11	1.29
Average acceleration time (sec)	11.5	12.5	14.3	17.0	16.1
Average acceleration distance (m)	101.1	110.3	148.6	211.7	201.5
85% acceleration distance (m)	149.1	158.6	216.5	291.1	243.3

These results provide a good estimation of drivers' acceleration behaviors when final speeds are unknown. One thing worth noting is that the average acceleration time and distance with speed limit of 72 km/h (45 mph) is less than that with speed limit of 64 km/h (40 mph). Again, the primary reason is that the sample size for the group with 72 km/h (45 mph) speed limit is much smaller compared with the other groups. Therefore, the results for this group may not be a representative reflection of drivers' acceleration behaviors.

4.1.3 Acceleration Speed Profile

With second-by-second acceleration profile, this dissertation evaluated average acceleration rates at one-second interval for all trips in each final speed group during the first 15 seconds prior to stop since the speed profiles indicated that most drivers accelerated in less than 15 seconds.

These rates were weighted by sample size at each second. Figure 4-3 shows the accelerating speed profiles with different final speeds. Figure 4-4 shows the average acceleration rates at each second. These figures indicate that drivers normally apply higher acceleration rates at the beginning and decrease acceleration rates with the increase of speeds.

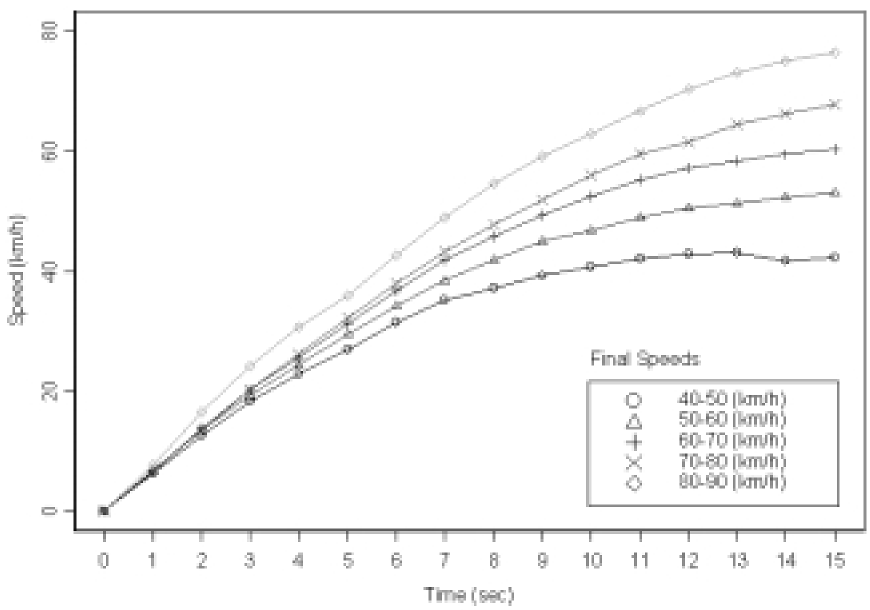

Figure 4-3 Acceleration Speed Profile with Different Final Speeds

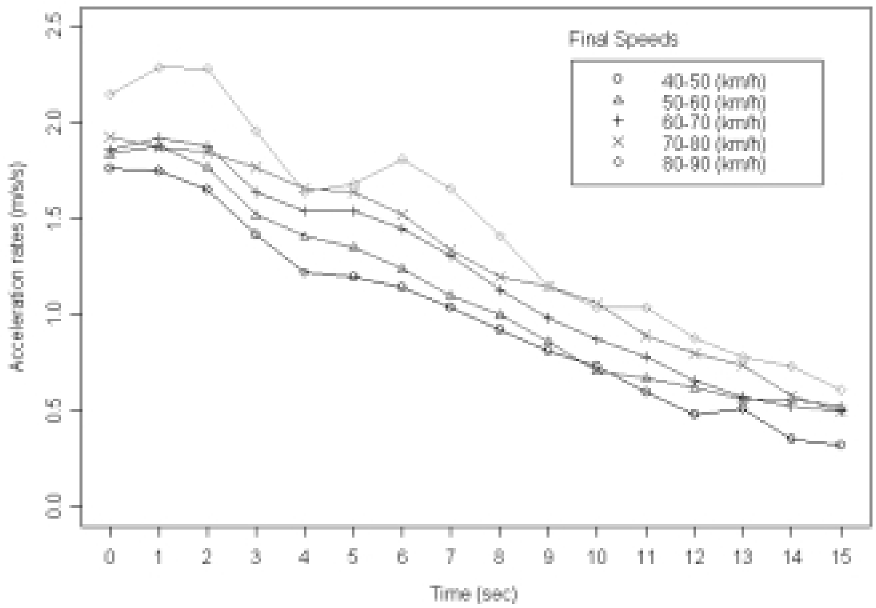

Figure 4-4 Average Acceleration Rates Profile with Different Final Speeds

4.1.4 Distribution of Acceleration Distance and Time

This dissertation also investigated the distribution of acceleration distance and time for all the collected trips. Figure 4-5 shows the distribution of acceleration distance. 85 percent of the trips have an acceleration distance less than 227 m (745 ft).

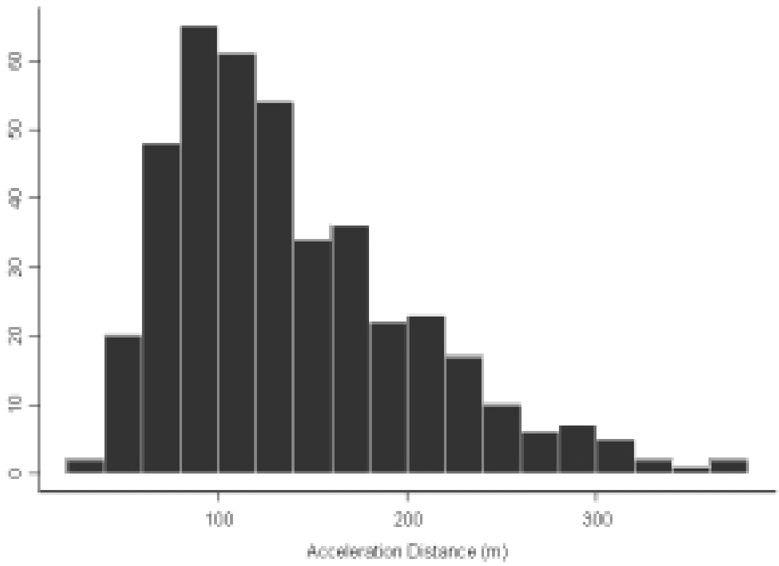

Figure 4-5 Distribution of Acceleration Distance

Figure 4-6 presents the distribution of acceleration time. 85 percent of the trips have an acceleration time less than 20 seconds.

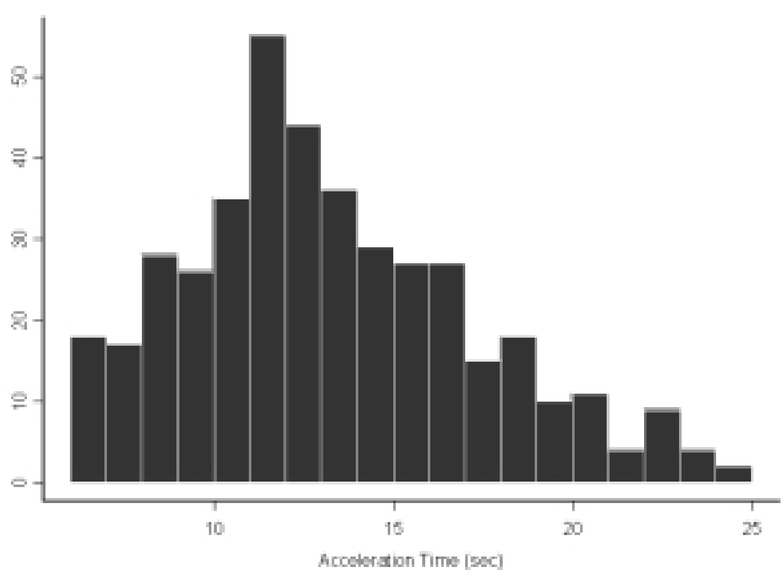

Figure 4-6 Distribution of Acceleration Time

4.2 Vehicle Deceleration Characteristics

This dissertation defines the deceleration profile as the part of a trip when drivers decelerate from an initial speed to a complete stop. A typical deceleration behavior for a single vehicle is more likely to occur regularly at stop sign controlled locations than at alternative traffic controlled intersections. At an intersection with a traffic control device such as a traffic signal, the driving behavior of non-free flowing vehicles could be affected by a slower downstream vehicle. Since the speed data are from individually equipped vehicles, the author does not know if a certain vehicle is the first vehicle stopped at the intersection or a trailing vehicle influenced by other vehicle's deceleration in the traffic stream. Therefore, this study only includes deceleration observations at all-way stop-sign-controlled locations under light traffic volume conditions where the driver executed a single stop at the intersection (indicating the vehicle was not in a queue).

The following are the deceleration profile selection criteria:

- The stop position is at an all-way or one-way stop-controlled intersection.
- The initial speed is higher than the speed limit.
- The final speed is zero.

Based on these criteria, the author analyzed a total of 428 deceleration trips on urban streets in the Atlanta metro area, including urban local streets, collectors, and arterials.

Since the speed data are collected at one-second interval, the deceleration rate for each one second interval can be calculated. The deceleration rate at the n^{th} second is estimated by the average deceleration rates of previous and successive seconds. Assuming a deceleration record with speeds of v_0, v_1, …, v_n, the authors use the following methods to calculate the deceleration rates at a specific second.

$$d_0 = |v_1 - v_0|/3.6 \tag{4-4}$$

$$d_i = |v_{i+1} - v_{i-1}|/(2*3.6) \qquad 0 < i < n \tag{4-5}$$

$$d_n = |v_n - v_{n-1}|/3.6 \tag{4-6}$$

where

d_i = estimated absolute deceleration rate at the i^{th} second (m/s^2), and

v_i = speed at the i^{th} second (km/hr)

4.2.1 Deceleration Statistics with Different Approach Speeds

This dissertation investigated the relationship between deceleration behaviors and approach speeds. The author divided the deceleration trips into five groups based on approach speeds in 10 km/h (6.3 mph) increments. For each group, the author first calculated the average deceleration rate for each trip, which is defined as the mean of second-by-second deceleration rates during a deceleration trip. Then, the author calculated the group-level means for average deceleration rates for each approach speed group, the deceleration time, and distance. The deceleration distance is defined as the distance traveled when drivers begin to decelerate to a stop position. Only the deceleration records with initial speeds larger than posted speed limits were included to increase the likelihood of observing unconstrained deceleration behavior.

Figure 4-7 and 4-8 indicate that approach speed have a significant influence on drivers' deceleration behavior. Drivers with higher approach speed normally decelerate over longer time and distance. Table 4-3 presents the average deceleration rate, time, and distance on roads with different approach speeds.

Table 4-3 Average Decelerate Rate, Time and Distance by Approach Speeds

Approach speed (km/h)	**40-50**	**50-60**	**60-70**	**70-80**	**80-90**
Number of trips	117	152	91	50	18
Average decelerate rates (m/s^2)	1.24	1.21	1.38	1.23	1.35
Average deceleration time (sec)	10.1	12.6	13.4	16.8	17.2
Average deceleration distance (m)	71.5	108.9	133.9	190.8	216.6
85% deceleration distance (m)	97.8	138.9	187.4	252.9	255.8

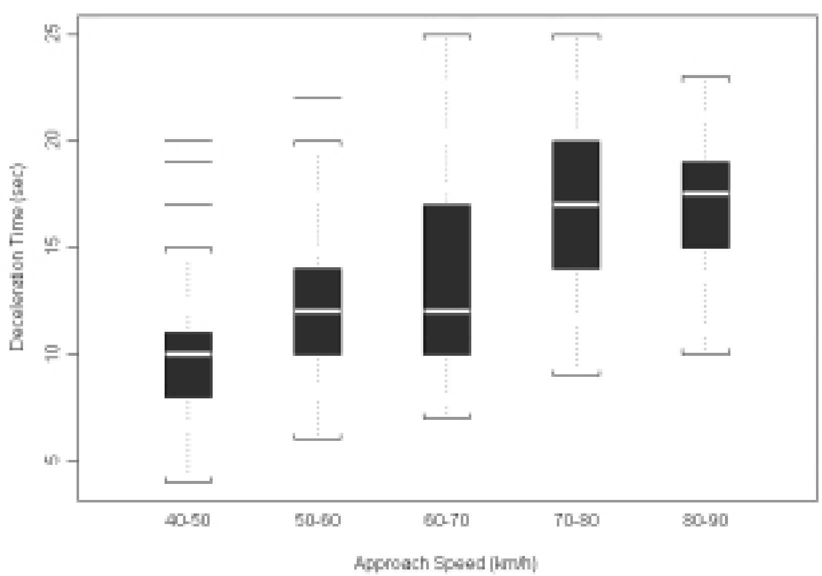

Figure 4-7 Average Deceleration Time with Different Approach Speeds

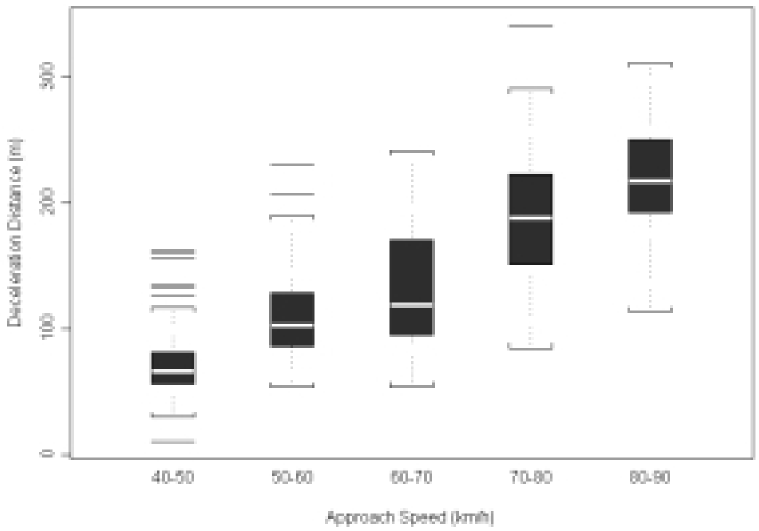

Figure 4-8 Average Deceleration Distance with Different Approach Speeds

4.2.2 Deceleration Statistics with Different Speed Limits

The author also divided the deceleration trips into different groups by associated speed limits. The results are similar as those observed for varying approach speeds. Higher speed limits are normally associated with longer deceleration distance and time. This observation is intuitive since drivers normally drive at higher speeds on roads with higher speed limits. Table 4-4 presents the average deceleration rate, time, and distance on roads with different speed limits.

Table 4-4 Average Decelerate Rate, Time and Distance by Speed Limits

Speed Limits (km/h)	**40**	**48**	**56**	**64**	**72**
Number of trips	234	68	104	11	11
Average decelerate rates (m/s^2)	1.28	1.20	1.26	1.30	1.22
Average deceleration time (sec)	11.2	12.9	15.1	16.1	18.2
Average deceleration distance (m)	92.6	115.9	156.7	198.0	228.8
85% deceleration distance (m)	131.9	150.6	221.8	255.3	273.8

These results provide a good estimation of drivers' deceleration behaviors when the approach speeds are unknown. One thing worth to note is that the sample size for the group with 64 and 72 km/h (40 and 45 mph) speed limit is much smaller compared with other groups. The results for these groups may not be representative reflection of drivers' deceleration behaviors.

4.2.3 Deceleration Speed Profile

With the second-by-second deceleration profile, this dissertation evaluated the average deceleration rates for each one-second interval for all trips in each approach speed group during the final 15 seconds prior to stop since the speed profiles indicated that most drivers decelerate in less than 15 seconds. These rates were weighted by the sample size at each second. Figure 4-9 shows the average deceleration rates at each second for different approach speeds. The figure shows that higher initial deceleration rates are associated with higher approach speeds. However, this relationship does not apply to the final three seconds prior to stopping. During the final three seconds, all drivers decelerate at similar rates regardless of the approach speeds. Figure 4-10 demonstrates the last 15 seconds observation in a plot of speed versus time.

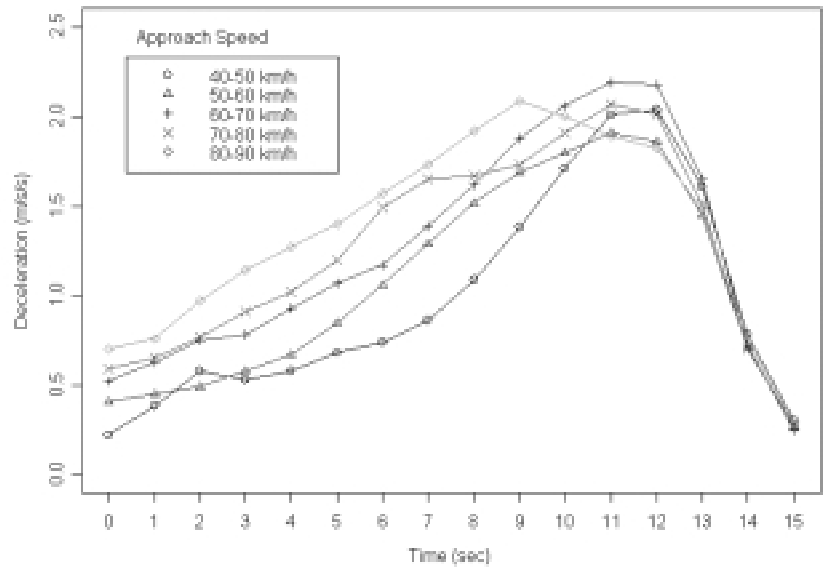

Figure 4-9 Average Deceleration Rate Profile with Different Approach Speeds

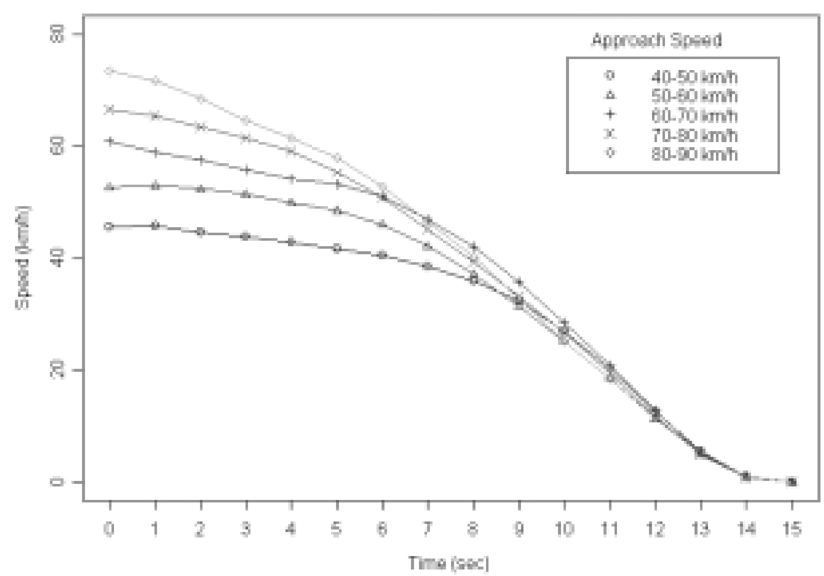

Figure 4-10 Deceleration Speeds Profiles with Different Approach Speeds

4.2.4 Distribution of Deceleration Distance and Time

This dissertation also investigated the distribution of deceleration distance and time from all collected trips. Figure 4-11 shows the distribution of deceleration distance on roads with different speed limits. 85 percent of the trips have a deceleration distance less than 181 m (594 ft).

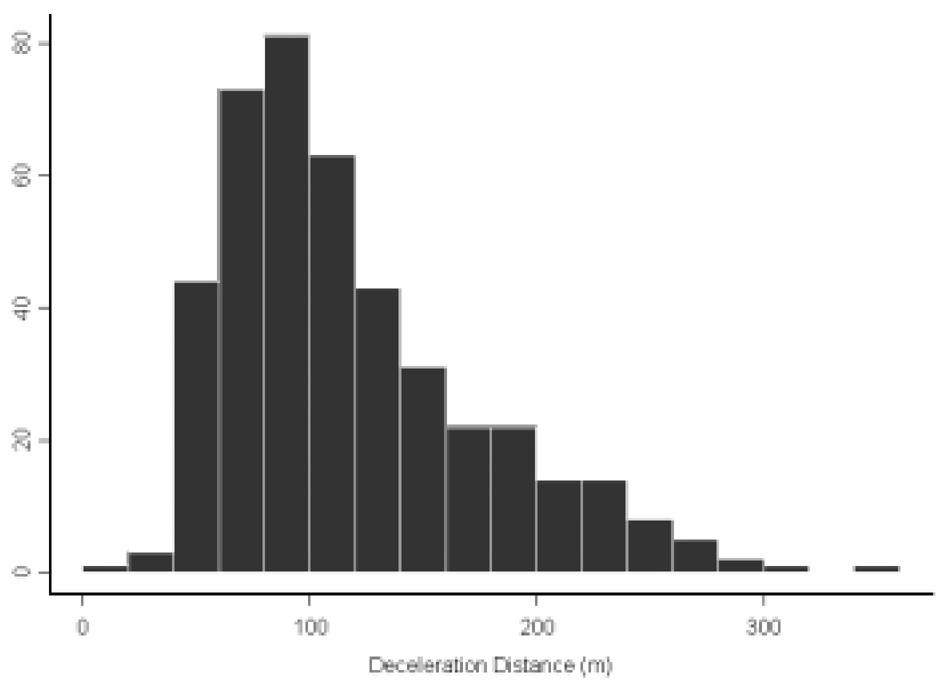

Figure 4-11 Distribution of Deceleration Distance

Figure 4-12 shows the distribution of deceleration distance on roads with different speed limits. 85 percent of the trips have a deceleration time less than 18 seconds.

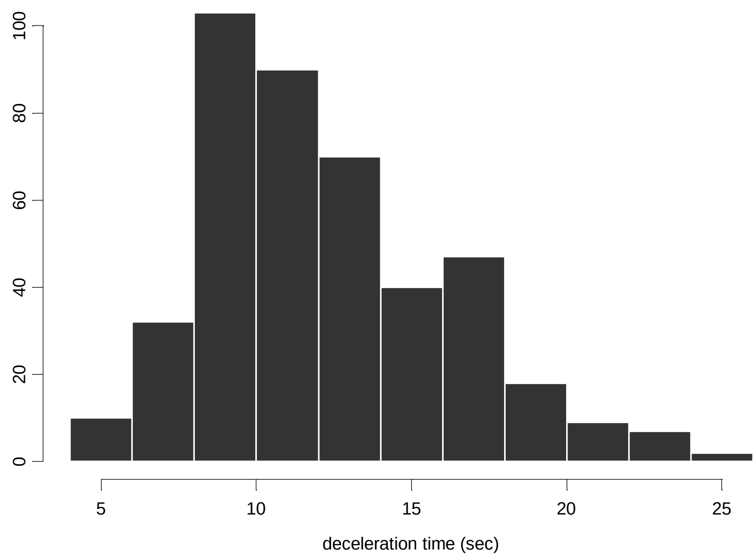

Figure 4-12 Distribution of Deceleration Time

CHAPTER 5

SELECTION OF STUDY CORRIDORS

5.1 Determination of Study Corridor Length

In this dissertation, a study corridor is defined as the roadway section between two intersections. The corridor has uninterrupted flow (no stop-control traffic control device such as a traffic signal or stop sigh present). If a study corridor is delimited by two intersections with traffic control devices, it must be long enough so that drivers can reach their desired speeds. If a study corridor is delimited by two intersections without traffic control devices, there is no minimum length requirement, but it must be located at a sufficient distance from the adjacent traffic control devices. Figure 5-1 demonstrates a typical study corridor.

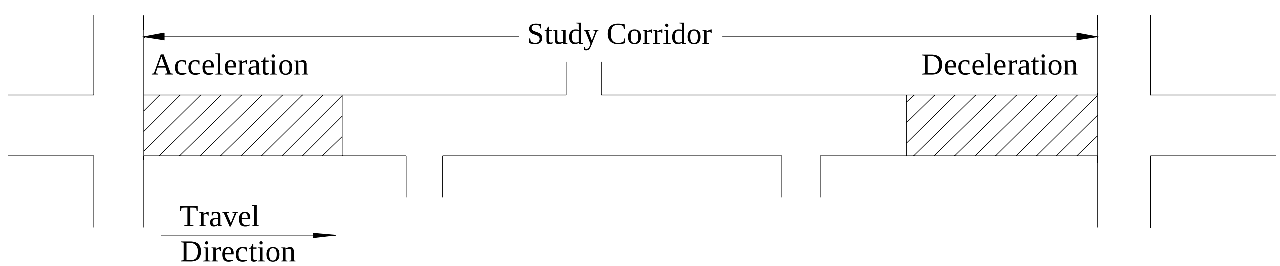

Figure 5-1 Example Study Corridor Layout

Several previous studies have indicated that the selected study corridors have to be long enough or sufficiently distant from the adjacent traffic control devices. Poe et al. (1996) investigated the relationship between urban road environment and vehicle speeds. In this study, the researchers defined a typical corridor as the entire roadway between the traffic control devices on both ends.

The corridors were typically 1 to 2 km (3280 ft to 6560 ft) in length. Fitzpatrick et al. (2001) evaluated the design factors that affected vehicle speeds on suburban streets. They defined the straight section/corridor as a straight portion of a suburban arterial between horizontal curves and/or traffic control devices. The straight sections selected were at least 200 m (656 ft) from an adjacent horizontal curve and 300 m (984 ft) from adjacent signal or stop sign. The length of these sections ranged from 149 to 1398 m (489 to 4585 ft). Another study by Fitzpatrick et al. (1997) investigated the operating speed on suburban arterials. In this study, there was at least 200 m (656 ft) between the study site and a signalized intersection to eliminate the effect of traffic control devices on vehicle speeds. Polus et al. (1984) suggested that the study site should be at least 500 m (1,640 ft) from any intersection to avoid the effect of traffic control devices on vehicle speeds. Schurr et al. (2002) studied the relationship between design, operating, and posted speeds at horizontal curves on rural two-lane highways in Nebraska. They suggested at least 300 m (984 ft) from the study site to any intersection or other elements which may affect operating speeds.

These previous urban studies indicated that the selected corridor should be located between two intersections and generally the corridor should exclude certain distances for each intersection in an effort to remove the influence of the traffic signal or similar traffic control devices on driver selected speeds. If the corridor includes the intersections, drivers may choose the vehicle speeds according to the status of traffic control devices at the intersection rather than the road environment. Speeds should also be measured for vehicles in traffic streams under free flow conditions to avoid the impact of traffic flow characteristics on specific vehicle speeds. These previous studies generally indicated selected corridor lengths and, if included, separation distances from proximate intersections. They did not, however, delve into the question of how to determine an adequate distance from the intersection influence regions or how to determine a minimum study corridor length so that drivers could reach their desired speeds without the influence of traffic control devices.

In previous chapter, the author evaluates drivers' acceleration and deceleration behaviors on low-speed urban streets. The research results provide guidance in determination of the minimum length of the studied corridors between two intersections with traffic signals or stop signs so that the selected streets are long enough that drivers are able to select and achieve their desired corridor speeds without the influence of adjacent traffic control devices.

The length of a selected study corridor should be at least equal to the length of acceleration zone plus the length of deceleration zone so that drivers are able to accelerate to their desired speeds under free-flow conditions. Since drivers' acceleration final speeds and deceleration approach speeds are generally unknown, this study uses the average acceleration and deceleration distance to estimate the minimum length of the study corridors for various speed limits. These values are depicted in Table 5-1.

Table 5-1 Minimum Length for Study Corridors

Speed Limit (km/h)	**Acceleration Distance (m)**	**Deceleration Distance (m)**	**Minimum Corridor Length (m)**
40	101	93	194
48	110	116	226
56	149	157	306
64	212	198	410
72	202	229	431

5.2 Selection of Study Corridors

5.2.1 Corridor Selection Criteria

Drivers shall be able to driver at free-flow speeds in the selected study corridors during low traffic volume conditions. Therefore, the corridor should be long enough or sufficiently distant from adjacent traffic control devices so that drivers can reach their desired speeds. In this study, the corridors are selected based on the following criteria:

1. Urban low speed streets with speed limit between 48 and 72 km/h (30 and 45 mph), which includes urban minor arterial, urban collector and local streets.

2. If the selected corridors are bounded by two intersections with traffic signs or stop signs, the corridors have to be long enough so that drivers can drive beyond the influence of traffic control devices. The selected study corridors should meet the requirement of minimum corridor length and intersection distance in Table 5-1.

3. The selected corridors should represent a variety of road geometry, roadside design, cross-section characteristics, adjacent land uses, and posted speed limits.

4. The selected corridors should have observations from as many different drivers as possible.

5.2.2 Identification of Study Corridor Candidates

The author first analyzed the received GPS data to identify the number of trips and the number of different drivers traveled on each road segment. Second, the author selected those road segments that have multiple trips from as many different drivers as possible to represent the study corridor candidates. Then, based on the other site selection criteria, the author selected the study corridors from these candidates.

In order to identify the number of trips and drivers on each road segments, the author analyzed the extensive one-second interval vehicle data. The total amount of data received from the 145 drivers during one year period is about 20 gigabytes. Thus, it was very important to develop efficient data process methods to handle the received data. The data process should be automated and without human intervention.

The author developed a perl script that processes the received data records and finds the traveled routes for each driver. This script has the following functions:

1. For each driver, the script finds all road segments along which that driver has traversed. The script sorts the traversed road segments by frequency. One road segment is defined as the road section with the same RCLINK and MILEPOINT combination. The results are stored as text files for the next step. These files are named as step_one_file, which only includes the information for each traversed segment, including RCLINK, MILEPOINT, and the number of data points along each segment.

2. The script then joins the Road Characteristics file (RC File) with the step_one_file based on RCLINK and MILEPOINT. This step creates a new txt file, step_two_file, which has road characteristic information, including speed limit and functional class, as well as all information in step_one_file. Next, the script divides the step_two_file into several smaller text files based on the functional class values since the researchers study the corridors of different functional classes separately. This step creates step_three_file. Each file would only contains the data on roads of a specific functional classification.

3. Based on the analyzed data for each driver from the previous step, the script summarizes the data for all drivers. The script finds the most frequently traversed road segments for all drivers, and the number of trips and number of drivers along each of these road segments. This step creates step_four_file, which includes the information for all drivers on roads of a specific functional classification.

The author identified the study corridor candidates based on these statistics obtained in the step_four_file.

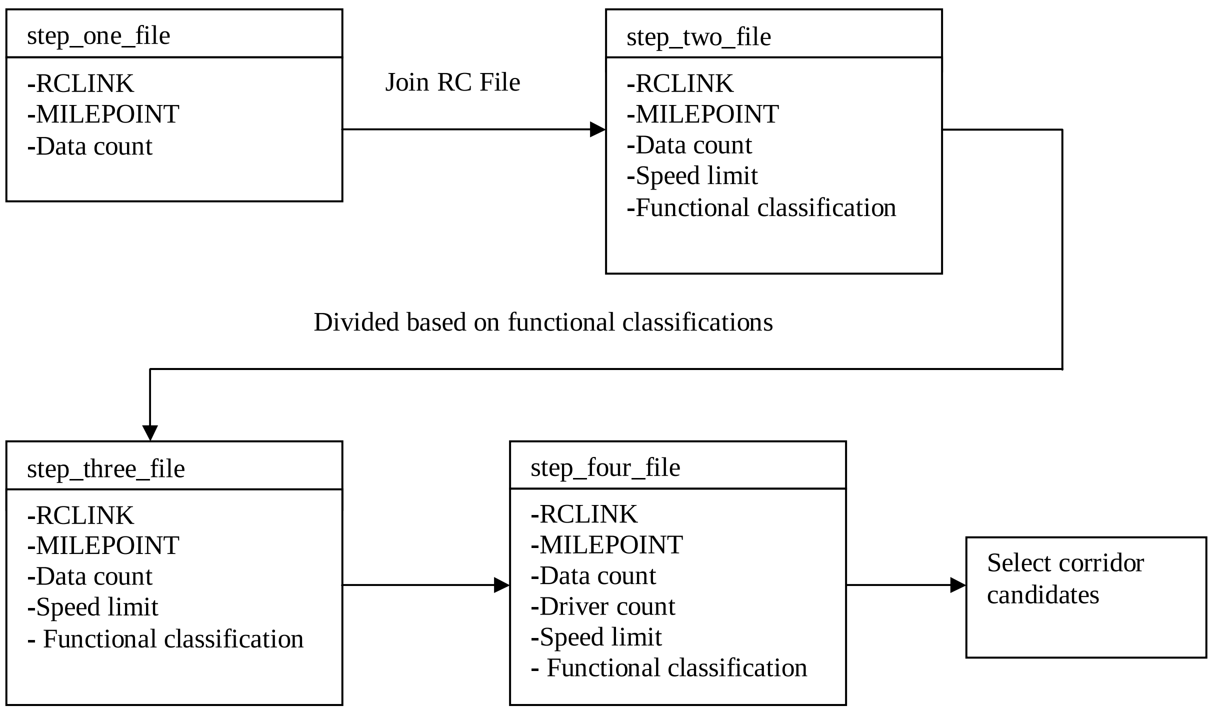

Figure 5-2 Study Corridor Candidate Selection Procedure

5.2.3 Selected Study Corridors

This dissertation evaluated four types of corridors including urban local streets, collectors, minor arterials, and principal arterials. The selected roads in each type vary in road geometry, roadside design, cross-section characteristics, adjacent land use, and posted speed limit. Table 5-2 summarizes the general characteristics of the collected corridors.

The evaluation summarized in this dissertation is based on the assumption that a driver is mostly influenced by the side of the road nearest to his/her vehicle, which is the drivers' right side. The author measured the road features on both sides of the selected roads. Each side of the road is considered as a separate corridor in this study. The analysis included 28 two-way low speed urban streets that correspond to 56 study corridors with the consideration that each corridor has two directions. 36 corridors are horizontal curves and 20 corridors are tangent sections. Based on the heading information of the GPS data, the author determined the direction a vehicle traveled.

Table 5-2 Summary of Selected Corridors

Site ID	Functional Class	Speed Limits (km/h)	Alignment	Number of lanes	Land Use
Urban local street					
C7	19	56	horizontal curve	1	residential
C8	19	56	horizontal curve	1	residential, park
C10	19	56	tangent	1	commercial
C11	19	56	horizontal curve	1	commercial
C15	19	56	tangent	1	residential
Urban connectors					
C3	17	48	horizontal curve	1	residential, park
C4	17	48	tangent	1	residential, park
C23	17	48	tangent	1	commercial
C24	17	48	horizontal curve	1	residential
C25	17	48	horizontal curve	1	residential
C26	17	48	tangent	1	residential
C96	17	48	horizontal curve	1	residential
C28	17	56	tangent	2	commercial, park
C81	17	56	horizontal curve	2	commercial
Urban minor arterials					
C32	16	56	horizontal curve	1	residential
C33	16	56	horizontal curve	1	residential park
C34	16	56	horizontal curve	1	residential
C35	16	56	tangent	1	residential
C91	16	56	horizontal curve	1-2	residential
C92	16	56	horizontal curve	2	commercial
C43	16	64	horizontal curve	2	residential
C44	16	64	tangent	2	residential
C94	16	46	horizontal curve	2	residential
Urban principal arterials					
C48	14	56	horizontal curve	1	residential
C53	14	64	horizontal curve	2	commercial
C49	14	56	horizontal curve	2	residential
C51	14	64	tangent	2	apartment school
C52	14	64	tangent	2	residential

5.3 Roadway Environment Characteristics

Roadway environment characteristics include roadway features, cross section/road side characteristics, traffic calming techniques, and adjacent land uses. Some road environment information is available from the Georgia Department of Transportation (GDOT) Road Characteristic (RC) file. Others were measured in the field.

Most previous studies are spot speed studies in which speed data are collected at specific location of a roadway, such as the middle point of tangents and horizontal curves. Point variables are measured at those locations to describe the road environment features at the specific locations, such as sight distance, grade, and curve radius. This study is based on continuous speed profile. A corridor variable should be appropriate to describe the characteristics of a roadway section as a whole, such as average sight distance, average grade, and average curve radius. These corridor variables were measured over the length of the study corridor. Therefore, it was necessary to identify an adequate, homogeneous corridor for the continuous speed study that demonstrates the influence of corridor features on drivers' speeds.

Table 5-3 presents the corridor variables and their measured values. All selected corridors are two-way one-lane or two-lane roads (per direction of travel), with the superelevation of zero. The horizontal curve radius ranges from 54 m (177 ft) to 172 m (565 ft). The average grade ranges from -4 to 4 degree. The site selection is not completely random in this study since it is based on the collected GPS speed data. The selected sites include limited road features.

Therefore, the results of this study should be only applied to low-speed urban streets with similar characteristics.

Table 5-3 Selected Corridor Characteristics

Corridor variables	Measured value
Alignment	
corridor length (m)	208 to 847
average curve radius (m)	54 to 172
horizontal curve length (m)	27 to 141
horizontal curve type	single curve, S-curve, compound curve
average grade (percent)	-4 to 4
vertical curve type	sag and crest
average sight distance (m)	54 to larger than 140
superelevation	0
Access Density	
driveway density (per km)	0 to 48
T-intersection density (per km)	0 to 14
Roadside Objects	
tree density (per mile)	0 to 13
tree offset (m)	0 to 1.3
utility pole density (per mile)	0 to 27
utility pole offset (m)	0 to 1.3
mailbox density (per mile)	0 to 19
mailbox offset (m)	0 to 1.1
Cross-Section Features	
average lane width (m)	2.8 to 4.6
number of lanes	1 to 2
median type	none, TWLT, painted, raised
sidewalk width (m)	0 to 3
shoulder type	none, curb and gutter, curb only
landscape buffer width (m)	0 to 1.5
on-street parking	none, on-street parking
Others	
land use	commercial, residential, park, forest
speed limits (km/h)	48 to 64
pavement quality	poor, fair, good

5.4 Curve Radius Estimation with GPS Data

The radius or degree of curvature of a horizontal curve is one of the most important geometric features in roadway safety studies. However, this information is not available in the GDOT road characteristics database (RC File). The author had to measure it in the field or refer to the original design plans. However, it is dangerous to measure the curve radius in the field since some low-speed roadways do not have sidewalks. For high-speed highways, it may be not feasible to measure the curve radius in the field. Another alternative method is to look the value in the original road design plans. However, sine these design plans are kept by different agencies at the states or county level, it is time consuming to find the needed information and often original plans are no longer in existence.

GPS technology provides an advanced way of road geometry measure. A GPS receiver can determine position (latitude, longitude, and attitude) and time at one-second interval. If a vehicle equipped with a GPS recorder is traveling on a roadway, the road alignment information can be obtained by analyzing the received GPS data. For example, if the researchers have repeated trips on one horizontal curve, they can mathematically calculate the average radius of the curve based on the latitude and longitude information.

This dissertation uses a nonlinear least square regression method, called curve fitting, to calculate the average curve radius. That is, the author tries to find a best circle to fit the collected

GPS data points so that the points are as close to the perimeter of the circle as possible. The radius of the best fitting circle would be the average radius of the horizontal curve. The goal is to determine the center and radius of the circle given a set of points so that the selected circle best fits the points, which means the sum of the squared distances between the points and the perimeter of the circle are minimized. Assuming that the center of the circle is at (Xc, Yc) and a given point is at (Xp, Yp), then the distance from the center of circle to the point is in Equation 5-1.

$$\sqrt{(Xp - Xc)^2 + (Yp - Yc)^2} \tag{5-1}$$

If assume that the radius of the circle is R, the distance from the perimeter to the point is equal to the distance from the center to the point less the distance from the center to the perimeter, which is show in Equation 5-2;

$$\sqrt{(Xp - Xc)^2 + (Yp - Yc)^2} - R \tag{5-2}$$

The sum of squared distance from the given points to the perimeter of the circle is calculated in Equation 5-3. Assume the given points are (Xi, Yi), i = 0, 1, 2, … n, the author tries to find the values of Xc, Yc, and R to minimize the total squared error of the dataset.

$$\sum_{i=1}^{n}(\sqrt{(Xi - Xc)^2 + (Yi - Yc)^2} - R)^2 \tag{5-3}$$

Several nonlinear regression and curve fitting software, such as Maple 9 and NLREG, are available to solve this nonlinear regression problem. This study used Maple 9 to do the curve fitting.

Figure 5-3 illustrates one horizontal curve without overlaid GPS data points. In contrast, Figure 5-4 illustrates the curve with overlaid GPS data points that come from multiple trips with different travel directions. The RCLINK and MILEPOINT of the selected curve are labeled in the two figures.

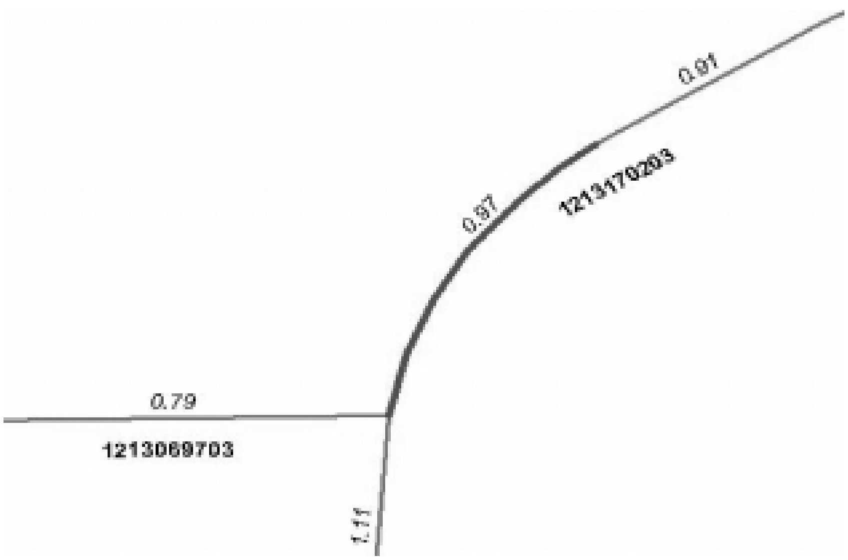

Figure 5-3 Horizontal Curve without Overlaid GPS Data Points

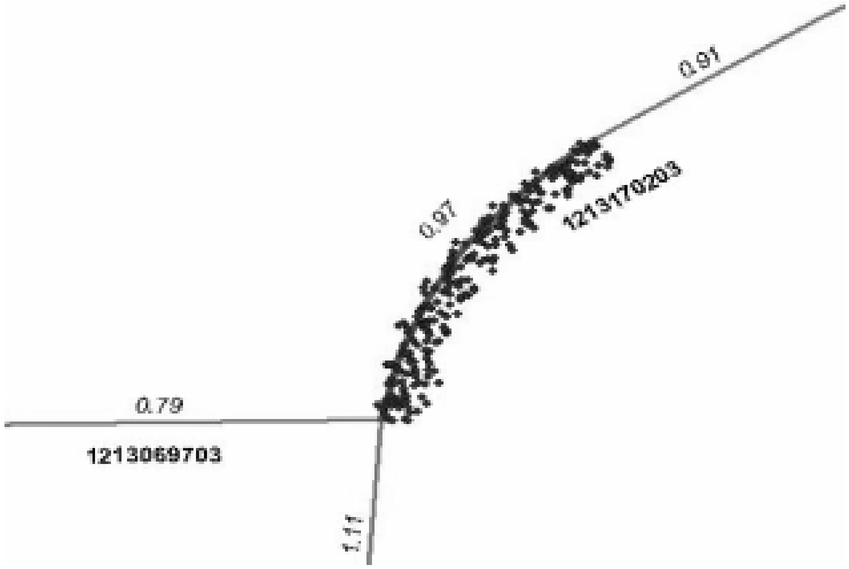

Figure 5-4 Horizontal Curve with Overlaid GPS Data Points

Table 5-4 lists a sample trip data on the selected horizontal curves. Each row of the table represents a point with the speed, time, location and travel heading information

Table 5-4 Sample Trip Data on Selected Horizontal Curves

Date	Time	Speed (km/h)	Latitude	Longitude	Heading (degree)
20020418	221432	48.432	33.777391	84.402311	14.04
20020418	221433	47.28	33.777504	84.402270	21.03
20020418	221434	48.352	33.777615	84.402211	26.81
20020418	221435	49.104	33.777721	84.402138	33.12
20020418	221436	48.528	33.777815	84.402048	42.00
20020418	221437	44.88	33.777896	84.401950	47.10

Location was identified by latitude and longitude in the decimal degree formats. Latitude is determined by the earth's polar axis. Longitude is determined by the earth's rotation. To use these data for circle fitting, this study converted the latitude and longitude in decimal degree format to coordinates in linear unit, such as meters.

This study uses a local coordinate system with the first GPS point as its origin and uses Equation 5-4 to convert the distance form decimal degree to meter.

$$\text{Distance (m)} = R * Pi * \text{Distance (degree)}/180 \qquad (5\text{-}4)$$

Where;

R = 6378137 (m), which is the equatorial radius, and

Pi = 3.1415926.

The converted GPS data are shown in Table 5-5. Latitude and longitude are in meters.

Table 5-5 Converted Sample Trip Data on Selected Horizontal Curves

Date	Time	Speed (km/h)	X	Y	Heading (degree)
20020418	221432	48.432	0.00	0.00	14.04
20020418	221433	47.28	12.58	4.56	21.03
20020418	221434	48.352	24.94	11.13	26.81
20020418	221435	49.104	36.74	19.26	33.12
20020418	221436	48.528	47.20	29.28	42.00
20020418	221437	44.88	56.22	40.19	47.10

The estimated curve radius is 112 m (367 ft) based on northbound trip data that includes 27 trips, and 118 m (387 ft) based on southbound trip data that includes 14 trips. Therefore, the average curve radius is 115 m (377 ft), which corresponds to the centerline of the curve. Table 5-6 shows that the field measured average curve radius is about 108 m (354 ft), which corresponds to the inner edge of the curve. Since the selected curve is a one-lane two-way roadway. Considering the lane width of 3.6 m (12 ft), the estimated radius using circle fitting is very close to the field measured value.

Table 5-6 Field-Measured Curve Radius

Chord Length (m)	Middle Ordinate (m)	Radius (m)
30.5	1.19	98
30.5	1.16	101
30.5	1.04	113
30.5	0.98	120
Average	1.10	108

The author used this method to estimate the average curve radius of a set of selected horizontal curves in this study. Table 5-7 indicates that curve fitting using GPS data provides an accurate estimation of horizontal curve radius.

Table 5-7 Comparison of Field Measured and GPS Estimated Average Curve Radius

Site_ID	Measured curve radius (m)	Estimated curve radius by GPS (m)	Difference (%)
C3_E	93	98	5.59
C3_W	89	97	9.25
C8_W	92	78	15.18
C8_E	92	73	21.12
C11_E	94	88	5.54
C11_W	96	94	2.22
C24_N	201	172	14.13
C24_S	201	171	14.89
C43_N	61	71	16.50
C43_S	76	72	5.24
C81_W	128	126	1.67
C81_E	138	140	1.77
C91_W	99	85	14.72
C91_E	104	85	18.24

5.5 Speed and Road Feature Database

Based on the collected road features, the author created a database that combined the speed data and the associated road features. This database was then used in data analysis and model development. Table 5-8 presents an example of the database. The records in one shadow area represent the data from one single trip.

Table 5-8 Example Database

Driver	Site_ID	Date	Time	Speed (km/h)	Speed Limit (km/h)	Lane Width (m)
T01006	1	20030306	165210	45.46	48	3.66
T01006	1	20030306	165211	47.22	48	3.66
T01006	1	20030306	165212	47.28	48	3.66
T01006	1	20030306	165213	48.06	48	3.66
T01006	1	20030306	165214	43.09	48	3.66
T01006	1	20030306	165215	43.55	48	3.66
T01006	1	20030306	165216	44.82	48	3.66
T01006	1	20021026	120703	48.90	48	3.66
T01006	1	20021026	120704	42.42	48	3.66
T01006	1	20021026	120705	49.07	48	3.66
T01006	1	20021026	120706	45.20	48	3.66
T01006	1	20021026	120707	48.93	48	3.66
T01006	1	20021026	120708	50.11	48	3.66
T01006	1	20021026	120709	49.28	48	3.66
T01006	4	20030117	145720	44.10	56	3.35
T01006	4	20030117	145721	59.87	56	3.35
T01006	4	20030117	145722	49.04	56	3.35
T01006	4	20030117	145723	52.78	56	3.35
T01006	4	20030117	145724	57.87	56	3.35
T01009	4	20021026	125910	49.18	56	3.35
T01009	4	20021026	125911	50.35	56	3.35
T01009	4	20021026	125912	52.37	56	3.35
T01009	4	20021026	125913	51.62	56	3.35
T01009	4	20021026	125914	52.83	56	3.35
T01009	4	20020202	165937	35.22	56	3.35
T01009	4	20020202	165938	36.75	56	3.35

CHAPTER 6

DATA ANALYSIS

6.1 Data Reduction

This dissertation generates speed profiles for each trip by plotting operating speed versus distance along each of the study corridors. Figure 6-1 is a typical speed profile of multiple trips on a selected corridor between two signalized intersections. Assume the traffic volume is low and the corridor is long enough, where drivers begin to accelerate from a stopped condition, the influence of the road environment is initially negligible because the speed is low and the acceleration rate is a factor of vehicle type and surrounding traffic conditions. After accelerating to a certain appropriate speed, drivers begin to adjust speeds according to their perceptions of the road environment as their vehicles approach their final desired speed. Drivers maintain their desired free-flow speeds until they approach the next stop controlled intersection. This dissertation is concentrated on the road environment's impact on drivers' selection of free-flow speeds.

Traditional speed studies often use five-second headway as the criterion to determine if a driver is driving at free-flow speeds. However, this criterion is not applicable in this study since the speed data are collected by the in-vehicle GPS instruments for study vehicles only. Therefore, the author uses only day time trips during off peak time period to increase the likelihood of sampling free-flow speeds. However, this criterion alone can not guarantee that drivers are driving under free-flow condition.

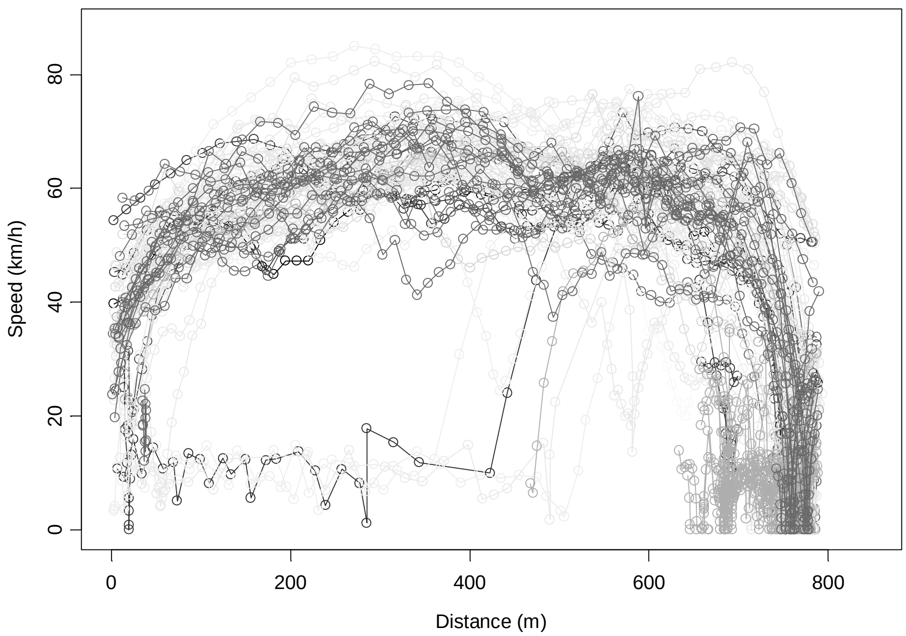

Figure 6-1 Speed Profile during Off-Peak Time

Figure 6-1 demonstrates the possibility that drivers may not always drive at free-flow speeds during off peak time period. A driver may have to slow down to accommodate turning vehicles or crossing pedestrians. In Figure 6-2, the speed distribution also indicates that drivers often drive at low speeds during off-peak time period.

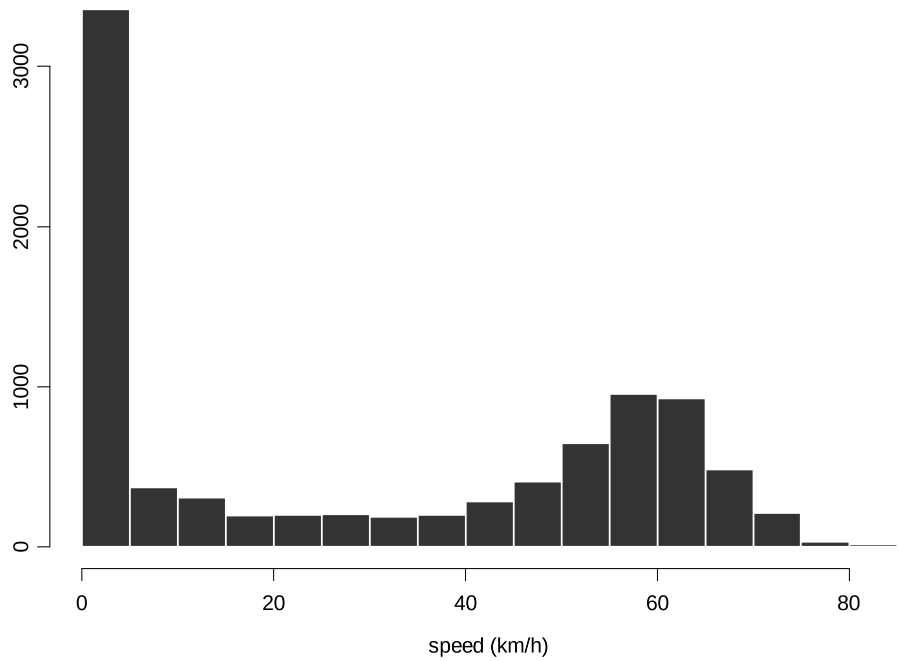

Figure 6-2 Speed Distribution during Off-Peak Time

Since weather conditions may affect drivers' speeds, the author removed the trips that occurred under raining conditions for the purpose of initial model development. The author obtained the historical weather information from the National Oceanic and Atmospheric Administration (NOAA). Based on the trip location, time, and date, the author determined if it had rained at a specific time and location.

After that, the author removed the speed data when drivers accelerate and decelerate. Based on the acceleration and deceleration study in Chapter 4, the author estimated an average acceleration and deceleration distance on each selected corridor based on its speed limit. Then, the author removed the acceleration and deceleration data of the trip based on the acceleration and deceleration distances if the site is delimited by two traffic signals or stop signs. In the next step, the author calculated the speed distribution at each study corridor. For an uninterrupted trip, this dissertation assumes that drivers' selected speeds (not in the acceleration and deceleration zone) should be greater than a certain threshold, assumed as two standard deviations from the mean speed at each study corridors for this modeling effort. Trips that do not satisfy this criterion were removed from the dataset. Figure 6-3 and 6-4 show the example of the speed profiles and distribution of trips on the same corridor as in Figure 6-1 and 6-2.

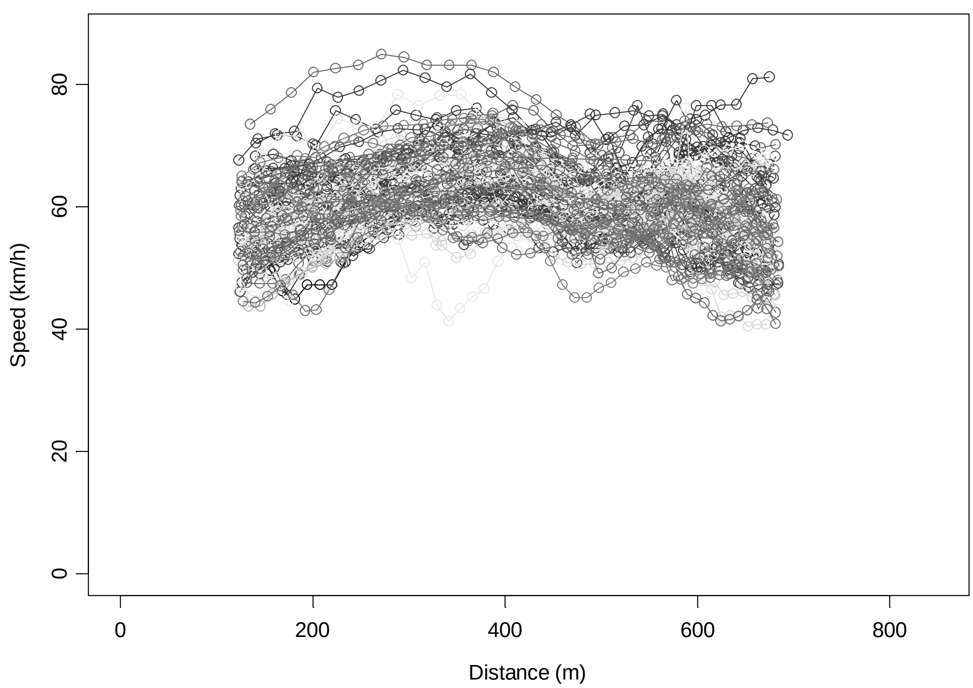

Figure 6-3 Example Speed Profile in the Final Dataset

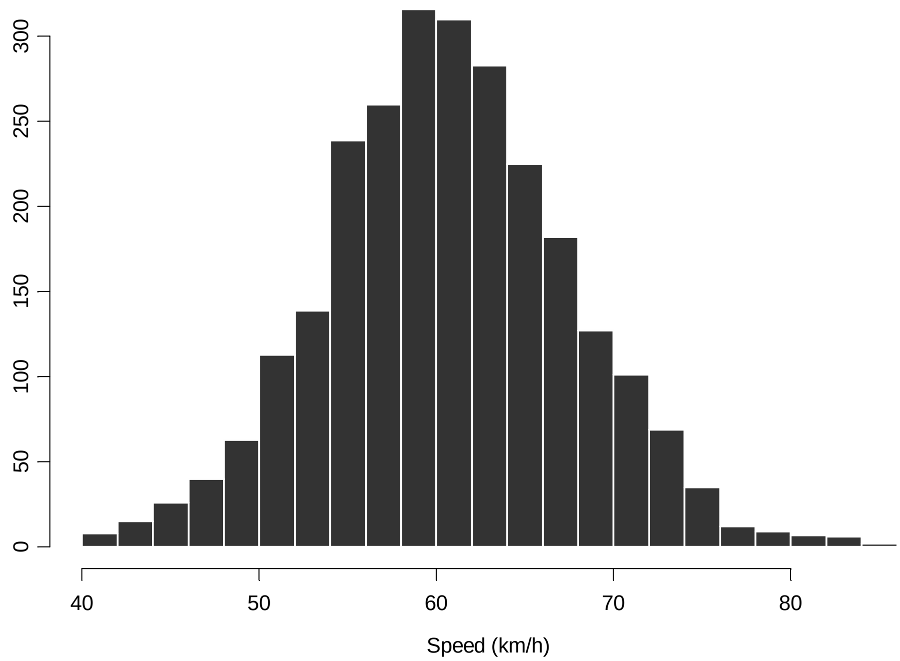

Figure 6-4 Speed Distribution in the Final Dataset

Figure 6-4 indicates that after filtering out acceleration, deceleration, and interrupted speed trips, drivers are more likely to drive at free-flow speeds, which follows a normal distribution.

To summarize, this dissertation uses the following steps to filter speed data:

1) Only select trips during off-peak time during daylight (10 am to 4 pm),
2) Remove trips that occurred under raining conditions,
3) Remove acceleration and deceleration data based on acceleration and deceleration distance if the selected corridor is ended by two traffic signals or stop signs,

4) Remove trips if any speed point (not in acceleration and deceleration zone) in the trip drops below two standard deviations from the mean because the driver is possibly affected by other drivers or is making turning movement.

6.2 Driver Selected Speeds on Tangents and Horizontal Curves

The driver selected speeds (desired speeds) on tangents or horizontal curves under free-flow conditions are the reflection of the impact of road environments on drivers' speed choices. Previous studies consider the maximum speeds along a tangent and the minimum speeds along a horizontal curve as the desired speeds since tangents do not have any geometric constraints on drivers. However, these studies assumed that drivers reach their maximum speeds at the middle point of tangents and reach their lowest speeds at the middle point of horizontal curves because traditional data collection methods (radar gun, detector) could only measure speeds at a few specific pre-selected locations along the roadway. With the second-by-second speed profile in this study, the author found that this assumption is not realistic for modeling operating speeds, especially on urban streets. Drivers reach their maximum speeds at different locations along the tangents. Even the same driver reaches his or her maximum speeds at different locations along the same tangent for different trips. This study found that drivers reach their minimum speeds at different locations, not just at the middle point of the horizontal curve. In this study, the second-by-second speed profile makes it possible to measure the true operating speeds along the selected corridors even when extreme values take place at different locations along the same corridor.

For each trip along a tangent corridor, this study captures a continuous speed profile at one-second interval, which provides more detailed information of drivers' driving behavior, such as speed variations, maximum and mean speeds.

For a tangent, this dissertation defines cruising speeds as the speeds at which drivers are driving along the tangent after they reached their maximum speeds. For a typical speed profile along a tangent, drivers would accelerate to their desired (maximum) speeds and then drive at the cruising speeds until they have to decelerate. The speed profiles collected in this study indicate that drivers still vary their cruising speeds along the corridor. Therefore, this dissertation used the 85th and 95th percentile of the cruising speed to estimate the driver selected speed for this trip, as shown in Figure 6-5. The author recommends the 85th and 95th percentile speed rather than the maximum speed because the collected speed profiles shows that drivers sometimes speed up to higher speed value and then adjust speed back to a preferred rate. The choice of maximum speeds during a trip may result in higher speeds than those actually maintained for the trip.

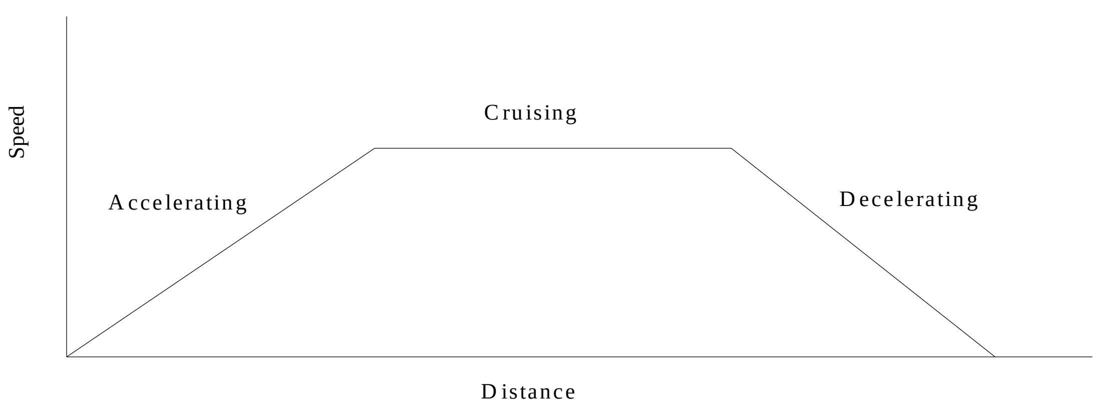

Figure 6-5 Speed Profile along Tangent

The minimum intersection length criterion guarantees that drivers can reach their desired speeds on selected corridors under free flow conditions. However, the situation is more complicated for horizontal curves because of different tangent lengths preceding the curves, as shown in Figure 6-6.

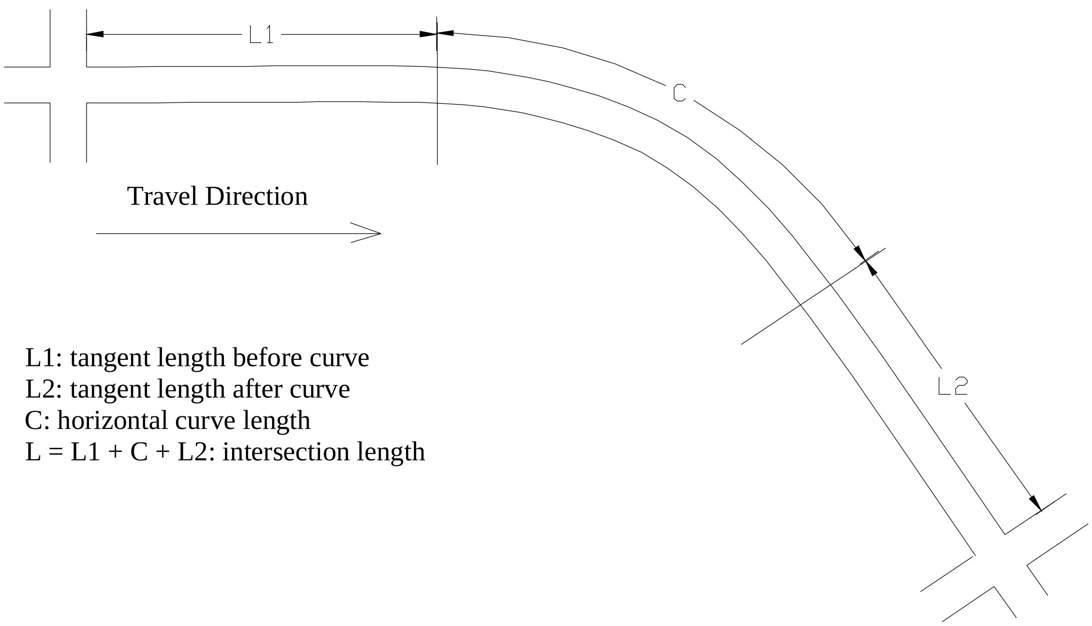

Figure 6-6 Horizontal Curve between Two Tangents

For a horizontal curve between two tangents, two situations exist. If the length of tangent before the horizontal curve (L1 in Figure 6-6) is long enough so that drivers reach their maximum speeds on the tangent, drivers normally decelerate when they start traveling along the curve. In this case, this dissertation defines cruising speeds as the speeds at which drivers are driving along the horizontal curve after they reach their minimum speeds.

Most of previous studies used minimum speeds (at midpoint of curve) as the driver selected speeds. However, the speed profiles collected in this study indicated that after drivers reached their minimum speeds, they tended to adjust (increase) their speeds when they were still traveling along the curves. Therefore, this dissertation uses the 85th and 95th percentile of the cruising speeds along the curve corridor to estimate the drivers' desired speeds. This speed choice is consistent with the author's assumptions for the tangent model. Figure 6-7 shows the speed profile under this condition.

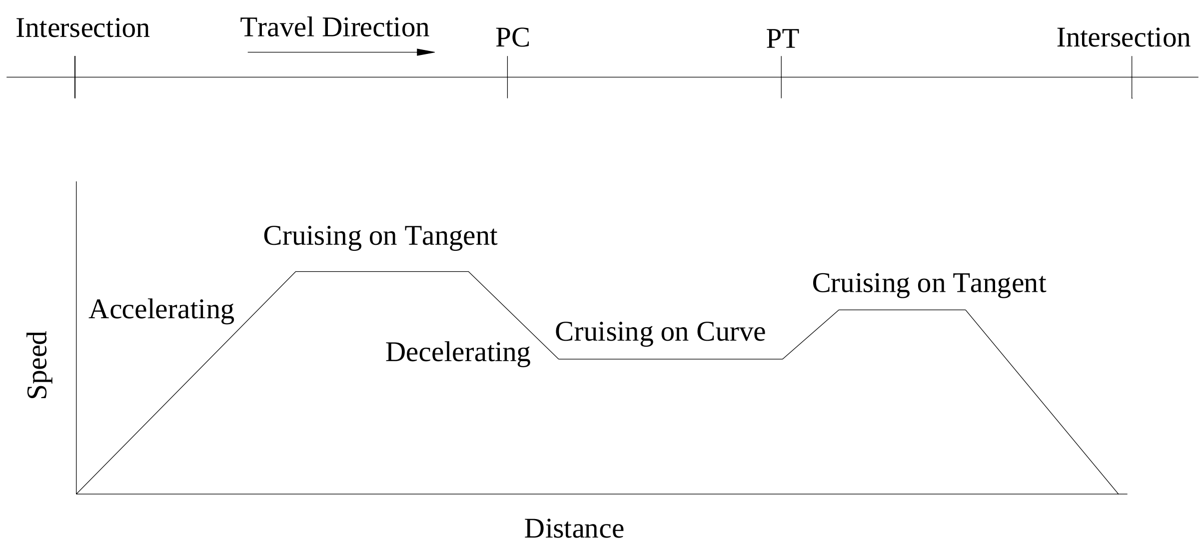

Figure 6-7 Speed Profile along Horizontal Curves with Long Leading Tangent

If the length of tangent before the horizontal curve is not long enough so that drivers still are driving at a low speed when they approach the horizontal curve, drivers continue to increase their speed along the curve. In this case, this dissertation defines cruising speeds as the speeds at which drivers are driving along the horizontal curve after they reach their maximum speeds. Therefore, this study uses the 85th and 95th percentile of the cruising speed along the curve to estimate drivers' desired speed. Figure 6-8 shows the speed profile under this condition.

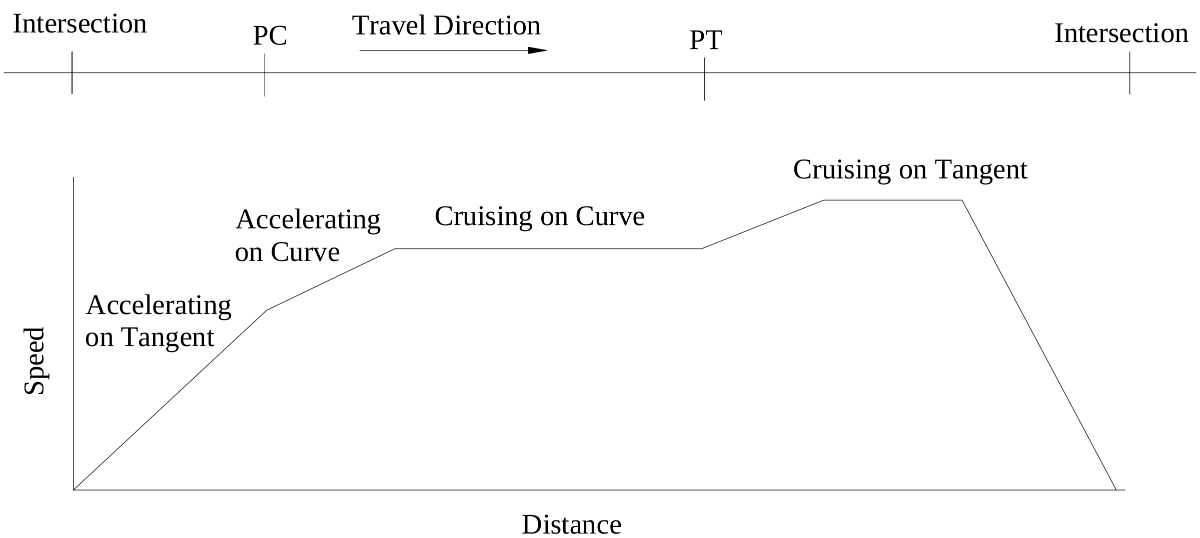

Figure 6-8 Speed Profile along Horizontal Curves with Short Leading Tangent

6.3 Speed Variation Components Analysis

The author assumes that source of speed variation comes from the following sources under free-flow conditions, including driver/vehicle characteristics, road environments, and other unknown or unobservable factors.

Most of previous operating speed models try to explain speed variation based on road feature variations. In those models, the dependent variable is operating speed while the independent variables are road features. Generally, speed data were collected at a specific location for each selected site, for example, the middle point of tangents or horizontal curves. Researchers calculated the aggregated speed statistics for each site (85^{th} percentile speed or mean speed), and used them as the dependent variable in model development. If the sample size of speed data collected at each site meets the minimum requirement, the aggregated speed is the representative speed for general drivers at each site. Therefore, the variation of the driver and vehicle characteristics were removed in aggregation. Hence the variation of speeds is caused by road features and other unknown factors, as shown in equation 6-1.

$$\sigma^2_{speed} = \sigma^2_{road} + \sigma^2_{unknown} \tag{6-1}$$

In this study, since the driver and vehicle information is available for each speed data point and multiple trips from the same drivers are available so that this dissertation is able to identify the variation caused by driver and vehicle. This dissertation included the influences of drivers and vehicles into model development, which is not possible in traditional speed studies. In this study, the source of speed variation includes road features, driver/vehicle characteristics, and other unknown factors, as shown in equation 6-2.

$$\sigma^2_{speed} = \sigma^2_{driver} + \sigma^2_{road} + \sigma^2_{unknown} \tag{6-2}$$

Where

σ^2_{speed} : speed variation,

σ^2_{driver} : driver/vehicle characteristics variation

σ^2_{road} : road feature variation, and

$\sigma^2_{unknown}$: unknown variation

6.4 Speed Data Aggregation

For each trip along a tangent corridor, the author calculated statistics including mean speed, 85th percentile speed, 95th percentile speed, maximum speed, and the standard deviation of the cruising speeds. This study uses the 85th (V85) and 95th (V95) percentile speed to estimate the driver selected speeds (desired speeds) along tangents.

Similarly, for each trip along a horizontal curve corridor, the author calculates the statistics including mean speed, 85th percentile speed, 95th percentile speed, maximum speed, minimum speed, and the standear deviation of the cruising speeds.

Since different drivers may need different tangent lengths to accelerate to their desired speeds. For example, aggressive drivers may need shorter acceleration distance. The author used the speed profiles to determine if a driver reaches his/her desired speed on the preceding tangent.

On a long tangent followed by a horizontal curve, if the speeds on the tangent are higher than that on the horizontal curve, the driver is very likely to reach his/her desired speeds on the tangent and decelerate on the horizontal curve. Therefore, the author used 85th and 95th percentile speed of the cruising speeds after drivers reached their minimum speeds on the horizontal curve to estimate the driver selected speeds on horizontal curves.

On a short tangent followed by a horizontal curve, if the speeds on the tangent are lower than that on the horizontal curve, the driver is very likely to keep accelerating on the horizontal curve since he/her hasn't reached his/her desired speeds on the tangent. Therefore, the author used the 85th and 95th percentile speed of the cruising speed after drivers reached their maximum speeds on the horizontal curve to estimate the desired speed on horizontal curves.

6.5 Study Data Layout

Table 6-1 presents the layout of this study's dataset. In this dataset, each subject (driver) has different observations (trips). The road feature variables (x_{ijk}) are the same if the observations (trips) are on the same site.

Table 6-1 Longitudinal Data Layout

Subject (i)	**Observation (j)**	**Response**	**Covariates (k)**		
1	1	y_{11}	x_{111}	$\cdots$	x_{11p}
1	2	y_{12}	x_{121}	$\cdots$	x_{12p}
.	.	.	.	$\cdots$	.
1	n_1	y_{1n_1}	$x_{1n_1 1}$	$\cdots$	$x_{1n_1 p}$
.	.	.	.	.	.
.	.	.	.	.	.
.	.	.	.	.	.
N	1	y_{N1}	x_{N11}		x_{N1p}
N	2	y_{N1}	x_{N21}		x_{N2p}
.	.	.	.		.
N	n_N	y_{Nn_N}	$x_{Nn_N 1}$		$x_{Nn_N p}$

In which

i = 1, 2, …, N subjects (drivers)

$j = 1, 2, \ldots, n_i$ observations (trips) for subject i,

$k = 1, 2, \ldots, p$ road feature variables ,

y_{ij} = response (aggregated speed statistic) for subject i on observation j, and

x_{ijk} = road feature variable k for observation j from subject i.

This dissertation used 1560 trips from 187 drivers on 35 tangent corridors and 2984 trips from 188 drivers on 45 horizontal curve corridors in the model development.

CHAPTER 7

MODEL DEVELOPMENT

7.1 Regression Techniques

Linear regression is a technique commonly used to describe a statistical relationship between a dependent variable and one or more explanatory or independent variables. The simple linear regression model has the following general form:

$$\mathbf{y} = \mathbf{X}\boldsymbol{\beta} + \boldsymbol{\varepsilon} \tag{7-1}$$

$\boldsymbol{\varepsilon} \sim N(0, \sigma^2\mathbf{I}_n)$

Where:

$\mathbf{y}$ is the dependent variable vector,

$\mathbf{X}$ is independent variable model matrix,

$\boldsymbol{\beta}$ are the regression parameters vector, and

$\boldsymbol{\varepsilon}$ is the random error term vector.

Linear regression assumes the error terms are uncorrelated. That is, the outcome of one observation has no effect on the error term of any other observations. Therefore, the response variables are assumed to be uncorrelated (Verbeke and Molenberghs, 1997).

An analysis of variance partitions the total sum of squares (SSTO) into the Regression Sum of Squares (SSR) and Error Sum of Squares (SSE) as follows:

$$SSTO = SSR + SSE \qquad (7\text{-}2)$$

Where

SSTO = total sum of squared deviation from the mean,

SSR = deviation of the fitted regression value from the mean, and

SSE = deviation of the fitted regression value from the observed value.

The coefficient of determination (R^2) represents the proportion of total variation explained by the predictor variables, as showed in Equation 7-3. The larger the R^2, the more of total variation is explained by the predictor variables.

$$R^2 = SSR/SSTO = 1 - SSE/SSTO \qquad (7\text{-}3)$$

Many previous studies have employed this statistical approach to predict drivers' speed choices based on physical conditions such as roadway geometry and roadside features. The 85th percentile speed is the general statistic used to describe operating speed when assessing the influence of the road environment on speed selection.

However, normal linear regression methods are not appropriate for this modeling effort because the speed data from the same driver at different sites are likely to be correlated. The dependent variables (y_i) are not independent with each other. Therefore, the assumption of linear regression is violated. This study uses a linear mixed effects (fixed-effects and random-effects) model, which is an extension of ordinary linear regression model. Liner mixed effects model adds another random variable to reflect the influence from each individual subject so that it allows within-subject correlations and accounts for the influence of both fixed and random-effects in explaining the response variable (speed).

- Fixed effects: factor levels in the sample are all levels to which reference will be made. In this study, street environment features are fixed effects.
- Random effects: factor levels represent a random sample from the population. In this study, drivers are random effects because drivers in this study were randomly selected from a set of all possible drivers in the Atlanta, Georgia region.

The linear mixed effects model is as follows (Verbeke and Molenberghs, 2000):

$$\mathbf{y}_i = \mathbf{X}_i\boldsymbol{\beta} + \mathbf{Z}_i\mathbf{b}_i + \boldsymbol{\varepsilon}_i \qquad (7\text{-}4)$$

$$\mathbf{b}_i \sim \mathbf{N(0,\psi)}$$

$$\boldsymbol{\varepsilon}_i \sim N(0, \sigma^2\mathbf{I}_n)$$

Where:

$\mathbf{y}_i$ is the response vector for response for subject i,

$\mathbf{X}_i$ is the fixed effects model matrix for subject i,

$\mathbf{Z}_i$ is the random effects model matrix for subject i,

$\mathbf{b}_i$ is the vector of random effects coefficients,

$\boldsymbol{\beta}$ is the vector of fixed effects coefficients,

$\boldsymbol{\varepsilon}_i$ is the vector of random error term, and

Ψ is the covariance matrix for the random effects.

The fixed effects are applicable if researchers are only interested in treatments observed in the study. The random effects are applicable if treatments are a random sample from a larger population of treatments, and researchers are interested in all treatment levels in the population. Since this study is interested in the whole driver population, drivers are modeled as random factors.

This dissertation views the selected drivers as being randomly drawn from the population at large. The driver (subject) variable is a random effect and, in this way, the author is able to incorporate the sampling variability into account and make inferences about the driver population from which the subjects were selected. On the other hand, if we view the subject variable as a 'fixed effect' then our inferences are only made to the selected sample drivers.

7.1.1 Random Intercept Mixed Effects Model

Random intercept mixed effects model is a simple mixed effect model with the following form:

$$y_{ij} = \beta_{0i} + \beta_1 x_{1j} + \beta_2 x_{2j} + \ldots + \beta_p x_{pj} + \varepsilon_{ij} \qquad (7\text{-}5)$$

$$\beta_{0i} = \beta_0 + v_{0i}$$

$$\varepsilon_{ii} \sim N(0, \sigma^2)$$

$$v_{0i} \sim N(0, \sigma^2_v)$$

where

y_{ij} is the response (speed) of subject (driver) i at site j,

β_{0i} is the intercept of subject i,

β_0 is the mean speed across the population,

v_{0i} is a random variable represents the deviation from the mean speed for subject i, which represents the influence of driver/vehicle characteristics on his/her speeds,

β_i is the coefficient for road feature variable i,

x_{ij} is road features variable,

ε_{ij} is the random error for subject i at site j.

σ^2 is within subject variance, and

σ^2_v is between subject variance.

This model indicates that the speed of driver i at site j is influenced by road features, driver characteristics, and vehicle characteristics. Each driver's initial speed (intercept) is determined by the population mean speed β_0, plus a unique contribution from that driver v_{0i}. Therefore, each driver has his or her own distinct initial speeds. The population intercepts and slope parameters (β_i) represent the overall trend while the subject parameter (v_{0i}) represents the deviation of each subject from the population trend. This model assumes that the influence of road features is the same for all drivers (the same coefficient β_i for all drivers). Figure 7-1 represents this model graphically with only one independent variable (lane width). In this figure, *Driver j* is driving more aggressively than *Driver i*.

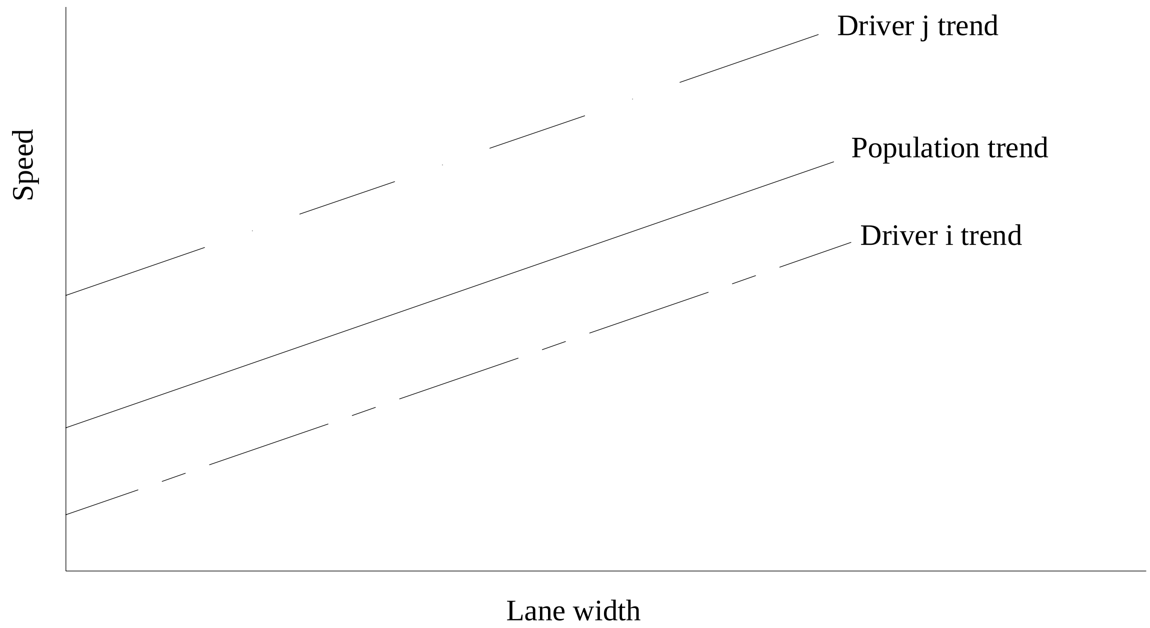

Figure 7-1 Random-Intercept Mixed Effects Model

The between-subject variance σ^2_v measures the variability of speeds from different drivers at the same site. The greater variability observed for the different drivers' mean speeds at the same site, the greater the σ^2_v. If all drivers traveled at the same speeds at the same site, the between-subject variance (σ^2_v) will be zero. The within-subject variance σ^2 measures the variability of speeds from the same drivers. The greater speed variability observed from different trips from the same driver, the greater the σ^2.

Random intercept mixed effects model is represented as a linear regression model with a random intercept. In this model, researchers are interested in estimating the coefficient of fixed effects (β_i) and testing hypothesis about the variance of random effects (σ^2_v).

The random intercept model implies a compound symmetry assumption for the variances and covariance of the longitudinal dataset. That is as follows:

$$\mathrm{Cov}(y_{ij}, y_{ij}) = V(y_{ij}) = \sigma^2 + \sigma^2_v \tag{7-6}$$

$\mathrm{Cov}(y_{ij}, y_{ik}) = \sigma^2_v$ if $j \neq k$

$\mathrm{Cov}(y_{ij}, y_{lk}) = 0$ if $i \neq l$ (observation from different drivers are independent)

The covariance between any two responses for the same driver is constant (σ^2_v). The covariance between any two responses for different drivers is zero. The reason why any two responses from the same driver are correlated is that the responses (speeds) observed from the same driver are expected to be similar.

For subject i, the covariate matrix is an (n_i x n_i) symmetric matrix Σ_i.

$$\Sigma_i = \begin{pmatrix} \sigma^2 + \sigma_v^2 & \sigma_v^2 & \cdots & \sigma_v^2 \\ \sigma_v^2 & \sigma^2 + \sigma_v^2 & \cdots & \cdots \\ \vdots & \vdots & \ddots & \vdots \\ \sigma_v^2 & \sigma_v^2 & \cdots & \sigma^2 + \sigma_v^2 \end{pmatrix}$$

For all subjects, the covariate matrix is an (M x M) matrix Σ.

$$M = \sum_{i=1}^{N} n_i$$

$$\Sigma = \begin{pmatrix} \underset{(n_1 x n_1)}{\Sigma} & 0 & \cdots & 0 \\ 0 & \underset{(n_2 x n_2)}{\Sigma} & \cdots & \cdots \\ \vdots & \vdots & \ddots & \vdots \\ 0 & \cdots & \cdots & \underset{(n_i x n_i)}{\Sigma} \end{pmatrix}$$

The intra-class correlation (ICC) represents the proportion of unexplained variance that is attributed to the individual subject. If ICC is near zero, differences in the mean speeds among different drivers at the same site are not significant. On the other hand, if ICC is large, much of the total variance is caused by the differences among different drivers.

$$\text{ICC} = \frac{\sigma_v^2}{\sigma_v^2 + \sigma^2} \tag{7-7}$$

7.1.2 Model Estimation

Maximum likelihood (ML) estimation is used in linear mixed effects models for estimating parameters. In the ordinary least squares regression, the objective in fitting a model is to estimate the parameters that minimize the sum of squared errors of predictions. In maximum likelihood, the objective in fitting a model is to estimate the parameters that make the observed data (y_{ij}) most likely to have occurred, in other words, maximize the likelihood (L) of observing the sample values. Generally, it is easier to work with the log of the likelihood function (log-likelihood). The maximum value of L can be derived by finding the point at which log-likelihood has a slope of zero (Harrell, 2001).

Assuming normal regression model

$y_i \sim N(\beta_0 + \beta_1 x_i, \sigma^2)$

The probability density function in Equation 7-8 represents the likelihood (probability) of y_i given the mean $(\beta_0 + \beta_1 x_i)$ and variance (σ^2).

$$p(y_i|\beta_0, \beta_1, \sigma^2) = \frac{1}{\sqrt{2\pi\sigma^2}} \exp\left\{-\frac{1}{2\sigma^2}\left(y_i - (\beta_0 + \beta_1 x_i)\right)^2\right\} \tag{7-8}$$

The likelihood is equal to

$$L(\beta_0, \beta_1, \sigma^2) = \left(\frac{1}{\sqrt{2\pi\sigma^2}}\right)^n \exp\left\{-\frac{1}{2\sigma^2}\sum_{i=1}^{n}\left(y_i - (\beta_0 + \beta_1 x_i)\right)^2\right\} \tag{7-9}$$

The log-likelihood is equal to

$$\mathrm{LogL}(\beta_0, \beta_1, \sigma^2) = -\frac{n}{2}\log 2\pi - \frac{n}{2}\log \sigma^2 - \frac{1}{2\sigma^2}\sum_{i=1}^{n}\left(y_i - (\beta_0 + \beta_1 x_i)\right)^2 \tag{7-10}$$

$$= -\frac{n}{2}\log 2\pi - \frac{n}{2}\log \sigma^2 - \frac{1}{2\sigma^2} SSE$$

In this case, minimizing SSE is equivalent to maximizing log-likelihood.

$$\hat{\beta}_0 = \overline{Y} - \hat{\beta}_1 (x) \tag{7-11}$$

$$\hat{\beta}_1 = \frac{\sum_{i=1}^{n} Y_i (x_i - \bar{x})}{\sum_{i=1}^{n} \left(x_i - \bar{x}\right)^2} \tag{7-12}$$

$$\hat{\sigma}^2 = \frac{1}{n} \sum_{i=1}^{n} \left(Y_i - \left(\hat{\beta}_0 + \hat{\beta}_1 x_i \right) \right)^2 \tag{7-13}$$

The MLE for β_0 and β_1 are the same as the least squares estimators. However the MLE for σ^2 is not. The least squares estimate of σ^2 is unbiased while the MLE of σ^2 is biased.

A model with large log-likelihood is preferred than one with small log-likelihood. However, a model with more parameters normally has a larger log-likelihood than a model with fewer parameters. Therefore, The Akaike Information Criterion (AIC) (Sakamoto et al., 1986) and Bayesian Information Criterion (BIC) (Schwarz, 1978) are used to compare models with the correction of the number of parameters. Normally, smaller AIC and BIC values indicate better models.

AIC = - 2 log-likelihood + 2n (7-14)

BIC = - 2 log-likelihood + n log(N) (7-15)

Where

n=number of parameters, and

N=number of observations.

Restricted maximum likelihood estimation (REML) has the same merits as ML but has the advantage of taking into account the loss of degrees of freedom involved in estimating the fixed effects (Verbeke et al., 1997). For example, the REML estimator of the error variance in the simple balanced one-way ANOVA model is SSE/(n – p), where SSE is the within-group sum of squares, n is the sample size and p is the number of the fixed effect parameters. In contrast, the ML estimator is SSE/n.

7.2 Operating Speed Models for Tangents

7.2.1 Dependent and Independent Variables

In this study, the predicting dependent variable is driver selected speeds on selected corridors, which is defined in section 6.2. Independent variables in the model include roadside objects, access density, cross-section features, grade and land use. Table 7-1 presents the independent variables.

Table 7-1 Description of Independent Variables

Variables	***Description***
roadside.d	trees and utility poles density (number of trees and utility poles per mile/average offsets (ft))
driveway	driveway density (number of driveways per mile)
intersection	t-intersection density (number of intersection per mile)
grade	average grade (degree)
lane width	lane width (ft)
lane.num	number of lanes
sidewalk.indicator	0: no sidewalk 1: sidewalk
parking.indicator	1: on-street parking 0: no on-street parking
median.indicator	0: no median 1: median (raised or TWLT)
curb.indicator	0: no raised curb 1: raised curb
land.use	0: commercial 1: residential 2: others
speed.limit	posted speed limit

Roadside objects include trees and utility poles. Generally, as tree density increases, drivers tend to decrease their speed. With the same tree density, drivers tend to decrease their speed with the decrease of the offsets (defined as the distance from the edge of travel lanes to roadside objects). Those two roadside objects tend to affect drivers' speed collectively. Thus, this study defines a new variable to consider their effects together, which is the roadside object density divided by their offsets. Generally, drivers should decrease their speed with the increase of this variable, which might be a result of the increase of roadside objects densities or decrease of their offsets.

Access density is defined as the number of driveways or T-intersections per mile. Only T-intersections are considered because they do not have traffic control devices, such as stop signs or traffic signals, on the study corridors.

Cross-section features include number of lanes, lane width, sidewalk, on-street parking, median, and curb. Sidewalk, on-street parking, median, and curb are represented as a dummy variable in the model development. Land use includes commercial, residential, industrial, park, forest, and schools. Since most of the selected corridors are either commercial or residential, this study divided the land use into three categories, commercial, residential, and others.

7.2.2 Model Development

Based on the methodology summarized in this dissertation, the author developed random intercept mixed effects models for each independent variable to test its significance at the 95% significance level. Table 7-2 lists the coefficients and p-values for each independent variable. Most variables were determined to be significant. The results in Table 7-2 indicate the following initial findings for urban tangent roads:

Table 7-2 Coefficients and P-values for Individual Variables of Tangent

Variable	**Coefficient**	**P-value**
speed.limit	1.45918	<.0001
roadside.d	-0.13978	<.0001
driveway	-0.18037	<.0001
intersection	-1.34817	<.0001
lane.width	-1.99387	0.0096
lane.num	8.37683	<.0001
sidewalk.indicator	-5.06362	<.0001
parking.indicator	-7.0677	<.0001
curb.indicator	1.45941	0.0113
median.indicator	5.65844	<.0001
land.use1	-0.87383	0.1151
land.use2	1.15901	0.0277

- Posted speed limits are highly correlated with drivers' speeds. Drivers tend to drive at higher speeds on roads with higher speed limits.
- Drivers tend to select lower operating speeds with the increase of the roadside object densities or the decrease of the roadside object offsets since driver may perceive that there is less room for maneuvering.
- Drivers tend to travel at lower speeds with the increase of access densities.
- The number of lanes has the most significant influence on drivers' speeds. Drivers travel at higher speeds when there are two lanes for one direction of travel rather than one lane. Drivers normally should drive at higher speeds for roads with wider lane width; however, in the selected corridors, lane widths on two-lane movement tended to be narrower than those for one-lane movements.
- The existence of sidewalk or on-street parking are associated with lower operating speeds, which is reasonable since sidewalks indicate the activities of pedestrians and on-street parking indicates potential hazards, such as possible car door opening and presence of pedestrians. On-street parking also makes a street looks narrower.
- Drivers tend to select lower speeds on roads with painted or raised medians. One possible reason for this observation is that the painted or raised median decreases the risk of conflicting with the traffic in the opposite direction.
- Grade does not affect drivers' speeds in this study since all selected roads have grades less than 4 percent, and the influence of grade is small when drivers are driving at low speed.

7.2.2.1 Final Tangent Model without Speed Limits

Although speed limit is highly correlated with speeds as shown in Table 7-2, this dissertation did not include speed limit as an independent variable due to the strong correlation of speed limit to design speed and, thereby to geometric design of the road. Generally, for an existing road, the posted speed limit is determined by the observed operating speeds. In contrast, for a new road, designers normally select design speed based on proposed functional classification and speed limit, such as 16 km/h (10 mph) above the speed limit.

This dissertation used forward stepwise regression method to select independent variables. Forward stepwise regression starts with no independent variable. At each step it adds the most statistically significant variable into the model until there are none left. In this dissertation, all selected variables are significant at the 95% level.

This dissertation tested different interactions between independent variables in the model development, but did not found significant improvement to the model. Therefore, the final model does not include any interaction between variables. Table 7-3 shows the final tangent model with the 85th percentile speed as the dependent variable.

Table 7-3 Final Tangent Model for 85th Percentile Cruising Speed

Variable	**Coefficient**	**P-value**
(Intercept)	50.50439	<.0001
lane.num	10.38624	<.0001
roadside.d	-0.07901	<.0001
driveway	-0.12987	0.0004
intersection	-0.21104	0.0516
curb.indicator	4.81645	<.0001
sidewalk.indicator	-6.82383	<.0001
parking.indicator	-5.10394	<.0001
land.use1	5.29963	<.0001
land.use2	5.23704	<.0001
AIC	10817.26	
BIC	10881.41	
logLik	-5396.632	
σ_v	5.231917	
σ	7.146813	
ICC	0.349	

The ICC of 0.349 indicates that 34.9 percent of the unexplained variance of speeds is caused by the characteristics of different drivers or vehicles.

The coefficient of determination of the model is calculated as follows:

$$SST = \sum_{i=1}^{n}(y_i - \bar{y}) = 213148.5$$

$$SSE = \sigma^2 \times df = 7.147^2 \times 1368 = 69873.25$$

Where

SST is total sum of squares,
SSE is error sum of squares,
σ is residual, and
df is the degree of freedom for residuals.

$R^2 = (SST – SSE)/SST = 0.67$

Table 7-4 presents the 95% confidence interval of the independent variables in the full model. For example, 95% of drivers are expected to have intercepts between 47.82 and 53.19 km/h (29.72 and 33.06 mph).

Table 7-4 Confidence Interval for Tangent Model

	Lower	**Estimate**	**Upper**
(Intercept)	47.8187	50.50439	53.19009
lane.num	9.256662	10.38624	11.51582
roadside.d	-0.09668	-0.07901	-0.06134
driveway	-0.19006	-0.12987	-0.06968
intersection	-0.42357	-0.21104	0.001483
shoulder.dummy	3.087721	4.816449	6.545176
sidewalk.indicator	-7.85877	-6.82383	-5.78889
parking.indicator	-7.27546	-5.10394	-2.93243
land.use1	3.736502	5.299626	6.86275
land.use2	3.93698	5.237037	6.537095

Table 7-5 shows the final tangent model with the 95th percentile speed as the dependent variable. The results are very similar to the 85th percentile speed model.

Table 7-5 Final Tangent Model for 95th Percentile Cruising Speeds

Variable	Coefficient	P-value
(Intercept)	49.82847	<.0001
lane.num	10.67294	<.0001
roadside.d	-0.07473	<.0001
driveway	-0.12214	0.0001
intersection	-0.1984	0.0684
curb.indicator	5.3188	<.0001
sidewalk.indicator	-7.07768	<.0001
parking.indicator	-4.58302	<.0001
land.use1	5.61051	<.0001
land.use2	5.40593	<.0001
AIC	10830.15	
BIC	10894.3	
logLik	-5403.077	
σ_v	5.292291	
σ	7.17231	
ICC	0.353	

$$SST = \sum_{i=1}^{n}(y_i - \bar{y}) = 215139$$

$$SSE = \sigma^2 \times df = 7.172^2 \times 1368 = 27489.34$$

$$R^2 = (SST - SSE)/SST = 0.67$$

7.2.2.2 Correlation of Posted Speed Limit and Other Independent Variables

In order to investigate the correlation between speed limits and driver selected speed, this dissertation also performed an analysis with speed limits as an independent variable to predict the 85th percentile cruising speed. The results are presented in Table 7-6. The correlations of posted speed limit with other independent variables are presented in Table 7-7.

With speed limit in the model, several variables that are previously significant now become not significant, including the intercept, roadside objects, driveway and T-intersection density, on-street parking. The intercept, number of lanes, roadside objects, and land use are correlated with posted speed limit. Especially, the intercept is the most significantly correlated to speed limits, which is expected. The intercept in the model represents a general population average speed in urban streets. It is not surprising that average speed on a road is highly correlated with speed limits. Drivers tend to drive faster on roads with higher speed limits.

Table 7-6 Final Tangent Model with Posted Speed Limits

Variable	**Coefficient**	**P-value**
Intercept	1.333251	0.7389
speed.limit	1.1198	<.0001
lane.num	1.87004	0.0256
roadside.d	-0.01188	0.2331
Driveway	-0.04077	0.1733
Intersection	-0.03748	0.718
curb.indicator	2.776627	0.0012
sidewalk.indicator	-5.42326	<.0001
parking.indicator	-1.35998	0.214
land.use1	0.086589	0.9183
land.use2	1.905757	0.0049
AIC	10661.5	
BIC	10730.98	
logLik	-5317.749	
σ_v	4.361366	
σ	6.856923	
ICC	0.288	

Table 7-7 Correlation of Speed Limits and Other Independent Variables

	speed.limit
Intercept	-0.947
lane.num	-0.762
roadside.d	0.517
driveway	0.221
intersection	0.145
curb.indicator	-0.173
sidewalk.indicator	0.213
parking.indicator	0.265
land.use1	-0.449
land.use2	-0.364

7.2.2.3 Final Model for Tangents

The resulting final model for tangents is as follows:

$$
\begin{aligned}
V85 = {} & 50.503 + (10.386 \times lane.num) - (0.079 \times roadside.d) - (0.129 \times driveway) - \\
& (0.211 \times intersection) + (4.816 \times curb.indicator) - (6.824 \times sidewalk.indicator) - \\
& (5.104 \times parking.indicator) + (5.299 \times land.use1) + (5.237 \times land.use2)
\end{aligned}
$$

$$
\begin{aligned}
V95 = {} & 49.828 + (10.673 \times lane.num) - (0.075 \times roadside.d) - (0.122 \times driveway) - \\
& (0.198 \times intersection) + (5.319 \times curb.indicator) - (7.078 \times sidewalk.indicator) - \\
& (4.583 \times parking.indicator) + (5.611 \times land.use1) + (5.406 \times land.use2)
\end{aligned}
$$

Where

- V85 = driver selected speeds represented by 85^{th} percentile cruising speeds (km/h),
- V95 = driver selected speeds represented by 95^{th} percentile cruising speeds (km/h),
- roadside.d = density of roadside objects (utility poles and trees) divided by their average

offsets from roadside (number of objects per km/offset (m))

- driveway = density of driveways (number of driveways per km)
- intersection = density of T-intersections (number of T-intersection per km)
- lane.num = number of lanes
- curb.indicator is as follows:

if there is no curb

curb.indicator = 0

otherwise

curb.indicator =1

- sidewalk.indicator is as follows:

if there is no sidewalk

sidewalk.indicator = 0

otherwise

sidewalk.indicator = 1

- parking.indicator is as follows:

if there is no on-street parking

parking.indicator = 0

otherwise

parking.indicator = 1

- land.use is as follows:

if land use is commercial

land.use1 = 0

land.use2 = 0

if land use is residential

land.use1 = 1

land.use2 = 0

else

land.use2 = 1

land.use1 = 0

This model indicates that the number of lanes has the most significant influence on drivers' speeds. Drivers travel at higher speeds when two-lanes are available for a direction of travel than when only one lane is available since two-lanes for a direction of travel provide more room for maneuvering.

Sidewalk and on-street parking and are the second and third significant variables that reduce drivers' speeds on urban tangent streets. Sidewalk indicates pedestrian activities, which makes drivers more cautious when driving. On-street parking indicates not only pedestrian activities but also potential hazards. On-street parking may make a driver feel that the lane width is narrower than its real width and that may contribute to this observed speed reduction.

The operating speed model also indicates that roadside object densities and offsets affect drivers' speed choice. Drivers tend to select lower operating speeds with the increase of roadside objects (trees and utility poles) densities or the decrease of the roadside objects offset, which is defined as the distance from the edge of curb face to the roadside object. It is important to note that the effect of roadside obstacles on speed reduction, however, may be offset by a greater exposure to hazardous roadside obstacles.

Higher driveway and T-intersection densities are also associated with lower operating speeds on low-speed urban streets. This may be due to the fact that higher access density creates a higher possibility of turning movements conflicting with through traffic.

In this study, drivers tend to drive faster on urban roads with raised curb than those without. The possible reason is that curbs may provide a barrier between the travel lane and roadside objects, like trees and utility poles. The tangent model also indicates that land use is a significant variable that affects drivers' speeds and drivers tend to drive faster on low volume residential streets than on the high volume commercial streets. Since the speed data used in model development are all from off-peak time, the traffic volume on residential streets are normally much lower than commercial streets during that period.

7.2.2.4 Comparison of Linear Mixed Effects Model and Ordinary Linear Regression Model

This dissertation compared the results from linear mixed effects (LME) model and ordinary linear regression (OLR) model with the same dataset. The author developed an ordinary linear regression with the same independent variables. The model has similar coefficients as the linear mixed effects model, as shown in Table 7-8.

Table 7-8 Comparison of LME Model and OLR Model for Tangents

Models	**Linear mixed effects**		**Ordinary linear regression**	
Variable	Coefficient	P-value	Coefficient	P-value
(Intercept)	50.50439	<.0001	58.7974	<0.0001
lane.num	10.38624	<.0001	8.1826	<0.0001
roadside.d	-0.07901	<.0001	-0.0855	<0.0001
driveway	-0.12987	0.0004	-0.1492	<0.0001
intersection	-0.21104	0.0516	-0.4837	<0.0001
curb.indicator	4.81645	<.0001	4.5345	<0.0001
sidewalk.indicator	-6.82383	<.0001	-8.4514	<0.0001
parking.indicator	-5.10394	<.0001	-5.9469	<0.0001
land.use1	5.29963	<.0001	2.6209	0.0006
land.use2	5.23704	<.0001	3.9013	<0.0001
Between subject variance	σ_v^2	27.37296		
within subject variance	σ^2	51.07694	σ^2	70.34177
Variance of speeds	$\sigma^2 + \sigma^2_v$	78.44989	σ^2	70.34177
R^2	0.67		0.48	

In the linear mixed effects model, the between-subject variance is excluded from random errors. What the ordinary regression model determined to be the error variance (70.34), the mixed model separates into within-subject variance (51.08) and between-subject variance (27.37). This is the reason why the linear mixed effects model has a higher coefficient of determination (R^2) than the ordinary linear regression model. If all drivers travel at the same speed at the same site, the between subject variance (σ_v^2) would be equal to zero.

7.2.3 Model Assumption Diagnostic

This dissertation verified the following two assumptions of linear mixed effects model:

- The within-group errors are normally distributed, and
- The random effects are normally distributed.

The normal plot of the within-group residuals (Figure 7-2) and normal plot of random effects (Figure 7-3) indicate that the within-group residuals and the random effects are normally distributed. The linear mixed effects assumptions are therefore appropriate for this modeling approach.

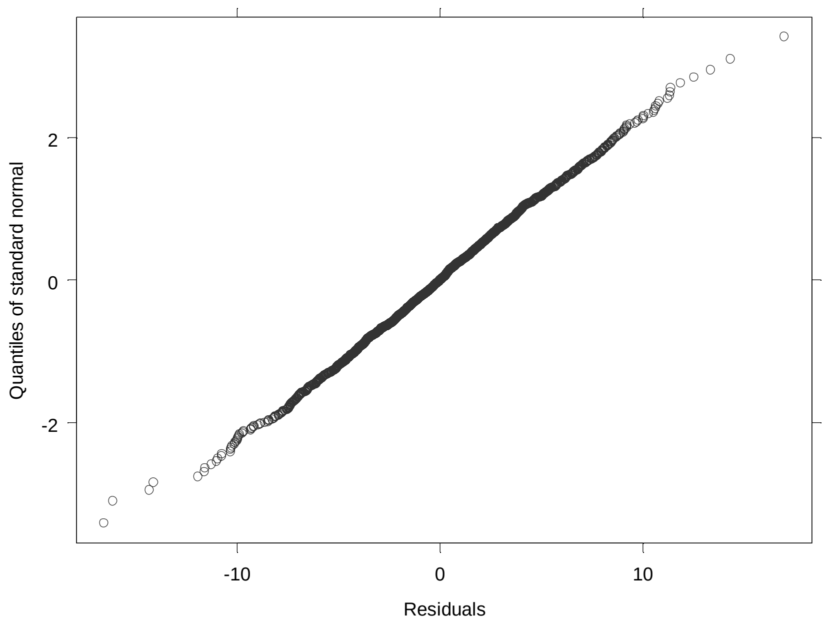

Figure 7-2 Normal Plot of Residuals of Tangent Model

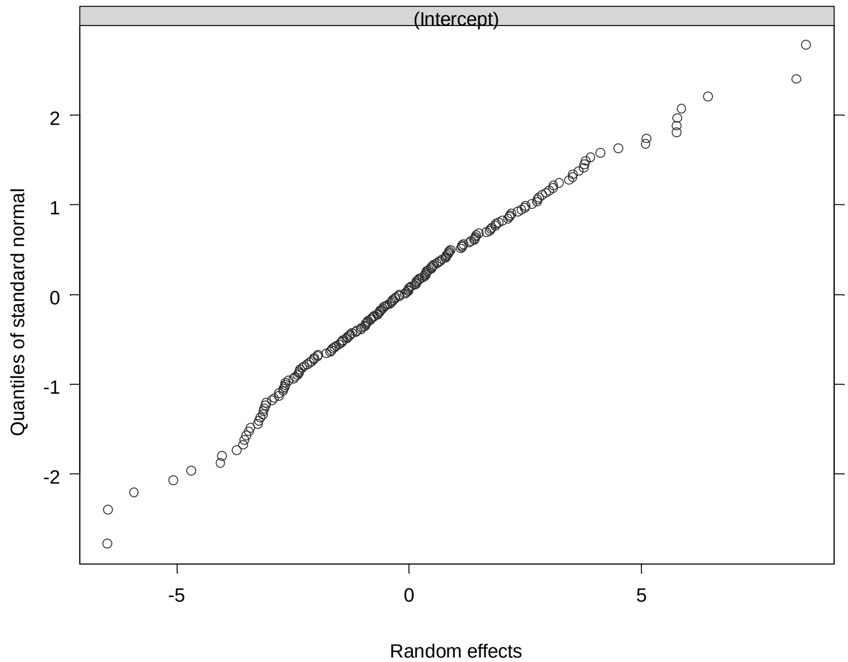

Figure 7-3 Normal Plot of Estimated Random Effects of Tangent Model

7.3 Operating Speed Models for Horizontal Curves

7.3.1 Model Development

This dissertation first developed random intercept mixed effects models for each independent variable to test its significance at the 95% significance level. Table 7-9 shows the coefficient and p-value for each significant independent variable. The results in Table 7-9 indicate the following initial findings for low-speed urban horizontal curves.

- Drivers tend to select lower speeds with the increase of roadside object densities or the decrease of roadside object offsets since driver may perceive that there is less room for maneuvering.

- Drives tend to driver at lower speeds with the increase of access densities, including driveways and T-intersections.
- Number of lanes has the most significant influence on drivers' speeds. Drivers travel at higher speeds when there are two-lanes for a direction of travel than one lane.
- Curve direction has a significant influence on drivers' speeds. Drivers tend to drive faster on horizontal curves to left than to right. This result is expected since the sight distance of horizontal curves to left is generally better than it on horizontal curves to right.
- The existence of sidewalk or on-street parking is associated with lower operating speeds.
- Drivers tend to select higher speeds on roads with painted or raised median with standard lane width.
- Grade does not affect drivers' speeds in this study.

Table 7-9 Coefficients and P-values for Individual Variables of Horizontal Curves

Variable	**Coefficient**	**P-value**
speed.limit	1.032434	<0.0001
roadside.d	-0.03878	<0.0001
driveway	-0.24148	<0.0001
intersection	-0.97057	<0.0001
lane.width	-1.93021	<0.0001
lane.num	4.36227	<0.0001
sidewalk.indicator	-2.41314	<0.0001
parking.indicator	-4.86406	<0.0001
radius	-0.00942	<0.0001
curve.direction	-1.01381	<0.0001
median.indicator	3.88605	<0.0001
curve.length	-0.01112	<0.0001
curb.indicator	-2.19211	<0.0001
land.use1	-1.96607	<0.0001
land.use2	-1.63161	<0.0001

7.3.1.1 Final Horizontal Curve Model without Speed Limit

This dissertation used forward stepwise regression method to select model variables. Forward stepwise regression starts with no model variable. At each step it adds the most statistically significant variable into the model until there are none left. In this dissertation, all selected variables are significant at the 95% level. Posted speed limits are not included in the model as previously discussed. The land use variable was found to be significantly correlated with median presence so that the land use variable was removed from the model.

The dissertation considered different interactions between independent variables in the model development, but did not find significant improvement to the model. Table 7-10 shows the independent variables in the final horizontal curve model.

Table 7-10 Final Horizontal Curve Model for 85th Percentile Cruising Speed

Variable	**Coefficient**	**P-value**
(Intercept)	57.5578	< 0.0001
lane.num	4.89939	< 0.0001
lane.width	1.19323	0.0023
driveway	-0.05969	0.0107
median.indicator	2.5572	0.0002
direction	-1.30803	< 0.0001
roadside.d	-0.07368	< 0.0001
parking.indicator	-7.80539	< 0.0001
sidewalk.indicator	-3.18695	< 0.0001
AIC	15093.37	
BIC	15156.54	
logLik	-7535.684	
σ_v	3.908052	
σ	5.954056	
ICC	0.301	

The coefficient of determination is calculated as follows:

$$\text{SST} = \sum_{i=1}^{n}(y_i - \overline{y}) = 195293.5$$

$$\text{SSE} = \sigma^2 \times \text{df} = 5.954056^2 \times 2120 = 29357.68$$

Where

SST is total sum of squares,

SSe is error sum of squares,

σ is residual, and

df is the degree of freedom for residuals.

$$R^2 = (\text{SST} - \text{SSE})/\text{SST} = 0.63$$

Table 7-11 presents the 95% confidence interval of the independent variables in the full model. For example, 95% of drivers are expected to have an intercept in the interval between 54.33 and 60.77 km/h (33.95 and 37.98 mph).

Table 7-11 Confidence Intervals for Horizontal Curve Model

	Lower	**Estimate**	**Upper**
(Intercept)	54.33646	57.5578	60.77915
lane.num	4.131795	4.89939	5.666986
lane.width	0.426316	1.193232	1.960149
driveway	-0.10553	-0.05969	-0.01386
median.indicator	1.228683	2.5572	3.885716
direction	-1.85332	-1.30803	-0.76274
roadside.d	-0.0847	-0.07368	-0.06265
parking.indicator	-8.96033	-7.80539	-6.65044
sidewalk.indicator	-3.92742	-3.18695	-2.44647

Table 7-12 shows the final tangent model with the 95th percentile speed as the dependent variable. The results are very similar to the 85th percentile speed model.

Table 7-12 Final Horizontal Curve Model for 95th Percentile Cruising Speed

Variable	**Coefficient**	**P-value**
(Intercept)	58.097	<.0001
lane.num	4.47662	<.0001
lane.width	1.35872	0.0006
driveway	-0.08288	0.0005
median.dummy	2.50013	0.0003
direction	-1.39652	<.0001
roadside.d	-0.07424	<.0001
parking.indicator	-8.0581	<.0001
sidewalk.indicator	-3.05358	<.0001
AIC	15140.41	
BIC	15203.58	
logLik	-7559.203	
σ_v	3.94057	
σ	6.015732	
ICC	0.300	

The coefficient of determination is calculated as follows:

$$SST = \sum_{i=1}^{n} (y_i - \overline{y}) = 198353.2$$

$$SSE = \sigma^2 \times df = 6.015732^2 \times 2120 = 76720.75$$

Where

SST is total sum of squares,

SSe is error sum of squares,

σ is residual, and

df is the degree of freedom for residuals.

$$R^2 = (SST - SSE)/SST = 0.62$$

7.3.1.2 Correlation of Posted Speed Limits and Other Independent Variables

This dissertation conducted a similar analysis with speed limits for the horizontal curve model. The results are presented in Table 7-13, and the correlations of posted speed limits with other independent variables are presented in Table 7-14.

With speed limit in the model, number of lanes and lane width variables are no longer significant. The intercept and number of lanes variables are highly correlated with posted speed limits. As expected, the intercept is the most significantly correlated to speed limits. The intercept in the model represents a general population average speed at horizontal curves on urban streets. It is not surprising that average speed on a road is highly correlated with speed limits. Drivers tend to drive faster on roads with higher speed limits.

Table 7-13 Final Horizontal Curve Model with Posted Speed Limits

Variable	**Coefficient**	**P-value**
(Intercept)	27.07202	<.0001
speed.limit	0.7541	<.0001
lane.num	-0.25601	0.6485
lane.width	-0.69903	0.087
driveway	-0.09473	<.0001
median.indicator	3.37401	<.0001
direction	-1.24167	<.0001
roadside.d	-0.03769	<.0001
parking.indicator	-4.19358	<.0001
sidewalk.indicator	-1.73232	<.0001
AIC	14954.36	
BIC	15023.26	
logLik	-7465.178	
σ_v	3.411492	
σ	5.802375	
ICC	0.257	

Table 7-14 Correlation of Posted Speed Limit and Other Independent Variables

	speed.limit
intercept	-0.844
lane.num	-0.741
lane.width	-0.378
driveway	-0.121
median.indicator	0.087
direction	0.024
roadside.d	0.481
parking.indicator	0.462
sidewalk.indicator	0.314

7.3.1.3 Final Model for Horizontal Curves

The resulting final model for horizontal curves is as follows:

$$V85 = 57.558 + 4.899\times lane.num + 1.193\times lane.width - 0.059\times driveway + 2.557\times median.indicator - 1.308\times direction - 0.074\times roadside.d - 7.805\times parking.indicator - 3.187\times sidewalk.indicator \quad (7\text{-}18)$$

$$V95 = 58.097 + 4.477\times lane.num + 1.359\times lane.width - 0.083\times driveway + 2.5\times median.indicator - 1.396\times direction - 0.074\times roadside.d - 8.058\times parking.indicator - 3.054\times sidewalk.indicator \quad (7\text{-}19)$$

Where

- V85 = driver selected speeds represented by 85th percentile cruising speeds (km/h),
- V95 = driver selected speeds represented by 95th percentile cruising speeds (km/h),
- roadside.d = density of utility poles (per km)/offsets (m)

- lane.num = number of lanes
- lane.width = average lane width (m)
- driveway = density of driveways (number of driveways per km)
- direction is as follows:

  if curve direction is left

    direction = 0

  else

    direction = 1

- median.indicator is as follows:

  if there is raised median or TWLT

    median.indicator = 1

  else

    median.indicator = 0

- parking.indicator is as follows:

  if there is on-street parking

    parking.indicator = 1

  else

    parking.indicator = 0

- sidewalk.indicator is as follow:

  if there is a sidewalk

    sidewalk.indicator = 1

  else

    sidewalk.indicator = 0

This model indicates that the number of lanes has the most significant influence on drivers' speeds in low speed urban horizontal curves. This finding is consistent with the number of lanes variable for the tangent model. Drivers travel at higher speeds when two lanes are available for a direction of travel than when only one lane is available. The lane width, which is not significant in the tangent model, has been found to be a significant variable for the curve model.

Similar to the tangent model, sidewalk and on-street parking are the second and third most significant variables that are associated with lower speeds on urban horizontal curves. On-street parking may make a driver feel that the lane width is narrower and this may contribute to the observed lower speeds.

The median presence encourages higher speeds on urban horizontal curves with standard lane width, which is reasonable since it isolates the conflicting traffic from the other direction. Drivers feel more comfortable to drive at higher speeds due to the reduced risks.

The operating speed model also indicates that roadside objects and access density influence drivers' speed choice. Drivers tend to select lower operating speeds with the increase of roadside objects (trees and utility poles) densities or the decrease of the roadside objects offsets. It is important to note that this effect of roadside obstacles on speed reduction, however, may be offset by a greater exposure to hazardous roadside obstacles. The model also indicates that drivers tend to decrease their speeds with the increase of driveway density.

Horizontal curve radius is not significant in this study, which is different from most of previous studies. This may be explained by the fact that there are much more factors influencing drivers speed choice on low speed urban streets compared to high speed rural highways. As a result, curve radius may be not as significant as it is in rural environments. Another possible reason is that drivers normally are driving at lower operating speeds on urban streets compared to rural highways. The geometric constraints of horizontal curves on drivers' speeds may not be significant. The third possible reason is that this study does not have enough sharp curves in the selected study corridors. In this study, curve direction is significant. Drivers tend to driver faster on horizontal curves to left than to right, which may be due to the factor that drivers' sight distance generally would be better on horizontal curves to the left than to the right.

7.3.1.4 Comparison of Linear Mixed Effects Model and Ordinary Linear Regression Model

This dissertation also compared the results from linear mixed effects (LME) model and ordinary linear regression (OLR) model with the same dataset. This study developed an ordinary linear regression with the same independent variables. The model has similar coefficients as the linear mixed effects model, as show in Table 7-15

Table 7-15 Comparison of LME Model and OLR Model for Horizontal Curves

Models	**Linear mixed effects**		**Ordinary linear regression**	
(Intercept)	57.5578	< 0.0001	59.2553	< 0.0001
lane.num	4.89939	< 0.0001	4.0749	< 0.0001
lane.width	1.19323	0.0023	1.6078	< 0.0001
driveway	-0.05969	0.0107	-0.1297	< 0.0001
median.indicator	2.5572	0.0002	3.4618	< 0.0001
direction	-1.30803	< 0.0001	-1.6635	< 0.0001
roadside.d	-0.07368	< 0.0001	-0.094	< 0.0001
parking.indicator	-7.80539	< 0.0001	-9.2134	< 0.0001
sidewalk.indicator	-3.18695	< 0.0001	-4.1343	< 0.0001
between subject variance	σ_v^2	15.27287		
within subject variance	σ^2	35.45078	σ^2	47.07332
variance of speeds	$\sigma^2 + \sigma^2_v$	50.72365	σ^2	47.07332
R^2	0.63		0.44	

In linear mixed effects model, the between-subject variance is excluded from random errors. What the ordinary regression model determined to be the error variance (47.07), the mixed model separates into within-subject variance (35.45) and between-subject variance (15.17).

7.3.2 Model Assumption Diagnostic

This dissertation verified the following two assumptions of linear mixed effects model:

- The within-group errors are normally distributed, and
- The random effects are normally distributed.

The normal plot of the within-group residuals (Figure 7-5) and normal plot of random effects (Figure 7-6) indicate that the within-group residuals and the random effects are normally distributed. The linear mixed effects assumptions are therefore appropriate for this modeling approach.

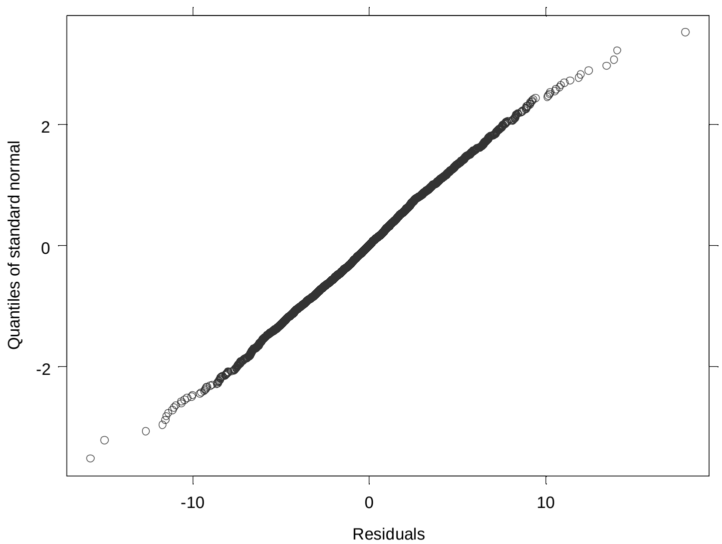

Figure 7-5 Normal Plot of Residuals of Horizontal Curve Model

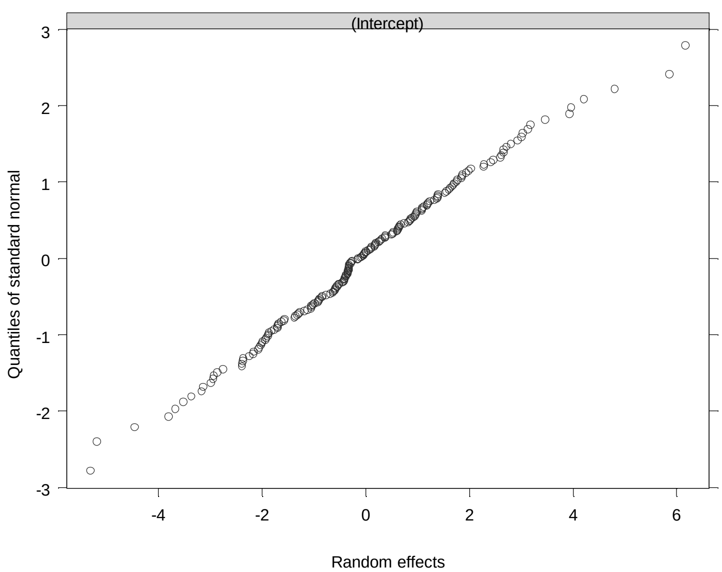

Figure 7-6 Normal Plot of Estimated Random Effects of Horizontal Curve Model

7.4 Application Example of the Operating Speed Model

This section gives an example that predicts a speed profile along a section of low-speed urban street by using the developed tangent model.

Assumes a tangent section with a length of 915 m (3000 ft) and bounded by two traffic signals have the following characteristics:

- number of lanes: 1
- lane width: 3.6 m (12 ft)
- number of trees: 10
- number of utility poles: 15
- average utility pole offset: 0.91 m (3 ft)
- number of driveways: 10
- number of T-intersections: 5
- curb type: curb and gutter
- sidewalk: yes
- on-street parking: no
- land use: residential

length = 0.915 km

lane.num = 1

roadside.d = (10 + 15)/0.915/0.91 = 30.02

driveway = 10/0.915 = 10.93

intersection = 5/0.915 = 5.46

curb.indicator = 1

sidewalk.indicator = 1

parking.indicator = 0

land.use1 = 1

land.use2 = 0

$$V85 = 50.503 + (10.386\times1) - (0.079\times30.02) - (0.129\times10.93) - (0.211\times5.46) + (4.816\times1) - (6.824\times0) - (5.104\times1) + (5.299\times1) + (5.237\times0) = 60.9 \text{ km/h}$$

$$V95 = 49.828 + (10.673\times1) - (0.075\times30.02) - (0.122\times10.93) - (0.198\times5.46) + (5.319\times1) - (7.078\times0) - (4.583\times1) + (5.611\times 1) + (5.406\times0) = 62.2 \text{ km/h}$$

Therefore, the estimated driver selected speed is from 60.9 to 62.2 km/h (38.1 to 38.9 mph). With the final speed of 60.9 km/h (38.1 mph), Table 4-1 gives an average acceleration distance of 124 m (407 ft), and Table 4-3 gives an average deceleration distance of 109 m (358 ft). The following Figure 7-13 presents the typical speed profile on this corridor.

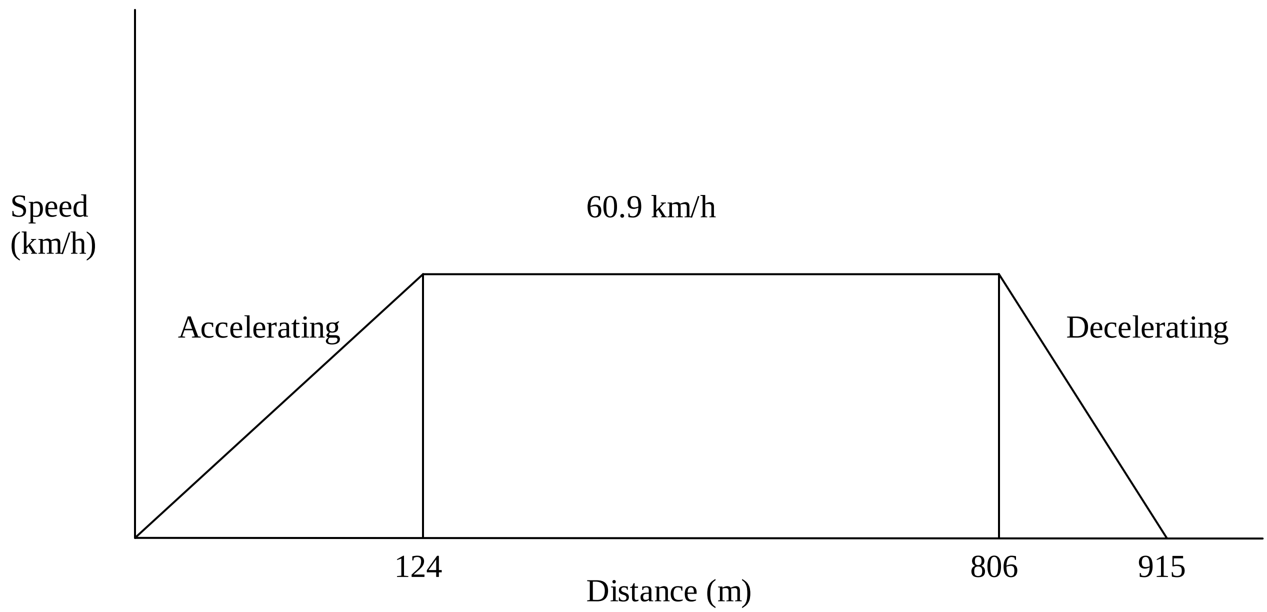

Figure 7-13 Estimated Speed Profile on the Low-Speed Urban Tangent Street

CHAPTER 8

CONCLUSIONS AND CONTRIBUTIONS

8.1 Contributions and Findings

This dissertation is the first large scale comprehensive speed study on low speed urban streets with the in-vehicle GPS technology. The author developed a methodology to study operating speed on urban streets with the GPS based vehicle activity data, including summarizing GPS trips, selecting study sites, filtering trips and speed data, and analyzing speed profiles.

This dissertation developed preliminary operating speed models to estimate drivers' selected speeds at tangent sections and horizontal curves on urban streets with speed limits ranging from 48 to 64 km/h (30 to 40 mph). These models include design features such as roadside objects, access densities, cross-section features, alignment characteristics, and adjacent land use. The results can help roadway designers and planners to better understand expected operating speeds and, as a result, design and evaluate proposed urban roadways accordingly.

For the tangent models, the following variables were found to be significant at the 95 percentile significance level: number of lanes, roadside object densities and offsets, the densities of T-intersections and driveways, raised curb presence, sidewalk presence, on-street parking, and land uses. For the horizontal curve models, the significant variables include number of lanes, lane width, roadside object densities and offsets, sidewalk presence, raised median or TWLT presence, and on-street parking. The following are the major findings from the developed models:

- The number of lanes per direction of travel has the most significant influence on drivers' speeds at both tangents locations and horizontal curves. Drivers normally travel at higher speeds on two-lane movements than for one-lane movements since two-lane for a direction of travel provide more room for maneuvering.

- On-street parking and sidewalk presence are the second and third most significant variables that affect drivers' speeds on urban streets, including tangents and horizontal curves. Drivers tend to select lower operating speeds on roads characterized by on-street parking and sidewalk presence. This may be due to increased pedestrian activity, parking maneuvers, or the perception of narrower travel lanes.

- Roadside objects, including trees and utility poles, influence drivers' speeds on urban streets since they indicate potential hazards to drivers and also reduce sight distance on horizontal curves. Drivers tend to select lower speeds with the increase of trees or utility pole densities, or with the decrease of their offsets, which is defined as the distance from the edge of curb face to roadside objects.

- Access density, including driveways and T-intersections also affect drivers' speed choices on urban streets. Drivers tend to select lower operating speed with the increase of driveways or T-intersection density on tangent sections. This may be due to the fact that higher access density creates a higher possibility of turning movements. However, the T-intersection density variable is only significant in the tangent speed model. For horizontal curve models, only driveway density is significant. The possible reason is that there are very few T-intersections on horizontal curves in the selected study corridors.

- The median presence encourages higher speeds at horizontal curves with standard lane width, which is reasonable since it isolates the opposing traffic from the other direction. Drivers feel more comfortable to drive at higher speed due to reduced risks.

- Curve direction is found to be significant. Drivers tend to select higher operating speeds on horizontal curves to the left than to the right, which may be due to the factor that drivers' sight distance generally are better on horizontal curves to left than curves to right.

Previous studies have argued whether posted speed limit should be included as an independent variable in operating speed model, and their findings have been inconclusive. The results in this dissertation suggest that posted speed limit should not be included in the model development due to the strong correlation of speed limit to design speed and, thereby to geometric design of the road. Generally, for an existing road, the posted speed limit is determined by the observed operating speeds. In contrary, for a new road, designers normally select the design speed based on the proposed functional classification and speed limit, such as 16 km/h (10 mph) above the speed limit. In this study, the following observations led the author to the conclusion that speed limit is not an independent variable and should, therefore, not be included.

- The intercept and number of lanes are highly correlated with posted speed limits. The intercept is the most significantly correlated to speed limits, which is expected. The intercept in the model represents a general population average speed on urban streets. It is not surprising that average speed on a road is highly correlated with speed limit. Drivers tend to drive faster on roads with higher speed limits.

- With speed limit in the horizontal curve model, number of lanes and lane width become statistically insignificant. Similarly, with speed limit in the tangent model, several variables that are previous significant become statistically insignificant, including the intercept, roadside objects, driveway and T-intersection densities, on-street parking. All of these variables are characteristics of the road functional classification and this classification is linked to the speed limit.

This dissertation is the first comprehensive attempt to develop operating speed models based on continuous speed profile on low-speed urban streets. Previous studies have developed numerous operating speed models. However, most of these models were based on spot speed and the researchers collected speed data at specific locations of a roadway, mostly at the middle point of tangents and horizontal curves. Most previous operating speed models were based on the assumptions that drivers reach their highest speeds at the middle point of tangents and reach their lowest speeds at the middle point of horizontal curves.

With the second-by-second speed profile in this study, this dissertation found that this point assumption is not realistic for operating speed model development, especially on urban streets. Drivers reach their maximum speeds at different locations along the tangents. Even the same driver reaches his or her maximum speeds at different locations along the same tangent for different trips. For horizontal curves after long tangents, drivers decelerate when they approach the horizontal curves. This study found that drivers reached their minimum speeds at different locations, not just at the middle point of the horizontal curve. In this study, the second-by-second speed profiles make it possible to measure the true operating speeds along the selected corridors even when extreme values take place at different locations along the same corridor

The most significant characteristic of urban streets is the presence of closely spaced intersections with traffic control devices. Drivers have to make frequent stops. Understanding vehicle acceleration and deceleration characteristics is a very important part of analyzing speed profiles on urban streets. Most previous acceleration/deceleration studies were based on outdated data rather than recent observations. Hence, the conclusions may not be reflective of today's drivers. Furthermore, due to the limitations of data collection methods, most previous studies could not provide accurate estimations of drivers' acceleration and deceleration behaviors, such as acceleration or deceleration time and distance. This is because different drivers may start to accelerate or decelerate at different time and location. With the second-by-second speed profile data from in-vehicle GPS equipments, this dissertation provided more accurate information about drivers' acceleration and deceleration behavior, such as acceleration time and distance, deceleration time and distance, average acceleration and deceleration rates, and their relationships with final speeds or approach speeds, and posted speed limits. These results provide guidance in the determination of minimum study corridor length.

This dissertation is the first comprehensive attempt to develop operating speed models with the consideration of driver and vehicle effects. Most previous studies could not collect drivers' information because of the limitation of the data collection methods since it is difficult to obtain drivers' information in field observations. In this study, each speed record has its associated driver/vehicle information and each study driver had multiple trips on the same site. These data enable the researchers to include driver and vehicle effects in the operating speed modeling. Although the driver/vehicle' characteristics are not included in the operating speed models as predictors, they are modeled as random effects in the linear mixed effects model used in this study while the road environment features are model as fixed effects. With traditional cross-sectional speed studies, all unexplained speed variation can be only considered as within-subject variation. Therefore, the researchers have no way to know if the variation across drivers is significant compared to within-subject variation. The mixed effects model used in this dissertation separates the unexplained speed variation into within-subject variation and between-subject variation, and calculate the proportion of speed variations that caused by individual driver.

This dissertation used mixed effects linear regression in the operating speed modeling, which is an extension of ordinary linear regression. The mixed effects model separates the total variance into within-subject variance and between-subject variance compared to the ordinary linear regression models. For urban tangents, this dissertation found that about 35 percent of the unexplained variance (by road features) was contributed to individual driver's effect. For urban horizontal curves, this dissertation found that about 30 percent of the unexplained variance (by road features) was contributed to individual driver's effect. The results indicate that the between-subject variance is about half of the within-subject variance.

GPS has been widely used in transportation research. This study demonstrates that GPS is an more effective data collection method for traffic operation studies compared to traditional data collection methods, such as radar guns and detectors. The GPS techniques provide second-by-second speed profile data, which enable the researchers to estimate acceleration and deceleration time and distance more accurately. Researchers can also more accurately estimate the drivers selected speeds on tangents and horizontal curves by second-by-second speed profiles without the assumptions that drivers reach their desired speeds at a specific location.

This dissertation also suggests GPS technology could be used in geometric measurements. The radius or degree of curvature of horizontal curve is an important geometric feature in roadway safety studies. It is always dangerous to measure the curve radius in field since most low-speed roadways do not have sidewalks. For high-speed highways, it may be not feasible to measure the curve radius in field. This dissertation implemented a method that estimates average curve radius based on GPS data using non-linear curve fitting method. The results indicate that this method is accurate and suitable for large scale geometric measurement practices.

8.2 Recommendation for Future Research

This dissertation is a preliminary study for an ongoing FHWA project, whose purpose is to investigate how urban roadway environments affect drivers selected speeds so that designers can use the knowledge to design safer roads. In this dissertation, the author has developed operating speed models for both tangents and horizontal curves with very promising results. But it is still recommended to select more study corridors for further analysis and modeling, especially for horizontal curves. This dissertation indicates that curve radius is not a significant variable, which is different from most previous studies. A possible reason is the limited sample size of horizontal curves. More horizontal curves with various radius and approach tangent length may be appropriate for future evaluation of the influence of curve radius on operating speeds on urban streets. The new dataset also could be used to validate the models developed in this dissertation.

The speed data used in this study have its associated driver and vehicle information, such as age, gender, and vehicle type. It is recommended to investigate the influence of these characteristics on drivers' speed. It is also desired to investigate how weather and light conditions affect drivers' speed.

APPENDIX A. Existing Operating Speed Models for Rural Conditions

Speed Prediction Model	**Location**	**R^2**
Lamm et al. (1990) $V85 = 93.85 – 1.82DC$	Two-lane rural highway curves, grades < 5%	 0.79
McLean (1979) $V85 = 53.8 + 0.464V_F – 3.26(1/R)*10^3 + 8.5(1/R)^2 *10^4$	Two-lane rural highway curves	 0.92
Passetti et al. (1999) $V85 = 103.9 – 3030.5(1/R)$	Two-lane rural highway curves	 0.68
Kanellaidis et al. (1990) $V85 = 129.88 – 623.1/(1/R)^{0.5}$	Two-lane rural highway curves	 0.78
Glennon et al (1983) $V85 = 150.08 – 4.14DC$	High-speed rural alignments, grades < 5%	 0.84
Ottesen et. al (2000) $V85 = 102.44 – 1.57DC + 0.012L – 0.01DC*L$ $V85 = 41.62 – 1.29DC + 0.0049L – 0.12DC*L + 0.95Va$	Two-lane rural highway curves, grades < 5%, 3 < degree of curvature < 12	 0.81 0.91
McFadden et al. (1997) $V85 = 104.61 – 1.90D$ $V85 = 103.13 – 1.58D + 0.0037L – 0.09\Delta$ $V85 = 54.59 – 1.50D + 0.0006L – 0.12\Delta + 0.81V_a$	Two-lane rural highway curves	 0.74 0.76 0.81
Andjus (1998) $V85 = 16.92 \ln R – 14.49$	Two-lane rural road curves, grades < 4%	 0.98
Islam et al. (1997) $V85_{(1)} = 95.41 – 1.48*DC –0.012*DC^2$ $V85_{(2)} = 103.03 – 2.41*DC – 0.029*DC^2$ $V85_{(3)} = 96.11 – 1.07*DC$	Two-lane rural highways (1) beginning of curve (2) middle of curve (3) end of the curve	 0.99 0.98 0.90
Schurr et al. (2002) $V85 = 103.3 – 0.1253DA + 0.0238L – 1.038G_1$	Two-lane rural highways	 0.46
Andueza (2000) $V85_{(1)} = 98.25 – 2795/R2 – 894/R1 + 7.486D + 9308L1$ $V85_{(2)} = 100.69 – 3032/R1 + 27819L1$	Two-lane rural highways (1) horizontal curves (2) tangents	 0.84 0.79
Jessen et al. (2001) $V_{mean}^{(1)} = 67.6 + 0.39V_p – 0.714G_1 – 0.00171\ T_{ADT}$ $V_{85}^{(1)} = 86.8 + 0.297\ V_p – 0.614G_1 – 0.00239\ T_{ADT}$ $V_{95}^{(1)} = 99.4 + 0.225\ V_p – 0.639G_1 – 0.0024T_{ADT}$ $V_{mean}^{(2)} = 55.0 + 0.5V_p – 0.00148\ T_{ADT}$ $V_{85}^{(2)} = 72.1 + 0.432V_p – 0.00212T_{ADT}$ $V_{95}^{(2)} = 82.7 + 0.379V_p – 0.002T_{ADT}$	Two-lane rural highways (1) crest vertical curve with limited stopping sight distance (2) approach tangent	 0.57 0.54 0.57 0.44 0.42 0.40
Fitzpatrick et al. (2000) $V85_{(1)} = 102.10 – 3077.13/R$ $V85_{(2)} = 105.98 – 3709.90/R$ $V85_{(3)} = 104.82 – 3574.51/R$ $V85_{(4)} = 96.61 – 2752.19/R$ $V85_{(5)} = 105.32 – 3438.19/R$ $V85_{(6)} = 103.24 – 3576.51/R$ $V85_{(7)}$ = assumed desired speed $V85_{(8)}$ = assumed desired speed $V85_{(9)} = 105.08 – 149.69/K$	Two-lane rural highway (1) horiz. curve, –9% < grade < –4% (2) horiz. curve, –4% < grade < 0 (3) horiz. curve, 0 < grade < 4% (4) horiz. curve, 4% < grade < 9% (5) horiz. curve with sag vertical curve (6) horiz. curve combined with limited sight distance crest vertical curve (7) sag vertical curve on horizontal tangent (8) vertical crest curve with unlimited sight distance on horizontal tangent (9) vertical crest curve with limited	 0.58 0.76 0.76 0.53 0.92 0.74 0.80

	sight distance on horizontal tangent	
Gibreel et al. (2001) $V85_{(1)} = 91.81 + 0.010R + 0.468\sqrt{Lv} - 0.006G_1^3 - 0.878\ln(A) - 0.826\ln(L_0)$ $V85_{(2)} = 47.96 + 7.217\ln(R) + 1.534(\sqrt{Lv}) - 0.258G_1 - 0.653A - 0.008L_0 + 0.020\exp(E)$ $V85_{(3)} = 76.42 + 0.023R + 2.300 * 10^{-4}K - 0.008\exp(A) - 1.230*10^{-4}L_0^2 + 0.062\exp(E)$ $V85_{(4)} = 82.78 + 0.011R + 2.067\ln(K) - 0.361G_2 - 1.091*10^{-4}L_0^2 + 0.036\exp(E)$ $V85_{(5)} = 109.45 - 1.257G_2 - 1.586\ln(L_0)$	Two-lane rural highway (1) Point 1 was set out at about 60-80 m on the approach tangent before the beginning of the spiral curve (2) Point 2 was the end of spiral curve and the beginning of horizontal curve in the direction of travel (SC) (3) Point 3 was the midpoint of horizontal curve (MC) (4) Point 4 was the end of horizontal curve and the beginning of spiral curve in the direction of travel (CS) (5) Point 5 was set out at about 60–80 m on the departure tangent after the end of the spiral curve.	0.98 0.98 0.94 0.95 0.79
Polus et al. (2000) $V85_{(1)} = 101.11 - 3420/GMs$ $V85_{(2)} = 105.00 - 28.107/e^{(0.00108*GML)}$ $V85_{(3)} = 97.73 + 0.00067*GM$ $V85_{(4)} = 105.00 - 22.953/e^{(0.00012*GML)}$	Two-lane rural highway tangents, (1) R1 and R2 = 250 m and TL = 150 m (2) R1 and R2 < 250 m and TL between 150 and 1000 m (3) R1 and R2 > 250 m and TL between 150 and 1000 m (4) TL > 1000 m	0.55 0.74 0.20 0.84
Liapis et al. (2001) $V85_{(1)} = -0.360839DC - 3.683548E + 75.161$ $V85_{(2)} = -0.472675DC - 3.795879E + 85.186$	Two-lane rural roads, passenger cars (1) off-ramps (2) on-ramps	0.75 0.73
Donnell et al. (2001) $V85_{(1)} = 51.5 + 0.137R - 0.779\ GAPT + 0.0127\ L1 - 0.000119\ (L1 * R)$ $V85_{(2)} = 54.9 + 0.123\ R - 1.07\ GAPT + 0.0078\ L1 - 0.000103\ (L1 * R)$ $V85_{(3)} = 56.1 + 0.117\ R - 1.15\ GAPT + 0.0060\ L1 - 0.000097\ (L1 * R)$ $V85_{(4)} = 78.7 + 0.0347\ R - 1.30\ GAPT + 0.0226\ L1$ $V85_{(5)} = 78.4 + 0.0140\ R - 1.40\ GDEP - 0.00724\ L2$ $V85_{(6)} = 75.8 + 0.0176\ R - 1.41\ GDEP - 0.0086\ L2$ $V85_{(7)} = 75.1 + 0.0176\ R - 1.48\ GDEP - 0.00836\ L2$ $V85_{(8)} = 74.7 + 0.0176\ R - 1.59\ GDEP - 0.00814\ L2$ $V85_{(9)} = 74.5 + 0.0176\ R - 1.69\ GDEP - 0.00810\ L2$ $V85_{(10)} = 82.8 - 2.00\ GDEP - 0.00925L2$ $V85_{(11)} = 83.1 - 2.08\ GDEP - 0.00934L2$ $V85_{(12)} = 83.6 - 2.29\ GDEP - 0.00919L2$ $V85_{(13)} = 84.1 - 2.34\ GDEP - 0.00944L2$	Two-lane rural highway, trucks (1) 200 meters prior to horizontal curve (2) 150 meters prior to horizontal curve (3) 100 meters prior to horizontal curve (4) 50 meters prior to horizontal curve (5) Beginning of horizontal curve (PC) (6) QP (7) Middle of horizontal curve (MC) (8) 3QP (9) End of horizontal curve (PT) (10) 50 meter beyond horizontal curve (PT50) (11) 100 meter beyond horizontal curve (PT100) (12) 150 meter beyond horizontal curve (PT150) (13) 200 meter beyond horizontal curve (PT200)	0.62 0.63 0.61 0.55 0.56 0.60 0.60 0.61 0.61 0.56 0.58 0.60 0.61
Cardoso et al. (1998) $V85_{(1)} = 49.220\ \frac{292736}{R^2} + 0.454Va$ $V85_{(2)} = 51.765\ \frac{337.780}{\sqrt{R}} + 0.6049Va$	(1) France horizontal curves (2) Finland horizontal curves (3) Greece horizontal curves (4) Portugal horizontal curves (5) France tangents (6) Finland tangents (7) Greece tangents	0.80 0.71 0.92 0.90 0.65 0.77

$V85_{(3)} = 41.363 \frac{294.000}{\sqrt{R}} + 0.699Va$ $V85_{(4)} = 25.010 \frac{271.500}{\sqrt{R}} + 0.877Va$ $V85_{(5)} = 97.737 + 0.007436\ L - 45.707\ Bend$ $V85_{(6)} = -17.17 + 0.02657\ L + 33.711LW - 21.936$ $V85_{(7)} = 134.069 - 3.799Hill - 126.59\ Bend$ $V85_{(8)} = -29.95 - 34.835LW - 0.0347PRad - 43.124\ Bend$	(8) Portugal tangents	0.92 0.82
Krammes et al. (1994) $V85 = 102.4 - 1.57D + 0.012L - 0.10\Delta$	Two-lane rural highway	0.82
Where: V85 = 85th percentile speed (km/h) Va = 85th percentile speed on approach tangent (km/h) Vp = posted speed (km/h) V_F = Desired speed of the 85 th percentile (km/h) R = horizontal curve radius (m) DC = degree of curve (degree/30 m) DA = deflection angle (degrees) L = length of curve (m) L1 = tangent length before the curve (m) L2 = tangent length after the curve (m) R1 = radius of the previous curve (m), R2 = radius of the following curve (m) S = minimum sight distance for the curve (m) GAPT = grade of approach tangent GDEP = grade of departure tangent T_{ADT} = ADT K = rate of vertical curvature E = superelevation rate *A* = algebraic difference in grades G_1 and G_2 = first and second grades in the direction of travel in percent L_0 = horizontal distance between point of vertical intersection and point of horizontal intersection (m) TL = tangent length (m) GMs = (R1 + R2)/2 (m) GML = $(TL^* (R1^*R2)^{0.5})/100$ (m^2) Bend = bendiness (degree/km) LW = land width (m) Hill = hilliness (percent) PRad = radius of the preceding curve (m) Δ = deflection angle (degree)		

APPENDIX B Existing Operating Speed Models for Urban Conditions

Speed Prediction Model	Location	R^2
Fitzpatrick et al. (1997) $V85_{(1)} = 56.34 + 0.808R^{0.5} + 9.34/AD$ $V85_{(2)} = 39.51 + 0.556\,(IDS)$	(1) suburban arterial horizontal curves, (2) suburban arterial vertical curves	 0.72 0.56
Fitzpatrick et al. (2001) $V85_{(1)} = 42.916 + 0.523PSL - 0.15DA + 4.402AD1$ $V85_{(2)} = 29.180 + 0.701PSL$ Or without speed limits $V85_{(1)} = 44.538 + 9.238MED + 13.029L1 + 17.813L2 + 19.439L3$ $V85_{(2)} = 18.688 + 15.050WD$	(1) suburban arterial horizontal curves, (2) suburban arterial straight sections	0.71 0.53 0.52 0.25
Bonneson (1999) $V85 = 63.5R\left(-B + \sqrt{B^2 + \frac{4C}{127R}}\right) \le Va$ $c = E/100 + 0.256 + (B - 0.0022)Va$ $B = 0.0133 - 0.0074I_{TR}$	Urban low speed, high speed roadways rural low speed, high speed roadways turning roadways -8.4% < grade < 8.0%	0.96
Tarris et al. (1996) $V85_{(1)} = 53.5 - 0.265D$ $V85_{(2)} = 53.8 - 0.272D$ $V85_{(3)} = 52.18 - 0.231D$	Low speed urban streets (1) aggregated speed data (2) individual speed data (3) panel analysis	0.82 0.63 0.80
Poe et al. (2000) $V85_{(1)} = 49.59 + 0.5*D - 0.35*G + 0.74*W - 0.74*HR$ $V85_{(1)} = 51.13 - 0.1*D - 0.24*G - 0.01*W - 0.57*HR$ $V85_{(1)} = 48.82 - 0.14*D - 0.75*G - 0.12*W - 0.12*HR$ $V85_{(1)} = 43.41 - 0.11*D - 0.12*G + 1.07*W + 0.3*HR$	Low speed urban streets (1) 150 ft before the beginning of curve (2) beginning of curve (PC) (3) middle of curve (MC) (4) end of curve (PT)	 0.99 0.98 0.90 0.90
Fitzpatrick et al. (2003) $V85_{(1)} = 8.666 + 0.963\,(PSL)$ $V85_{(2)} = 21.131 + 0.639\,(PSL)$ $V85_{(3)} = 36.453 + 0.517\,(PSL)$	(1) Suburban/urban arterial (2) Suburban/urban collector (3) Suburban/urban local	0.86 0.41 0.14
Where: V85 = 85th percentile speed (km/h) Va = 85 th percentile speed on approach tangent (km/h) R = horizontal curve radius (m) AD = approach density (approaches per km) IDS = inferred design speed (km/h) PSL = posted speed limit (km/h) MED = if raised or TWLTL then 1, otherwise 0 L1 = if school then 1, otherwise 0 L2 = if residential then 1, otherwise 0 L3 = if commercial then 1, otherwise 0 W = lane width (m) HR = hazard rating (0 to 4) E = superelevation rate D = degree of curve (degree) I_{TR} = indicator variable (= 1.0 if Va > V85; 0.0 otherwise)		

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