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AUTOMATED CALCULATION OF PASSING SIGHT DISTANCE USING GLOBAL POSITIONING SYSTEM DATA

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16 Abstract

Most of the rural highways in the United States of America are two-lane, two-way highways. In order to ensure smooth flow of traffic, maximum-passing opportunities must be provided on these highways, where the fast moving vehicles can overtake slow mowing vehicles. However, due to the geometric characteristics of the highways and cost limitations, passing opportunities cannot be provided throughout the length of the entire highway. Hence there is a need to find the segments of the highway that are not safe for overtaking called “no passing zones.”

The accurate placement of no passing zones on two-lane highways is critical to ensure safety of the travelers and also to protect the department of transportations of various states from lawsuits. Literature review shows that current methods used to mark the passing zones and no passing zones are very tedious and time consuming. The objective of this study was to develop a suitable model for measuring passing sight distance on two-lane, two-way highways using Global Positioning System (GPS) data and identify no passing zones. The model was converted into a computer algorithm and coded in Matlab version 6.5 and requires database toolbox. The algorithm has been tested on 10 highway segments and the results obtained are in agreement with the existing conditions. This model can be used to identify the no passing zones on any highways where the GPS data is available. It is accurate and cost effective when compared to the existing methods. The proposed model (procedure) has been automated, right from the importing the raw GPS data to the transferring of results into the final database.

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AUTOMATED CALCULATION OF PASSING SIGHT DISTANCE USING GLOBAL POSITIONING SYSTEM DATA

Final Report

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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ABSTRACT

Most of the rural highways in the United States of America are two-lane, two-way highways. In order to ensure smooth flow of traffic, maximum-passing opportunities must be provided on these highways, where the fast moving vehicles can overtake slow mowing vehicles. However, due to the geometric characteristics of the highways and cost limitations, passing opportunities cannot be provided throughout the length of the entire highway. Hence there is a need to find the segments of the highway that are not safe for overtaking called “no passing zones.”

The accurate placement of no passing zones on two-lane highways is critical to ensure safety of the travelers and also to protect the department of transportations of various states from lawsuits. Literature review shows that current methods used to mark the passing zones and no passing zones are very tedious and time consuming. The objective of this study was to develop a suitable model for measuring passing sight distance on two-lane, two-way highways using Global Positioning System (GPS) data and identify no passing zones. The model was converted into a computer algorithm and coded in Matlab version 6.5 and requires database toolbox. The algorithm has been tested on 10 highway segments and the results obtained are in agreement with the existing conditions. This model can be used to identify the no passing zones on any highways where the GPS data is available. It is accurate and cost effective when compared to the existing methods. The proposed model (procedure) has been automated, right from the importing the raw GPS data to the transferring of results into the final database.

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Introduction

1.1 Background

Despite the present day emphasis on freeways, expressways, superhighways, etc., the bulk of the rural highway network through out the United States is still the two-lane, two-way highway (AASHTO, 2001). Each highway has a maximum speed limit and a minimum speed limit. As a result of this fast moving vehicles have to overtake slower moving vehicles. According to Valkenburg & Michael, at least 90% of the total rural mileage is of the two-lane type and much of this mileage was constructed before modern geometric design standards were established. Consequently the horizontal and vertical alignments create hazards that frequently are the causes for many accidents while overtaking.

This contributing factor to accidents is due to the limited sight distance, which is available on some of the road segments due to poor alignment. Sight distance is especially important on two-lane, two-way highways more than on four lane highways or freeways because the passing maneuver requires the use of the lane normally occupied by on-coming traffic. This constitutes a constant danger to the two-lane highway user.

1.2 Problem Identification

The overtaking maneuver is a critical stage of the traffic flow on two-lane roads. The accidents occurred during the overtaking maneuvers are equal to about 10% of the total, with a slightly increasing trend in the past few years (Crisman et al., 2000). The fact of guaranteeing some segments on a two-lane road where overtaking can be safely made also means improving traffic flow quality. The current methods used to find the zones or segments of the highway that are safe for driving are manual and time consuming. For example the current method used by Kansas Department of Transportation (KDOT) involves two employees walking through a highway separated by a distance equal to the Passing Sight Distance (PSD). The employee at the front end holds a pole having a mark at the standard driver height at the end of the rope. A no-passing zone begins when the employee at the rear end cannot see the mark on the lead range pole and the no-passing zone ends when the mark on the lead pole comes into the visibility of the employee at the rear end. One can imagine how much time and labor it takes to find the no-passing zone segments on a highway using the current method.

1.3 Research Objective

Since finding no-passing segments is a laborious and time-consuming procedure and prone to error due to human intervention an accurate and fast procedure is required that would locate no-passing zones for all the highways. The purpose of this research is to develop a model for determining no-passing zones based on 3-D combined horizontal and vertical alignments.

1.4 Research Scope

The scope of the research is to develop a new model for automated calculation of passing sight distance from Global Positioning Systems (GPS) data. Software is developed based on this model and the algorithm is tested on several highways in Kansas on the data obtained from KDOT. The algorithm can identify the no-passing zone segments for all the highways in Kansas and write the results into a database.

1.5 Organization of Report

Chapter 2 presents a summary of the literature review. It explains in depth the concept of PSD, the origin and the models developed on determining the passing sight distance. It also explains about the Global Positioning System and how this can be used by the Kansas Department of Transportation to find the PSD.

Chapter 3 explains the parameters, which are used in calculating the PSD. The parameters used are height of the driver's eye, height of the object, lane width, clear zone, speed of the vehicles, headlight height, acceleration of the passing vehicle, length of the passing sections etc. It also discusses some of the assumptions made to develop the model.

Chapter 4 discusses the procedure used to calculate the existing PSD. It gives an in-depth explanation of how the PSD is determined, the concept and theory behind each stage of the algorithm. It also explains the procedure to determine the required PSD.

Chapter 5 examines the working of the algorithm. The algorithm has been tested on several highways and the results obtained by this method are compared to the existing conditions. Locations of no-passing zones in both directions obtained by this algorithm are shown for highway US 24.

Chapter 6 presents the conclusions of this research. It also shows the importance of PSD and also the scope for future research.

Chapter 2

Literature Review

Two-lane, two-way highways constitute a large portion of road networks on which vehicles frequently overtake slower moving vehicles, the passing of which must be accomplished on lanes regularly used by opposing traffic. The driver must be able to see a sufficient distance ahead, clear of traffic, to complete the passing maneuver without cutting off the passed vehicle in advance of meeting an opposing vehicle appearing during the maneuver (AASHTO 2001).

2.1 Introduction

Most roads and numerous streets are considered to qualify as two-lane, two-way highways. Overtaking a vehicle is one of the most difficult maneuvers a driver performs. Drivers cannot accurately estimate available distances and the speed of oncoming vehicles. Hence, accurately determined no-passing zones are essential to assist the driver in deciding when and where to pass (Gordon and Mast, 1968). The correct demarcation of passing/no passing zones is crucial because these zones affect the safety and capacity of the roadways. If there are enough gaps in opposing traffic, strategically located passing zones could greatly increase the capacity of a two-lane road. At the same time, inappropriate traffic signs and inaccurate pavement markings can compromise the safety (Forbes, 1990).

The oldest document for present operations Passing Sight Distances (PSDs) that is used in the current Manual on Uniform Traffic Control Devices (MUTCD) is the 1940 American Association of State Highway Officials (AASHO) publication, *A Policy on Criteria for Marking and Signing No-Passing Zones on two and Three-Lane Roads* (AASHO, 1940). In the paper the policy is referred to as the “no-passing zone policy”. The no-passing zone policy references a passing model in a second 1940 AASHO document, *A Policy on Sight Distance for Highways* (AASHO, 1940). This PSD model is based on several assumptions that are required to calculate the operations for PSDs. One of the key assumptions is that once a driver begins a pass he/she has no opportunity but to complete it. This assumption is believed to result in exaggerated PSD requirements. American Association of state Highway and Transportation Official's (AASHTO's) model is based on the field studies carried out from 1938 to 1941 by Prisk (1941). The passing model is based on a passing maneuver in which a single car passes another car. Measurements were taken on sections with regular traffic flows without researcher intervention in the passing process and without introducing any control factors. The resulting AASHTO model is able to represent the behavior of a large percentage of drivers instead of the average driver. It is based on a "delayed beginning and hurried return" assumption – that is, the passing car accelerates, and its average speed while it occupies the left lane is 15 km/h higher than that of the overtaken car. But this model does not reflect the impact of the increase in vehicle length and other geometric parameters and also the model does not reflect the rapid development in vehicle performance and the resulting change in driver behavior.

Because of the complexity of this type of field studies, no further large-scale studies were conducted for about 30 years, except for Normann's follow up work in late 50s which examined the validity of the recommended values in view of the great improvement in the performance of cars in the intervening period (Normann, 1958).

One of the major developments in the calculation of PSD was the introduction of the "critical point" in the model (Weaver & Glennon, 1971). According to this model, a driver can initiate a pass when the opportunity presents itself, but it does not imply that the driver is committed to complete the pass until he/she is almost abreast of the impending vehicle. Before reaching this point, if the driver finds an opposing vehicle approaching, he could easily decelerate and continue following the vehicle he/she intended to pass. After crossing this point, it would be best for the driver to complete the pass than to abort it. At the critical point, the driver can either make a pass or abort it based on personal judgment. The safety would be the same at this point if the pass is completed or aborted. The critical point occurs when the passing vehicle has two-thirds of the total passing distance remaining. The concept of the “point of no return” that is synonymous with the “critical point” was also introduced at the same time by Valkenburg & Michael (Valkenburg & Michael, 1971). Valkenburg & Michael’s model assumes that the pass is a delayed pass and that the average speed of the passing vehicle during the occupancy of the left lane is 10 mph higher than the impending vehicle speed, irrespective of the design speed. Neither study, however, attempted to mathematically define this critical position. Glennon and Harwood attempted to better explain the state-of-the-art concerning PSD requirements (Harwood & Glennon, 1976). This paper contributed further definition of the critical position as that point where the PSD needed to complete the pass is equal to the PSD needed to abort the pass.

These concepts laid the foundation for the present day models that derive the PSDs more scientifically. Valkenburg & Michael identified the point of return by producing the same factor of safety, whether the pass is completed or aborted, while Lieberman’s model used the same clearance value whether the pass is completed or aborted to identify the critical point (Lieberman, 1982). He added further insight by developing a mathematical time-distance model that identified the critical position and the critical PSD as a function of design speed. According to Glennon’s model, the critical position is specifically defined as the point where the distance to complete the pass equals the distance to abort the pass (Glennon, 1988). Glennon assumes that the difference in speed between the passing and impeding vehicles is not constant for all design speeds. He suggested that the differences in speed decreases with increasing design speed. The speed differentials are shown in Table 2.1.

Table 2.1: Speed Differential Between Passing and Impending Vehicles

Design Speed (mph)	**Speed Differential (mph)**
30	12
40	11
50	10
60	9
70	8

(Glennon, 1988)

The maximum or critical PSD is that needed at the critical position. Newer models have been developed to accommodate changes in the traffic conditions, road conditions and the vehicle capabilities. A comparison of passing sight distances calculated using different models is shown in Table 2.2.

Table 2.2: Minimum Passing Sight Distances for Operations

Design Speed or 85th Percentile Speed (mph)	**MUTCD (1988) (feet)**	**Van Valkenburg And Michael (1971) (feet)**	**Weaver and Glennon (1971) (feet)**	**Glennon (1987) (feet)**
30	500	-	-	-
40	600	1050	-	670
50	800	1300	1135	830
60	1000	1600	1480	990
70	1200	1900	1825	1140
80	-	-	2170	-

(Forbes, 1990)

Further research was carried out in the mechanics of the passing maneuver and the location of the critical point. Rilett carried out the studies on the location of the critical point for design velocities from 50 to 100 km/h for impending vehicle lengths of 5 to 25 m (Rilett, 1990). Here the passer is not arbitrarily assumed to have reached the design speed at the critical point as the earlier approaches have assumed.

A new model for PSD on two-lane highways takes into consideration the effect of having larger trucks on the roads into consideration. This is to accommodate sustained increases in the truck weights and dimensions that have occurred over the past 20 years in North America and other parts of the world. The principal stimulus behind these changes is the reduction in unit transportation costs with increasing payload. Today vehicles of a maximum length of 31 m are allowed in United States and up to 25m are permitted on Canadian roads (Wang & Cartmell, 1998). This increase in vehicle length has important implications on the costs for providing highway infrastructure and safety. One of them is the PSD, which is very crucial in overtaking maneuvers. The model designed can determine the safe PSD taking into account the presence of trucks on the roads and also 11 other input parameters –

- Velocity of the passing vehicle at the start of pass,
- Maximum velocity achieved,
- Speed of the impending vehicle,
- Speed of the opposing vehicle,
- Maximum acceleration of passing vehicle,
- Clearance between the head of impending vehicle and tail of the passing vehicle at the end of the pass,
- Clearance between the head of passing vehicle and the tail of the impending vehicle at the start position,
- Length of impending vehicle,
- Length of the passing vehicle,
- Clearance between the heads of passing vehicle and the opposing vehicle at the end of the pass and width of the lane.

Variation of each of these parameters has an affect in the determination of PSD (Wang & Cartmell, 1998).

In the recent years researches have come up with many models that try to give more ample opportunities to the drivers to pass. But from investigations made by Vera Mijuskovic based on more than 800 unsuccessfully accomplished overtakings within a certain region during a year period, it was found that 35% of the total number of all types of reported accidents resulted in injuries or/and fatalities and 92% of reported overtaking accidents had such consequences. In the review of accidents involving overtaking, database showed that over 90% of such accidents occurred on locations where overtaking was not prohibited (Mijuskovic, 1998).

Therefore it is very essential to permit passing opportunities only at locations where it is very safe to do so.

2.2 Need for Developing the Model

The current problem is to provide passing zones that are safe for overtaking - both with respect to the oncoming traffic and also any unexpected obstruction that may lie on the road and can be dangerous for the driver if undetected. This can be achieved by taking into consideration the passing sight distance criteria and also the stopping sight distance criteria, so that no accidents occur while overtaking. Based on the passing sight distance calculations, the minimum passing sight distance needed is obtained and from the stopping sight distance criteria the available and the required sight distance needed on the highway are obtained. The actual PSD requirement at the beginning of the zone is something less than minimum PSD, however, because passing operations vary widely by speed differentials, opposing vehicle speeds, and vehicle lengths, an added safety factor would be incorporated by starting the passing zone where the minimum PSD first becomes available. The next step is to identify the location of the passing zones on the highway. Therefore a model is needed which would give the passing sight distance and also identification of the location of the passing zones.

2.3 Passing Sight Distance

The AASHTO Policy defines PSD as the length of visible highway needed to safely complete normal passing maneuvers when a passing vehicle overtakes a slow-moving vehicle by moving into a lane that is normally used by opposing traffic. It is basically, the distance required for a vehicle to safely overtake a slower vehicle on a two-lane roadway by maneuvering into the lane of opposing traffic and then back into the right lane when past the slower vehicle.

2.4 Criteria for Design

Passing sight distance for use in design should be determined on the basis of the length needed to safely complete normal passing maneuvers. While there may be occasions to consider multi-vehicle passing, where two or more vehicles pass or are passed, it is not practical to assume such conditions in developing minimum sight criteria. Instead sight distance is determined for a single vehicle passing a single vehicle (AASHTO, 1990). Longer sight distances occur in design and these locations can accommodate an occasional multi-vehicle pass.

2.5 Assumptions for the Design

- When computing minimum passing sight distances on two-lane highways for design use, certain assumptions for traffic behavior are necessary. The assumed control for the driver behavior should be that practiced by a high percentage of drivers, rather than the average driver. Such assumptions follow (AASHTO, 2001):
- The over taken vehicle travels at uniform speed.
- The passing vehicle has reduced speed and trails the overtaken vehicle as it enters a passing section.
- When the passing section is reached, the driver requires a short period of time to perceive the clear passing section and to react to start his maneuver.
- Passing is accomplished under what may be termed a delayed start and a hurried return in the face of opposing traffic. The passing vehicle accelerates during the maneuver, and its average speed during the occupancy of the left lane is 10 mph higher than that of the overtaken vehicle.
- When the passing vehicle returns to its lane, there is a suitable clearance length between it and an oncoming vehicle in the other lane.
- Extraordinary maneuvers are ignored and passing sight distances are developed with the use of observed speeds and times that fit the practices of a high percentage of drivers. For example, some drivers accelerate at the beginning of a passing maneuver to an appreciably higher speed and then continue at a uniform

speed until the passing is completed. But many drivers accelerate at a fairly high rate until just beyond the vehicle being passed and then complete the maneuver either without further acceleration or at a reduced speed

2.6 Components of Passing Sight Distance

The minimum passing sight distance for two-lane highways is determined as the sum of the four distance components. Most of the research carried out in the recent years has calculated the passing sight distances as the sum of four component distances (Valkenburg & Michael, 1971; Weaver & Glennon, 1971; Glennon, 1987; Hassan & Easa, 1996), although there is a little variation in the definition of the components.

d_1 – Distance traversed during the perception and reaction time and during the initial acceleration to the point of encroachment on the left lane.

d_2 – Distance traveled while the passing vehicle occupies the left lane.

d_3 – Distance between the passing vehicle at the end of its maneuver and the opposing vehicle.

d_4 – Distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane (AASHTO, 2001).

2.6.1 Initial Maneuver Distance (d1)

The initial maneuver period has two components, a time for perception and reaction, and an interval during which the driver brings his vehicle from the trailing speed to the point of encroachment on the left or passing lane. To a great extent the two overlap. As a passing section of highway comes into view of a driver desiring to pass, he may begin to accelerate and maneuver his vehicle towards the centerline of the highway while deciding whether or not to pass. Studies show that average passing vehicle accelerates at less than its maximum potential, indicating that the initial maneuver period contains an element of time for perception and reaction. However, some drivers may remain in normal lane position while deciding to pass. The exact position of the vehicle during initial maneuver is unimportant because differences in resulting passing distances are insignificant.

The distance d_1 traveled during the initial maneuvering period is computed from the following formula: (AASHTO, 2001)

$$d_1 = 1.47\, t_1 \left(v - m + (a\, t_1 /2)\right) \qquad \text{(Equ 2.1)}$$

where:

t_1 = time of initial maneuver, sec;
a = average acceleration, mph/sec;
v = average speed of passing vehicle, mph; and
m = difference in speed of passed vehicle and passing vehicle, mph.

2.6.2 Distance While Passing Vehicle Occupies Left Lane (d_2)

The distance d_2 traveled in the left lane by the passing vehicle is computed by the following formula:

$$d_2 = 1.47\, v\, t_2 \qquad \text{(Equ 2.2)}$$

where:

t_2 = time passing vehicle occupies the left lane, sec; and
v = average speed of the passing vehicle, mph.

2.6.3 Clearance Length (d3)

The clearance length between the opposing and passing vehicles at the end of the maneuvers found in the passing study was found to vary from 110 feet to 300 feet. This length is usually taken to be 300 feet for design speeds of 70 mph. The clearance lengths for different design speed groups are as shown in Table 2.3 (AASHTO, 2001).

Table 2.3: Clearances for Various Speed Groups

Speed Group (mph)	Clearance (feet)
30 – 40	100
40 – 50	180
50 – 60	250
60 – 70	300

(AASHTO, 2001)

2.6.4 Distance Traversed by an Opposing Vehicle

Passing sight distance includes the distance traversed by an opposing vehicle during the passing maneuver to minimize the chance of a passing vehicle meeting an opposing vehicle while in the left lane. Conservatively, this distance should be the distance traversed by an opposing vehicle during the entire time it takes to pass or during the time the passing vehicle is in the left lane, but such distance is questionably long. During the first phase of the passing maneuver the passing vehicle has not yet pulled abreast of the vehicle being passed, and even though the passing vehicle occupies the left lane, its driver can return to the right lane if he sees an opposing vehicle. It is unnecessary to include this trailing time interval in computing the distance traversed by an opposing vehicle. The time interval, which can be computed from the relative positions of passing and passed vehicle, is about one-third the time the passing vehicle occupies the left lane, so that the passing sight distance element for the opposing vehicle is the distance it traverses during the two-thirds of the time the passing vehicle occupies the left lane.

The opposing vehicle is assumed to be traveling at the same speed as the passing vehicle, so $d_4 = 2d_2/3$ (AASHTO, 2001).

2.7 Stopping Sight Distance

One of the factors to be given top most priority while designing a model for passing sight distance is the safety of the drivers. Hence the safety of the driver is taken into account not only with regards to the opposing traffic but also with respect to any obstruction that may exist on the highway. Sight distance is defined as the length of the roadway ahead that is visible to the driver (AASHTO, 2001). From a geometric design standpoint, the minimum sight distance available on a roadway should be long enough to enable a vehicle traveling at design speed to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at each and every point along the highway should be at least the same as that required for a below-average driver or vehicle to stop (AASHTO, 2001).

American Association of State Highway and Transportation Officials defines the stopping sight distance (SSD) as the sum of two components, brake reaction distance (distance traveled from the instant of object detection to the instant the brakes are applied) and braking distance (distance traveled from the instant the brakes are applied to when the vehicle is decelerated to a stop). Conceptually, required stopping sight distance can be expressed by the following equation:

$$SSD_{req} = \text{Brake Reaction Distance} + \text{Braking Distance}$$

2.8 Locating the Passing Zones

The accurate placement of no-passing zones on two-lane highways is critical for motorist safety and for the protection of the departments of transportation (DOTs) from lawsuits after accidents. Agencies may need to identify the passing zones when a highway is resurfaced, or the speed limit is changed or when roadside vegetation has grown and block lines of sight or other reasons. There are several methods for locating the passing zones which the DOTs use. Some of the commonly used methods are:

- The Walking Method,
- Two-Vehicle Method,
- One-Vehicle Method,
- Eyeball Method
- Videolog or Photolog Method.

Brown and Hummer have studied in-depth about these methods by reviewing literature, interviewing engineers in 13 leading state DOTs (Brown & Hummer, 2000). These methods are described briefly below.

2.8.1 Walking Method

In this method, two employees walk through a site separated by a rope (some times a chain) whose length is equal to the required passing sight distance. Each one holds a range pole, marked at the standard driver height of 3.5 feet. A no-passing zone begins when the rear person can no longer see the mark on the lead range pole and the no-passing zone end where the mark on the lead range pole comes back into view. The DOT engineers of North Carolina interviewed during this project felt that this method to be very accurate and is used in North Carolina and Iowa. However this method is time-consuming and requires two people (Brown & Hummer, 2000).

2.8.2 Two-Vehicle Method

Two trucks equipped with two-way communication, distance-measuring instruments (DMIs), and a paint sprayer operated from within the truck. Upon starting, one truck moves the required passing sight distance ahead of the other. After each truck resets its DMI to zero, they both begin to drive through the section of the road to be measured for sight distance. The drivers use two-way communication to try and maintain the required separation. When the rear driver loses sight of a point (a standard height above the ground on the lead vehicle), he/she administers a short paint mark, which indicates the start of no-passing zone. Again, a paint mark is placed to mark the end of no-passing zone once the lead truck comes back into view. Pennsylvania, New Jersey, Texas, Michigan, California, and Colorado DOTs use this method. This method requires two persons, specialized equipment and presents other problems (Brown & Hummer, 2000).

2.8.3 One-Vehicle Method

This method requires one vehicle equipped with a DMI and one person. To measure a curve or a hill the observer (driver) drives slowly through it. When the observer reaches the point where the vista opens up and the observer is sure there is a length of road ahead sufficient for safe passing, he/she stops the vehicle and places a paint mark on the right side of the highway. This point is the end of the no-passing zone in the direction of travel. The observer then sets the DMI to zero and drives the required passing sight distance stops to place a mark on the left side of the road. This marks the beginning of no-passing zone in the opposite direction. Currently 5 of the 14 North Carolina Department of Transportation (NCDOTs) divisions use this method (Brown & Hummer, 2000).

2.8.4 Eyeball Method

A simple eyeball method consists of an observer being driven toward a curve or a hill and estimating where the no-passing zone should begin. The estimate can then be verified by setting a DMI to zero and traveling to the point where approaching vehicles would appear. Experienced observers apparently can estimate the beginning of the no-passing zone to within 15 – 30 m of where it should begin for horizontal curves but this method is difficult to use for the vertical curves (Brown & Hummer, 2000).

2.8.5 Videolog or Photolog Method

Videologs or photologs are pictorial images of the roadway and roadside integrated with georeferences. Sight distances can be computed from videologs or photologs of the roadway. This equipment is usually a part of a highly specialized data collection vehicle. This method is generally an accurate way to compute sight distances and the vehicles can collect a wealth of data besides sight distances. But the vehicles are very expensive and can cost between $400,000 to $1.2 millions, much of which is for the software and system integration. Virginia, New York, Arizona, Wisconsin and Connecticut DOTs use this method (Brown & Hummer, 2000). Although there are several methods for identifying the passing zones, each one has a set back because of the costs involved or the accuracy obtained. However if Global Positioning System (GPS) is available, the location of the passing zones and no-passing zones can be obtained very quickly and accurately.

2.9 Highway Models Derived Using the Global Positioning System (GPS)

Global Positioning System (GPS) is a satellite-based navigation system made up of a network of 24 satellites placed into orbit by the U.S. Department of Defense. GPS works in any weather conditions, anywhere in the world, 24 hours a day (www.garmin.com). Figure 2.1 shows the network of satellites around the earth.

Figure 2.1: GPS Satellite system
(Source: www.garmin.com)

GPS was originally intended for military applications, but in the 1980s, the government made the system available for civilian use. GPS satellites circle the earth twice a day in a very precise orbit and transmit signal information to earth. GPS receivers take this information and use triangulation to calculate the user's exact location. Essentially, the GPS receiver compares the time a signal was transmitted by a satellite with the time it was received. The time difference tells the GPS receiver how far away the satellite is. Now, with distance measurements from a few more satellites, the receiver can determine the user's position and display it on the unit's electronic map (www.garmin.com).

A GPS receiver must be locked on to the signal of at least three satellites to calculate a 2D position (latitude and longitude) and track movement. With four or more satellites in view, the receiver can determine the user's 3D position (latitude, longitude and altitude) (Nehate, 2002).

Chapter 3

Parameters for Measuring Passing Sight Distance

Passing Sight Distance for use in design should be determined on the basis of the length needed to complete normal passing maneuvers in which the passing driver can determine that there are no potentially conflicting vehicles ahead before beginning the maneuver. While there may be occasions to consider a multi-vehicle pass, where two or more vehicles can pass or are passed, it is not practical to assume such conditions in developing minimum sight criteria. Instead, sight distance should be determined for a single vehicle passing a single vehicle (AASHTO, 2001). Longer sight distances occur in design and such locations can accommodate an occasional passing. To ensure the passing maneuver to be completely safe, both the stopping sight distance and the passing sight distance are taken into consideration. When the criteria for both stopping sight distance and the passing sight distance are met, then passing is permitted in that zone.

3.1 Height of Driver's Eye

For sight distance calculations, the driver's height is considered to be 3.5 ft above the road surface. This value is based on the studies carried out by Fambro, Fitzpatrick and Koppa. The average vehicle heights decreased to 4.25 ft with a comparable decrease in the eye heights to 3.5 ft. (Fambro, Fitzpatrick & Koppa, 1997). Because of various factors that appear to place practical limits on further decreases in passenger car heights and the relatively small increases in the lengths of the vertical curves that would result from further changes that would occur, 3.5 feet is considered to be an appropriate height of driver's eye for measuring both passing sight distance and also for stopping sight distance. For large trucks the driver's eye height ranges from 5.9 to 7.9 feet. The recommended value for trucks is 7.6 feet (AASHTO, 2001).

3.2 Height of Object

For passing sight distance the height of the object is considered to be 3.5 feet above the road surface and for stopping sight distance the height of the object is taken as 2.0 feet above the surface of the road (AASHTO, 2001).

3.2.1 Passing Sight Distance Object

An object of height 3.5 feet is adopted for passing sight distance. This object height is based on a vehicle height of 4.35 feet, which represents the 15 percentile of the vehicle heights in the current car population, less an allowance of 0.82 feet, which represents a near maximum value for the portion of the vehicle height that needs to be visible for another driver to be able to recognize the vehicle as such (Harwood et al., 1996).

The passing sight distances calculated on this basis are also considered adequate for night conditions because headlight beams of opposing vehicles can be seen from a greater distance than a vehicle can be recognized during the day time (AASHTO, 2001). Choosing an object height equal to the height of the driver's eye makes the passing sight distance reciprocal (i.e., the driver of the passing vehicle can see the opposing vehicle and the driver of the opposing vehicle can also see the passing vehicle) (AASHTO, 2001). The object height adopted for the model is 3.5 feet, which supercedes the 4.5 feet object height during the 1940s and object height of 4.25 during the 1990s (AASHTO, 1990).

3.2.2 Stopping Sight Distance Object

The basis for selection of a 2 feet object height was largely an arbitrary rationalization of the size of object that might potentially be encountered on the road and of a driver's ability to perceive and react to such situations. It is considered that an object 2 feet high is representative of an object that involves risk to drivers and can be recognized by a driver in time to stop before reaching it. Using object heights of less than 2 feet height for stopping sight distance calculations would result in a longer crest vertical curves without documented safety benefits (Fambro et al., 1997). Object height of less than 2 feet (600 mm) could substantially increase construction costs because additional excavation would be needed to provide the longer crest vertical curves. It is also doubtful that the driver's ability to perceive situations involving risk of collisions would be increased because recommended stopping sight distances for high-speed design are beyond most driver's capabilities to detect small objects (Fambro et al., 1997).

3.3 Lane Width

No feature of a highway has a greater influence on the safety and comfort of driving than the width and condition of the surface (AASHTO, 2001). In general, 10 feet to 13 feet lane widths are used, with a 12 feet lane predominant on most typical highways. A 24 feet surface is required to permit desired clearance between commercial vehicles. It is generally recommended that lane widths of 12 feet should be provided on main highways (AASHTO, 2001).

3.4 Clear Zone

The term "clear zone" is used to designate the unobstructed, relatively flat area provided beyond the edge of the traveled way for the recovery of errant vehicles. The traveled way does not include shoulders or auxiliary lanes (AASHTO, 2001).

The growth of an urban area typically extends outward along major arterial highways. The nature of the land use along the highways gradually changes from rural and agricultural to suburban with strip commercial developments, such as service stations, fast food restaurants and shopping centers. The resulting growth in traffic volume and frequent turning movements can cause congestion and increase accident experience, which may necessitate widening the existing tow-lane highways to four or more lanes. Also, in anticipation of future growth, these suburban arterial highway sections are designed with curb-and-gutter cross sections and often with two-way left-turn center lanes, typical of urban type roadways. However, these highway sections will remain suburban in nature for a period of time, i.e. with moderate traffic volume and high speed, and speed limits ranging from 80.5 to 88.6 km/hr (50 to 55 mph). The land use and resulting traffic volume will continue to grow until these highway sections become urban roadways with high traffic volume and lower traffic speed limits [i.e. 72.4 km/hr (45 mph) or less] (Mak et al., 1995).

These suburban arterial highway sections pose some interesting problems because they serve as a transition from rural to urban type highways at the fringes of urban areas. Under current AASHTO design guidelines, low speed [i.e. 72.4 km/hr (45 mph) or less] urban roadways with curb-and-gutter cross sections and no shoulders must have a minimum clear zone width of 0.46 m (18 in) beyond the face of the curb. On the other hand, high-speed rural arterial highways with shoulders and parallel drainage ditches are typically required to have a clear zone width of 9.1 m (30 ft) or more beyond the edge of the travel way (i.e. edge line or edge of pavement) (Mak et al., 1995). The AASHTO Roadside Design Guide (1989) discusses clear zone widths as related to speed, volume and embankment slope (Nehate, 2002).

3.5 Headlight Height

In general, headlight height of 600 mm and a 1° upward divergence of the light beam from the longitudinal axis of the vehicle are used (AASHTO, 2001).

3.6 Speed of the Vehicles

The actual driver behavior in passing maneuver varies widely. To accommodate the behavior of a high percentage of drivers, rather than just the average driver certain assumptions are made with respect to the speeds of the passing vehicle, opposing vehicle and the impending vehicle.

The ranges of speeds of the passed and the passing vehicles are affected by traffic volume. When the traffic volume is low, there are few vehicles that need to be passed, but as the volume increases there are few, if any passing opportunities. The speed of the passed vehicle has been assumed to be the average running speed at a traffic volume near capacity. The speed of the passing vehicle is assumed to be 10 mph greater. The assumed speeds for the passing vehicles represent the likely passing speeds on two-lane highways. Passing sight distances for these passing speeds would accommodate a majority of the desired passing maneuvers. The values for these speeds are given in Table 3.1.

Table 3.1: Speeds of Vehicles Involved in Passing Maneuver

Design Speed (mph)	Passed Vehicle (mph)	Passing Vehicle (mph)	Opposing Vehicle (mph)
20	18	28	20
25	22	32	25
30	26	36	30
35	30	40	35
40	34	44	40
45	37	47	45
50	41	51	50
55	44	54	55
60	47	57	60
65	50	60	65
70	54	64	70
75	56	66	75
80	58	68	80

(AASHTO, 2001)

3.7 Acceleration of the Passing Vehicle

The acceleration of the passing vehicle is based on the studies carried out by Prisk (Prisk, 1941). These values were examined again in 1957 by making observations in three of the locations where Prisk study was performed. There were very little changes noticed in the passing practices (AASHTO, 1990). Weaver and Woods conducted a study to evaluate the parameters used by Prisk in his study. They found that the values of the parameters used by Prisk was conservative and for modern vehicles (Weaver & Woods, 1978). Hence AASHTO adopted these values (AASHTO, 2001). There are four speed groups – the 30 – 40 mph; the 40 - 50 mph; the 50 – 60 mph and the 60 – 70 mph groups. The values for the acceleration are as shown in the Table 3.2.

Table 3.2: Acceleration Values Based on the Speed

Speed Group (mph)	**Average Acceleration (mph/sec)**
30 - 40	1.4
40 - 50	1.43
50 - 60	1.47
60 - 70	1.5

(AASHTO, 2001)

The acceleration rates were obtained for the first three groups from the passing study data and these results were extrapolated to the fourth group.

3.8 Frequencies and Length of Passing Sections

Sight distance adequate for passing should be encountered frequently on two-lane highways along the length. Each passing zone must have a sight distance ahead equal to or greater than the minimum passing sight distance.

Roads are designed to provide passing sections where there is little or no cost involved to provide a passing zone. It is not practical to directly indicate the frequency with which passing zones must be provided due to the physical and cost limitations.

The minimum length of a passing zone must be at least 400 feet for any highway with design speeds of 60 mph (MUTCD, 2000). The basis for this minimum value of PSD is not documented in any of the MUTCD editions. Guidelines are provided by MUTCD based on the design speeds as shown in Table 3.3

Table 3.3: Recommended PSDs Based on the Posted Speeds

Posted Speed Limit (mph)	Minimum Passing Sight Distance (feet)
25	450
30	500
35	550
40	600
45	700
50	800
55	900
60	1000
65	1100
70	1200

(MUTCD, 2000)

3.9 Summary

Based on the speed groups, AASHTO & MUTCD recommend minimum values for the length of a passing zone. But to obtain accurate results the minimum length of the passing zones should be calculated. The length of the passing zone varies depending on the design speed. For example the passing length required for a design speed of 65 mph would be different from that of the passing length required for a design speed of 60 mph or 70 mph although they are in the same speed group according to AASHTO 2001 (Refer to table 2.3).

Chapter 4

Passing Sight Distance

4.1 Introduction

Most of the models developed so far calculate the minimum length required for passing based on the design speeds and the vehicle parameters. However, to mark a passing zone, two factors are needed.

- Minimum Sight Distance required for passing
- Minimum length for passing zones

But to ensure safety for the driver while overtaking due to objects or obstacles that might lie in the path and can cause major accidents if collided with, Stopping Sight Distance (SSD) is also calculated in this model.

4.1.1 Minimum Sight Distance Required for Passing

Sight distance is obstructed either by vertical alignments or horizontal alignments. On vertical alignments, if the sight line from the driver's eye to the object should pass over a crest curve, it may intersect the road itself. The point of intersection may be a tangent segment, a crest curve, or a sag curve. Subsequently, the sight distance is limited by having the sight line tangent to the crest curve. In addition, the distance ahead covered by the vehicle headlight limits sight distance on sag curves at nighttime. On horizontal alignments, sight distance is limited by lateral obstructions such as trees, buildings and cut slopes. Analysis done during on SSD showed that most of the deficiencies in sight distances available on the roads are due to vertical alignments.

4.1.2 Minimum Length for Passing Zones

The required minimum length for passing zones is calculated based on AASHTO's assumptions and standard values for the parameters. In order to permit passing, the minimum passing zone length is determined for one car overtaking another car as described in section 2.4.

4.1.3 Stopping Sight Distance

To ensure the safety of the drivers, the minimum sight distance available on a roadway should be long enough to take evasive action in sufficient time to enable the passing vehicle to stop before reaching a stationary object in its path while on the opposing traffic lane during the over taking maneuver.

4.2 Obtaining Data for the Highways

GPS is used to collect data for the highways in Kansas in the form of latitude, longitude and altitude. Every year the Kansas Department of Transportation (KDOT) collects the data and stores it into the database. This data is arranged in the database based on the year in which it was collected and within each year's collection that data is organized by driving direction. If the general direction of the road is either North-South, then South to North is taken as the normal forward driving direction and North to South is taken as the reverse direction. Similarly, if the general direction of the road is East-West, then West to East is taken as the normal forward driving direction and East to West is taken as the reverse direction. This data is sorted in order of increasing route mileage for each direction.

4.3 Refining the Data for Useful Purposes

The data obtained by GPS is in the form of terrestrial coordinates and needs to be transformed into Cartesian coordinates for mathematical calculations. Lambert conformal projection with two standard parallels transformation method is used for coordinate transformation.

The data thus obtained needs to be checked for outliers. First the elevation is checked for outliers and those found are eliminated. The outliers are found by using the following methodology (Ben-Arieh et. al., 2002):

- z(j) is the elevation of the j^{th} point of the sorted data,
- z(j+1) elevation of the point following z(j),
- if z(j+1) - z(j) > 5 meters, then the $(j+1)^{th}$ point is an outlier.
- The $(j+1)^{th}$ point is then eliminated.

Control points are thus obtained from the data. These control points are used for PSD analysis. Due to realignments of highways, the geometric profiles of a small portion of the highway network may change from year to year. To obtain current information of the highways, the GPS data obtained during the current year or previous year is used for the analysis.

Figure 4.1 shows the plot of raw data for a section of highway US 77 in Riley County. The red color and blue color show the plots based on GPS raw data collected in the year 2001 and 2002 respectively. Figure 4.2 shows the data for a section of highway 75 in Jackson County, Kansas, USA. The circles show the raw GPS data collected by KDOT for the year 2002. Using the procedure given above, the data is cleaned; the control points are shown in pluses (+).

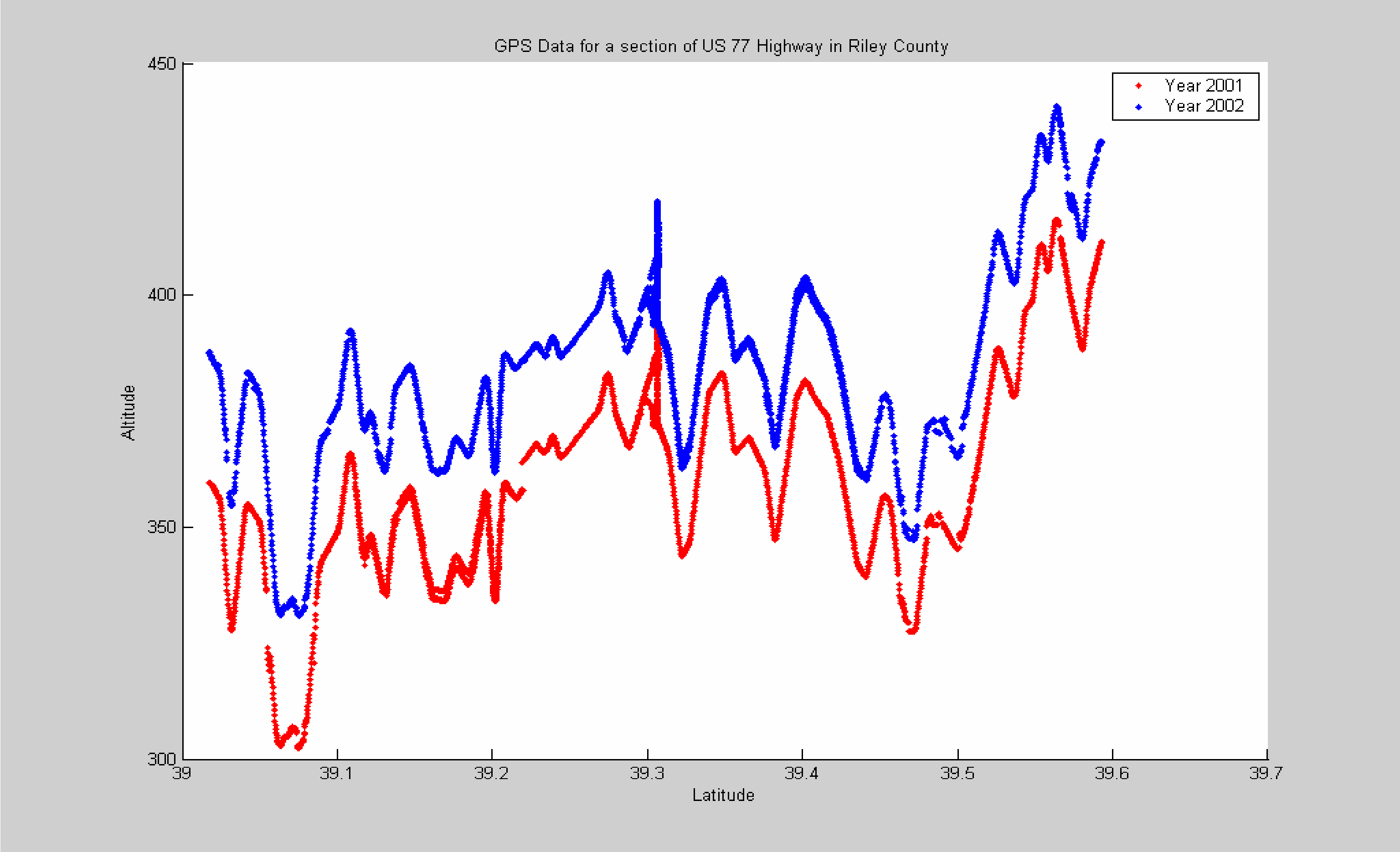

Figure 4.1: Data for a Section of Highway US-77 in Riley County, Kansas.

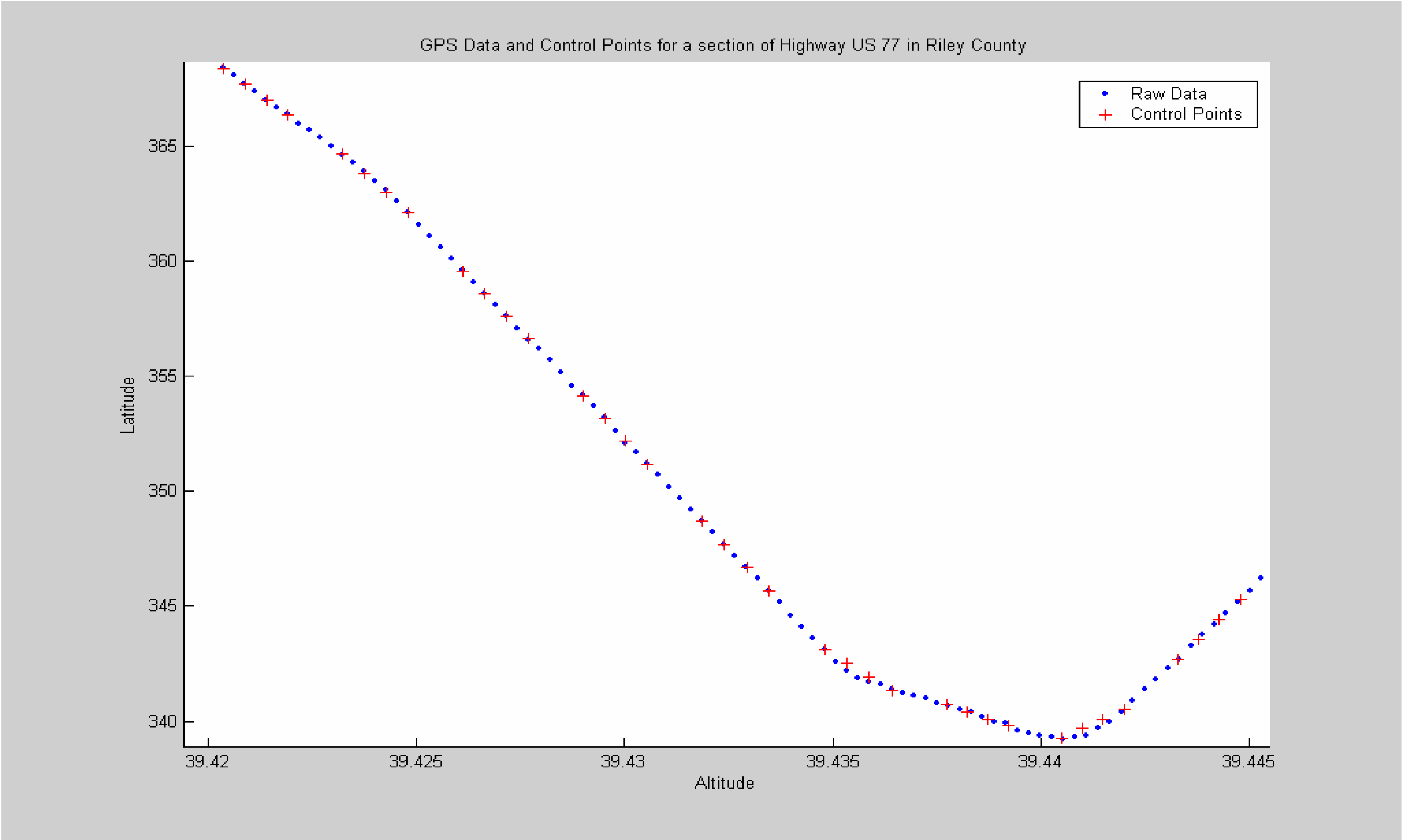

Figure 4.2: Data for a Section of Highway US-77 in Riley County, Kansas.

Table 4.1 shows the control points data for a section of highway US 77 in Riley County, Kansas, USA.

Table 4.1: Control Points Data for a Section of Highway US-77 in Riley County, Kansas

Latitude (°)	Longitude (°)	Altitude (m)
39.22	-96.93	357.82
39.22	-96.93	357.59
39.22	-96.93	357.34
39.22	-96.93	357.08
39.22	-96.93	356.42
39.22	-96.93	356.29
39.22	-96.93	356.21
39.22	-96.93	356.19
39.21	-96.93	356.46
39.21	-96.93	356.52
39.21	-96.93	356.58
39.21	-96.93	356.64
39.22	-96.93	357.30
39.21	-96.93	356.90
39.22	-96.93	359.83
39.22	-96.93	361.91
39.22	-96.93	363.34

4.4 Obtaining the Parametric Equations

The road obtained using GPS data is represented as a continuous curve through the use of interpolation equations that rely on parametric equations. Parametric equations are a set of equations that express a set of quantities as explicit functions of a number of independent variables, known as "parameters" (Stroud, 2001). For example, while the equation of a circle in Cartesian coordinates can be given by

$$r^2 = x^2 + y^2 \qquad \text{(Equ 4.1)}$$

one set of parametric equations for the circle are given by,

$$x = r*\cos(t) \qquad \text{(Equ 4.2a)}$$

$$y = r*\sin(t) \qquad \text{(Equ 4.2b)}$$

where ‘t’ is called a governing parameter and the two expressions for x and y are parametric equations.

Parametric equations provide a convenient way to represent curves and surfaces. Parametric equations come about because it is sometimes convenient to express a cartesian function by expressing x and y separately in terms of a third independent variable.

For example: $y = \cos(2t)$, $x = \sin(t)$

In this case any value that we give to t will produce one point on the curve $y = f(x)$. So if $t = 60$; $y = \cos(120°)$, $x = \sin(60°)$. Thus, the point on the curve will be $y = -0.5$, $x = 0.866$ or (-0.5, 0.866). (Nehate, 2002)

4.5 Constructing the B-Spline Curve

According to the information given by Heriot-Watt University in their website **http://www.cee.hw.ac.uk/~ian/hyper00/curvesurf/bspline.html** “A B-spline curve is a set of piecewise (usually cubic) polynomial segments that pass close to a set of control points.”

Given control points P_0, $P1$,… P_n, $n \geq 3$, a cubic B-spline curve consists of $(n\text{-}2)$ cubic curve segments. The i^{th} curve segment is defined by the points P_{i-1}, P_i,P_{i+1},P_{i+2} as shown in Figure 4.3. In addition the four points only define the curve shape between P_i and P_{i+1}.

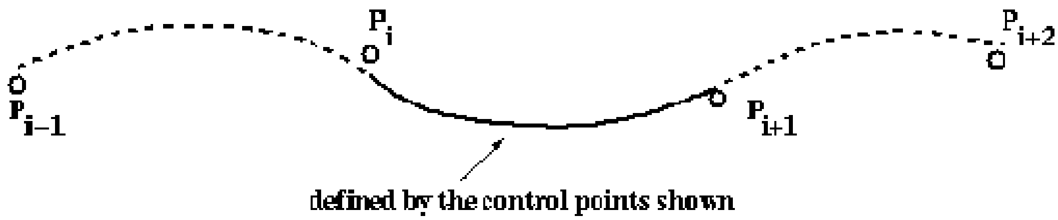

Figure 4.3: B-Spline Curve
(Source:***http://www.cee.hw.ac.uk/~ian/hyper00/curvesurf/bspline.html***)

The curve between P_i and P_{i+1} is defined by:

$$(C_i(t)) = \frac{1}{6}\begin{bmatrix} t^3 & t^2 & t & 1 \end{bmatrix}\begin{bmatrix} -1 & 3 & -3 & 1 \\ 3 & -6 & 3 & 0 \\ -3 & 0 & 3 & 0 \\ 1 & 4 & 1 & 0 \end{bmatrix}\begin{bmatrix} P_{i-1} \\ P_i \\ P_{i+1} \\ P_{i+2} \end{bmatrix} \qquad \text{(Equ 4.3)}$$

Where: $0<t<1$

$P_i = (x_i, y_i, z_i)$;

4.5.1 B-Spline Curve

A piecewise polynomial curve is defined by a set of control points through which the curve ordinarily does not pass. The degree of its polynomial parametric equations is defined independently of the number of control points. Local control of curve shape is possible because changes in control point location do not propagate shape change globally, and control points influence only a few nearby curve segments. However the curve does not pass through these control points, it only passes close to them as shown in figure 4.4 B-spline curve exhibits positional, first-derivative and second-derivative continuity (http://www.olympus.net/personal/mortenson/preview/definitionsb/bsplinecurve.html).

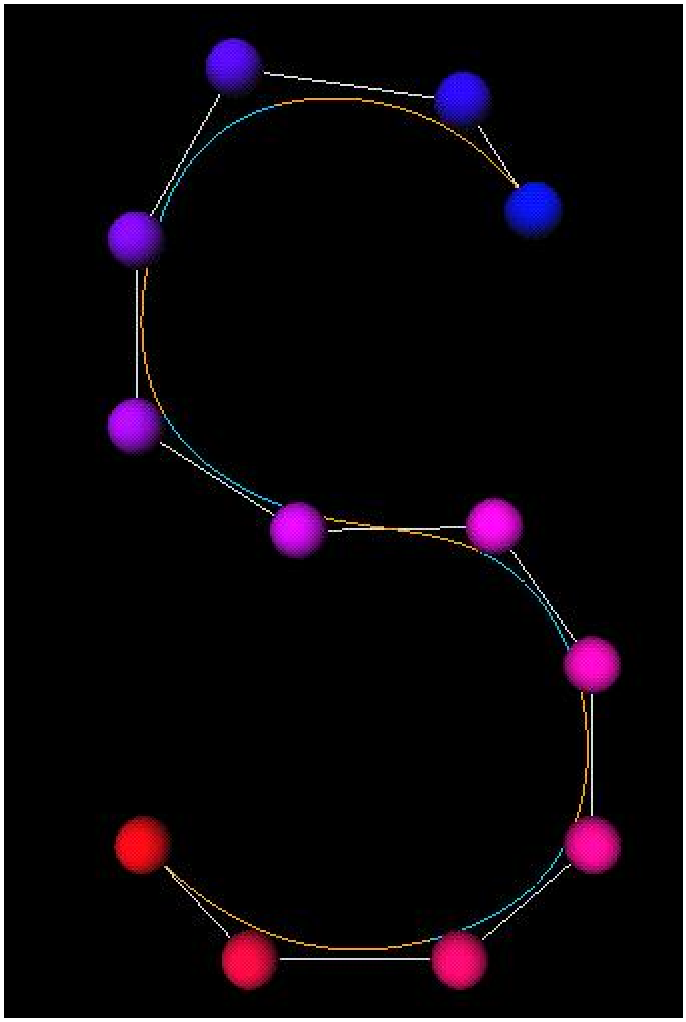

Figure 4.4: B-Spline Curve

(Source: http://graphics.stanford.edu/courses/cs348c-95-fall/software/scurvy/scurvy.jpg)

Four consecutive points generate a B-spline curve that fit these four points. These points form the control sequence $P_0P_1P_2P_3$ the curve between $P_j(x_j,y_j,z_j)$ and $P_{j+1}(x_{j+1},y_{j+1},z_{j+1})$ is given by:

$$(x(t), y(t), z(t)) = \frac{1}{6}\begin{bmatrix} t^3 & t^2 & t & 1 \end{bmatrix}\begin{bmatrix} -1 & 3 & -3 & 1 \\ 3 & -6 & 3 & 0 \\ -3 & 0 & 3 & 0 \\ 1 & 4 & 1 & 0 \end{bmatrix}\begin{bmatrix} x_0 & y_0 & z_0 \\ x_1 & y_1 & z_1 \\ x_2 & y_2 & z_2 \\ x_3 & y_3 & z_3 \end{bmatrix} \qquad \text{(Equ 4.4)}$$

where $0<t<1$.

A control point sequence $P_0P_1P_2P_3$ will generate a curve that passes through the end points P_0 and P_3. By fitting consecutive points in this way, the profile of the road is obtained. By varying the value of t between 0 and 1, the intermediate points are obtained. In the algorithm, t is varied by 0.05 i.e. 20 points are generated between two control points. These points are used for calculation of sight distances and determination of passing zones (Nehate, 2002).

4.6 3-D Coordinates Representation

Now that the curve is obtained to make mathematical calculations it is essential that the terrestrial coordinates be converted into Cartesian coordinates. A point in space can be represented by 3-dimensions namely (x,y,z) to determine its position with reference to the origin. Figure 4.5 shows a point P in space with coordinates (x, y, z). OP makes an angle α with the XOZ plane and an angle β with the XOY plane.

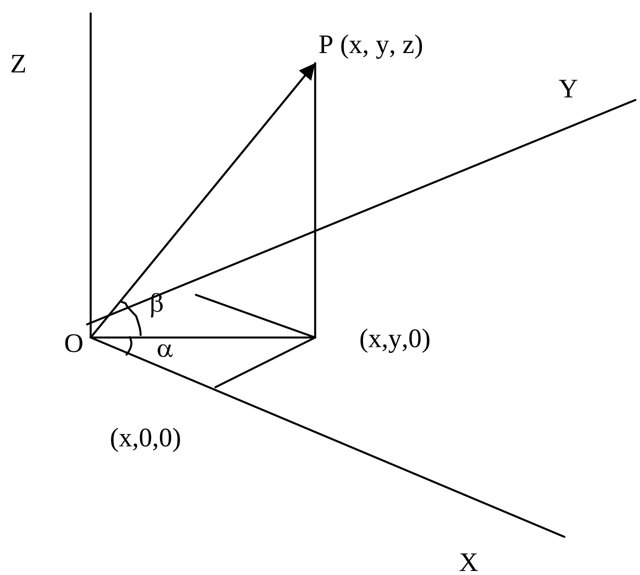

Figure 4.5: 3D-Coordinate Geometry

In Figure 4.6, curve AB represents the road with the start point A and end point B and the travel direction A to B. At the point S, the tangent is represented by dS. dS has components dx, dy and dz in directions X, Y, and Z respectively. dS makes an angle θ in the horizontal plane with the X-Z plane and an angle ϕ in the vertical plane with the X-Y plane. X-Z and X-Y planes are the planes of references for the calculation of angles θ and ϕ.

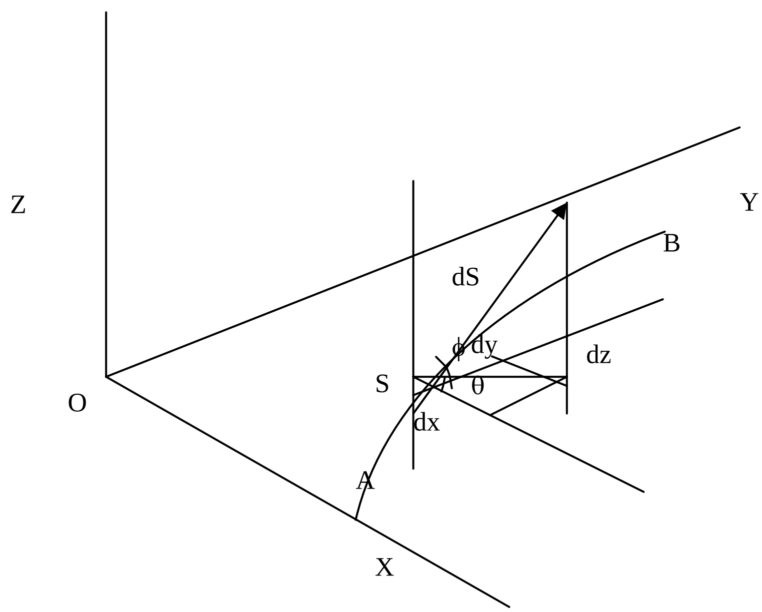

Figure 4.6: 3D Representation of Curve AB

4.6.1 Intermediate Points Calculations, Slopes for Each Point

Equation 4.5 gives the location of the intermediate points by varying the value of t between 0 and 1.

$$S = (x(t), y(t), z(t)) = \frac{1}{6}\begin{bmatrix} t^3 & t^2 & t & 1 \end{bmatrix} \begin{bmatrix} -1 & 3 & -3 & 1 \\ 3 & -6 & 3 & 0 \\ -3 & 0 & 3 & 0 \\ 1 & 4 & 1 & 0 \end{bmatrix} \begin{bmatrix} x_{j-1} & y_{j-1} & z_{j-1} \\ x_j & y_j & z_j \\ x_{j+1} & y_{j+1} & z_{j+1} \\ x_{j+2} & y_{j+2} & z_{j+2} \end{bmatrix} \qquad \text{(Equ. 4.5)}$$

where 0<t<1.

By taking the differential of the equation 4.5, we get the components in the three directions x, y and z at each point as shown in equation 4.6.

$$dS = (dx(t), dy(t), dz(t)) = \frac{1}{6}\begin{bmatrix} 3t^2 & 2t & 1 & 0 \end{bmatrix}\begin{bmatrix} -1 & 3 & -3 & 1 \\ 3 & -6 & 3 & 0 \\ -3 & 0 & 3 & 0 \\ 1 & 4 & 1 & 0 \end{bmatrix}\begin{bmatrix} x_{j-1} & y_{j-1} & z_{j-1} \\ x_j & y_j & z_j \\ x_{j+1} & y_{j+1} & z_{j+1} \\ x_{j+2} & y_{j+2} & z_{j+2} \end{bmatrix} \quad \text{(Equ 4.6)}$$

where 0<t<1.

For example, we have data for a section of Highway US 77 in Riley County as shown in Table 4.2.

Table 4.2: Data for a Section of Highway US-77 in Riley County, Kansas

	X (m)	Y (m)	Z (m)
P_{j-1}	35.373	2185.9	357.98
P_j	35.468	2164.9	357.9
P_{j+1}	35.601	2143.8	357.8
P_{j+2}	35.752	2121.6	357.71

For t = 0.05, substituting in equation 4.5, we get

(x, y, z) = (35.5, 2163.8, 357.9)

and substituting in equation 4.6, we get

(dx, dy, dz) = (0.1159, -21.0562, -0.0910)

Converting the components of derivatives from Cartesian to spherical coordinates, we get the values of ϕ and θ as

$(\theta, \phi) = (-1.5647, -0.004557)$

Note that the values of ϕ and θ are in radians.

Using the above logic for calculations of intermediate points and angles, data for the entire road are calculated and stored in computer generated memory arrays.

The length of the road from the first point is calculated by an incremental method. The increment in length between two consecutive points is calculated and added to the length, giving the length of the road from the first point. This length is also stored in an array. Thus, the distance between any two points can be easily found by just subtracting the values of the length at each of these points.

The increment in length (l) between two consecutive points $A(x_a, y_a, z_a)$ and $B(x_b, y_b, z_b)$ is calculated as

$$l = \sqrt{(x_b - x_a)^2 + (y_b - y_a)^2 + (z_b - z_a)^2} \qquad \text{(Equ 4.7)}$$

The total length obtained from this method is approximately equal to the length of the curve AB when the increment is very small (Nehate, 2002).

4.7 Determination of the Required PSD

The passing maneuver is one of the most significant yet complex and important driving tasks. This process is difficult to quantify because of the many stages involved and the length of the section of road that typically is needed to complete the maneuver. There are several factors that influence the PSD like the volumes of through and opposing traffics, speed differential between the passing and the passed vehicles, highway geometry etc. But one important factor to be taken into consideration is the type of pass drivers make – A flying pass or a delayed pass.

4.7.1 Flying Pass

These are maneuvers that take place when a passing vehicle “flies by” a slower vehicle at a considerably higher speed, without-any follow-up procedure. In other words, a flying pass is the one in which the passing vehicle is not required to lower its speed before passing an impending vehicle (Forbes, 1990). This kind of a pass is also referred to as “accelerative passing” (Polus et al., 2000).

4.7.2 Delayed Pass

A delayed pass occurs when the passing vehicle slows down and follows the impending vehicle because of an opposing vehicle or the absence of a passing opportunity. In a delayed pass, the passing vehicle slows down, trails the slower vehicle and then accelerates (Polus et al., 2000).

The suggested PSDs for the marking of no-passing zones are a compromise between the PSDs for the delayed and the flying pass for most models. Both the delayed and flying passes occur frequently (Forbes, 1990). In the current method, delayed pass is used as a factor in the model since using a flying pass could result in extremely hazardous accidents.

4.7.3 Calculation of the Required PSD

The required PSD is calculated based on the discussion and formulas given in chapter 2. The minimum distance required for overtaking is calculated as the sum of four components.

d_1 – Distance traversed during the perception and reaction time and during the initial acceleration to the point of encroachment on the left lane.

$$d_1 = 1.47\, t_1 \left(v - m + (a\, t_1 /2)\right) \qquad \text{(Equ 2.1)}$$

where:

t_1 = time of initial maneuver, sec;
a = average acceleration, mph/sec;

v = average speed of passing vehicle, mph; and
m = difference in speed of passed vehicle and passing vehicle, mph.

For illustration the parameters of highway 18 in Riley County are given below:

t_1 = time of initial maneuver, sec;
a = 1.47 mph/sec;
v = 54 mph; and
m = 10 mph

d_1 for this highway would be $= 1.47 * 4.3 (54 – 10 + (1.47* 4.3 /2)$
$= 298.10$ feet
d_2 – Distance traveled while the passing vehicle occupies the left lane.

$$d_2 = 1.47\, v\, t_2 \qquad \text{(Equ 2.2)}$$

where:
t_2 = time passing vehicle occupies the left lane, sec; and
v = average speed of the passing vehicle, mph.

Substituting the values t_2 & v based on AASHTO 2001 standards,
t_2 = 10.7 sec; and
v = 54 mph

d_2 for this highway would be $=1.47 *54* 10.7$
$= 849.36$ feet

d_3 – Distance between the passing vehicle at the end of its maneuver and the opposing vehicle.
d_3 = 100, 180, 250 or 300 feet depending on the speed group

Substituting the value based on AASHTO 2001 standards,
d_3 for this highway would be = 250 feet
d_4 – Distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane

$$d_4 =(2/3)* d_2 \qquad \text{(Equ 2.3)}$$

Using these parameters for the analysis of highway 18 in Riley County,
d_2 = 849.36 feet; and
d_4 for this highway would be $= (2/3)*849.36$
$= 566.24$ feet

Total minimum length of a passing zone $= D = d_1 + d_2 + d_3 + d_4$ (Equ 4.8)

The total minimum length of a passing zone on Highway 18 in Riley County according to the analysis should be = D = 298.10 + 849.36 + 250 + 566.24 = 1963.7 feet

4.8 Availability of Passing Zones on the Highways

The next step would be to check for the availability of the sight distance for a length that is equal to or greater than the minimum passing sight distance along the road. The available sight distance on vertical alignments may be restricted at daytime by the road surface and at nighttime by the headlight beam of the vehicle. On horizontal alignments, the sight distance may be governed by the lateral obstructions. Sight distance is calculated at each point for vertical as well as horizontal alignments and the lower bound is taken as the available sight distance at that point. In these calculations, the eye height and the object height on the road surface are applied as 3.5 ft. Since our objective is to have a clear line of sight to a vehicle (smallest is a motorcycle with a driver on it), object height is considered as 3.5 ft. In most calculations, an average vehicle height on the road surface is accepted as 4.25, however a lower value like 3.5 feet will assist to ensure in finding a safe curve length. The required length of the road ahead of the driver and visible to him/her for an average speed group of 50 to 60 mph is 912 ft (Tayfun, 1998). The following procedure is used for calculating the sight distance for a highway with respect to the vertical geometry and horizontal geometry (Nehate, 2002).

4.8.1 Sight Distance on Crest Vertical Curves

The road itself may obstruct the sight line if the vehicle should pass over a crest vertical curve. The road cannot obstruct the sightline unless there is a crest curve, a sag curve or a tangent segment. The available sight distance using vertical geometry is determined assuming an initial value of 0 for the sight distance. The following steps explain this procedure with reference to Figure 4.7.

1. Determine the coordinates of the point A at which sight distance is to be calculated. The coordinates of P are then calculated by adding the height of driver's eye (3.5 feet).
2. Determine the point B along the path such that the line PB and the tangent at B coincide. An iterative procedure is followed to get the location of B. The angle made by the line PB with the horizontal plane i.e. ϕ_{PB} is found and compared with the angle ϕ_B made by the tangent at B with the horizontal plane. Here, some approximate means of determining whether the condition is met is necessary. For this purpose, the following approximation is used:

$$|\phi_{PB} - \phi_B| < \delta \qquad \text{(Equ 4.9)}$$

for some small value of tolerance δ.

1% tolerance means that any answer, which is within 1% deviation in either direction from the correct answer, will be accepted as correct. In the proposed application developed, tolerance value of 0.001 is accepted to give reasonable results.

A sight line is thus established which is obstructed by the road surface itself.

If no point of intersection is found till a distance of 400 m, then the sight distance is assumed as 400 m.

3. Point C is determined when the following condition is met. The condition being the tangent at B should coincide with line BQ. An iterative procedure is followed to get the location of C. The coordinates of point Q are found by adding the height of the car or motor cycle (3.5 feet) to a point C. Again, the angle made by line BQ with the horizontal plane i.e. ϕ_{BQ} is found and compared with the angle ϕ_B made by the tangent at B with the horizontal plane. For this purpose, the following approximation is used:

$$|\phi_{BQ} - \phi_B| < \delta \qquad \text{(Equ 4.10)}$$

where δ is the tolerance.

4. The sight distance is the length AC along the road.

Since the above method only makes use of ϕ, it is similar to taking vertical projections of the road for the calculations and makes no difference even if the road has horizontal curvature at the same time.

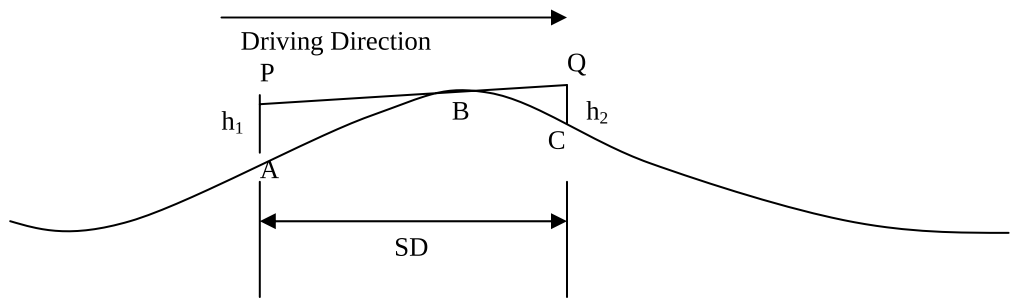

Figure 4.7: Sight Distance Limited by Crest Vertical Curve (Profile view)
(h_1, height of driver's eye; h_2, height of car; SD, sight distance).

4.8.2 Sight Distance on SAG Vertical Curves

At nighttime, the available SSD on sag vertical curves is limited to the farthest point lighted by the vehicle's headlight. Fig 4.8 shows the graphical methodology followed for finding the sight distance limited by headlight on sag vertical curves. A simple procedure, as given below is followed to find the sight distance due to headlight control.

1. Determine the coordinates of the point A at which sight distance is to be calculated. The coordinates of P are then calculated by adding the height of driver's eye.
2. Determine the point B along the path such that the line PB makes an angle equal to sum of the measures of the angle made by the tangent at point A with the horizontal plane i.e. ϕ and the measure of the angle β made by the headlight in the upward direction. An iterative procedure is followed to get the location of B. The angle made by the line PB with the horizontal plane i.e. ϕ_{PB} is found and compared with the angle $(\phi+\beta)$ made at A with the horizontal plane. Therefore, some approximate means of determining whether the condition is met is necessary. For this purpose, the following approximation is used:

$$\phi_{PB} - (\phi+\beta)| < \delta \qquad \text{(Equ 4.11)}$$

Here δ is the tolerance. A sight line is thus established which is obstructed by the road surface itself. If no point of intersection is found till a distance of 400 m, then the sight distance is assumed as 400 m.

3. The sight distance is length AB along the road.

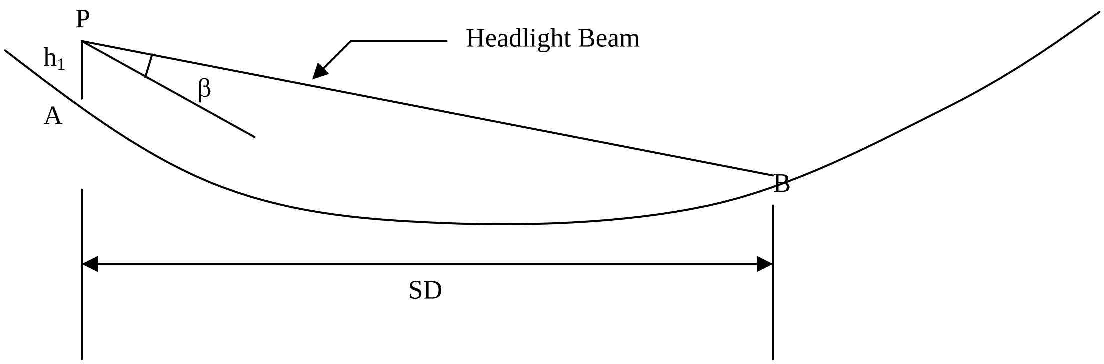

Figure 4.8: Sight Distance Limited by Headlight on Sag Vertical Curves in Profile View
(h1, height of vehicle's headlight; β, angle of headlight in upward direction; SD, sight distance)

4.9 Sight Distance on Horizontal Alignments

When the sight distance is limited by lateral obstructions on horizontal alignments, the following notation is being used:

d_left = distance of the lateral obstruction on the left from center of the driving lane.
d_right = distance of the lateral obstruction on the right from center of the driving lane.

For a two way lane as shown in Figure 4.9,

d_left = clear zone + lane_width*1.5
d_right = clear zone + lane_width*0.5

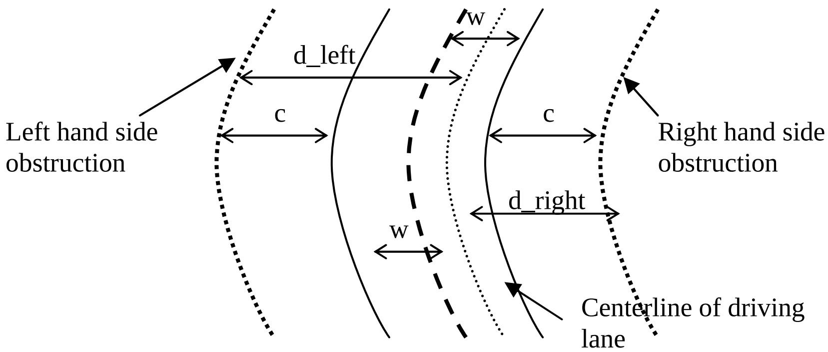

Figure 4.9: Roadway Dimensions in Plan View for a Two Way Lane
(w: lane width; c: clear zone; d_left: distance from left obstruction; d_right: distance from right obstruction)

The coordinates of the left and right side lateral obstructions for the point A are given by (x_l, y_l) and (x_r, y_r). Fig 4.10 shows the diagrammatic representation of the road and the tangent at point A in the horizontal plane for $0 \leq \theta \leq \pi/2$.

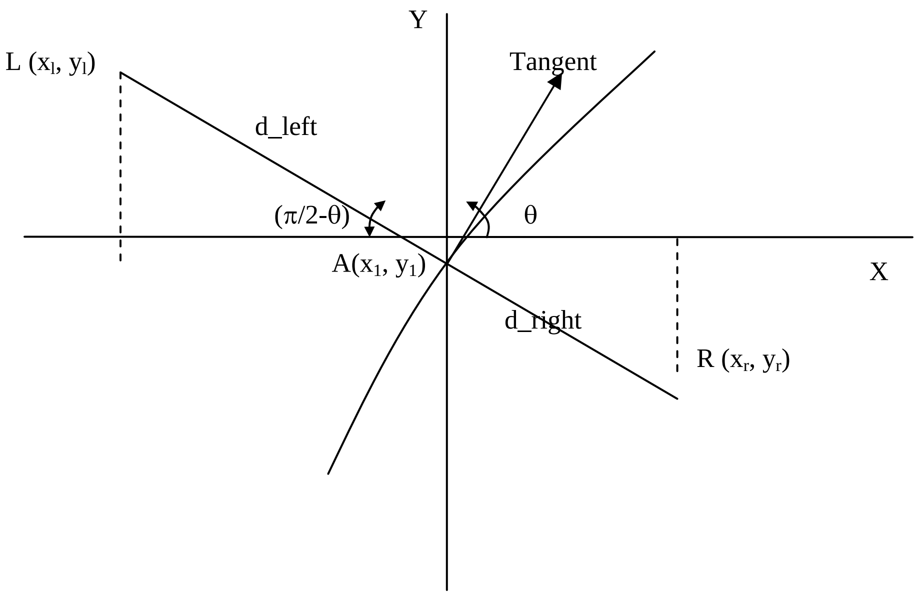

Figure 4.10: Calculation of the Coordinates of the Left and Right Side Obstructions for $0 \leq \theta \leq \pi/2$

For $0 \leq \theta \leq \pi/2$, (x_l, y_l) and (x_r, y_r) are calculated as:

$x_r = x_1 + d\_right*\cos(\pi/2 - \theta)$ (Equ 4.12a)

$y_r = y_1 - d\_right*\sin(\pi/2 - \theta)$ (Equ 4.12b)

$x_l = x_1 - d\_left*\cos(\pi/2 - \theta)$ (Equ 4.13a)

$y_l = y_1 + d\_left*\sin(\pi/2 - \theta)$ (Equ 4.13b)

These equations change depending upon the angle θ.

For $\pi/2 < \theta \leq \pi$, (x_l, y_l) and (x_r, y_r) are calculated as:

$x_r = x_1 + d\_right*\cos(\theta - \pi/2)$ (Equ 4.14a)

$y_r = y_1 + d\_right*\sin(\theta - \pi/2)$ (Equ 4.14b)

$x_l = x_1 - d\_left*\cos(\theta - \pi/2)$ (Equ 4.15a)

$y_l = y_1 - d\_left*\sin(\theta - \pi/2)$ (Equ 4.15b)

For $\pi < \theta \leq 3\pi/2$, (x_l, y_l) and (x_r, y_r) are calculated as:

$x_r = x_1 - d\_right*\cos(3\pi/2 - \theta)$ (Equ 4.16a)

$y_r = y_1 + d\_right*\sin(3\pi/2 - \theta)$ (Equ 4.16b)

$x_l = x_1 + d\_left*\cos(3\pi/2 - \theta)$ (Equ 4.17a)

$y_l = y_1 - d\_left*\sin(3\pi/2 - \theta)$ (Equ 4.17b)

For $3\pi/2 < \theta < 2\pi$. (x_l, y_l) and (x_r, y_r) are calculated as:

$x_r = x_1 - d\_right*\cos(\theta - 3\pi/2)$ (Equ 4.18a)

$y_r = y_1 - d\_right*\sin(\theta - 3\pi/2)$ (Equ 4.18b)

$x_l = x_1 + d\_left*\cos(\theta - 3\pi/2)$ (Equ 4.18c)

$y_l = y_1 + d\_left*\sin(\theta - 3\pi/2)$ (Equ 4.18d)

The lateral obstructions are constructed so that they have the same profile as that of the road. Thus, the slope at L and R is the same as that of A.

The following steps explain the procedure to find the sight distance for horizontal alignment on the right with reference to Figure 4.11.

1. Determine the coordinates of the point A at which sight distance is to be calculated.
2. Determine the point B along the path of lateral obstruction on the right such that the line AB and the tangent at B coincide. An iterative procedure is followed to get the location of B. The location of the lateral obstruction is obtained from the procedure explained above. The angle made by the line AB in the horizontal plane i.e. θ_{AB} is found and compared with the angle θ_B made by the tangent at B in the horizontal plane. Here, some approximate means of determining whether the condition is met is necessary. For this purpose, the following approximation is used:

   $(\theta_{AB} - \theta_B) < \delta$ (Equ 4.19)

   where δ is the tolerance.

   A sight line is thus established which is obstructed by the lateral obstruction on the right. If no point of tangency is found till a distance of 400 m, then the sight distance is assumed as 400 m.
3. Point C is determined when the following condition is met. The condition being the tangent at B should coincide with line BC. An iterative procedure is followed to get the location of C. Again; the angle made by line BC in the horizontal plane i.e. θ_{BC} is found and compared with the angle θ_B made by the tangent at B in the horizontal plane. For this purpose, the following approximation is used:

   $(\theta_{BC} - \theta_B) < \delta$ (Equ 4.20)

   where δ is the tolerance.
4. The sight distance is the length AC along the road.

The same procedure is followed for finding the sight distance for lateral obstruction on the left-hand side. The lower bound of the two values is the sight distance on horizontal alignments.

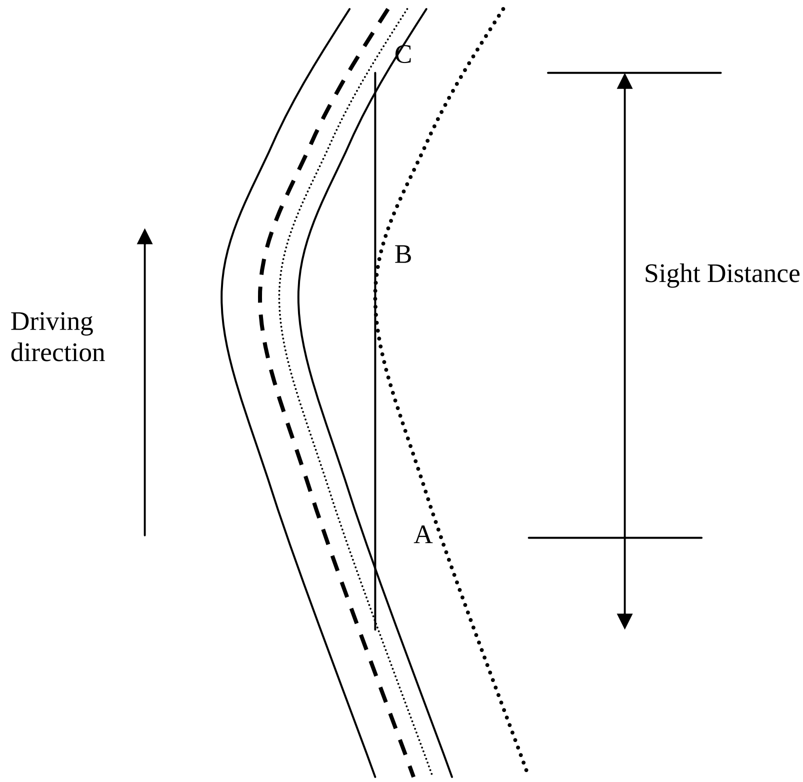

Figure 4.11: Sight Distance Limited by Lateral Obstructions on Horizontal Alignments

4.10 Determining the No Passing Zones

From the calculation of the available sight distances on the highway, no passing zones are identified. The segments of the highway where the available sight distances are greater than or equal to 912 feet (0.172 mile) are identified. From these segments the segments with lengths greater than or equal to the minimum passing sight distances are identified. These segments constitute the passing zones. Likewise segments of the highway where the sight distances are less than 912 are identified. When the distance between the segments of the highways that are no-passing zones is less then the minimum passing sight distance required then the segments are combined to make a single no passing zone. This is in accordance with the AASHTO's recommended value of a passing zone. The final database would contain the no-passing zones for a highway with the following information.

- Beginning of a no passing zone (location of the beginning of the no-passing zone from the start of the highway in miles)
- Ending of the no-passing zone (Location of the ending of the no-passing zone from the start of the highway in miles)
- Direction (For which traffic the highway segment is a no-passing zone, for forward direction or the reverse direction)
- A-index Number (Reference number for each highway segment in Kansas, used by KDOT to identify the highway number and the county)

A sample database for Highway 77 in Riley County for 5.87 miles is shown in Table 4.3.

Table 4.3: Database Showing the No-Passing Zones in Riley County for Highway 70

Beginning of No-Passing Zone (From the start of the highway in miles)	Ending of No-Passing Zone (From the start of the highway in miles)	Direction of Travel (1 –Forward; 2-Reverse)	A-index
0.21	0.74	1	455
3.60	3.80	1	455
3.79	3.98	2	455
0.48	0.92	2	455
0.40	0.44	2	455

4.11 Calculation of the Stopping Sight Distance

The overtaking maneuver can result in hazardous accidents if the sight distance available on the road is not sufficient enough to see any stationary obstructions of heights greater than or equal to

1 foot, which might lie on the road (AASHTO, 2001). So sight distance available on the highway is calculated based on 3-D coordinate geometry. Sight distance for vertical as well as horizontal geometry is calculated. Since these calculations are done in 3-D coordinate geometry, the sight distance at a point is the minimum value of sight distance of vertical and horizontal geometry. The segments of the highways where the required SSD is not available are identified based on previous research done (Namala et al., 2003).

By using the lower bound of both the Passing Sight Distance and the Stopping Sight Distance calculations we get the final no-passing zones. Due to inaccuracy in the GPS data collected there is a possibility of having no-passing zones of length zero miles. The no-passing zones are checked for the lengths if a no-passing zone is of zero length, then it is deleted. A snap shot of no-passing zones for highway #70 in Riley County is given in Table 4.4.

Table 4.4: Final database showing the no passing zones in Riley County for Highway #77

Beginning of Stopper (From the start of the highway in miles)	Ending of Stopper (From the start of the highway in miles)	Direction of Travel (1 –Forward; 2-Reverse)	A-index
0.18	1.58	1	455
3.57	3.82	1	455
5.30	5.36	1	455
0.81	0.81	2	455
0.81	1.76	2	455
3.76	4.01	2	455
5.50	5.56	2	455

A step-by-step explanation of the working of the algorithm would be as shown in Figure 4.12.

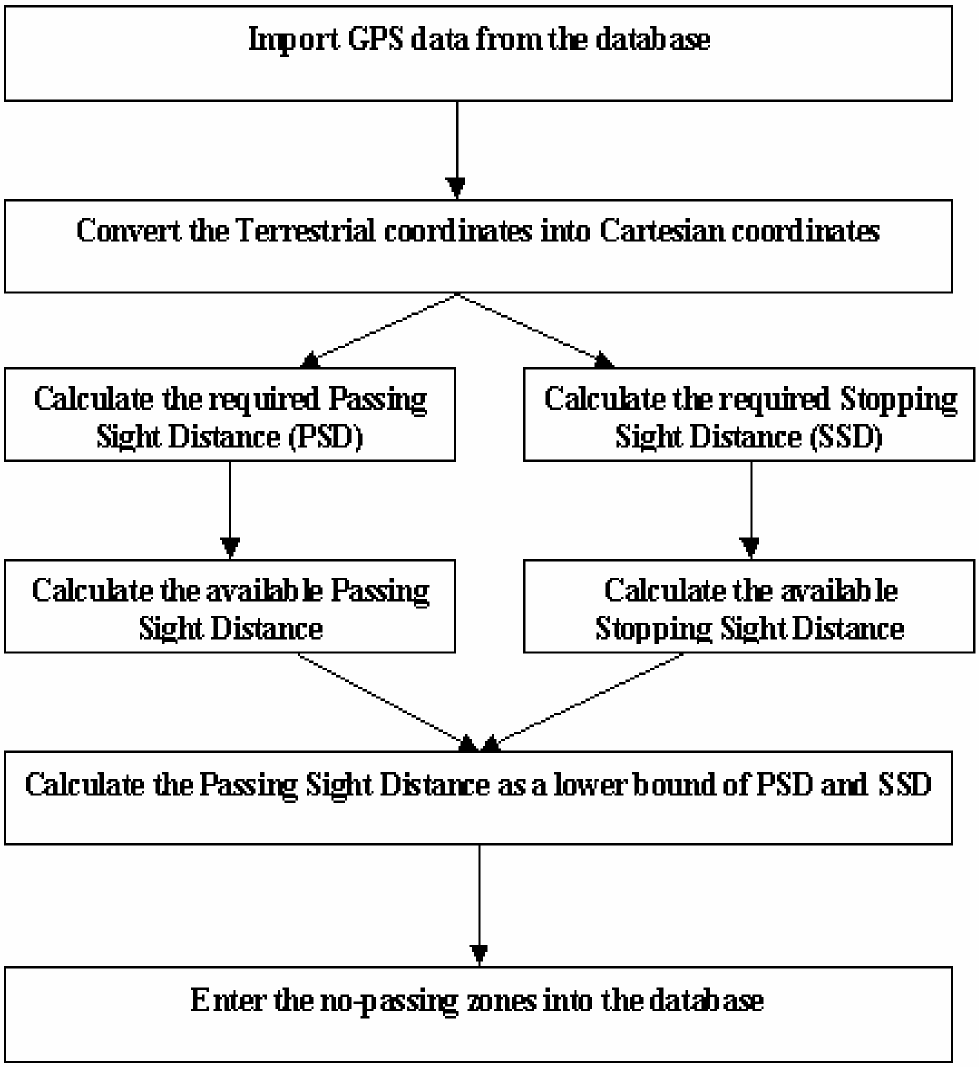

Figure 4.12: Breakup of the Algorithm into Modules

4.12 Discussion

The algorithm has been developed to calculate the PSD and identify the no-passing zones based on AASHTO's standards, however the algorithm can be changed to calculate the passing sight distance and to identify the no-passing zones based on MUTCD standards too. The algorithm has been executed for MUTCD standards and the results obtained are shown in Table 4.5.

<u>Table 4.5: No-Passing Zones on Highway 18 in Riley County based on MUTCD Standards</u>

Beginning of Stopper (From the start of the highway in miles)	Ending of Stopper (From the start of the highway in miles)	Direction of Travel (1 –Forward; 2-Reverse)	A-index
1.85	1.91	1	453
8.18	8.44	1	453
9.13	9.49	1	453
9.77	10.25	1	453
10.97	11.67	1	453
0.72	0.84	2	453
1.55	2.27	2	453
2.85	3.42	2	453
7.86	7.96	2	453
8.36	8.92	2	453
9.07	9.17	2	453

Now the location of the no-passing zones obtained through the algorithm based on MUTCD standards are compared to the existing conditions on the Highway 77 in Riley County. Driving on the highway and taking the readings gives the existing locations of the no-passing zones. A comparison of the two values is given in Table 4.6.

Table 4.6a: Comparing the Results of the Analysis with the Existing Conditions for Highway 18 in Riley County for Forward Direction

KDOT				Experiment		
***Start* (miles)**	***End* (miles)**	***Length* (miles)**		***Start* (miles)**	***End* (miles)**	**Length (miles)**
0.8	2.5	1.7		1.85	1.91	0.6
6.9	7.3	0.4	VS.	8.18	8.44	0.26
7.8	8.3	0.5		9.13	9.49	0.36
8.5	9.4	0.9		9.77	10.25	0.47
X	X	X		10.97	11.67	0.7

Note that the values reported by KDOT database are offset from the values measured in the field. This is because the KDOT geometric base model extents outside of the county for a short distance for any route that crosses the county line. [At the time of the study, the 3D model had not yet been cross referenced to the KDOT Linear Referencing System (LRS system).]

Table 4.6b: Comparing the Results of the Analysis with the Existing Conditions for Highway 18 in Riley County for Reverse Direction

KDOT				Experiment		
Start (in miles)	**End (in miles)**	**Length (in miles)**		**Start (in miles)**	**End (in miles)**	**Length (in miles)**
0.5	0.6	0.1		0.72	0.84	0.12
1	2.7	1.7		1.55	2.27	0.67
				2.85	3.42	0.57
6.9	7.3	0.4	VS.	X	X	X
7.5	7.6	0.1		7.86	7.96	0.1
8	8.5	0.5		8.36	8.92	0.56
8.8	9.4	0.6		9.17	9.17	0.1

From this comparison, it is evident that there is an agreement between the values obtained through the algorithm and those currently used on the roads. However there is a small offset in mileage (usually between 0.05 to 0.2 miles) between that of the experiment values and the existing conditions on the highways, this offset is due to the reasons stated earlier. Another interesting observation is that whenever the highway passes through any town or village, the highway is designated as a no-passing zone, even though the required sight distance and the minimum passing zone length exists.

Chapter 5

Application of the Algorithm

5.1 Algorithm

Computer software is developed to find the Passing Sight Distance based on the theoretical procedures discussed in the previous chapter. The algorithm developed in Matlab v6.5., was used to find the PSD for several highways in Kansas, USA. This algorithm can be used to find the PSD based on AASHTO and MUTCD standards. GPS data obtained from KDOT was used for the analysis. KDOT has GPS data for each highway in Kansas for every year since 1997. The 2003 data obtained was used for the analysis and where there was no sufficient data, 2002's data was used.

5.2 Testing the PSD Algorithm

In the determination of the no-passing zones, KDOT uses a manual method called the "Walking Method" which involves two employees walking long the road separated by a rope equal in length to the Passing Sight Distance according to MUTCD standards. But the current algorithm calculates the PSD based on the AASHTO standards. Hence this model was tested for both AASHTO and MUTCD standards. The results obtained by this analysis were compared to the existing conditions on 10 highway segments and the results were in mutual agreement.

5.2.1 Illustration for US-77 in Riley County

The algorithm is run as per the AASHTO 2001 standards on the highway US-77 in Riley County, Kansas. Table 5.1 gives the parameters used for the application example.

Table 5.1: Parameters Used Calculating the PSD – AASHTO 2001 Standards

Parameter	Value
Height of driver's eye	1080 mm (3.5 ft)
Height of object for PSD	1080 mm (3.5 ft)
Height of object for SSD	600 mm (2 ft)
Height of vehicle's headlight	600 mm (2 ft)
Angle of headlight beam	1°
Lane width	3.6 m (12 ft)
Clear zone	9.1 m (30 ft)

Figure 5.1 shows the location of the no passing zones on highway US-77 in Riley County according to the AASHTO 2001. The figure shows the length of the road in miles from start along X-axis and the elevation in feet along the Y-axis. The profile of the road is plotted in black color. The segments of the road visible in yellow color (as solid yellow lines along the profile) represent the no-passing zones for the traffic traveling in direction 1.

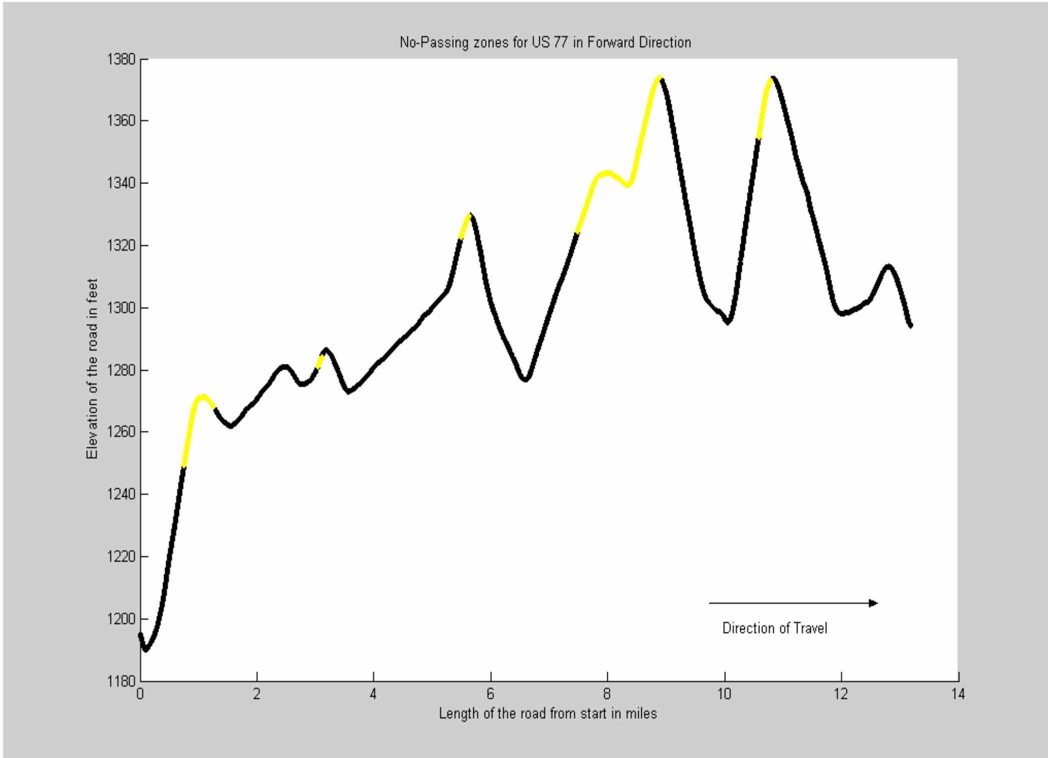

Figure 5.1: Location of No-Passing Zones in US-77 in Riley County for Forward Direction

Likewise Figure 5.2 shows the no-passing zones for the same highway in the reverse direction (direction 2). As the Figure 5.1 shows most of the no-passing zones are due to the vertical geometry of the earth in which the road surface itself blocks the view of the driver. At the 8.5 mile mark the no-passing zone is due to the presence of sag on the road.

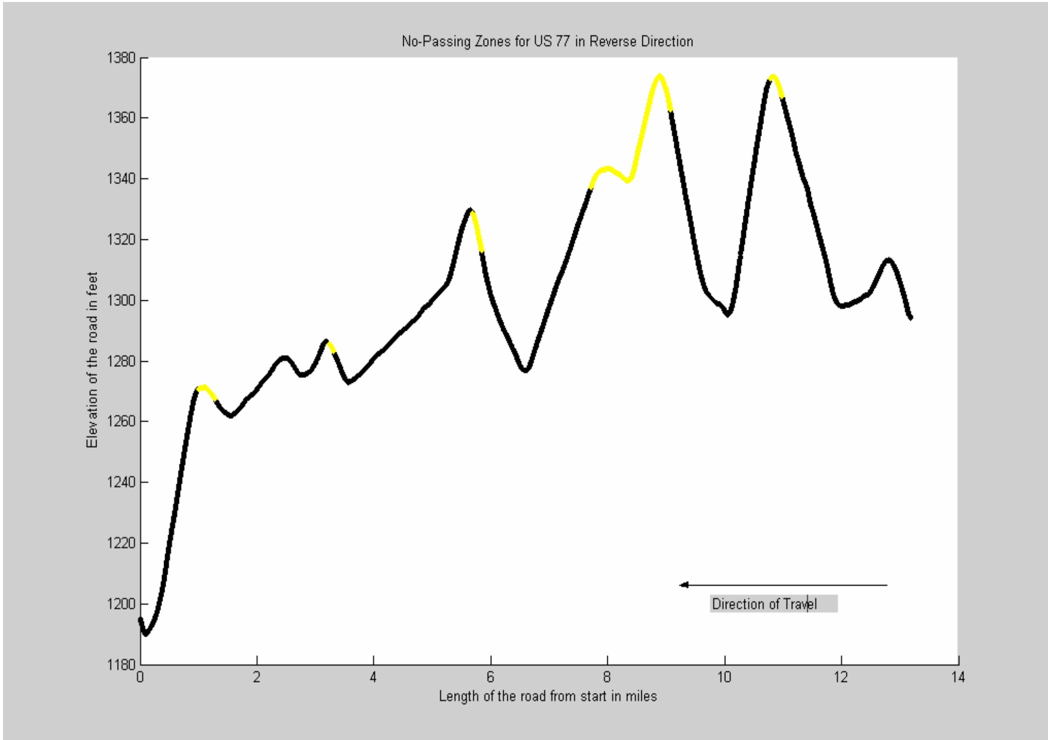

Figure 5.2: Location of No-Passing Zones in US-77 in Riley County for Reverse Direction

The no-passing zones for the section of highway US-77 in Riley County for both directions are as shown in Figure 5.3. The profile of the road is shown in black color. If a no passing zone is in the forward direction, it is shown in blue color. If a no passing zone is in the reverse direction, then it is shown in light blue color. Some times there exist no passing zones for traffic traveling in both directions, if a no passing zone is in both directions it is shown in red color. The results obtained are reasonable because the no-passing zones exist only when the road surface blocks the view of the driver or there is a sag curve obstructing the view of the driver during the nighttime.

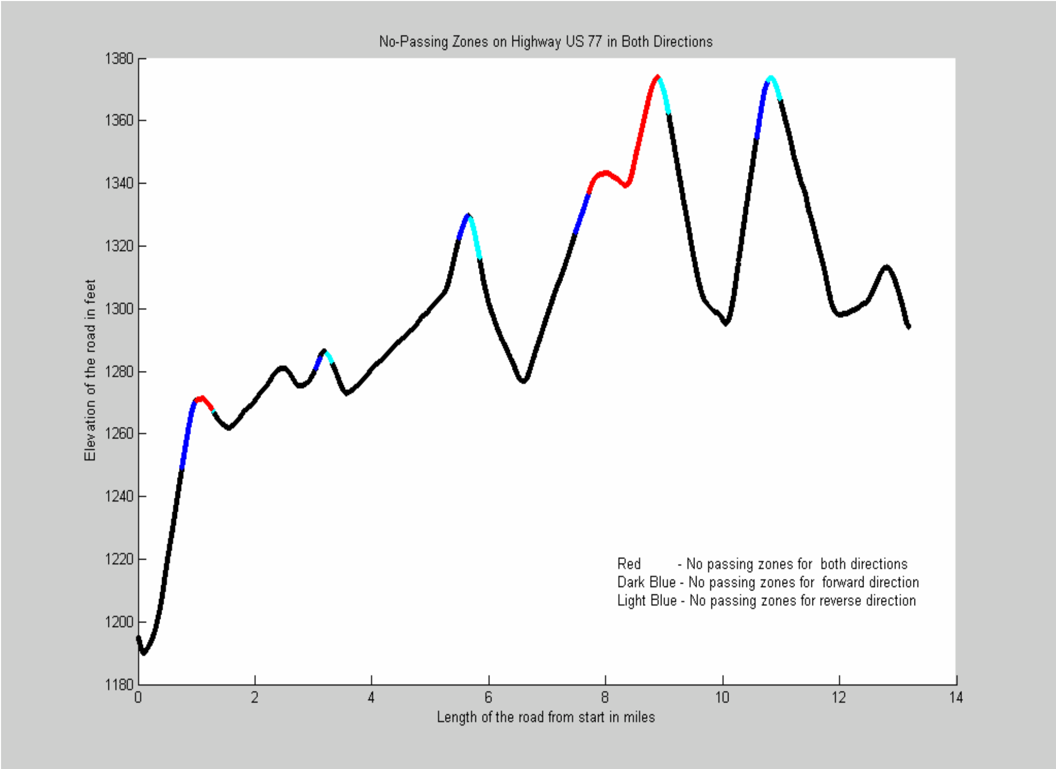

Figure 5.3: Location of No-Passing Zones in US-77 in Riley County

5.2.2 Testing the Algorithm

Testing the results obtained from the algorithm was difficult since KDOT had no database available which contained information on the no passing zones for the highways. Hence to check the results, a car was driven on the highway and the location of the no passing zones were taken for both directions by traveling along the highway from one end to the other end of the highway in a county. The values obtained from the field studies were based on MUTCD standards; hence the algorithm was run based on MUTCD standards. The algorithm was tested for ten highways and compared. Table 5.2 gives a comparison of the results obtained through the algorithm and the existing locations of the no passing zones for one such highway segment in Riley County.

Table 5.2: Comparison of No Passing Zones for a Section of US-24 in Riley County

Experiment Results				Existing Locations	
Starting Point (Miles)	**Ending Point (Miles)**	**Direction**	**A-index**	**Starting Point (Miles)**	**Ending Point (Miles)**
0.00	0.53	1	454	0	0.5
1.08	1.20	1	454	1	1.2
2.03	2.17	1	454	X	X
2.40	2.59	1	454	2.4	2.8
2.98	3.44	1	454	X	X
3.76	4.28	1	454	3.7	3.9

5.2.3 Discussion of the Results

As shown in the above table, there is a general agreement in the results obtained from the analysis using theoretical procedures with that of the existing ones. However, it can be observed that the values obtained through the analysis are slightly greater than those of the existing ones. According to the analysis, some no passing zones generated from our model do not exist on the road currently. The reason for this to occur is due to the conservativeness of the model. In this model, no passing zones are determined not only by using PSD criterion but also the SSD criterion. This is done to provide safety to the over taking vehicles not only from the approaching vehicles but also from any stationary objects of considerable height (2 feet according to AASHTO, 2001) on the surface of the road and can cause accident to the driver if not taken into consideration. Having this criterion in the design of the model ensures safety to the driver during the overtaking maneuver.

Chapter 6

Conclusions

6.1 Conclusions

An overtaking maneuver is a critical stage of the traffic flow on two-lane roads. The accidents occurred during overtaking maneuvers are equal to about 10% of the total, with a slightly increasing trend in the past few years (Source: Istat 1992 – 1997). The fact of guaranteeing some segments on a two-lane road where overtaking can be safely made also means improving traffic flow quality (Crisman, 2000).

The current methodology used by KDOT to determine the no-passing zones is called the ‘Walking Method’ discussed in section 2.6.1. But this method is time consuming and expensive. Since it is done manually there is every chance of making mistakes in the identification of passing zones due to human errors. The purpose of this research was to provide an efficient way to identify the no-passing zones and also to improve the safety and efficiency of two-lane, two-way highways through improved procedures for establishing no-passing zones. Since KDOT has GPS data, this data used to identify segments of the highway where the sight distance available is less than that required to either complete/abort a pass or to stop the vehicle due to any dangerous obstruction on the road. Such segments were identified and reported to ensure safety on highways while overtaking. The available sight distance is estimated analytically by examining the intersection between the sight line and the elements representing the highway surface and sight obstructions. A profile of available sight distance is thus established and used to evaluate sight distance deficiency. The entire procedure has been automated and the no-passing zones can be determined for all the highway segments in the state of Kansas. The computer software developed can be used to assist the highway designers and professionals locate the actual passing and no-passing segments.

6.2 Scope for Future Research

Another version of the algorithm can identify the no-passing zones on a highway by taking into consideration the following parameters

- Velocity of the passing vehicle at the start of pass,
- Maximum velocity achieved,
- Speed of the impending vehicle,
- Speed of the opposing vehicle,
- Maximum acceleration of passing vehicle,
- Clearance between the head of impending vehicle and tail of the passing vehicle at the end of the pass,
- Clearance between the head of passing vehicle and the tail of the impending vehicle at the start position,
- Length of impending vehicle,
- Length of the passing vehicle,
- Clearance between the heads of passing vehicle and the opposing vehicle at the end of the pass and width of the lane.

Variation of each of these parameters has an affect in the determination of PSD and this can be obtained by executing the algorithm.

The research presents a mathematical model for the overtaking maneuver of two vehicles driven by drivers. But this scope can be extended to include even the Intelligent Transportation Systems (ITS). It requires automation of the control of the vehicles. Especially the steering, acceleration, and braking of the vehicles should be automatically controlled. This would require a detailed description of the PSD procedure including steering and speed controls. Since extensive research has already been done in SSD and PSD, more research can be carried out in

Decision Sight Distance (DSD). Decision sight distance is the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its potential threat, select an appropriate speed and path, and initiate and complete the required safety maneuver safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to maneuver their vehicles at the same or reduced speed rather than to just stop, its values are substantially greater than the stopping sight distance (AASHTO, 2001). It is up to the engineer to decide when to use the decision sight distance. Providing the extra sight distance will probably increase the cost of a project, but it will also increase safety. The decision sight distance should be provided in those areas that need the extra margin of safety, but it isn't needed continuously in those areas that don't contain potential hazards.

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