

# Accounting for Risk and Level of Service in the Design of Passing Sight Distances

by

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## Abstract

Current design methods in transportation engineering do not simultaneously address the levels of risk and service associated with the design and use of various highway geometric elements. Passing sight distance (PSD) is an example of a geometric element designed with no risk measures. PSD is provided to ensure the safety of passing maneuvers on two-lane roads. Many variables decide the minimum length required for a safe passing maneuver. These are random variables and represent a wide range of human and vehicle characteristics. Also, current PSD design practices replace these random variables by single-value means in the calculation process, disregarding their inherent variations.

The research focuses on three main objectives. The first goal is to derive a PSD distribution that accounts for the variations in the contributing parameters. Two models are devised for this purpose, a Monte-Carlo simulation model and a closed form analytical estimation model. The results of both models verify each other and differ by less than 5 percent. Using the PSD distribution, the reliability index of the current PSD criteria are assessed.

The second goal is to attach risk indices to the various PSD lengths of the obtained distribution. A unique microscopic simulation is devised to replicate passing maneuvers on two-lane roads. Using the simulation results, the author is able to assess the risk of various PSD lengths for a specific design speed. The risk index of the AASHTO Green Book and the MUTCD PSD standards are also obtained using simulation.

With risk measures attached to the PSD lengths, a trade-off analysis between the level of service and risk is feasible to accomplish. The last task is concerned with applying the Highway Capacity Manual concepts to assessing the service measures of the different PSD lengths. The results of the final trade-off analysis show that for a design speed of 50 mph, the AASHTO Green Book and the MUTCD standards overestimate the PSD requirements. The criteria can be reduced to 725 ft and still be within an acceptable risk level.

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*John El Khoury*

*To The Memory of My Father, Said*

*To The Hard Work of My Mother, Layla*

*To My Lovely Sister, Jacky*

*And Finally To My Dear Brothers, Jack & Joud*

*For Their Continuous Love and Support*

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## **CHAPTER 1:**

### **INTRODUCTION**

## **1.1 Introduction**

Passing sight distance (PSD) is the distance traveled by a driver while trying to pass a slower vehicle ahead on a two-lane road. It is provided to ensure that passing vehicles have a clear view ahead for a sufficient distance to minimize the possibility of a collision with an opposing vehicle (Harwood and Glennon 1989). Thus, PSD is designed to guarantee the safety of the passing maneuvers on two-lane roads. Mainly, the design criteria of the PSD define the standards by which passing and no-passing zones are marked. Those design criteria outlined in the AASHTO's "Green Book" *A Policy on the Geometric Design of Highways and Streets* and the operational criteria as stated in the FHWA *Manual on Uniform Traffic Devices for Streets and Highways* (MUTCD) have remained virtually unchanged for more than five decades. Besides, the two criteria are inconsistent with each other. They also provide different minimum PSD for similar conditions. So, research was initiated by NCHRP to provide consistent PSD standards that will be valuable for highway design and operations nationwide. The new standards will be considered for incorporation in future editions of both the Green Book and the MUTCD.

Design standards provide a benchmark for the development of elements that compose a highway design. Ideally, every highway design would meet the appropriate standards. Realistically, designers are sometimes faced with situations where adherence to standards may not be practical from an engineering, environmental, community, or benefit-cost perspective. In such cases, designers must make decisions regarding the impacts and risks associated with meeting or exceeding the design standards or allowing exceptions to them.

## **1.2 Background**

Researchers over the past three decades have recognized the inconsistencies between AASHTO and MUTCD policies and have investigated alternative formulations of PSD criteria. These investigators retained the overall deterministic structure of the "Green Book" models and improved on them by including the concept of critical position, where the sight distances required to abort the pass and to complete it are equal. Harwood et al. (2003) in their NCHRP report 15-21; *Review of Truck characteristics as Features of Roadway Design* studied the previous research done on PSD and concluded that the Glennon model (1988) provides the best safety conservative approach for marking passing zones on two-lane highways. They revised the model to make it applicable to trucks and used it to determine the sight distance requirements for passing by trucks. In one of their conclusions, they state:

“In order to complete a passing maneuver at speeds of 100 km/h or more under the stated assumptions, trucks require passing zones at least 610 m long. There are relatively few such passing zones on two-lane highways and, yet, trucks regularly make passing maneuvers. The explanation of this apparent paradox is that, since there are very few locations where a truck can safely make a delayed pass<sup>\*</sup>, truck drivers seldom attempt them. Most passing maneuvers by trucks on two-lane highways are flying passes that require less passing zone length than the delayed passes. Thus, there may be no need to change current PSD criteria to accommodate a truck passing a passenger car or a truck passing a truck. It makes little sense to provide enough PSD for delayed passes by trucks when passing zones are not generally long enough to permit such maneuvers.”

In other words, if the proper truck PSD is used very few zones will be marked as passing zones on the two-lane highways, thus decreasing the level of service to the traveling public. However if shorter PSD is used, then trucks will be subjected to collision risks if they passed, and consequently will decrease the safety level of the road.

The question that needs to be raised is how to make a trade-off analysis between safety and level of service in determining the minimum PSD on a particular two-lane road? This question addresses the main intent of providing the PSD, which is to ensure the *safety* of the passing driver, and provide an adequate *level-of-service* to the traveling public. Driver safety could be measured by the risk level encountered in conducting the passing maneuver. It is characterized by the probability of being involved in a collision or a crash with the opposing vehicle. The level of service concept could be measured by the average travel speed of all drivers on that road section and by the percentage of time that these drivers spend following slower vehicles on a particular two-lane road. Both these parameters are well discussed in the Highway Capacity Manual 2000. The latter parameter is denoted by the Percent-Time-Spent-Following (PTSF). It's importance is elaborated in the AASHTO's Green Book (2004):

“Sight distance adequate for passing should be encountered frequently on two-lane highways.....Frequency and length of passing sections for highways depend principally on the topography, the design speed...and the cost.....The importance of passing sections is illustrated by their effect on the service volume of a two-lane, two-way highway. Table 8.1 of the HCM (1985) shows, for example, that, for an average travel speed of 90 Km/h over level terrain the service flow rate is reduced from approximately

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\* Delayed pass: the passing driver slows down to the speed of the impeding vehicle before initiating a pass.

760 passenger cars per hour where there are no sight restrictions to about 530 where PSD is available on only 40 percent of the highway.”

The Highway Capacity Manual (2000) also emphasizes the importance of providing adequate/enough passing zones on two lane highways in Chapter 20 (HCM 2000). The calculation of the LOS of the highway is affected by the percent of no-passing zones. Recently, the effects of time drivers spend following slower traffic are being researched to quantify their impacts on the traffic stream. Moshe Pollatschek and Abishai Polus are trying to model the impatience of drivers in passing maneuvers on two lane highways. In short, they have tried to quantify the impatience level of drivers based on the amount of time they are delayed while following slower traffic. Drivers become more aggressive, and thus take higher risks, in conducting a passing maneuver had they been trailing for a considerable amount of time. This projects a clear idea of the importance of providing enough passing zones, especially on two lane highways that stretch for miles. Perhaps the impatience of drivers who are trailing slower traffic within no-passing zones is causing drivers to take riskier pass attempts, which explains the high fatality rates within no-passing zones. Table 1.1 presents the crash statistics of the state of Virginia using the DMV website and the Fatality Analysis Reporting System (FARS) provided by the National highway Traffic Safety Administration (NHTSA).

**Table 1.1 – Traffic Crash Statistics of Virginia (DMV and FARS Databases)**

Year	1999	2000	2001	2002	2003	Mean
Total # of Accidents	139573	141650	144585	147737	154848	145679
Accidents within No-passing Zones	11147	11183	11401	11611	12047	11478
% Accidents within No-passing Zones	8.0%	7.9%	7.9%	7.9%	7.8%	7.9%
Total Fatal Accidents	795	851	855	830	861	838
Fatal Accidents within No-passing Zones	172	168	204	172	185	180
% Fatal Accidents within No-passing Zones	21.6%	19.7%	23.9%	20.7%	21.5%	21.5%

Previous research did not address the concepts of safety and level of service simultaneously, and the trade off between them in determining the PSD. They only considered the worst case scenario and determined the minimum safe PSD accordingly. The considered scenarios did not encompass all possible conditions and parameters, such as driver characteristics, roadway characteristics, vehicle characteristics and traffic characteristics. For example the traffic volume using the roadway is not considered. Hence, the level of service concept could not be implemented. The vehicle characteristics are not considered which may influence the vehicle acceleration and its passing speed and consequently the length of PSD. In other words, the current design criteria are not descriptive of the passing conditions. Also, current design requirements are based on criteria obtained during 1938 to 1941. It is clear that

vehicle dynamics have changed dramatically, rendering vehicles much more reliable, responsive, and easily maneuverable. Acceleration and deceleration rates have changed drastically, not to mention engine power and efficiency.

In addition, the criteria used for the current PSD standards assume averages rather than the whole distribution of the influencing parameters. That is, single values are replaced into the formulation to represent, for example, the driver perception reaction time, the passing/passed/opposing vehicles' speeds, the clearance and gap distance, the vehicle length, the acceleration/deceleration rates of vehicles. Vehicle types and properties may add up to hundred(s), rendering acceleration /deceleration performances extremely variable. Even vehicle lengths vary sharply among vehicle classes and types. Still, vehicle properties are fortunately countable and can still be represented by simple distributions, sometimes even discrete probability functions. However, human properties and drivers' characteristics are infinitely variable. No single value can closely represent all groups of drivers' perception reaction time, speed preference, or following distance. These need to be represented by continuous probability functions. For this reason, no single PSD can be the minimum "safe" PSD for all drivers. There exists a variation of the minimum PSD that is based on the interaction of all the variations of the influencing parameters.

### **1.3 Problem Statement**

A comprehensive assessment of the safety and operational impacts of trade-offs in PSD design elements is needed to guide designers in weighing appropriate trade-offs in design elements against safety and operational concerns for two-lane road design.

### **1.4 Research Objectives**

The objectives of this research are to develop a methodology and a model that would provide highway designers and the highway operation engineers with minimum PSD that would provide a certain level of service for a certain risk level based on the roadway characteristics, traffic characteristics, vehicle characteristics, and driver characteristics that best describe the roadway under consideration and its users. Thus, the objectives are two-fold:

1. Quantify the safety and operational impacts of the PSD design element trade-offs and their associated risks and
2. Develop guidelines to assist designers in making reasonable choices among possible PSD element trade-offs.

## **1.5 Research Approach**

The first task of this research presents a new approach for the design of PSDs. The new approach will provide a distribution of the minimum PSD which accounts for the variability of the influencing parameters. Most of the aforementioned parameters are random numbers. Their variations can be captured by certain probability density functions based on data collected in the field or from the literature. A computer simulation is devised to replicate the passing maneuver under the varying conditions. The simulation will utilize the Monte Carlo process to randomly sample values for the different parameters from their corresponding distributions. Thousands of sampling runs are conducted to capture most of the possible scenarios. The results of the simulation will be a distribution of minimum PSD requirements for each selected design speed.

In order to accomplish the main goal of this research, risk and level-of-service measures need to be attached to the values of the PSD distributions. The second task focuses on assessing the risk index of the PSD distribution. Also, a computer simulation will be used to replicate the passing maneuver. The purpose of the simulation is to conduct virtual passing attempts and assess the probability of accidents under certain selected PSD lengths. This will demonstrate the risk level of the PSD length. A unique microscopic simulation is developed for this goal. The simulation replicates the behavior of three vehicles in a passing maneuver. Thousands of passing attempts are simulated for each PSD length under varying traffic, driver, and vehicle conditions. The results of the simulation are weighted risk indices relative to the various PSD lengths.

The third and final task of this research is to assess the level of service of the different PSD lengths. To accomplish this task, a two-lane road section is analyzed using the Interactive Highway Safety Design Model (IHSDM) software utilities and the concepts provided by the Highway Capacity Manual (HCM 2000). The main purpose is to compute the delay time that the traffic stream incurs relative to the selected PSDs. The delay is based on the variation in the average travel speed due to the variation in the no-passing zone percentage. The results of this task summarize the service measures of the two lane section stretch relative to the selected PSD lengths.

Finally, the results of the two simulations will be combined in order to conduct a trade-off analysis. The final recommendations of the research will be useful to highway design and operations engineers.

## **1.6 Dissertation Layout**

The dissertation is divided into 8 chapters. The first chapter introduces the problem, describes the background, and presents a solution approach. The second chapter discusses the history of the PSD and its design methods. The third chapter brings about the need for risk in designing highway elements, of which is the PSD. The fourth chapter details and describes the different parameters that influence the formulation and the length of the PSD. In this chapter, the variations and characteristics of each of the parameters are listed based on data from the field or from the literature. The fifth chapter discusses the Monte-Carlo simulation which is used to obtain the PSD distribution curve. Also, in this chapter, a closed form analytical formulation is presented to verify the results of the Monte-Carlo simulation. The sixth chapter of the dissertation discusses in details the unique microscopic simulation and its results. In this chapter, the risk levels of each of the PSD values are computed. The seventh chapter demonstrates the use of the well known Highway Capacity Manual (HCM 2000) concepts in order to assess the operational effects of the different PSDs. The eighth and final chapter presents the combined results of both simulations. Risk and level of service measures are tabulated for each PSD value. The research conclusions are also discussed in this chapter along with future research recommendations.

## **CHAPTER 2:**

### **LITERATURE REVIEW 1: PASSING SIGHT DISTANCE**

## **2.1 Introduction**

Passing sight distance (PSD) is needed when passing is permitted on two-lane, two-way highways. It is provided to ensure that passing vehicles using the lane normally used by opposing vehicles have a clear view ahead for a distance sufficient to minimize the possibility of a collision with an opposing vehicle (AASHTO 2004). The intent of PSD is to ensure the *safety* of the passing drivers and to provide an adequate level of service to the traveling public on two-lane highways. The criteria adopted by the *Green Book* were based on extensive field observations conducted by Prisk during 1938 to 1941. These observations were used as the basis for the current PSD formulation. Though many studies have showed that these criteria are mostly conservative, they were kept virtually unchanged even until the late editions of the “Green Book”.

## **2.2 Two Different Passing Sight Distance Criteria**

The design of four lane highways is not mainly concerned with passing maneuvers, for example. However, passing criteria are critical in the design of two-lane two-way highways. The capacity of a two-lane roadway is greatly increased if a large percentage of the roadway's length can be used for passing. Providing sufficient PSDs over large portions of the roadway can be very expensive due to the increased civil work that accompanies that. Thus, it is a matter of adequate compromise between safety and level of service. However, when defining the PSD, design books (MUTCD and AASHTO) only mention the word safety. Simply put, the PSD is the length of roadway that the driver of the passing vehicle must be able to see initially, in order to make a passing maneuver *safely*. The goal is to provide most drivers with a sight distance that gives them a feeling of *safety* while passing slower vehicles (AASHTO 2004). As simple as it is, the criteria for PSD and the passing zone length differed between the Green Book and the MUTCD.

### **2.2.1 The Green Book Formulation**

The Green Book formulation uses a simple but very conservative model to calculate the required PSD on a two way two lane road. The model was tested during the period between 1938 and 1941, and later validated in 1958 (Harwood et al. 2003). The model incorporates the interaction of three vehicles; the passing, the passed, and the opposing vehicle. It is based on six assumptions:

- 1) The vehicle being passed travels at a constant speed throughout the passing maneuver.

- 2) The passing vehicle follows the slow vehicle into the passing section.
- 3) Upon entering the passing section, the passing vehicle requires some time to perceive that the opposing lane is clear and to begin accelerating.
- 4) While in the left lane, the passing vehicle travels at an average speed that is 10 mph faster than the vehicle being passed.
- 5) An opposing vehicle is coming toward the passing vehicle.
- 6) There is an adequate clearance distance between the passing vehicle and the opposing vehicle when the passing vehicle returns to the right lane.

Based on AASHTO, the PSD can be divided into four quantifiable portions that are listed separately below. Table 2.1 illustrates the derivation of the PSD criteria, representing the sum of the distances  $d_1$  through  $d_4$  for specific speed ranges.

- a)  $d_1$  -- The distance that the passing vehicle travels while contemplating the passing maneuver, and while accelerating to the point of encroachment on the left lane.
- b)  $d_2$  -- The length of roadway that is traversed by the passing vehicle while it occupies the left lane.
- c)  $d_3$  -- The clearance distance between the passing vehicle and the opposing vehicle when the passing vehicle returns to the right lane.
- d)  $d_4$  -- The distance that the opposing vehicle travels during the final 2/3 of the period when the passing vehicle is in the left lane.

**Table 2.1 – Elements of the PSD for the design of two-lane highways**

Component of passing maneuver	Metric				US Customary			
	Speed range (km/h)				Speed range (mph)			
	50-65	66-80	81-95	96-110	30-40	40-50	50-60	60-70
	Average passing speed (km/h)				Average passing speed (mph)			
	56.2	70.0	84.5	99.8	34.9	43.8	52.6	62.0
Initial maneuver:								
$a$ = average acceleration <sup>a</sup>	2.25	2.30	2.37	2.41	1.40	1.43	1.47	1.50
$t_1$ = time (sec) <sup>a</sup>	3.6	4.0	4.3	4.5	3.6	4.0	4.3	4.5
$d_1$ = distance traveled	45	66	89	113	145	216	289	366
Occupation of left lane:								
$t_2$ = time (sec) <sup>a</sup>	9.3	10.0	10.7	11.3	9.3	10.0	10.7	11.3
$d_2$ = distance traveled	145	195	251	314	477	643	827	1030
Clearance length:								
$d_3$ = distance traveled <sup>a</sup>	30	55	75	90	100	180	250	300
Opposing vehicle:								
$d_4$ = distance traveled	97	130	168	209	318	429	552	687
Total distance, $d_1 + d_2 + d_3 + d_4$	317	446	583	726	1040	1468	1918	2383

The formula used in the calculation of the distances  $d_1$  and  $d_2$  are shown in equations 2.1 and 2.2 (Metric units):

$$d_1 = 0.278 \times t_i \left( v - m + \frac{a \times t_i}{2} \right) \quad (2.1)$$

$$d_2 = 0.278 \times v \times t_2 \quad (2.2)$$

Where,  $t_i$  = time of initial maneuver, (seconds);

$a$  = average acceleration, (Km/h/s);

$v$  = average speed of passing vehicle, (Km/h);

$m$  = speed differential between passing and impeding vehicles, (Km/h);

$t_2$  = time the passing vehicle occupies the left lane, (seconds);

The assumptions adopted by the Green Book in formulating the problem and obtaining the aforementioned distances are as follows (AASHTO 2004):

- a) The time for the initial maneuver ( $t_i$ ) falls within the 3.6 to 4.5 s range.
- b) The average acceleration rate during the initial maneuver ranges from 1.38 to 1.51 mph/s [2.22 to 2.43 km/h/s].
- c) The distance ( $d_2$ ) is estimated assuming that the time the passing vehicle occupies the left lane ranges from 9.3 to 11.3 s for speed ranges from 30 to 70 mph [50 to 110 km/h].
- d) The clearance distance ( $d_3$ ) is estimated to range from 100 to 300 ft [30 to 90 m].
- e) The distance ( $d_4$ ) is estimated as two-thirds of the distance traveled by the passing vehicle in the left lane.
- f) The passing vehicle could abort its pass and return to the right lane if an opposing vehicle should appear early in the passing maneuver.

The design values, as obtained by the *Green Book* policy formulation, range from 710 to 2,680 ft [200 to 815 m] for design speeds ranging from 20 to 80 mph [30 to 130 km]. The PSD design criteria set by the *Green Book* are used in the design of two-lane two-way highways. The MUTCD criteria are used to mark passing and no-passing zones on these types of roads.

### **2.2.2 The MUTCD Marking Criteria**

The MUTCD standards are used to mark passing and no-passing zones on two-way two lane highways. The MUTCD virtually uses an inverse method to mark passing zones. The MUTCD standards define where no-passing zones are warranted. The rest of the road is then marked as a passing zone. The speeds used in the MUTCD design criteria are the prevailing off-peak 85th-percentile speeds rather than the design speeds. Table 2.2 presents the MUTCD PSD warrants for no-passing zones. However, the derivation leading to these distances are not stated in the MUTCD. They are identical to those of the 1940 AASHTO policy on marking no-passing

zones (Harwood et al. 2003). These warrants represent a subjective compromise between distances computed for flying passes and distances computed for delayed passes. So, they do not represent any particular passing situation. Nevertheless, the MUTCD sets a minimum passing zone length of 400 ft [120m], while the *Green Book* does not.

**Table 2.2 – Minimum PSD for marking purposes**

Metric		U.S. Customary	
85th Percentile Speed (Km/h)	Minimum PSD (m)	85th Percentile Speed (mph)	Minimum PSD (ft)
40	140	25	450
50	160	30	500
60	180	35	550
70	210	40	600
80	245	45	700
90	280	50	800
100	320	55	900
110	355	60	1,000
120	395	65	1,100
		70	1,200

### 2.2.3 Critique of Passing Sight Distance Concepts

Clearly, the *Green Book* and the MUTCD criteria are not compatible in calculating PSDs. The *Green Book* criteria are not clear when estimating the four component distances of the PSD for speeds higher than 105 Km/h (65 mph). Also, in obtaining the PSD, the passing driver is assumed to be committed to the passing maneuver at an early stage of the action. In fact, observations of two lane highway operations show that passing drivers frequently abort passing maneuvers (Harwood et al. 2003). On the other hand, the MUTCD criteria represent a subjective compromise between distances computed for flying passes and distances computed for delayed passes. So, they do not represent any particular passing situation. By definition, a delayed pass is a maneuver in which the passing vehicle slows to the speed of the passed vehicle before initiating the passing maneuver. While a flying pass is a maneuver in which the passing vehicle comes up behind the passed vehicle at a speed higher than the passed vehicle and initiates the passing maneuver without slowing down to the speed of the passed vehicle (Harwood et al. 2003). Furthermore, both the AASHTO and MUTCD criteria are based on field data collected nearly 50 years ago. They also do not account for trucks in obtaining the PSD criteria. They only considered passenger cars which are obviously more powerful and shorter in length than the average truck.

## **2.3 Overview of the PSD Research Endeavors**

Over the last three decades, researchers have recognized the inconsistencies between the AASHTO and MUTCD policies. Many have investigated alternative formulations for the PSD criteria. A total of 13 studies published since 1970 have questioned the premises of the AASHTO and MUTCD models and/or suggested revisions to those models. In 1971, Weaver and Glennon, and Van Valkenberg and Michael, independently recognized that a key stage of a passing maneuver occurs at the point where the passing driver can no longer safely abort the pass and is, therefore, committed to complete it. One study called this the “point of no return” and another called it the “critical position” (Glennon 1988). In 1976, Glennon and Harwood added great insight to the notion of the “critical point”, but they did not give it any mathematical form. From then on, the “critical position” is defined as the point where the distance needed to complete the passing maneuver is equal to the distance needed to abort it and return to the right lane behind the impeding vehicle. Beyond the critical position, the driver is committed to complete the pass, because the sight distance required to abort the pass is greater than the sight distance required to complete the pass (Glennon 1988). This formulation obviously assumes more responsibility on the driver’s side. In 1982, Lieberman added further insight by developing a mathematical time-distance relationship that identified the critical position and the critical PSD as a function of design speed. Thus, his formulation was the first related to the critical PSD. Lieberman assumed in his formulation that the total PSD is the critical distance plus the distance that the vehicle needs to reach that critical point. This was later criticized by Glennon (1988). Nevertheless, Glennon also points out that the Lieberman formulation ignored the effects of the vehicle length and the perception reaction time during the abort maneuver. M. Saito highlighted the importance of the abort maneuver in calculating the PSD criteria (Saito 1984). However, he did not compromise between the abort and the completed maneuvers. Saito assumed that the critical distance is the point where the passing vehicle is right behind the impeding vehicle.

### ***2.3.1 A Logical Model***

Several of the studies cited above formulated PSD models based on the critical position concept. Glennon formulated a new PSD model that accounts for the kinematics’ relationships between the passing, passed, and opposing vehicles (Glennon 1988). He modeled the critical PSD bearing in mind that its maximum value is at the point where the distance needed to safely abort the passing maneuver is equal to the distance needed to safely complete the maneuver. The formulation derived by Glennon is as follows (Glennon 1988):

$$\Delta_c = L_p + m \left\{ \frac{(2m + L_l + L_p)}{(2V - m)} - \sqrt{\frac{4V(2m + L_l + L_p)}{d(2V - m)}} \right\} \quad (2.3)$$

$$PSD_c = 2V + \frac{2V(m + L_p - \Delta_c)}{m} \quad (2.4)$$

Where,

$\Delta_c$  = Critical separation which is the distance from the front of the passing vehicle to the front of the passed vehicle at critical position, (ft).

V = Speed of the passing vehicle and opposing vehicle, (ft/sec).

m = Speed difference between passing and passed vehicle, (ft/sec).

d = Deceleration rate when aborting the passing maneuver, (ft/sec<sup>2</sup>).

$L_p$  = Length of passing vehicle, (ft).

$L_l$  = Length of passed vehicle, (ft).

Glennon made some assumptions about the three vehicles involved in the maneuver, which are summarized as follows:

- 1) The opposing vehicle maintains constant speed during the maneuver and that is the design speed of the highway.
- 2) The passing vehicle accelerates to the design speed at or before the critical position and maintains it unless the maneuver is aborted.
- 3) The impeding vehicle also travels at a constant speed that is less than the design speed by the value of the speed differential (m).
- 4) The passing vehicle has the capability to attain the design speed on or before the critical position.
- 5) In case the maneuver is aborted, the perception reaction time of the passing vehicle before decelerating is assumed to be 1 second.
- 6) The perception reaction time prior to initiating a pass is also 1 second.
- 7) The minimum clearance time between the passing vehicle and the opposing vehicle is 1 second, (that is,  $C = 2V$ ).
- 8) The gap between the passing vehicle and the passed vehicle, in case the maneuver is aborted, is also 1 second, (that is,  $G = m$ ).

Using the model, Glennon and Harwood were able to conduct sensitivity tests in order to evaluate the results. The PSD formulation includes the lengths of the passing and passed vehicles and their speeds. So, different passing scenarios have been analyzed. The scenarios include passenger car passing another passenger car, passenger car passing a truck, truck passing a

passenger car, or even truck passing a truck. The study showed that the current PSD provided by the MUTCD criteria are similar to those of the proposed model for the scenario where a passenger car is passing another passenger car (Harwood and Glennon 1989). Application of the Glennon model indicates that successively longer PSDs are required for a passenger car passing a truck, a truck passing a passenger car, and a truck passing a truck. However, all the derived design criteria for PSDs were less than the AASHTO design criteria, which are obviously based on very conservative assumptions.

## **2.4 Critique of Other Transportation Design Concepts**

Highways are designed according to standards set over the years by successive committees of professional engineers. It is common belief that design criteria are set in order to attain maximum safety within any constructed project, whether roads, buildings, dams, etc. The author aims at showing that this belief is somehow naïve. The first anecdote is about the standard that pertains to the design criteria of roads. It shows how a “preconceived idea about why crashes occur has shaped the evolution of a standard in which factual knowledge of safety was neither required nor played a discernible role” (Hauer 1999). Two highway design areas are chosen to support the argument. These are the criteria used in the design of vertical curves and lane widths. Then, the author will contrast the two aforementioned ideas to the design criteria of PSD.

### ***2.4.1 The Crest Curve Design Argument***

The road is made up of straight section and curves. Curves could be vertical or horizontal, or sometimes combined in the case of ramps. Vertical curves are composed of straight lines that are connected by parabolic curves to smooth the edges where the two lines meet. Long ago, designers noticed that sight is limited when going up a vertical curve. Their notion of attaining safety on these curves got attached to the concept of sight distance. Design standards emerged to specify the adequate vertical alignment that allows for enough sight distance. All road design standards prescribe that the parabola be sufficiently shallow so that, if there is some object of specified height in the path of the vehicle, it can be seen from far enough for the driver to stop safely. In this manner, the standard is driven by an explicit concern for safety. Thus, the core of the standard is the “design speed” and a few “parameters” (the reaction time, pavement-tire friction, eye height and object height) (Hauer 1999). The rest is a matter of computation based on physics and mathematics. The designer can compute what shape of the parabola will satisfy the stopping sight distance requirement. This looks logical, but where in this formulation did designers question the reasons for crashes frequency and severity? They only

imagined what would lead to crashes on vertical curves. In this case the conjecture was that sight distance limitations are an important cause of crashes on crest curves.

It is surprising that designers did not need any knowledge about crash frequencies on curves to devise a standard procedure for “safe” design. The procedure is based on a plausible conjecture. Conjectures, no matter how plausible, are not usually acceptable when it comes to matters affecting health. For example, a drug will not be approved for use unless its effect is carefully tested and its curative benefits as well as harmful side effects are known. Yet, the design of vertical crest curves is based not on empirical fact but on plausible conjecture. Thus, unintentionally, the design of crest curves became a ritual founded on a preconceived idea of what causes ‘failures’ (i.e., crashes) to occur on crest curves, and not on a proven fact. Another thing worth mentioning is the talk about the object height that is used in the design formula of the crest curves. The American engineering standards committee set the height criteria to be 4 inches above the road. Their assumptions were not based on any safety study but on the fact that by raising the height criteria to 4 inches instead of 0, the construction costs will be reduced by 40% (AASHTO 1954). At that time, no one had ever investigated the relation between object height and road accidents. Still, many years later this criterion needed to be changed but did not. The fact that newer car models resulted in a lower driver’s eye height meant that a 4 inch object can no longer be seen by drivers at the prescribed stopping sight distance. A reevaluation process of all the crest curves had to be done. However, the standards committee dealt with this dilemma by increasing the design object height to 6 inches, instead, which is a more economical way of solving the problem. Although the original motivating concern is safety, the committee usually recognizes that the relationship of sight distance on crests to safety has never been established. So, different perception and evaluation of the criterion were shaped by various judgments. That is why, designers use obstacles that are 0" high in Germany, 4" and later 6" in the USA, 8" in Australia, and 15" in Canada.

To be clear about what *Road Safety* means in general, consider two points A and B. Of two alternative highway designs connecting these points, that highway design which is likely to have fewer and less severe crashes is the safer one. Thus, the safety of a road is measured by the frequency and severity of crashes expected to occur on it (Hauer 1999). The safety of any road can be measured by some degree that has units of Safety or Risk. But, current highway design is based on set standards in which no premeditated level of safety can be insured. For example, consider the design criterion of object height used in the design of stopping sight distance on vertical curves. It is said, “It is more expensive to build highways to ensure that all obstacles are

visible and it is cheaper to build roads to ensure only the visibility of taillights.” (Hauer 1999). In conclusion, roads designed to meet standards are neither ‘as safe as they can be’ nor ‘as safe as they should be’. Therefore, designers can in no sense claim that their design is safe.

#### ***2.4.2 The Lane Width Design Argument***

It has been assumed that the critical situation that might lead to failure (crash) on crest curves is related to the ability of the driver to stop in time when seeing an object in the vehicle’s path. Similarly, the clearance distance between two oncoming vehicles has been assumed to be the critical variable in the design of lane widths. It has been described as follows: the loss of clearance is when drivers tend to move to the right most of the lane when faced by oncoming traffic. Thus, the measurable properties for lane width design are separation between oncoming vehicles and how much drivers tend to shift to the right (Hauer 1999).

The policies for the design standard of lane width have been set during the period between 1938 and 1944 by the Committee on Planning and Design Policies of the American Association of State Highway Officials. Later, the policies were assembled into a single volume in 1950 and published with revisions as a ‘Policy on Geometric Design of Rural Highways’ in 1954. The ‘Policy’ was revised and reissued in 1965, 1984, 1990, 1994, 2001, and 2004 but kept the same criteria related to lane width. The basic criteria have been set by Taragin in his paper which was published in 1944. His research was based on extensive empirical studies related to vehicle speeds and vehicle placement as a function of pavement width. Taragin thought that an “adequate pavement width” is when drivers do not shift toward the edge of the pavement when meeting an oncoming vehicle or when being passed by one. Taragin mentions nothing about crashes or their severity and frequency, but he finally concludes that lane widths are safer if designed over 11 ft. He speculates that if drivers feel the need to shift to the right when meeting an oncoming vehicle, a hazard exists; when they no longer shift to the right, the lane-width related hazard is of no concern. Suddenly, drivers’ behavior becomes the measure of safety when designing lane widths instead of a safety equation related to the occurrence of crashes. Once again conjecture is substituted for fact. Standards are then written to govern the occurrence of ‘situations’ rather than the occurrence of safety outcomes (Hauer 1999). If the safety portion is based on the conjecture about separation between oncoming vehicles, and since the relationship between separation and safety is unknown, safety is not really being taken into account. The resulting standard builds an unpremeditated amount of safety into roads. The sad thing is that in the course of half a century and five Policy revisions, all committees quoted one single study done in 1944. The study contained no evidence of the link between vehicle separation and crash

occurrence. Similarly, no edition of the Policy refers to any study of how many more crashes are expected on 9 foot lanes than on 10 foot lanes, for example. Yet, somehow all committees found it possible to make the tradeoffs necessary to decide under what conditions are 9 foot lanes the permissible minimum and when should a minimum of 10, 11 or 12 foot lanes be used. The first part of the problem lies in the false assumptions that the design of the lane width is based on.

The second wrong assumption is related to the assumed nature of the parameters embedded in the formulation. It is known that a road is a man-made product dedicated to be used by man. Civil engineers are trained to deal with inanimate matter. They deal with loads, flows, modulus of elasticity, stress, strain, porosity etc. Once the physics of the situation and the properties of the materials are known, the results of certain events can be calculated using the known formulas. This is the basis on which reasoned design choices are made. In geometric design, however, the story is different. As mentioned earlier, roads are built for road users. Unlike inanimate matter, road users adapt to the prevailing situations. Thus, in geometric design, one should not assume that speed, reaction time and similar ‘design parameters’ are quantities that do not depend on the design itself. There is no parallel to this in other civil engineering design. One does not assume that the load will adapt to the strength of the beam or that it will rain less if the diameter of a culvert is small. The consequence of this fundamental misconception is that speed, reaction time, and similar parameters are treated as constants in all the formulae and computation that are at the root of geometric design standards (Hauer 1999).

### ***2.4.3 The Need for Risk Studies***

Driving on any road is not 100 percent safe but is known to be harmful. Thus, it is not acceptable to produce roads and to put them into use without providing for a premeditated level of safety. Besides, failure in highway design is not a matter of guessing, but a matter of degree. It is not like the collapse of a roof or the flooding of a culvert, but more like the deflection of a beam exceeding the allowable amount. Accordingly, safety/failure should be defined straightforwardly and directly in terms of the expected frequency of crashes or crash consequences. There are two kinds of safety, the nominal and the substantive (Hauer 1999). *Nominal safety* is judged by compliance with standards, warrants, policies and sanctioned procedures. It ensures that most road users can behave legally, that design does not make road use difficult for significant minorities and provides protection from moral, professional and legal liability. To reform how nominal safety is dealt with, the faulty design paradigm has to be replaced by a new one, and genuine safety information should be incorporated in highway design standards. The concept of *substantive safety* is measured by expected crash frequency and

severity. Substantive safety is a new concept which needs to be introduced into highway design process (Hauer 1999). It is the measure of expected crash frequency and severity. It is a matter of degree where a road could be said to be safe or less safe. The degree of substantive safety attainable is relative to the available resources. Having mentioned the degree of safety in design, the notion of risk needs to be discussed since risk and degree of safety come together. The talk about safety brings up in our mind the degree of risk associated with that. That is, when asked how safe this column is, the response of a structural engineer would be as follows: the risk of it failing is 1/1000, for example. In addition, when one asks about the risk of failure, the common response is that the design is safe. So, these terms are highly and inherently correlated. So, the next chapter of this dissertation talks about the history of risk and its relation to design procedures and elements.

## **CHAPTER 3:**

### **LITERATURE REVIEW 2: RISK AND RELIABILITY OF DESIGN**

### **3.1 Introduction**

Decisions will be made; however, the complete information needed to make informed decisions often does not exist. In these situations, risk analysis can be used to facilitate and improve the decision making process. It is important that risk analyses be as accurate and objective as possible in order to arrive at the best possible decisions. Risk analysis is the link between science and the society which looks to science for advice. Thus, the quality of risk analysis is very important because the long term success of a society is dependent on the quality of its decisions (Cumming 1981).

### **3.2 History of Risk**

#### ***3.2.1 Background***

The mastery of the idea of risk in the current time is considered one of the defining boundaries between modern times and old ages. The fact that people accepted the notion of risk to replace the whim of the Gods is an indication of modernization (Bernstein 1996). The coming section of the dissertation deals with ancient history of risk and its primary recognition by societies in the past versus the changed perception of risk that came within the few last decades.

#### ***3.2.2 Ancient Perception of Risk***

Covello and Mumpower (Covello et al. 1986) trace the beginnings of risk perception by humans to 3200 years B.C. to the Ashipu society. Those people lived in the Tigris-Euphrates valley. They were the primary consultants for risky, uncertain, or difficult decisions. If a decision needed to be made concerning a future forthcoming risky venture, marriage, arrangement, or a suitable building site, they were called upon. They would identify the dimensions of the problem, enumerate the alternatives, and collect data about likelihood of each alternative. They would give plus or minus signs to certain alternatives and then finally decide which alternative has the most favorable outcome. Then they issue a report etched upon a clay tablet. Their practices have marked the first recorded instances of dealing with risk analysis, though in a very simplified way. Then, the talk about risk got associated with the need for insurance which appeared first in Mesopotamia 3000 B.C. Records of interest rates have been found when farmers loaned a portion of their excess production in exchange of a share of the return. Then, different interest rates have been founded ranging from low in personal loans to high in agricultural loans. Later, the Code of Hamurabi in 1950 B.C. established several doctrines of risk management and laid the basis for the institutionalization of insurance. One of the code statements dealt with the safety in buildings. It said that if a building falls down on its residents, then the life of the builder is

forfeited. Also, the Code formalized the concepts of *Bottomry* and *respondentia* (Covello et al. 1986). The latter was highly developed by the Greek in 750 B.C. with 10-25% premiums depending on the riskiness of the venture.

Other sources believe that the origins of risk go back to 3500 B.C. to the concept of gambling, which is the very essence of risk-taking (Bernstein 1996). Humans have long been infatuated with gambling because it puts us head-to-head against fate with no holds barred. People used to think they have an ally when gambling, Lady Luck, who is supposed to interpose herself between us and fate so that we win. The earliest known form of gambling was called the astragalus, or Knuckle-bone. It is assumed to be the early ancestor of today's dice. It is a squarish bone taken from the ankles of sheep or deer; it was very solid and virtually indestructible. It was found in many parts of the world. Egyptian tomb paintings picture games played with the astragali. The Greek vases show pictures of young men tossing bones into a circle. The Greek methodology drew on a giant game of craps to explain what modern scientists call the Big Bang. Three brothers rolled the dice for the universe, with Zeus winning the heavens, Poseidon the seas, and Hades, the loser, going to hell as the master of the underground (Bernstein 1996). Later on cards became famous as gambling tools, and they were first developed from various forms of fortune telling within the Hindu society.

### **3.2.3 Risk Progression A.D.**

Without numbers there are no odds and no probabilities. And without probabilities the study of risk is a mere matter of gut. We live in a world of numbers, without which we are paralyzed. Thus, it is crucial to trace the history of numbers. The origin of the numbering system that we have been using for around 1500 years goes back to the Hindus 500 AD. Ninety years after the Muslims established their strong nation; they invaded India and brought back this important revelation with them. This had a great effect of the intellectual activity, especially in Baghdad which was already an intellectual research city. Then, the Arabs carried this great invention to, as far as, Spain. The center piece of the Hindu-Arabic system was the invention of the Zero (*cifr*). The Arabic scientist, Al Khwarizmi, was the one to invent the rules of subtracting, multiplying, adding and dividing numbers around 825 AD. It is from his name that the word algorithm was invented. Then, Muslim followers and mathematicians, such as Omar AL Khayyam, added a lot to mathematics as early as 1050 AD. By the year 1000 AD, the new numbering system was being taught by Moorish universities in Spain (Bernstein 1996). The use of numbers became very easy and suitable after the rules of algebra and calculus were developed.

This prepared the way for the development of the sophisticated statistical theory which is associated to the notion of risk.

### ***3.2.4 Modern Stages of Risk***

The modern conception of risk is rooted in the Hindu –Arabic numbering system that reached the West 700 – 800 years ago. But, the serious study of risk began during the Renaissance period, when people broke loose from constraints and accepted the challenge (Covello et al. 1986). That was a time of religious turmoil, nascent capitalism, and vigorous approach to science and the future. In the year 1654, during the Renaissance period, a French nobleman, Chevalier De Mere, who had a taste for gambling and mathematics, challenged Blaise Pascal to solve a puzzle (Bernstein 1996). The same puzzle had confused mathematicians for two hundred years earlier when posed by Luca Paccioli. Pascal asked for Pierre de Fermat for help. They teamed up to solve the puzzle. That led to the discovery of the theory of probability, the heart of risk concepts. Their solution meant that people, from now on, can make decisions based on probabilities and numbers. As the years passed, mathematicians transformed the theory of probability from a gambler's toy to a powerful instrument for organizing, interpreting, and applying information. In 1703, Bernoulli invented the law of Large numbers and methods of statistical sampling. By 1725, mathematicians were competing with one another in devising tables of life expectancies. In 1730, Abraham de Moivre discovered the structure of the normal distribution and the concept of standard deviation, which together form the Law of averages that is the essential part of the modern risk quantification (Covello et al. 1986). The marine insurance followed in the middle of the same century as a sophisticated business in London. One hundred years after Pascal and Fermat's collaboration, a dissident English minister, Thomas Bayes, made striking advances in statistics by demonstrating how to make better informed decisions by blending new information into old information. Basically, all the tools and techniques currently used in risk management and decision analysis are based on the inventions made between the years 1654 and 1760, with only two exceptions. In 1875, Francis Galton, a first cousin of Charles Darwin, discovered the regression of the mean. And 73 years later in 1952, a young graduate at the University of Chicago, Nobel Laureate Harry Markowitz, demonstrated mathematically why putting your eggs in one basket is an unacceptably risky strategy (Bernstein 1996). That revelation touched off the intellectual movement that revolutionized Wall Street, corporate finance, and business decisions around the world; its effects are still being felt till today (Bernstein 1996).

### ***3.2.5 The Current History of Risk***

Risk analysis is currently being applied to many sectors, including transport, construction, energy, chemical processing, aerospace, the military, and even to project planning and financial management. In many of these areas, Probabilistic Risk Analysis (PRA) techniques have been used as part of the regulatory framework. PRA tools are growing in sophistication and becoming ever more widely used. In this section, the author discusses the recent past of risk relative to three main areas. These latter are the aerospace, nuclear, and the chemical sectors.

A systematic concern with risk assessment methodology began in the aerospace sector following the fire of the Apollo test AS-204 on January 27<sup>th</sup>, 1967. On that event, three astronauts were killed. This one event set the National Aeronautics and Space Administration (NASA) back 18 months, involved considerable loss of public support, cost NASA salaries and expenses for 1500 people involved in the subsequent investigation, and ran up \$410 million in additional costs (Bedford and Cooke 2001). Prior to the accident, NASA relied on good engineering practices of its contractors. On April 5, 1969 the Space Shuttle Task group was formed in the Office of Manned Space Flight of NASA. The developed task group suggested criteria for evaluating safety quantitatively. The probability of mission completion was to be at least 95% and the probability of injury or death per mission was not to exceed 1%. Though, these safety goals were not accepted from the group because of the low numerical assessments of accident probability that could not guarantee safety. Another accident occurred, the Challenger in January 28, 1986, whose expected failure risk was quantified three years earlier by a study commissioned by the US Air Force. The study estimated the failure rate as 1 per 35. This was rejected by the NASA management which relied on its own engineering judgment which said that the failure is 1 in 100. Their distrust in risk numbers was not the reason behind abandoning quantitative risk assessment, but because the numbers obtained from such assessment studies made the NASA operations look bad. They also threatened the political viability of the entire space program. For example, a General Electric full numerical probabilistic risk assessment on the likelihood of successfully landing a man on the moon showed that the probability is less than 5% (Bedford and Cooke 2001). The NASA administrator thought that these numbers could do great harm to the program. So, he banned the study. Since then, the NASA programs of quantifying risk to support safety during the design and operations of manned space travel were increased. It reached high point with the publication of the SAIC Shuttle Risk Assessment (Bedford and Cooke 2001). This report showed that the probability of failure causes had been significantly reduced. Same efforts were progressing in other space programs around the world.

Through out the 1950's, the American Atomic energy Commission (AEC) pursued a philosophy of risk assessment based on the maximum credible accident. Residual risk was estimated by studying the hypothetical consequences of incredible accidents because credible accidents were covered by plant design. In 1957, a study of the effects of an incredible accident (release of radioactive materials) in a 200 Megawatt nuclear plant operating 30 miles from a population was developed. The results have shown that no one will ever know the exact magnitude of this low probability. The thought of introducing nuclear reactors close to large populated areas provoked the introduction of probabilistic risk analysis into the nuclear field. It was introduced to quantify the risk of such attempts and the effects to increasing safety measures on certain power reactors. The first full scale application of these methods was undertaken in the Reactor Safety Study WASH-1400 published by US Nuclear Regulatory Commission (NRC) in 1975. This was considered the first modern PRA study. It was considered turbulent after being reviewed by the American Physical Society (APS) (Bedford and Cooke 2001). In 1977 the US Congress passed a bill creating a special review panel of external reactor safety experts to review the achievements and the limitations of the Reactor Safety Study. The panel came up with the Lewis report, named after the leader of the panel, Professor Harold Lewis. The Lewis report acknowledged the validity of the basic concepts of PRA in the Reactor Safety Study. It also found some deficiencies in the treatment of probabilities. Shortly after that, in 1979, a dramatic event led to the enhancement of the PRA studies when a nuclear reactor in Three Mile Island had two of the TMI nuclear generating units suffer severe core damage. The damage was fairly predicted by the Reactor Safety Study (Bedford and Cooke 2001). After studying the incident, a new generation of PRAs appeared in which some of the methodological defects of the Reactor Safety Study were avoided. The US NRC released The Fault Tree Handbook in 1981 and the PRA Procedures Guide in 1983 which standardized much of the risk assessment methodology. Despite all the studies in this field, the two unfortunate accidents, Chernobyl and Three Mile Island, formed a mood of distrust in the nuclear power plants within the public opinions. The development of numerical safety goals in the late 1980s and 1990s has aided the technical advances in the methodology of risk analyses. Examples of which are the USNRC policy statement of 1986 and the UK Tolerability of Risk document, which sought to place the ALARP "as low as reasonably possible" principle into numerical framework. It defined the upper levels of intolerable risk and lower levels of broadly tolerable risk.

In the chemical process sector, the government became interested in the use of the PRA as a tool for estimating public exposure to risk in the context of licensing and citing decisions.

Important European efforts in this direction include two studies of refineries on Canvey Island in 1978 and 1981 in the UK, a German sulphuric acid plant study in 1983, and the Dutch LPG and COVO studies 1982 and 1983. The COVO study was a risk analysis of six potentially hazardous objects in the Rijnmond area. The group which performed the study later formed the consulting firm Technica, which has since played a leading role in risk analysis. The incentive behind most of this work is the dramatic incident where a chemical plant in Italy released a dioxin. Thus, the Post-Seveso Directive was adopted by the European Community. This directive mandates that chemical industries each should have a risk management methodology. The Dutch were leading in this field. Their legislation requires that the operator of a chemical facility dealing with hazardous substances to submit an external safety report (EVR) which is to be updated every five years. This report specifies the upper levels of acceptable risk of death of people exposed to chemical materials as well as the group risk (Bedford and Cooke 2001).

Risk based regulations are now common in many different sectors, such as the environmental sector, the structural engineering sector, the manufacturing sector, and many others. Now that the origins and incentives behind risk concepts were identified, the next section of this dissertation defines the concept of risk and risk analysis.

### **3.3 Definition of Risk**

In the context of risk analysis, there is no single definition of risk that pertains to all fields to which it applies. A common aspect of risk across disciplines is that risk incorporates the potential for a particular consequence to occur and the probability or likelihood of that consequence occurring (Bahr 1997). Using the term risk usually implies that the consequence being considered is an undesirable one. Webster's New World Dictionary defines risk as "the chance of injury, damage, or loss." Kaplan defines risk as the product of damage multiplied by uncertainty, leading to the conclusion that if there is no damage, there is no risk. Pierre Corneille, in his *Le Cid*, said that "we triumph without glory if we win without danger (risk)." Many definitions of risk also include a factor dealing with the magnitude of the negative outcome. The greater the magnitude is the greater the risk. Various actions and safeguards can be implemented which will mitigate the risk and reduce it. As long as the hazard exists at a level greater than zero, risk does exist and can not be zero.

More formally, risk is the severity of the consequences of an accident times the probability of its occurrence. In an even more technical sense, risk can be defined as the potential occurrence of unwanted adverse consequences to human life, health, property, and/or the

environment. The estimation of risk is usually based on the expected value of the conditional probability of the event occurring, multiplied by the consequences of the event, given that it has occurred (Cumming 1981).

It should be noted that the social sciences and the natural sciences understand risk in different ways. While the probability of a technological or ecological risk - for example, an earthquake - is scientifically calculable, the likelihood of a terrorist attack occurring cannot be predicted. Some researchers in the social sciences differentiate between risks with known probabilities and risks with unknown probabilities. They tend to call the latter "uncertainties" (Society of risk analysis website).

### ***3.3.1 Risk Analysis***

Risk analysis is the formalized process of identifying hazards and estimating the risks presented by those hazards (risk assessment), making a decision regarding the hazards (risk management), and communicating throughout the process with others (risk communication). Thus, it is divided into three main parts.

### ***3.3.2 Risk Assessment***

The primary goal of a risk assessment is to provide scientific and objective information to decision makers. Risk assessment should identify the possible hazards of a situation and provide a clear picture of the risk involved and the likelihood of an adverse event occurring given the uncertainty of the future. It should also evaluate the magnitude of any potential impact from the identified risks (Bedford and Cooke 2001).

The process of completing a risk assessment is broken down into several steps which vary between disciplines, but follow a general pattern. The first step is the identification of all potential hazards. This can be accomplished by answering the question "what can possibly go wrong?" Basically, a hazard is any event which has the potential to result in harm. All possible scenarios must be identified leading to all possible hazards. The next step to risk assessment is determining the likelihood of each hazard occurring. This step also takes into account any mitigation measures which may be applied to the process. The final step in risk assessment is to characterize the risk or determine what the consequences would be if things went wrong. This portion of the assessment includes a determination of the magnitude of the impact that would be felt as a result of any negative consequences. How that magnitude is expressed will vary depending on the nature of the hazards being assessed. It might be stated in terms of loss of life, amount of contamination of the environment, or dollars spent or loss (Bahr 1997).

Uncertainty is a factor that plays a role in all risk assessments. It is a key to what makes them unique from other types of epidemiological analysis (Cumming 1981). Risk assessments try to predict future events and their impacts. However, there is never sufficient knowledge or data available for a given situation to complete a risk assessment with total certainty of the outcome. Else one would not need to be doing the assessment in the first place. As one works through the steps of the risk assessment process, the amount of uncertainty associated with each calculation and probability should be determined, documented, and incorporated into the assessment (Bedford and Cooke 2001). In addition to containing the previously mentioned components, a risk assessment should be accurate, timely, transparent, and objective. An inaccurate risk assessment can result in bad decisions by management. Decisions can only be as good as the information upon which they are based. If a risk assessment is not completed in a timely fashion, the decision may have to be made without the benefit of the assessment. By being transparent, others can see the methodologies used and the assumptions which went into the risk assessment. And, if the risk assessment is not done in an objective manner, meaning it is based on subjective opinion, the risk assessment is open to criticism and challenge (Bahr 1997).

### ***3.3.3 Risk Management***

Risk management is the process of utilizing the information gained during the risk assessment to weigh policy alternatives and select the most appropriate action. In other words, making a decision regarding the issue at hand based upon a risk assessment (Society of risk analysis website). Typically those dealing with risk management are not the same people that were involved in the risk assessment. This separation of science from management has the benefit of protecting the objectivity of those doing the risk assessment. It does, however, increase the requirement for effective communication between the risk assessors and decision makers to ensure understanding and to prevent misinterpretation of the information provided. Considerations other than science come into play during the decision making phase of risk analysis. Most decisions made also involve economic, social, and political factors. Another aspect of the risk management phase is deciding what mitigation actions will or will not be added to the process being examined. Mitigation actions are those actions that have the effect of lowering the risk of an adverse outcome (Bahr 1997). Examples of mitigation actions are the addition of safety equipment, quarantine placement, pre- and post-import testing, and additional training of personnel. Once a decision is made, appropriate actions are carried out. This is not the end of risk management. After a period of time, the program and actions that derived from the decision making process must be evaluated. From the evaluation will come ways to improve the

process. It is a continuous cycle that should continue as long as the process is in effect (Bahr 1997).

#### ***3.3.4 Risk Communication***

Risk communication is the effective exchange of information, leading to a better understanding of risk, risk assessment, and risk management. It is a critical, yet often overlooked, aspect of risk analysis. Even if the best possible decision is made and is based on a well done, scientifically sound risk assessment, if it is ineffectively communicated, its reception will be negative (Cumming 1981). It has been said that regulatory agencies have three areas of responsibility when setting acceptable risk levels for society, two of which deal with risk communication. These two items are that all affected parties are heard from and that decisions made are visible for all to examine and question. The third item is that the best available technical information and expert opinion is used (Bahr 1997).

An important aspect of risk communication is involving those impacted by or interested in the issue from the very beginning of the analysis process. In addition to the final decision being communicated, the process used to arrive at that decision, including all steps from the identification of hazards to the factors that were utilized in the risk management phase, should be communicated with input sought.

#### ***3.3.5 Economic Analysis***

An important, but rarely discussed, component of a risk analysis is an economic analysis. The purpose of the economic analysis is to identify, quantify, and value all relevant benefits and costs associated with the issue at hand. Adding economics enhances risk analysis by providing a means to value diverse policy outcomes and by providing a common unit, usually the dollar, allowing comparison of various outcomes. An economic analysis can also help the decision maker better understand the implications and impacts of both favorable and unfavorable outcomes of various decision options (Bahr 1997).

Economic factors are probably most often taken into account in the risk analysis process during the risk management phase. However, by completing the economic assessment in conjunction with the biological portion of the risk assessment, the information provided to the decision maker will be better coordinated and therefore more useful. If the biological and economic portions of the assessment are completed independent of each other, they can be disjointed, leaving gaps in the information provided to the decision maker. It should be noted that while it is very important to consider the economics of a situation, it should scarcely be the sole basis for the risk decision. When dealing with health issues, there are often times when other

factors may over ride the economics of the situation. These factors are often intangibles or incommensurables that are difficult to measure or apply dollar values to, and are therefore difficult to include in an economic analysis (Bahr 1997).

### **3.4 Reliability of Design and Risk of Failure**

#### ***3.4.1 Introduction***

When asked about the safety of a particular design, highway engineers could not provide any single meaningful measure of safety, as a structural engineer, geotechnical, or environmental engineer could more than a decade ago. So, transportation engineers wondered why such safety measures are not developed in the transportation field while they are extensively used in other fields of engineering, and specifically the civil engineering practice. Lately, transportation engineers were able to develop the margin of safety and safety measures for the different highway components (Navin 1990). Highway engineers simply copied the knowledge of risk analysis and reliability measurements from other fields into the transportation field. This was something bound to happen sooner or later. The following section of the dissertation discusses the aspect of safety and reliability in design. Although these two concepts are not the same, they mainly complement one another in the field of design. Assessing the risk and safety of a certain design component is equivalent in the eyes of the engineer to quantifying the probability or chance of failure of the design with respect to the ongoing time. Thus, while defining reliability, the author intends to encompass the notion of failure in design and its associated probability within the same context (Hart 1982).

#### ***3.4.2 Definitions***

Reliability is the ability of a system or component to perform its required functions under stated conditions for a specified period of time (Webster's Dictionary). Through the use of probability and statistics, it is possible to make assertions about a system's performance especially when certain aspects of information are missing, incomplete, or random. One is capable of quantifying the probability of failure based on the designer-required reliability level using the probability and statistics concepts. Uncertainties exist in many aspects of any design. Such uncertainties may be classified as reducible or irreducible. Reducible uncertainties are usually caused by lack of data, modeling simplifications, human errors, etc. These can be reduced by collecting more data, better understanding of the problem, and stricter quality control. Irreducible uncertainties are caused by phenomenon of a random nature and can not be reduced by possession of more knowledge or data (Hart 1982).

### ***3.4.3 Reliability/Failure Rate in Structural Design***

In structural analysis, the use of measures of safety such as the probability of failure and the safety index goes back for a while ago. The structural engineers knew about the eminent uncertainties in their design and tried to quantify them through the use of probability theory and statistical inferences. Because of the existence of such uncertainties in the life cycle of a structure, the structural response and life also show scatter. To design structures that can perform their intended function with desired confidence, the uncertainties involved must be taken into account (Hart 1982). The traditional way of dealing with the uncertainties is to use conservative values of the uncertain quantities and/or safety factors in the framework of deterministic design. A more rigorous treatment of the uncertainties can be found in reliability-based design philosophies. Such philosophies have been under development for the last half of a century and are gaining more and more momentum. Now that the existence of uncertainties is acknowledged, resistance forces and applied loads are then analyzed as random numbers with certain probability density functions (PDF). One can now describe the response given a probabilistic description of the structural parameters. It is a known fact that engineering failures occur and have been occurring for a long time. The reason behind that is that loads happen at their high values while resistances are at their low values, leading to insufficient resistance and thus design failure (Hart 1982). Evidently, the structural safety was first viewed from the rate of failure angle. It has been captured by describing the failure term and the probability of failure associated with the design components. Generally, failure is defined as the inability of a system or system component to perform a required function within specified limits. For this reason, failure is considered to be subjective based on the designer's perspective or duty (Navin 1990).

### ***3.4.4 Special Case Formulation in Structural Design***

A simple case study of the probability of failure will be discussed in the following section in the area of structural design. The same concepts will later be used in assessing the probability of failure of the PSD criteria in highway design. A simplified structural component will be analyzed in this section. Consider a single structural member that has a certain uncertain resistance due to the variations in its material properties, casting techniques, human errors, etc. The member is acted upon by a random load that has a certain probability density function. Thus, the mean  $\bar{S}$  and the standard deviation  $\sigma_S$  of the load/stress are easily computed based on the given probability density function. Similarly, the resistance of the material is also considered

random and has a mean  $\bar{R}$  and standard deviation  $\sigma_R$ . Failure is assumed to occur when the load induced stress exceeds the component resistance (Hart 1982). Define a new variable F, where

$$F = R - S \quad (3-1)$$

Consequently, F is a random variable since both R and S are random. Assuming that the aforementioned random variables are normally distributed, the resulting distribution of F is also normally distributed with a probability density function P(f). Then, we can write:

$$\bar{F} = \bar{R} - \bar{S} \quad (3-2)$$

$$\sigma_F^2 = \sigma_R^2 + \sigma_S^2 \quad (3-3)$$

Since failure occurs when F is less than or equal to 0, then we can define the probability of failure  $P_f$  as

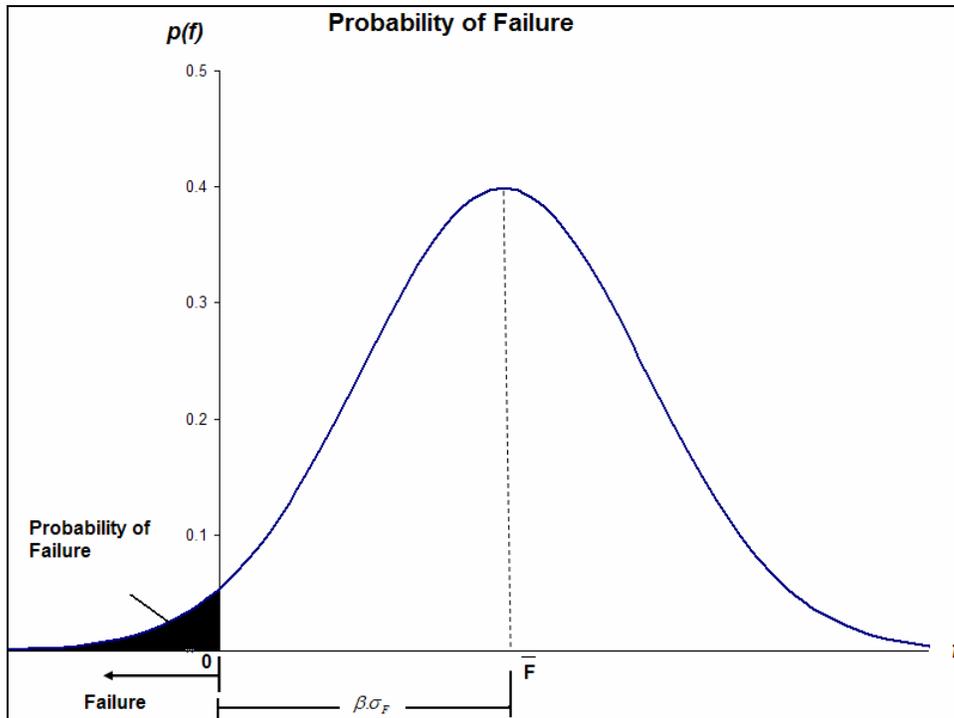
$$P_f = \Pr[F \leq 0] = \int_{-\infty}^0 p(f).df \quad (3-4)$$

This quantity is shown in figure 3.2. It is represented by the solid colored area under the curve. It is apparent from the figure that  $F=0$  occurs at  $(\bar{F} - \beta \sigma_F)$ . Therefore,  $\beta$  is directly related to the probability of failure. Then, by knowing  $\bar{F}$  and  $\sigma_F$  and setting:

$$F = 0 = \bar{F} - \beta \sigma_F \quad (3-5)$$

$$\beta = \frac{\bar{F}}{\sigma_F} = \frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} \quad (3-6)$$

Where,  $\beta$  is called the *reliability* index or *safety* index (Navin 1990). If the probability density function of F is transformed into a standardized normal density function, the probability of failure can then be obtained for a specific value of  $\beta$ .  $\beta$  can be specified or calculated by the designer.

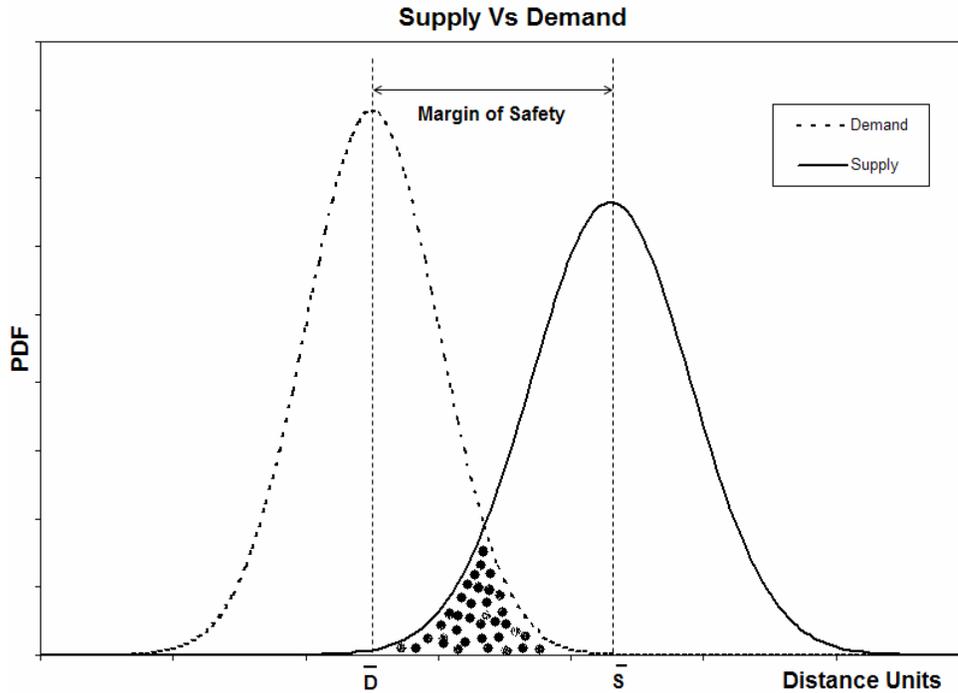


**Figure 3.1 – Probability of failure**

### **3.4.5 Generalized Formulation**

The ideas utilized in the limit state design used in structural engineering constitute the fundamentals of the formulation that will be discussed in this section. The analysis is concerned with the transportation design. Reliability could be applied within the engineering fields in two manners. The first case is to assess or evaluate the reliability of an existing design parameter or component and its probability of failure, as well as its safety index based on its current conditions. The second case deals with designing the proposed system component in a way to achieve a preset safety index, or reliability measure (Easa 1994). Simply, we can either check for a certain safety measure or design our system in order to achieve it. The formulation described in this section aims at providing the designer with the tools to assess the safety index/risk index/or probability of failure of an existing two lane road relative to its provided PSD. It will also allow the designer to compute the PSD required for a certain desired safety/risk level.

Two components will be defined, the demand  $D_0$  of the driver-vehicle system versus the supply  $S_0$  provided by the road system based on the current highway design standards (Navin 1990). The two parameters are assumed, and usually are, random variables with their distributions arranged as shown in Figure 3.3. Failure of the system occurs when the demand exceeds the supply. Thus, failure criteria are defined by the engineer or the specified standard and not necessary that resulting in an accident (Hart 1982).



**Figure 3.2 – Sight distance supply versus demand**

The simplest measure of safety is the central factor of safety ( $SF_0$ ). It is defined as the ratio of the mean supply parameter ( $\bar{S}_0$ ) to the mean demand ( $\bar{D}_0$ ). It is given by the equation:

$$SF_0 = \frac{\bar{S}_0}{\bar{D}_0} \quad (3-7)$$

However, it is rarely used in current engineering practices. The most common measure of safety is the conventional factor of safety. The conventional factor of safety ( $SF_C$ ) is computed as follows: the mean demand is increased by a factor of its standard deviation while the mean supply is reduced by a factor of its standard deviation. The ration of the reduced supply to the increased demand yields  $SF_C$ .

$$SF_C = \frac{\bar{S}_0 - k \cdot \sigma_{S_0}}{\bar{D}_0 + k \cdot \sigma_{D_0}} \quad (3-8)$$

This idea is based on the fact that the parameters we are dealing with contain uncertainty and randomness. So, there is a finite chance that the demand will exceed the supply at some point. This occurs when the sight distance needed by the driver is more than the distance provided by the highway engineer. The methods needed to calculate the means and variances of common functions are provided by Ang and Tang (1984), and describing them is beyond the purpose of this dissertation. The first measure of safety can be defined as the margin of safety (M), and is given by the following equation:

$$M = E [S_0] - E [D_0] = \bar{S}_0 - \bar{D}_0 \quad (3-9)$$

In addition, a better measure of safety, that is the safety index, has already been defined in equation (3-6). It is applied to general types of distributions, not only the normal distribution. The evaluation of the safety index is feasible using simple probability methods if the variables of the basic equation are linearly related; otherwise, the appropriate methods given by Ang and Tang (1984) should be used.

#### **3.4.6 Optimized Reliability-Based Formulation**

Design and optimization tasks are traditionally focused on the low-cost objective. Safety is ensured in the design codes of practice by introducing the well-known "partial safety factors". Its purpose is to over-estimate loading and to under-estimate strength. In this way, optimized structures may have lower reliabilities than the initial ones. The aim of the reliability based optimization is to find the best compromise between cost reduction and safety insurance. With the huge progress in the fields of Optimization and Reliability, the coupling of these methods leads to new multidisciplinary optimization. A global analysis of the design process shows that the total cost cannot be limited to the initial cost. It should also include the expected failure cost as well as maintenance costs. The coupling of reliability and optimization can improve the design by looking for the minimum total expected cost (Wang et al. 2000).

This section discusses the possibility of applying optimized reliability design to the transportation field as it is applied in the structural design. In reliability-based design,  $F$  in equation (3-1) is called the limit state function or failure function.  $F = 0$  divides the design space into two regions, the safety region ( $F > 0$ ) and the failure region ( $F \leq 0$ ). Because of the uncertainties in loads and yield strength,  $F$  is a random variable itself. As a result, we can not be certain in advance whether  $F$  falls into the safe region or the failure region. We can only hope that the beam is designed such that the probability that  $F$  is positive and sufficiently high. In mathematical terms, this is expressed as: Reliability = Probability [ $F > 0$ ]  $\geq$  Target Reliability.

In engineering practice, the safety index,  $\beta$ , instead of structural reliability, is often used to represent the reliability level. When  $F$  has a normal distribution, the safety index ( $\beta$ ) has a one-to-one correspondence with the structural reliability and it is given by equation (3-6) as :

$$\beta = \frac{\bar{F}}{\sigma_F} = -\Phi(1 - \text{Reliability}),$$

Where,  $\Phi$  is the cumulative distribution function for the standard normal distribution and the rest of the variables are already defined. In the case where  $F$  has other distributions, equation (3-6) is

not valid. In general, a larger  $\beta$  corresponds to a higher reliability level. Depending on the goal of the design, different formulations can be used to achieve the design objective. For example, if the goal is to achieve maximum reliability as long as the weight of the structural component (that is the cost) is within some bounds, the optimized design requirement can be expressed as:

Maximize: Safety Index ( $\beta$ )

Subjected to: Design Area < Maximum Area

For example, if the concern is with the weight, the design can be formulated as:

Minimize: Area =  $g(x)$

Subjected to: Safety Index ( $\beta$ ) > Target Safety Index ( $\beta_T$ )

The selection of a target safety index,  $\beta_T$ , is problem dependent.

This chapter defined the aspects of risk and safety and reliability of design. The concepts described in this chapter will be later used in the dissertation. The following chapter discusses the input parameters that influence the calculation and formulation of the PSDs.

## **CHAPTER 4:**

### **INPUT PARAMTERS FOR THE PASSING SIGHT DISTANCE**

## **4.1 Introduction**

A passing maneuver is the process in which a faster vehicle tries to overtake a slower vehicle on a two-lane two-way road. The process implies that the passing vehicle uses the opposite lane for some time while conducting the overtaking maneuver. The speed differential between the passing and the passed vehicle should be enough for the passing vehicle to initiate the pass. The greater the speed differential the less time the passing vehicle spends in the opposite lane. This maneuver obviously includes some risk. So, the concept of the Passing sight distance (PSD) has emerged to ensure the safety of the passing drivers while conducting such maneuvers. PSD is the distance traveled by a driver while trying to pass a slower vehicle ahead on a two-lane road. It is provided to ensure that passing vehicles have a clear view ahead for a sufficient distance to minimize the probability of a crash with oncoming vehicles. Too many variables are involved in a passing maneuver rendering the formulation of the PSD a complex one. Drivers, vehicles, road sections and traffic flows are all part of the design process. The characteristics of each affect the length and placement of the passing zone.

## **4.2 Background**

The design of most highway elements includes parameters that are random variables. These parameters traditionally describe the range of human and vehicle behaviors including road characteristics. Examples of some of these parameters are driver's perception reaction time, acceleration rate, deceleration rate, vehicle speed, vehicle length, and others. They should be represented by random variables with adequate probability density functions. Instead, current highway design formulations use one-value averages to replace these parameters in the design equations. An example of a geometric design element that is influenced by random parameters is the PSD requirement. The PSD design requirement is dependent on many parameters such as, the passing vehicles' speeds, the passed vehicles' speeds, vehicle lengths, deceleration rate, etc. They are random variables and represent a wide range of human and vehicle characteristics. Current PSD design practices replace these random variables by single-value means in the calculation process disregarding their inherent variations. The use of the current design practice results in a single-value PSD design criteria. A method that accounts for the randomness of each of the parameters in the development of the PSD requirement is proposed in this dissertation.

As mentioned earlier, the main objectives of this research effort are to assess the risk and the level of service of various PSDs in order to conduct a design trade-off analysis. Three different simulation setups are devised and used to achieve these goals. The first simulation is a

Monte Carlo simulation that aims at determining the probability distribution of the PSD by accounting for the variations in all the influencing parameters. The second simulation focuses on attaching risk measures to the values of the PSD obtained from the first simulation. The third simulation is concerned with finding the level of service of the various PSDs already obtained from the first simulation. By combining the results of the three simulations, a design trade-off analysis is feasible. Inherent to the first two simulations are the input parameters that influence the PSDs. This chapter discusses the input parameters, their characteristics, and their probability density functions based on data collected in from the literature or the field.

### **4.3 Probability Distributions of the Contributing Parameters**

Literature and field data sources are used to assign distributions for each of the contributing parameters. The contributing parameters may be grouped as follows:

- a) *Driver/Demand related parameters*, such as:
  - Regular traffic speed relative to various design speed levels
  - Speed differential between the passing and the impeding vehicle
  - Driver perception reaction times
  - Clearance distances required to complete a passing maneuver
  - Minimum gap distances required to abort a passing maneuver
- b) *Vehicle related parameters*, such as:
  - Deceleration rates
  - Acceleration rates
  - Percent of vehicles in each vehicle class
  - Vehicle lengths classified by vehicle class

Some of the parameters have known probability density functions. Others were assigned density functions based on their characteristics.

#### **4.3.1 Vehicle Speeds ( $V$ )**

In most cases, the probability density function of vehicle speeds follows a normal distribution. To avoid negative values of speeds, a truncated normal distribution is usually used. Vehicle speed profiles vary with the various posted speed limits. Three speed profiles are considered in this dissertation where the design speeds are 40, 50 and 60 mph. Field data were only collected on a two-lane road in VA, for the design speed of 50 mph. The other two data sets were derived by subtracting/adding 10 mph to each speed value. For the collected speed counts, a truncated normal distribution was found to best fit the filtered speed data with a mean of 53.6

mph and standard a deviation of 4.05 mph, as shown in Figure 4.1 and 4.2. The other two sets were similarly fit.

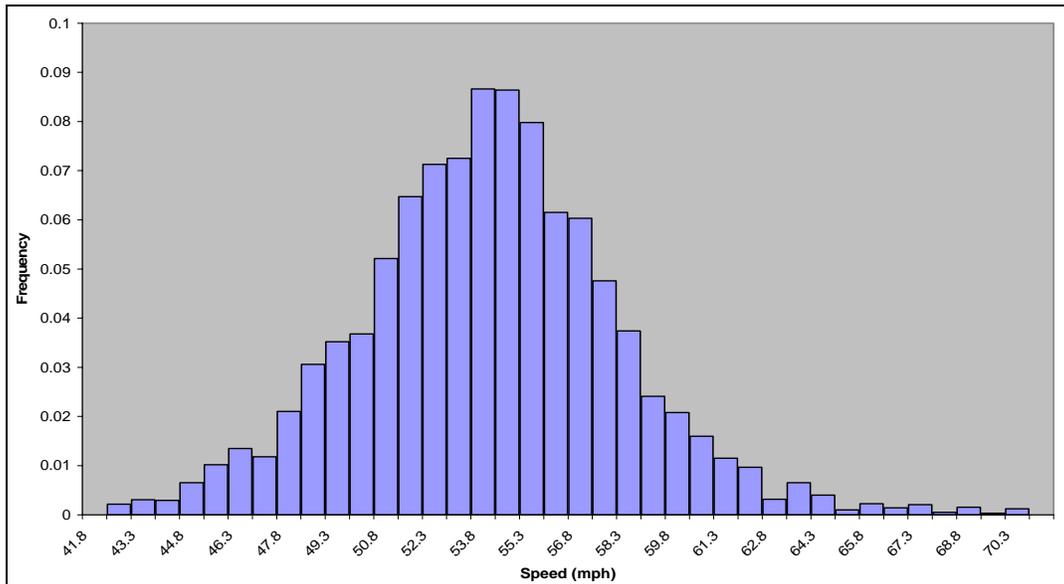


Figure 4.1 – Fitted speed profile at 50 mph design speed

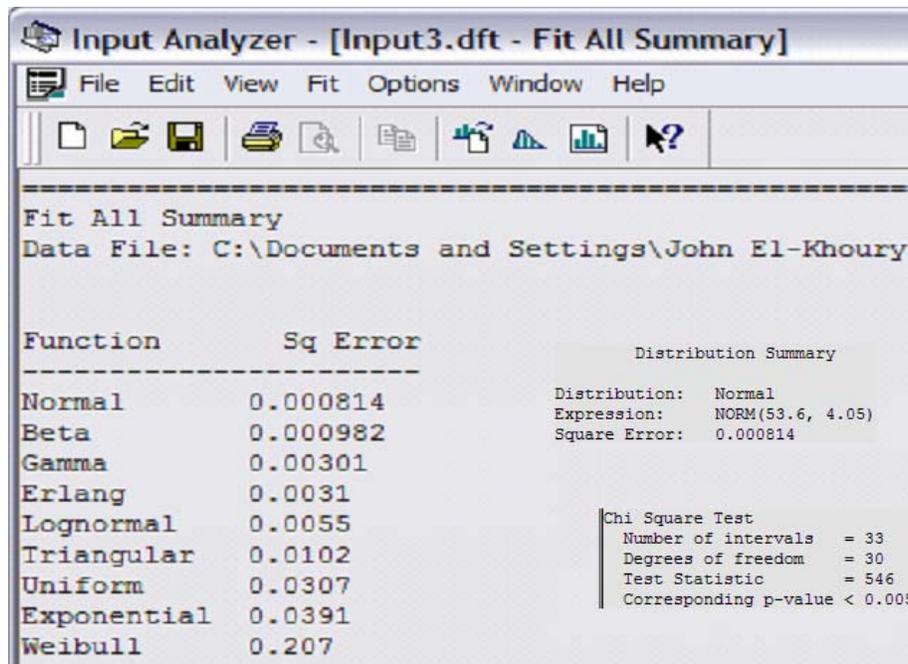


Figure 4.2 – Fitted speed profile parameters

### 4.3.2 Speed Differential (m)

Speed differential is the difference in speed between the passing vehicle and the impeding vehicle. Many studies have discussed this parameter but ended up assuming one value for it. The author believes that a distribution of the speed differentials better describes the behavior of a wider range of drivers. Various modelers have assumed different values of ‘m’ in

their calculations. AASHTO (2004) for example assumed a fixed speed differential of 10 mph. MUTCD (FHWA 2000) criteria specify various speed differentials for various design speeds, where the difference increases as the design speed increases. MUTCD values increased from 10 to 25 mph for speeds ranging from 30 to 70 mph, respectively. Glennon (1988) assumed that the speed differential is negatively correlated with the design speed. The values decreased from 12 to 8 mph for speeds increasing from 30 to 70 mph, according to the following formula:

$$m = 15 - \frac{V_p}{10} \quad (4.1)$$

Where,

m = Speed differential, (mph)

V<sub>p</sub> = Passing vehicle speed, (mph).

The values adopted by Harwood were close to those collected by Polus. In his study, Polus analyzed 1,500 passing maneuvers (Polus et al. 2000). He collected data about the different parameters influencing the passing maneuver, one of which is the speed differential. To account for all the aforementioned assumptions, a Log-Normal distribution was found to best fit the variations in the speed differential parameter, with a mean and standard deviation of 11 and 2.0 mph, respectively. This mainly ensures that the most values are concentrated between 8 and 15 mph. The minimum and maximum values are assumed to be 5 and 25 mph, respectively. Figure 4.3 portrays the log-normal curve of the speed differential. Note that the units are changed to ft/sec instead of mph.

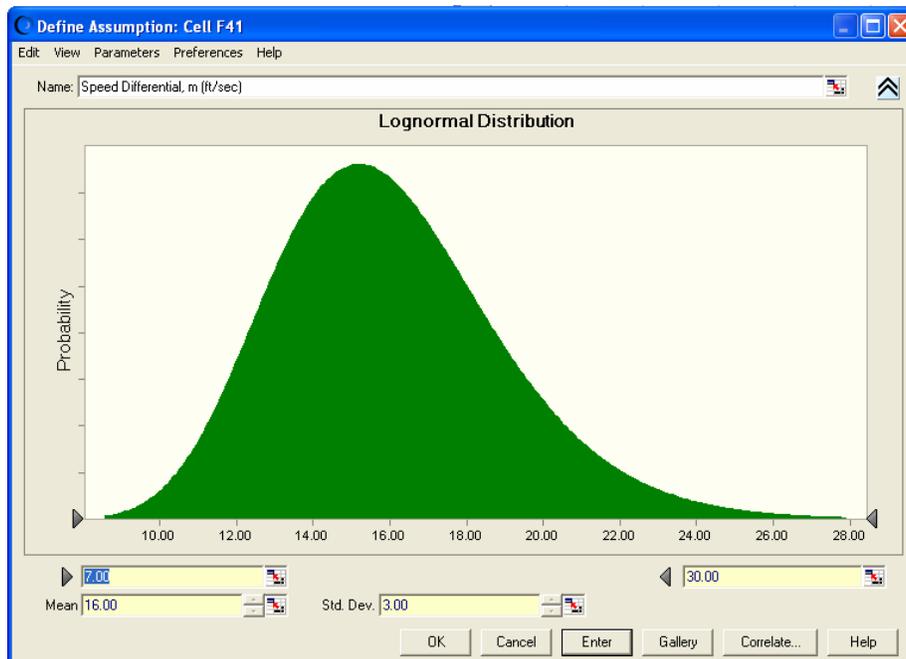


Figure 4.3 – Speed differential profile fit at 50 mph design speed

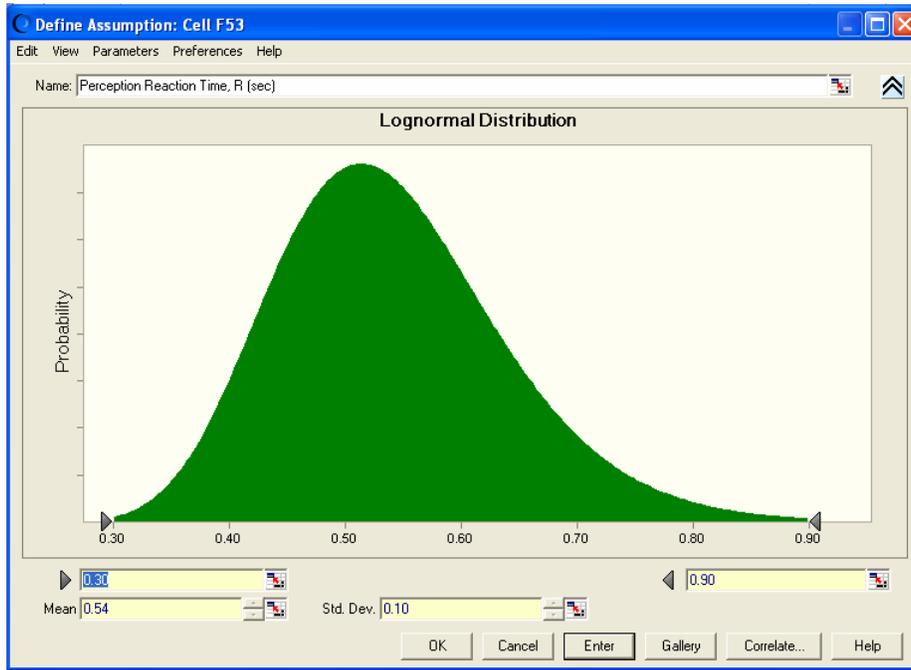
### 4.3.3 Braking Perception Reaction Time (R)

Perception-reaction time is the time lag between the detection of an input (stimulus) and the initiation of a driving response, in this case, braking. Driver's braking response is composed of two parts, the perception-reaction time (PRT) prior to the actual braking of the vehicle, and the movement time immediately following. Several PRT models were formulated. Hooper and McGee (1983) formulated a very typical model with such components for braking response time (including movement time MT). Neuman (1989) has proposed perception-reaction times (PRT) for different types of roadways, ranging from 1.5 seconds for low-volume roadways to 3.0 seconds for urban freeways (Lerner 1995). Lerner summarized the values of brake PRT from a wide variety of studies. Two types of response situations were summarized. The first case is when the driver does not know, ahead of time, if the stimulus for braking will occur. That is, he or she is surprised. The second case is when the driver knows a braking condition might occur and thus is expecting it. The obtained results are presented in Table 4.1.

**Table 4.1 - Brake PRT Comparison (in Seconds)**

	<b>“Surprised”</b>	<b>“Expected”</b>
Mean	1.31	0.54
Standard Deviation	0.61	0.1
50 <sup>th</sup> Percentile	1.18	0.53
85 <sup>th</sup> Percentile	1.87	0.64
95 <sup>th</sup> Percentile	2.45	0.72
99 <sup>th</sup> Percentile	3.31	0.82

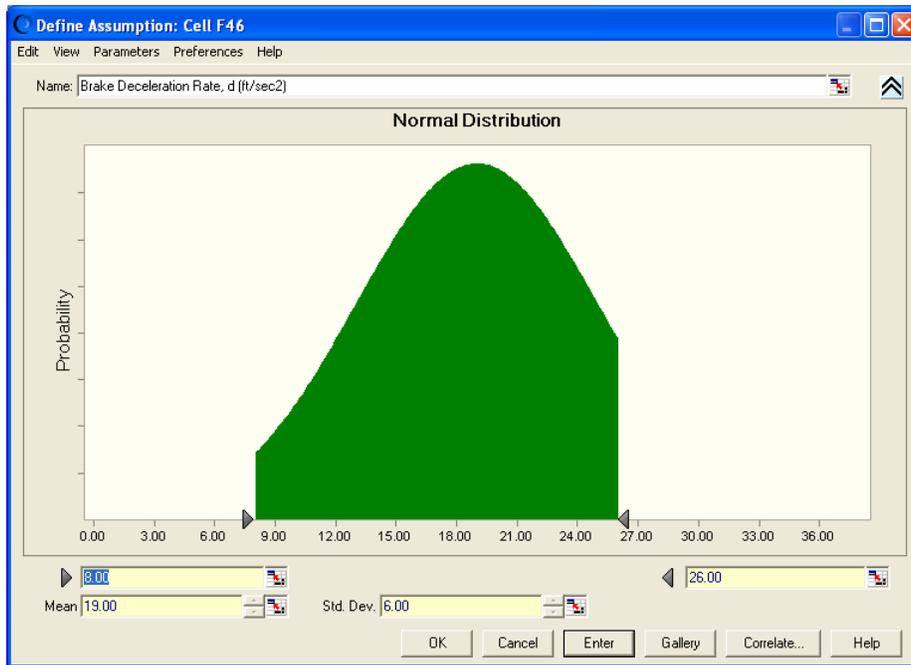
The composite data of sixteen studies of braking PRT were converted to a Log-Normal transformation. The “Expected” values of the braking PRT are used in this study since the driver conducting a passing maneuver is obviously aware of the fact that he/she might need to brake at any time. The corresponding mean and standard deviation of the braking PRT are 0.54 and 0.1 seconds, respectively. Figure 4.4 portrays the curve which was fit to the braking perception reaction times.



**Figure 4.4 – P DF fit of the perception reaction times**

#### **4.3.4 Deceleration Rate (*d*)**

Deceleration rate is mainly composed of the driver's foot-action against the brake pedal and the vehicle's braking capabilities. It is a human initiated action which has shown to have wide range of variations. Hence, it is a random response among different drivers or even among same drivers using different vehicles or under different driving conditions. Fambro studied the variations in the braking performance (Fambro 1994, et al. 200). He distinguished between two cases of braking performance where, in the first, it is assumed that the driver is surprised by the action while, in the second, he/she is expecting to brake or decelerate. The summary of the study showed that 95 percent of the drivers will produce a deceleration rate of at least 0.3g on wet pavements without Anti-lock Braking System (ABS). For the same conditions, the mean maximum deceleration rate was found to be 0.75g. Fambro noted that the mean deceleration rate for all drivers is 0.6g with a standard deviation of 0.19g. In this research, a normal distribution is used since only the mean and the standard deviation are known. The normal distribution is truncated by a minimum and a maximum value of 0.25g and 0.8g, respectively. Figure 4.5 presents the truncated normal distribution of the deceleration rate in units of  $\text{ft}/\text{sec}^2$ .



**Figure 4.5 – PDF of the deceleration rate of the passing driver**

#### ***4.3.5 Vehicle Lengths and Percentages***

The distribution of vehicle lengths to be used in the modeling is based on the percentage of each of the vehicle classes driving on the road. The national vehicle composition is used in this paper, which is based on the 2002 United States Census Bureau of Vehicle Inventory and Use Survey (US Census bureau 2004) and the FHWA Highway Statistics 2002 (FHWA 2002). The frequency of each of the vehicle classes are calculated based on the Vehicle Miles of Travel (VMT), which represents the percent of road usage by these types of vehicles. The data is shown in Table 4.2. The data could not be adequately fit with any common continuous distribution. Thus, four vehicle classes are assumed in the simulations to represent most of the vehicles. The vehicle classes, lengths, and percentages are as follows:

- Light Vehicles (LV): 19 ft long and amounting to 91 % of traffic.
- Medium Vehicles (MV): 26 ft long and constituting 5% of traffic.
- Heavy Vehicles (HV): 41 ft long and are 1% of the traffic.
- Very Heavy Vehicles (VHV): 66 ft long and are 3% of the traffic.

**Table 4.2 – Frequency of vehicle classes by vehicle lengths**

Length Range (ft)	Length (ft)	Vehicle-Miles	Frequency	
less	20	19	2524452.7	0.9102
20	27.9	24	101006.5	0.0364
28	35.9	32	39685.1	0.0143
36	40.9	38.5	13676	0.0049
41	44.9	43	4322.8	0.0016
45	49.9	47.5	5118.2	0.0018
50	54.9	52.5	5414.1	0.0020
55	59.9	57.5	10063.2	0.0036
60	64.9	62.5	17561.5	0.0063
65	69.9	67.5	21406.1	0.0077
70	74.9	72.5	20632.7	0.0074
75	79.9	77.5	8955.8	0.0032
80	more	80	1073.3	0.0004

#### 4.3.6 Vehicle Following Gap ( $G$ )

The space between the passing vehicle and the passed vehicle at the end of the maneuver is referred to as the following gap ( $G$ ). AASHTO does not mention this distance in the design of the PSD, neither does MUTCD. Glennon (1988) assumes a minimum following gap equal to 1 sec of time headway, thus, using distances ranging from 11 to 18 ft for speeds ranging from 30 to 70 mph. To simplify the formulation, Glennon assumes the same clearance distance for both the completed and aborted passes. The authors propose to use two clearance distances to represent each case.

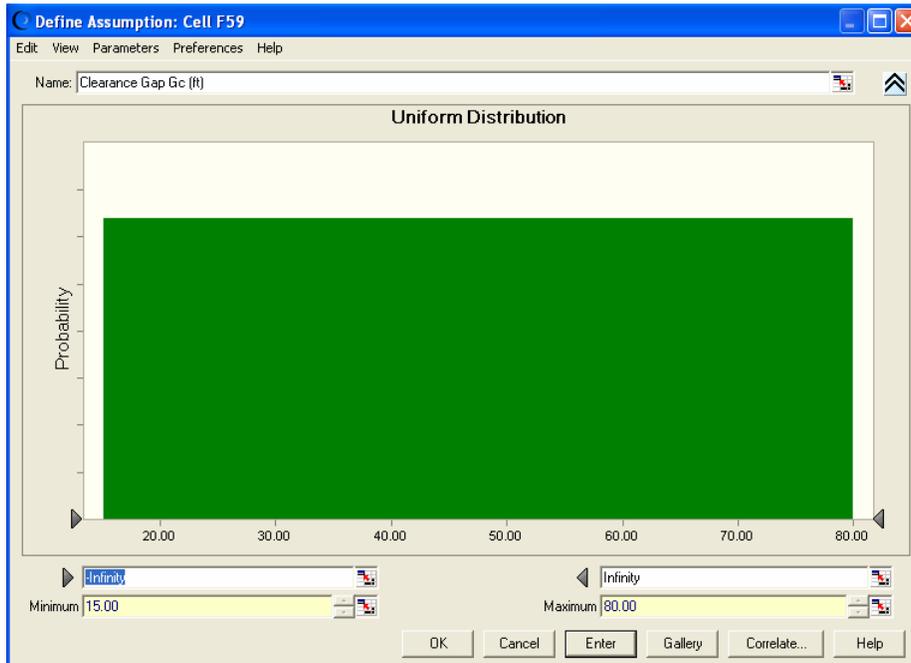
##### 4.3.6.1 Completed Pass Gap ( $G_C$ )

This is the clearance distance between the passing and passed vehicle towards the end of the completed pass. Data collected by Polus show that the passing vehicle merges ahead of the impeding vehicle at an average distance of 70 ft (Polus et al. 2000). These values represent passing maneuvers on highway stretches with unlimited sight distances. They are not representative of the critical passing conditions where sight distance is limited. Hence, to account for most situations, a Uniform distribution is used to represent the variation of  $G_C$  with a minimum of 15 ft and a maximum of 80 ft. The probability density function of  $G_C$  is shown in Figure 4.6.

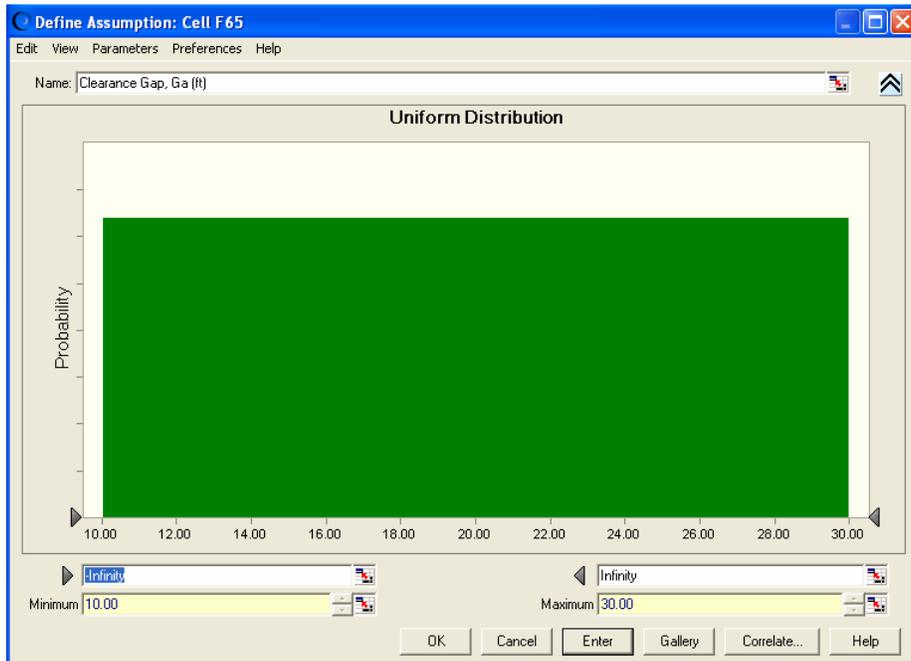
##### 4.3.6.2 Aborted Pass Gap ( $G_A$ )

This is the clearance distance between the passing and passed vehicle in case the pass is aborted. In this case, the passing situation is obviously critical since the passing driver is trying to avoid oncoming traffic by aborting the pass. Data is also lacking about this parameter. Emergency setback distances as low as 10 ft are mentioned in the literature. A Uniform

distribution is used to account for the variation of  $G_A$ , where the minimum and maximum values are 10 and 30 ft, respectively. Figure 4.7 presents the probability density function of  $G_A$ .



**Figure 4.6 – PDF of the completed pass gap,  $G_C$**

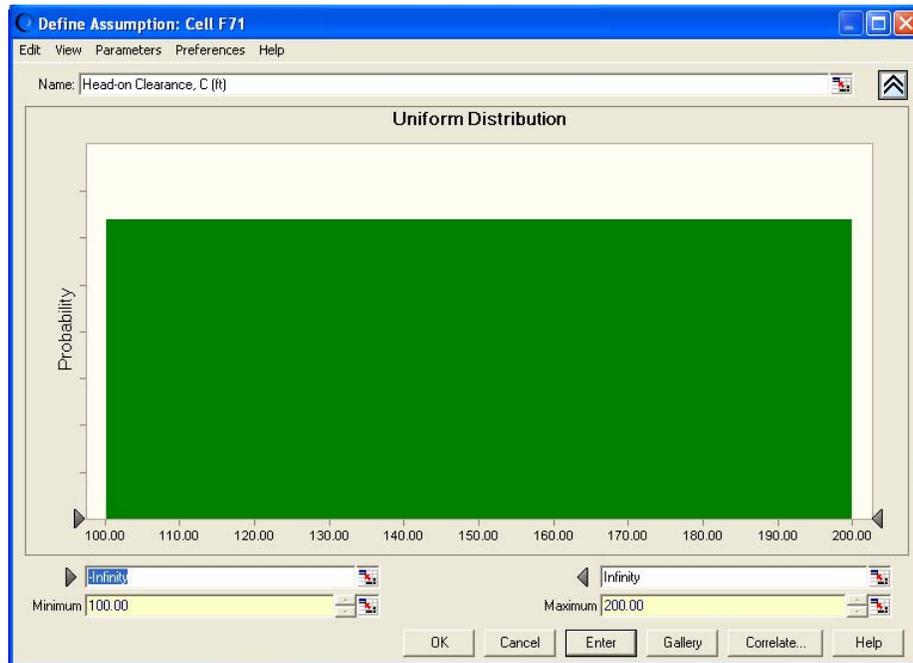


**Figure 4.7 – PDF of the aborted pass gap,  $G_A$**

#### 4.3.7 Clearance Gap ( $C$ )

This is the clearance distance between the passing and opposing vehicles at the end of the passing maneuver. In deriving his model, Glennon assumed a minimum head-on clearance of 1 second time headway, which is about 118 ft for a 40 mph design speed, 160 ft for a 50 mph, and

176 ft for a 60 mph (Glennon 1988). AASHTO 2004 considered values ranging from 100 to 300 ft for passing speeds of 35 to 62 mph. Polus uses head-on clearance of about 120 ft for the different speed levels (Polus et al. 2000). A Uniform distribution is used to represent the values of C with minimum and maximum bounds of 100 ft and 200 ft, respectively. The probability density function used for C is shown in Figure 4.8.



**Figure 4.8 – PDF of the Clearance gap, C**

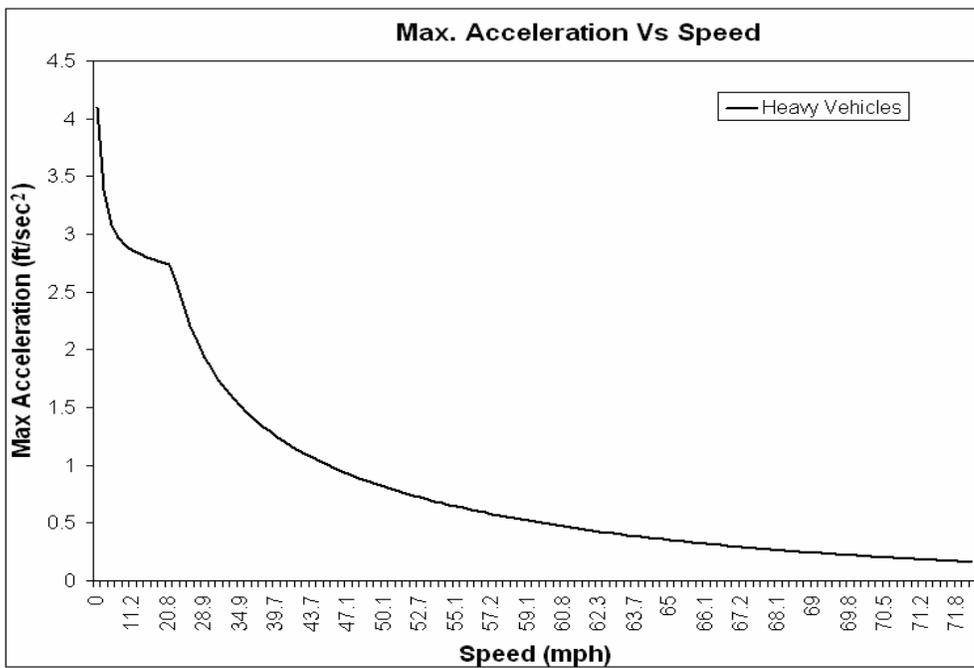
#### **4.3.8 Acceleration Rate**

The acceleration rate parameter is used in one of the simulations only. It is not used in the Monte Carlo simulation since the adopted PSD formulation does not include acceleration in the equation. The second simulation which is devised in order to attach risk measures to the values of the PSD distribution accounts for the acceleration parameter. The passing vehicle is the only vehicle which accelerates during the maneuver. Research has demonstrated that overtaking acceleration is typically 65 percent of the maximum acceleration for a vehicle under “unhurried” circumstances (Halati et al. 1997). The passing driver is assumed, in the simulation, to adopt maximum acceleration for two reasons. First, the driver is passing while there is an oncoming vehicle in the opposite direction. Second, all the simulated passes are assumed to be accelerative; thus, the driver is in a hurried situation. An accelerative pass is a pass in which the faster vehicle slows to the speed of the impeding vehicle before initiating the passing maneuver. This accounts for the worst case scenario of the passing maneuvers. The constant power model is utilized to derive the acceleration of the light and medium vehicles (Rakha et al. 2004), while the variable

power model is used to obtain the acceleration for the two heavy vehicle classes (Rakha et al. 2002, 2001). The model produces different maximum accelerations depending on the current vehicle speed. Table 4.3 presents the values of maximum acceleration adopted in the simulation. Figure 4.9 presents a sample curve that was generated for heavy vehicles. The curve plots the maximum acceleration of a heavy truck versus the current speed using the variable power model (Rakha et al. 2002).

**Table 4.3 – Maximum acceleration (ft/sec<sup>2</sup>)**

Design Speed (mph)	Vehicle Class			
	LV	MV	HV	VHV
40	11.0	7.5	1.5	1.2
50	10.6	5.6	1.2	0.8
60	10.0	4.2	0.6	0.5



**Figure 4.9 – Maximum acceleration curve for heavy vehicles (Rakha et al. 2002)**

#### 4.4 Pre-Simulation Setup

The input parameters which have been discussed in this chapter will be used in the first two simulations. The current design methods do not account for the variations in the PSDs which are accrued from the variations of the different influencing parameters. These variations will be captured by using a Monte Carlo simulation technique. The Monte Carlo method has been in use for the last couple decades. It mainly involves random number generation and random sampling from the various input probability density functions. Since the author plans to simulate multiple scenarios to assess the robustness of the simulation and check its sensitivity to various parameters,

variance reduction techniques come in handy. These techniques are also used in the setup for the second simulation, which is dedicated to obtaining the risk measures.

#### 4.4.1 Variance Reduction Techniques

Modelers try to compare results that are generated out of random inputs and apply intensive statistical measures to bring the variation in the results to a minimum. These statistical measures might sometimes be very expensive that the results are not as desirable. Thus, the Variance Reduction Techniques (VRT) are very important in increasing the efficiency of the model by implementing certain defined procedures to the input in order to bring the output of different random inputs to a minimum variance. This means obtaining smaller confidence intervals with the same simulation time and trials. Thus, the results will exhibit higher precision (Law and Kelton 2000). The five most famous techniques used for variance reduction are:

- *Common Random Numbers (CRN)*: This is the simplest but mostly used Variance reduction technique. The basic idea behind the CRN is comparing the results of the simulation for two dependent sets of input random variables of circumstances that are generated based on the same input random variables. In other words, the variation of the simulation results would be due to the system configuration and not to the fluctuation of the input “circumstances”. Based on Law and Kelton’s rationale (Law and Kelton 2000), the proof is very simple and logical. Consider two sets of observations,  $X_{1j}$  and  $X_{2j}$  where we want to calculate the estimator  $\zeta = \mu_1 - \mu_2 = E(X_{1j}) - E(X_{2j})$ . If we do  $n$  replications of each system, then the estimator  $Z_j = X_{1j} - X_{2j}$  for  $j = 1, 2, \dots, n$ , and then  $E(Z_j) = \zeta$ . Thus,

$$\bar{Z}(n) = \frac{\sum_{j=1}^n Z_j}{n}$$

then,

$$Var[\bar{Z}(n)] = \frac{Var(Z_j)}{n} = \frac{Var(X_{1j}) + Var(X_{2j}) - 2Cov(X_{1j}, X_{2j})}{n}$$

If the simulation of the two setups is done independently, then  $X_{1j}$  and  $X_{2j}$  will be independent, which means that the covariance will be zero. But if the Common Random Numbers technique is used, then the configurations will be dependent of each other and thus the variables  $X_{1j}$  and  $X_{2j}$  will be dependent and the covariance will be a positive number subtracted from the Variance of the two observations, thus decreasing it to a lower confidence level. This means achieving higher precision.

- *Antithetic Variates*: This is the second VRT where negative correlation between a sample pair is sought so that taking the average of the sample pair will result in a closer value to the mean. For the sample pair, a random variable for one pair is used while the complement of that is used for the other pair, which is still valid. That is we use  $U_k$  for one system configuration and  $1-U_k$  for the corresponding pair of that system configuration. The two complement pairs of random numbers should also be synchronized for this technique to work. Then, the random number and its complement are used for the same purpose. The rationale is similar to the first technique (Law and Kelton 2000). Consider two sets of observations,  $X_{1j}$  and  $X_{2j}$  where the first set is generated from the regular random number and the second set is generated from the antithetic of that first set (the complement). For  $n$  trials of the system, we can calculate the average of both values as  $X_j = (X_{1j} + X_{2j})/2$ , for  $j = 1, 2, \dots, n$ , and then,

$$\bar{X}(n) = \frac{\sum_{j=1}^n X_j}{n}$$

then,

$$Var[\bar{X}(n)] = \frac{Var(X_j)}{n} = \frac{Var(X_{1j}) + Var(X_{2j}) + 2Cov(X_{1j}, X_{2j})}{4n}$$

If the simulation of the two setups is done independently, then  $X_{1j}$  and  $X_{2j}$  will be independent, which means that the covariance will be zero. But if the Antithetic Variate technique is used, then the configurations will be dependent and thus the variables  $X_{1j}$  and  $X_{2j}$  will be dependent. So, the covariance in this case will be a negative number added to the Variance of the two observations, thus decreasing it to get a better confidence interval. The fundamental requirement of any model to work under Antithetic Variate technique is that it should have a monotonic response to random numbers.

- *Control Variates*: This is the third technique in the list of VRT. This is very simple technique too. Consider an output random variable  $X$  that is dependent on some random input variables. Then,  $\mu = E(X)$ . Consider another random variable that is correlated with  $X$ , to be  $Y$  where the mean  $\nu = E(Y)$ .  $\nu$  is supposed to be known since the simulation was conducted and the results of the random output  $Y$  are known. If  $X$  and  $Y$  are really correlated then when  $X$  fluctuates away from its mean,  $Y$  will also vary away from its mean in a direction either similar to that of  $X$  or opposite (positively or negatively related to  $X$ ). If for a certain run,  $Y$  is larger than its mean  $\nu$ , then the random mean of  $X$  of that

run is also larger than its mean  $\mu$  (assuming that they are positively related). Then, varying  $X$  for that run downward will bring it closer to its mean and will also affect the result of  $Y$  bringing it closer to its mean. To know how much  $X$  should be varied, the difference between  $Y$  and its mean is calculated and then the following equation is used to vary  $X$ :

$$X_c = C - a(Y - \nu),$$

where  $a$  is a positive number. Then the variance of  $X_c$  is as follows:

$$\text{Var}(X_c) = \text{Var}(X) + (a^2) \cdot \text{Var}(Y) - 2 \cdot a \cdot \text{Cov}(X, Y)$$

Then  $X_c$  is less variable than  $X$  if:  $2 \cdot a \cdot \text{Cov}(X, Y) > (a^2) \cdot \text{Var}(Y)$

Solving for the optimal value (Variance-minimizing),  $a^* = \text{Cov}(X, Y) / \text{Var}(Y)$ .

But,  $\text{Cov}(X, Y)$  and the  $\text{Var}(Y)$  are not always known. and so on, so what we can do is get the mean of the variables  $X_j$  and  $Y_j$  where  $j=1, 2, \dots, n$ . Then, we can assign a new value for  $a^*$  that is:

$$a_1^*(n) = C_{XY}(n) / S_y^2(n) \rightarrow \overline{X_c^*}(n) = \overline{X}(n) - a_1^*(n)[\overline{Y}(n) - \nu]$$

Although, this might be a little biased since  $a_1^*(n)$  is not totally independent from mean of  $Y_j$ 's.

- *Indirect Estimation*: This is the fourth Technique in the VRT list of procedures which was developed for queuing type simulations (Law and Kelton 2000). Thus, it will not be discussed further since it exceeds the scope of this research.
- *Conditioning*: This is the fifth and last type of VRT (Law and Kelton 2000). It aims at replacing an estimate of the system by the exact analytical value in order to decrease the variance of the output result. The idea is to observe a random variable say  $E[X/Z]$  rather than observing directly the random variable  $X$  in order to get a smaller variance. The trick is to have  $Z$  easily and efficiently generated since it needs to be simulated. Also, it is required to have the value  $E[X/Z = z]$  as a function of  $z$  easily and analytically computed for any possible value of  $z$ . It is clear that this type of VRT is heavily dependent on the form of model used. The basic proof formula for this technique is:  $\text{Var}_Z[E(X/Z)] = \text{Var}(X) - E[Z \cdot \text{Var}(X/Z)] \leq \text{Var}(X)$  (thus reducing the variance of  $X$  into a smaller value.)

Only the first method on variance reduction technique is used in both simulation setups. The same random streams were used by specifying the same seed values for the various simulation scenarios. This way, when the results of the simulation are compared, the variations in the results are only caused by the variation in the parameters in the simulation and not by the random

number sampling process. In other words, the results of the simulations are based on dependent random numbers and thus the Common Random Number techniques apply.

## **CHAPTER 5:**

### **PASSING SIGHT DISTANCE DISTRIBUTION**

## **5.1 Introduction**

Most of the geometric design formulations provide limit values as design requirements for certain highway elements. The design of most highway elements includes parameters that are random variables. These parameters traditionally describe the range of human and vehicle behaviors including road characteristics. They should be represented by random variables with adequate probability density functions. Instead, current highway design formulations use one-value averages to replace these parameters in the equations. One of the geometric design elements that is influenced by random parameters is the passing sight distance (PSD) requirement. The PSD design requirement is dependent on many parameters such as, the passing vehicles' speeds, the passed vehicles' speeds, vehicle lengths, deceleration rate, etc. They are random variables and represent a wide range of human and vehicle characteristics. Current PSD design practices replace these random variables by single-value means in the calculation process, disregarding their inherent variations, which results in a single-value PSD design criteria. This chapter focuses on describing a new approach that accounts for the random distribution of each of the parameters in the development of the PSD requirements. Two models are devised for this purpose, a Monte-Carlo simulation model and a closed form analytical estimation model. The Monte-Carlo simulation model uses random sampling to select the values of the contributing parameters from their corresponding distributions in each run. A different PSD value is calculated in each trial representing a different set of conditions. The analytical model accounts for each parameter variation by using their means and standard deviations in a closed form approximation method.

## **5.2 Monte Carlo Method**

### ***5.2.1 Definition and Background***

Monte Carlo methods are one type of statistical simulation methods. The latter is defined to be any method that utilizes sequences of random numbers to perform the simulation. Monte Carlo methods have been used for centuries. But it gained the status of a full-fledged numerical method in the past several decades. It is capable of addressing the most complex applications. The name "Monte Carlo" was coined by Metropolis during the Manhattan Project of World War II for two reasons. The first is due to the similarity of statistical simulation to games of chance. The second is because the capital of Monaco was a center for gambling and similar pursuits. Monte Carlo is now used routinely in many diverse fields, from the simulation of complex physical phenomena to the mundane.

Statistical simulation methods may be contrasted to conventional numerical discretization methods. Typically, numerical methods are applied to ordinary or partial differential equations that describe some underlying physical or mathematical system (Law and Kelton 2000). In many applications of Monte Carlo, the physical process is simulated directly. There is no need to write down the differential equations that describe the behavior of the system. The only requirement is that the physical (or mathematical) system be described by probability density functions (pdf's). Once the pdf's are known, the Monte Carlo simulation can be performed by random sampling from the pdf's. Many simulations are then performed and the desired result is taken as an average over the number of observations. In many practical applications, one can predict the statistical error (the “variance”) in the result, and thus, estimate the number of Monte Carlo trials that are needed to achieve a given error (Law and Kelton 2000). Assuming that the evolution of the physical system can be described by probability density functions (pdf's), then the Monte Carlo simulation can proceed by sampling from these pdf's. This necessitates a fast and effective way to generate random numbers uniformly distributed on the interval  $[0,1]$ . The outcomes of these random samplings must be accumulated in an appropriate manner to produce the desired result. An essential characteristic of the Monte Carlo is the use of random sampling techniques to arrive at a solution of the physical problem. In contrast, a conventional numerical solution approach would start with the mathematical model of the physical system, discretizing the differential equations and then solving a set of algebraic equations for the unknown state of the system (Law and Kelton 2000).

### ***5.2.2 Major Components of a Monte Carlo Method***

This section describes briefly the major components of a Monte Carlo method. These components comprise the foundation of most Monte Carlo applications. An understanding of these major components will provide a sound explanation of why this method is used in the context of this research. The primary components of a Monte Carlo simulation method include the following:

- *Probability distribution functions (pdf's)* - the physical (or mathematical) system must be described by a set of pdf's. The pdf's of the parameters influencing the PSD are presented in the previous chapter of the dissertation. The data has been collected from field investigations as well as the literature.
- *Random number generator* - a source of random numbers uniformly distributed on the unit interval must be available. A computational random number generator is more

accurately called a “pseudorandom” number generator since the sequence is generated by a specific algorithm. It can be replicated exactly to yield an identical sequence. A numeric value, called a seed, is used in the algorithm to produce the random number stream. This seed is often based on the computer's local time at the moment the computer code is executed, thus reducing the probability of producing the same starting sequence twice (Law and Kelton). However, the same seed was used in to apply the Common Random Number technique, which was explained in chapter 4 of this dissertation. Various methods were invented to generate random numbers of which the most famous are the Linear Congruential method and the Combined Linear Congruential method. Since many random generators were introduced, tests were devised to assess their goodness, such as the Frequency test, the Runs test, the test for Autocorrelation, the Gap test, and the Poker test among others. The two software packages that are considered in this research are the Crystal Ball and ARENA, which have been validated for all these tests and been recognized for adequate random number generation.

- *Scoring (or tallying)* - the outcomes must be accumulated into overall tallies or scores for the quantities of interest.
- *Error estimation* - an estimate of the statistical error (variance) as a function of the number of trials and other quantities must be determined. The two major estimates to be calculated are the estimate of the mean and that of the variance. Then, using these two values, a confidence interval will be calculated for the PSD. The PSD curve which will be obtained from the Monte Carlo simulation will be subjected to numerous tests to assess its goodness-of-fit to a certain distribution using Chi-Square test, Kolmogorov-Smirnov test, and the  $p$ -Values test.
- *Variance reduction techniques (VRT)* – are methods for reducing the variance in the estimated solution to reduce the computational time for Monte Carlo simulation. There are various VRTs described in the literature, as was discussed in an earlier chapter.
- *Parallelization and vectorization* - algorithms to allow Monte Carlo methods to be implemented efficiently on advanced computer architectures. This will not be utilized in this research and thus is out of the dissertation scope.

### **5.3 The Devised Simulation**

Based on the aforementioned concepts, the author devised a unique Monte Carlo simulation. It is conducted using the software package, Crystal Ball. Crystal Ball is used to

perform the random sampling from the pdfs of the different parameters according to the revised Glennon's formulation of the PSD.

### ***5.3.1 Crystal Ball Suite***

Crystal Ball is a full suite of Microsoft® Excel-based applications for Monte Carlo simulation, time-series forecasting, optimization and real options analysis. Crystal Ball has an embedded best-fit tool that helps predict the distribution of the forecasted cell(s). It is the easiest way to perform risk analysis in your own spreadsheets. With one integrated toolset, one can use historical data to build accurate models, automate "what if" analysis to understand the effect of underlying uncertainty and search for the best solution or project mix. Crystal Ball also includes improved reports, charts and graphs that let one vividly present and communicate the results of the analysis, and give a credible picture of risk. This software suite includes **OptQuest**, which automatically search for the optimal solution, accounting for uncertainty and constraints; and **CB Predictor**, which analyzes historical data to build the model, with time-series forecasting and multiple linear regression.

### ***5.3.2 Revised PSD Model***

Glennon's formulation logically represents the mechanics of the passing maneuver but is based on many assumptions. The model and all its parameters are discussed in Chapter 2 of this dissertation. The detailed derivation of the model can be reviewed in 'Glennon 1988'. In his calculation, Glennon substitutes single-value averages for the parameters used in the formulation when in fact they are random variables. These parameters should be assigned adequate probability distributions. By using Glennon's formulation, highway engineers are limited to one PSD value when designing a two lane road, with no risk measure or level of service attached to it. However, by accounting for the variability of each of the parameters, a distribution of PSD is obtained. With it, the traffic engineer could select a value from the distribution based on the trade-off analysis between a desired level of service and safety index.

In this research, the author proposes a new version of Glennon's model by minimizing the number of assumptions throughout the derivation. The last four simplifying assumptions that Glennon used are not adopted. A new variable is introduced to represent the perception reaction time (R), which has been assumed as constant in Glennon's formulation. Glennon's assumption, that the clearance between the passing and impeding vehicles, G, is the same for the abort and for the complete pass scenarios, is too simplistic. Two different variables are used to represent the clearance distances in the new formulation, which are  $G_A$  and  $G_C$ . Following the same

derivation procedure as that of Glennon, the new formulation is expressed as follows (note that all other variables have been defined in Chapter 2 of the dissertation):

$$\Delta_c = L_p + G_c - m \times R + m \left\{ \frac{(G_c + G_A + L_l + L_p)}{(2V - m)} - \sqrt{\frac{4V(G_c + G_A + L_l + L_p)}{d(2V - m)}} \right\} \quad (5.1)$$

$$PSD_c = C + \frac{2V(G_c + L_p - \Delta_c)}{m} \quad (4)$$

where, R = Perception reaction time in case of an aborted pass, (sec)

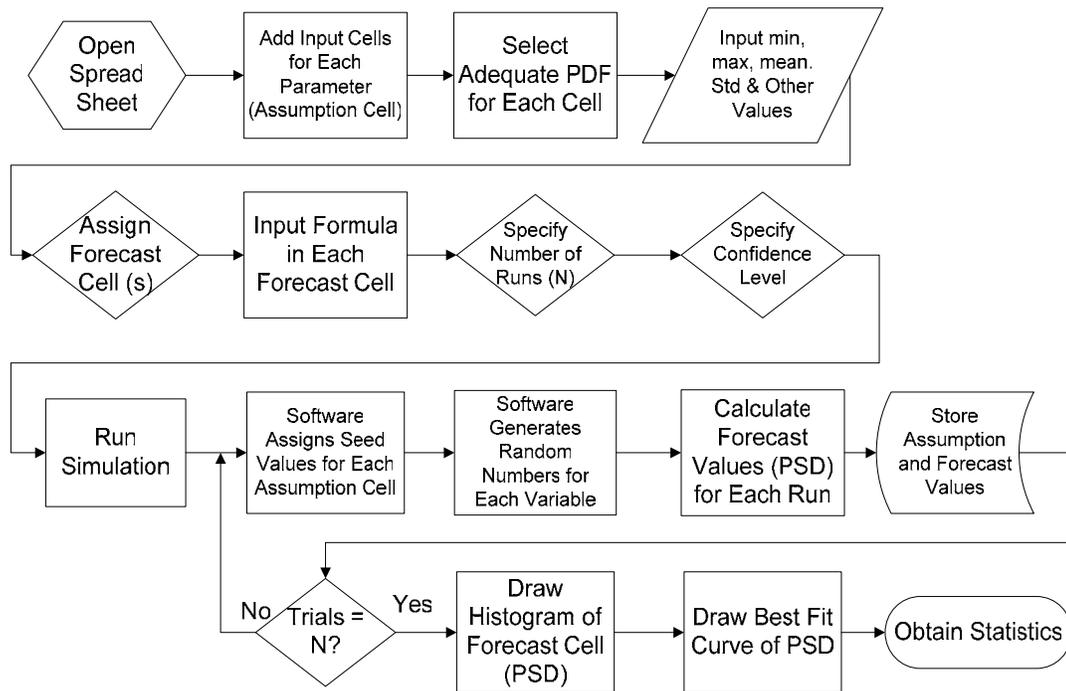
G<sub>C</sub> = Clearance distance between passing and impeding vehicle at the end of the completed pass, (ft)

G<sub>A</sub> = Clearance distance between passing and impeding vehicle at the end of the aborted pass, (ft).

### 5.3.3 The Monte Carlo Simulation Model

A spreadsheet model is build to incorporate all the parameters with their distributions. Each parameter is given an “assumption” cell in the spreadsheet. The assumption cell is the cell which incorporates the characteristics of the parameter’s distribution. Twenty different distributions are available within Crystal Ball, plus one custom fit distribution which allows for incorporating uncommon distributions. These parameters are then related using a specific equation; in this case, it is the Glennon formulation. The equation is embedded into the “forecast” cell. The forecast cell is the cell which provides the modeler with the results. Once the model is run, the random sampling occurs from the different probability density functions of the influencing parameters, and the results are stored into the forecast cell. It is similar to an output module in any other simulation. Output statistics about the forecast cell are collected based on the modeler’s preferences. The flowchart of the Monte Carlo simulation is shown in Figure 5.1. A snapshot of the actual input model used in the Monte Carlo simulation is shown in Figure 5.2. Figure 5.3 presents the output model which includes the forecast cells. There could be more than one forecast cell in a certain model. One or more forecast cells could be part of the calculation of another forecast cell. Once the forecast cell is calculated for one trial, it becomes as an assumption cell relative to the final forecast cell. Two forecast cells are specified in the Monte Carlo model. Based on Glennon’s formulation, the calculation of the critical passing sight distance (PSD<sub>C</sub>) requires calculating the value of the critical point (Δ<sub>C</sub>). So, the first forecast cell is the critical point (Δ<sub>C</sub>). The second and final forecast cell is the passing sight distance (PSD<sub>C</sub>). In this research effort, twenty thousand runs are performed for each setup. Three setups are

considered, one for each design speed considered in the research. The design speeds are 40, 50, and 60 mph.



**Figure 5.1 – Flowchart of the Monte-Carlo model**

In each trial of a certain setup, the simulation picks a different value for each of the contributing parameters from their corresponding distributions. Then, using the equation of the forecast cell, it computes the design values of the PSD. The modeler can specify the confidence level needed before the simulation can stop. The target confidence level has been selected to be 95 percent. The model will continue running until both conditions are satisfied, the number of trials and the confidence level. Otherwise, a message will pop up saying that the assigned number of trials is not enough to reach the required confidence level. Following is a discussion of the results of the three setups based on the input parameters presented in chapter 4 of this dissertation.

	A	B	C	D	E	F	G	H	I	J
27	<b>1- Random Variables Specified PDFs :</b>									
28										
29	a) V	: is the speed of the passing vehicle and opposing vehicle, (ft/sec). Usually, V is normally distributed between some minimum and maximum values. Thus, various speed values will be considered based on the design speed of the road.								
30										
31										
32										
33		Min	Max	Mean	STD	PDF				
34		80	110	92	6.5	90				
35										
36	c) m	: is the speed difference between the passing and the passed vehicle, (ft/sec). Usually, V is normally distributed between some minimum and maximum values. Thus, m will be based on the speed range considered for V.								
37										
38										
39										
40		Min	Max	Mean	STD	PDF				
41		7	35	16	3	12				
42										
43	d) d	: is the deceleration rate when aborting the passing maneuver, (ft/sec <sup>2</sup> ).								
44										
45		Min	Max	Mean	STD	PDF				
46		8	26	19	6	18				
47										
48	e) R	: is the perception reaction time of the driver when aborting the pass, (sec). The PRT is based on the fact that the driver is anticipating to brake and thus is in the expected category for PRT, after the study by Fambro 1994.								
49										
50										
51										
52		Min	Max	Mean	STD	PDF				
53		0.3	0.9	0.54	0.1	0.55				
54										
55	f) Gc	: is the clearance distance between the passing and the impeding vehicle at the end of the pass maneuver (ft).								
56										
57				Min	Max	PDF				
58				15	80	50				
59										
60										
61	f) Ga	: is the clearance distance between the passing and the impeding vehicle at the end of the abort maneuver (ft).								
62										
63				Min	Max	PDF				
64				10	30	20				
65										
66										
67	g) C	: is the clearance distance between the passing and the opposing vehicle at the end of the passing maneuver, (ft).								
68										
69				Min	Max	PDF				
70				100	200	150				
71										
72										
73	h) L(l)	: is the length of the impeding vehicle, (ft). This depends on the percentage of each vehicle class in the traffic stream.								
74										
75										
76						PDF				
77						20				
78										
79	i) L(P)	: is the length of the passing vehicle, (ft). This depends on the percentage of each vehicle class in the traffic stream.								
80										

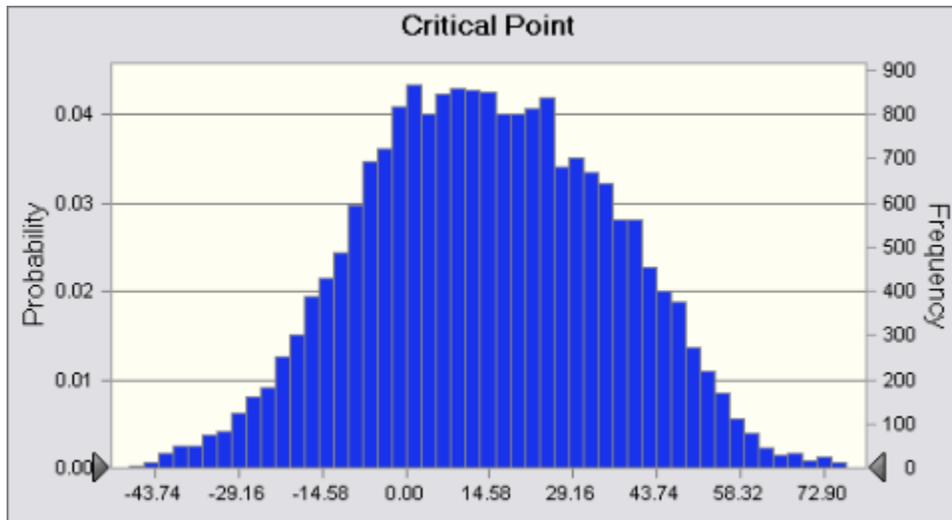
Figure 5.2 – Input model within the Monte Carlo Simulation

	A	B	C	D	E	F	G	H	I	J
85	<b>Output Parameters</b>									
86	The output parameters are the forecast cells which will be the result of "Monte Carlo" process using the formulation that is described earlier.									
87										
88										
89										
90	<b>Critical Point :</b>	is the point at which the distance to complete the passing maneuver equals the distance needed to abort the maneuver. Beyond the critical point, the driver ought to complete the pass, and before it, the driver needs less distance to abort the pass.								
91										
92										
93										
94										
95		$\Delta_c =$	27.83	ft						
96										
97										
98										
99	<b>Critical Passing Sight Distance :</b>	is the minimum distance needed to complete the passing maneuver under most conditions.								
100										
101										
102		PSD =	782.5	ft						
103										

Figure 5.3 – Output model within the Monte Carlo Simulation

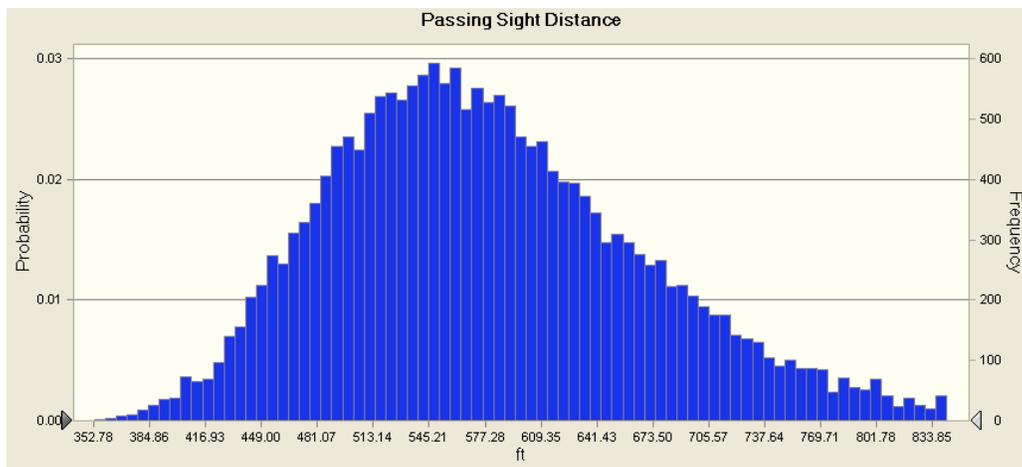
### 5.3.3.1 Results for 40 mph Design Speed

The first design speed considered is 40 mph. Twenty thousand runs are performed for this setup. Separate speed profiles for the passing and impeding vehicles are input relative to this design speed. Consequently, the speed differential (m) varies as well. Two probability distributions constitute the output of this simulation at a design speed of 40 mph. The first distribution is the critical point while the second is the PSD distribution. The critical point output distribution is shown in Figure 5.4. The PSD distribution is shown in Figure 5.5. The cumulative distribution of the PSD is shown in Figure 5.6.

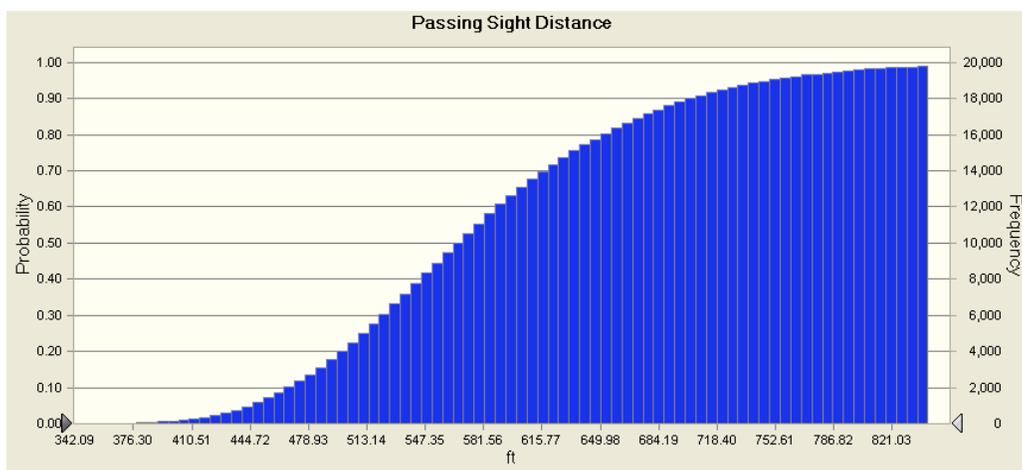


**Figure 5.4 – Histogram of the Critical Point at 40 mph design speed**

Crystal Ball has an internal curve fitting capabilities. “Fit All” option fits all known distributions to the available data histogram. The fit all statistics of the output data at 40 mph design speed is shown in Table 5.1. Gamma distribution is found to be the best distribution to fit the output points. Three ‘goodness of fit’ tests are performed which are the Chi-Square test, the Kolmogorov-Smirnov test, and the Anderson-Darling test. The selected distributions are ranked based on their goodness of fit scores relative to one of three tests. Tables 5.2 and 5.3 respectively, presents the percentiles and statistics of the gamma distribution relative to the actual forecast values of the PSD.



**Figure 5.5 – Histogram of the PSD at 40 mph design speed**



**Figure 5.6 – Cumulative distribution of the PSD at 40 mph design speed**

**Table 5.1 – Goodness-of-fit tests of the PSD distribution**

Distribution	A-D	Chi-Square	K-S	Parameters
Gamma	0.98	107.58	0.01	Location=299.71,Scale=30.88,Shape=9.10853
Beta	1.93	134.11	0.01	Minimum=343.86,Maximum=2,204.01,Alpha=5.51262,Beta=37.73757
Lognormal	14.08	245.65	0.02	Mean=580.88,Std. Dev.=91.83
Max Extreme	18.68	330.78	0.02	Likeliest=537.12,Scale=78.90
Weibull	54.29	672.84	0.04	Location=347.24,Scale=262.80,Shape=2.70294
Logistic	78.93	1229.99	0.04	Mean=575.03,Scale=52.76
Normal	111.39	1394.43	0.05	Mean=580.95,Std. Dev.=93.26
Student's t	164.17	2307.71	0.07	Midpoint=569.94,Scale=75.50,Deg. Freedom=2.22054
Triangular	1540.97	5863.02	0.2	Minimum=351.31,Likeliest=510.69,Maximum=1,027.78
Min Extreme	676.18	6308.44	0.12	Likeliest=630.12,Scale=107.43
Uniform	4767.16	22638.96	0.37	Minimum=353.33,Maximum=1,024.11
Pareto	4176.72	27592.8	0.35	Location=353.36,Shape=2.06306
Exponential	6583.6	78013.87	0.5	Rate=0.00

**Table 5.2 – Percentiles of the Gamma distribution**

Forecast: PSD		
Percentile	Fit: Gamma dist.	Forecast values
0%	299.71	353.36
10%	470.07	470.49
20%	501.07	501.34
30%	525.69	525.73
40%	548.28	547.88
50%	570.73	569.93
60%	594.49	593.03
70%	621.41	620.34
80%	654.91	655.43
90%	705.02	706.28
100%	Infinity	1,024.07

**Table 5.3 – Statistics of the Gamma distribution**

Forecast: PSD		
Statistic	Fit: Gamma dist.	Forecast values
Trials	---	20,000
Mean	580.95	580.95
Median	570.73	569.93
Mode	550.07	---
Standard Deviation	93.19	93.26
Variance	8,683.92	8,696.83
Skewness	0.66268	0.65684
Kurtosis	3.66	3.5
Coeff. of Variability	0.16041	0.16052
Minimum	299.71	353.36
Maximum	Infinity	1,024.07
Mean Std. Error	---	0.66

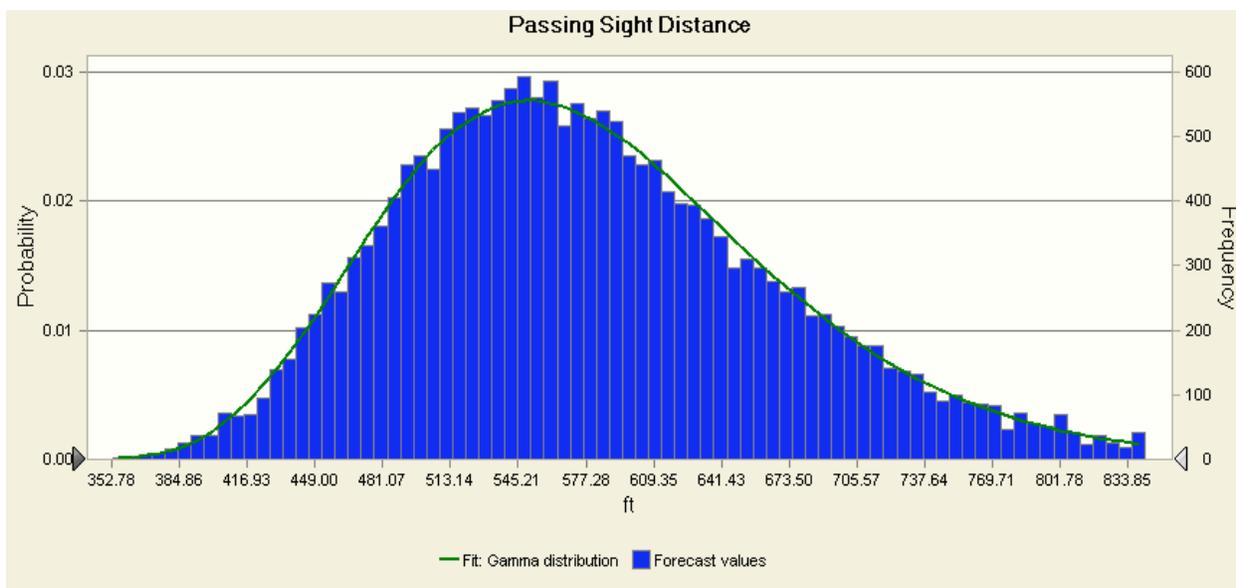
The Gamma-fit curve of the PSD histogram is shown in Figure 5.7. The corresponding probability density function is:

$$f(x) = \frac{\left(\frac{x-\mu}{\beta}\right)^{\gamma-1} e^{-\left(\frac{x-\mu}{\beta}\right)}}{\beta\Gamma(\gamma)}; \quad x \geq \mu \quad \& \quad \gamma, \beta > 0 \quad (6)$$

$$\Gamma(\gamma) = \int_0^{\infty} t^{\gamma-1} e^{-t} dt \quad (7)$$

Where,

- $\beta$  = Scale parameter for the Gamma distribution = 30.88
- $\gamma$  = Shape parameter for the Gamma distribution = 9.11
- $\mu$  = Location parameter for the Gamma distribution (ft) = 299.71



**Figure 5.7 – Gamma fit to the PSD distribution at 40 mph design speed**

The software also computes and ranks the correlation and contribution to variance of each of the influencing parameters. Tables 5.4 and 5.5 present the sensitivity of the critical point and the PSD values to the various contributing parameter, respectively. Positive coefficients indicate that an increase in the assumption (parameter) is associated with an increase in the forecast (PSD). Negative coefficients imply the opposite situation. Clearly, as the deceleration rate increases, the PSD needed to abort the pass decreases showing negative correlation, which is logical. All other variables show positive correlation with the PSD variation. Intuitively, the speed differential is supposed to have the most effect on the PSD, as it does on the critical point. The equation of the PSD contains the speed differential in the numerator and the denominator, as well. So, its sensitivity to this parameter decreases.

**Table 5.4 – Sensitivity of the critical point to the various parameters**

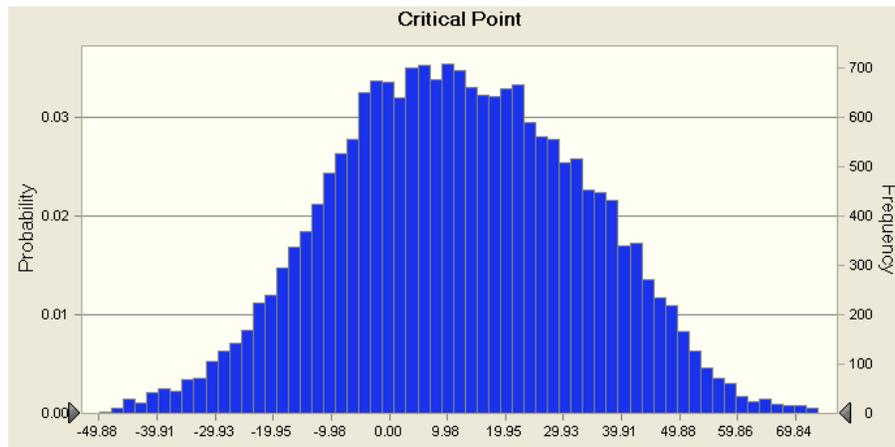
Assumptions	ContributionToVariance	RankCorrelation
Clearance Gap $G_c$ (ft)	0.630666954	0.761252975
Speed Differential, $m$ (ft/sec)	0.199385757	-0.428031795
Brake Deceleration Rate, $d$ (ft/sec <sup>2</sup> )	0.125140364	0.339099908
Passing Vehicle Lengths, $L_p$ (ft)	0.035699996	0.181118598
Perception Reaction Time, $R$ (sec)	0.003333868	-0.055348157
Passing Vehicle Speed, $V$ (ft/sec)	0.003142777	-0.053738529
Clearance Gap, $G_a$ (ft)	0.001889009	-0.041662558
Impeding Vehicle Lengths, $L_i$ (ft)	0.000575266	-0.02299128
Head-on Clearance, $C$ (ft)	0.00016601	0.012350846

**Table 5.5 – Sensitivity of the PSD to the various parameters**

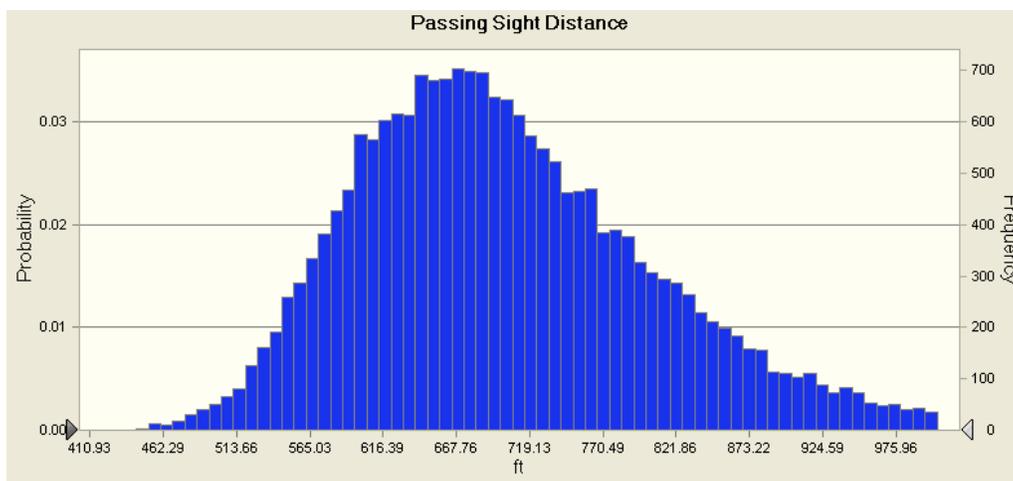
Assumptions	ContributionToVariance	RankCorrelation
Brake Deceleration Rate, d (ft/sec <sup>2</sup> )	0.490865198	-0.676363004
Passing Vehicle Speed, V (ft/sec)	0.315302855	0.542079115
Head-on Clearance, C (ft)	0.106284076	0.314726147
Clearance Gap Gc (ft)	0.055547977	0.227526951
Perception Reaction Time, R (sec)	0.01848692	0.131259578
Clearance Gap, Ga (ft)	0.005429356	0.071133285
Passing Vehicle Lengths, Lp (ft)	0.004341738	0.063610751
Impeding Vehicle Lengths, Li (ft)	0.003446231	0.056672314
Speed Differential, m (ft/sec)	0.000295648	0.016599161

*5.3.3.2 Results for 50 mph Design Speed*

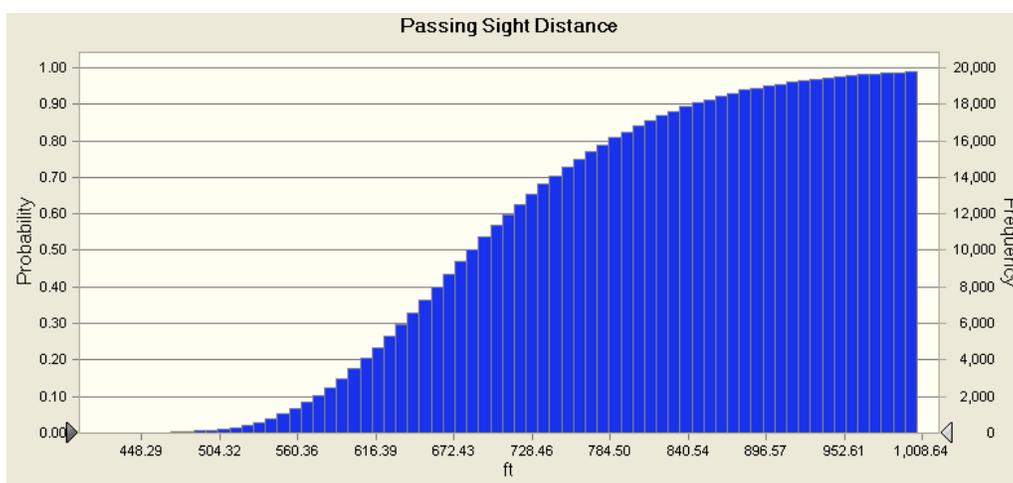
The second design speed considered is 50 mph. Twenty thousand runs are also performed for this setup. Separate speed profiles for the passing and impeding vehicles are input relative to this design speed, which varies the speed differential (m). Two probability distributions constitute the output of this simulation at a design speed of 50 mph similar to those obtained for the 40 mph design speed setup. The critical point output distribution is shown in Figure 5.8. The PSD distribution is shown in Figure 5.9. The cumulative distribution of the PSD is shown in Figure 5.10. The fit all statistics of the output data at 50 mph design speed is shown in Table 5.6. Gamma distribution is also found to be the best distribution to fit the output points. The same ‘goodness of fit’ tests are performed. Tables 5.7 and 5.8 respectively, presents the percentiles and statistics of the gamma distribution relative to the actual forecast values of the PSD.



**Figure 5.8 – Histogram of the Critical Point at 50 mph design speed**



**Figure 5.9 – Histogram of the PSD at 50 mph design speed**



**Figure 5.10 – Cumulative distribution of the PSD at 50 mph design speed**

**Table 5.6 – Goodness-of-fit tests of the PSD distribution**

Distribution	A-D	Chi-Square	K-S	Parameters
Gamma	3.25	106.39	0.01	Location=386.46,Scale=36.14,Shape=8.78878
Beta	3.75	137.94	0.01	Minimum=448.63,Maximum=2,452.64,Alpha=4.79589,Beta=32.83372
Max Extreme	12.03	260.55	0.02	Likeliest=653.76,Scale=90.17
Lognormal	25.99	391.87	0.03	Mean=703.94,Std. Dev.=105.45
Weibull	102.08	1238.71	0.05	Location=406.90,Scale=332.68,Shape=3.01457
Logistic	92.22	1403.25	0.04	Mean=696.65,Scale=60.70
Normal	133.28	1588.12	0.06	Mean=704.04,Std. Dev.=107.53
Student's t	193.59	2738.75	0.08	Midpoint=689.59,Scale=82.68,Deg. Freedom=1.89978
Triangular	1463.3	6431.24	0.19	Minimum=415.47,Likeliest=633.19,Maximum=1,212.81
Min Extreme	719.6	6752.14	0.12	Likeliest=760.93,Scale=124.76
Uniform	4343.7	24088	0.35	Minimum=418.03,Maximum=1,208.58
Pareto	4627.6	32651.28	0.38	Location=418.06,Shape=1.96093
Exponential	6708.7	83905.41	0.51	Rate=0.00

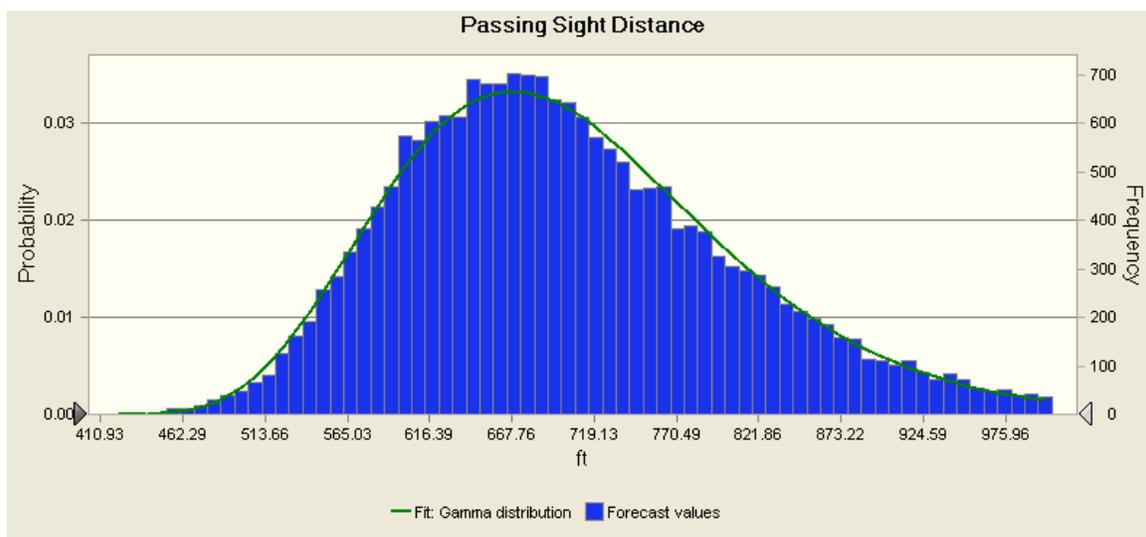
**Table 5.7 – Percentiles of the Gamma distribution**

Forecast: PSD		
Percentile	Fit: Gamma dist.	Forecast values
0%	386.46	418.07
10%	576.79	578.72
20%	612.22	612.54
30%	640.41	640.45
40%	666.31	665.44
50%	692.08	689.58
60%	719.39	716.37
70%	750.36	748.78
80%	788.94	788.71
90%	846.72	849.16
100%	Infinity	1,208.54

**Table 5.8 – Statistics of the Gamma distribution**

Forecast: PSD		
Statistic	Fit: Gamma dist.	Forecast values
Trials	---	20,000
Mean	704.04	704.04
Median	692.08	689.58
Mode	667.91	---
Standard Deviation	107.13	107.53
Variance	11,476.14	11,561.87
Skewness	0.67463	0.70079
Kurtosis	3.68	3.57
Coeff. of Variability	0.15216	0.15273
Minimum	386.46	418.07
Maximum	Infinity	1,208.54
Mean Std. Error	---	0.76

The Gamma-fit curve of the PSD histogram is shown in Figure 5.11. The parameters of the Gamma probability density function are  $\beta = 36.14$ ;  $\gamma = 8.79$ ; &  $\mu = 386.46$ .



**Figure 5.11 – Gamma fit to the PSD distribution at 50 mph design speed**

**Table 5.9 – Sensitivity of the critical point to the various parameters**

Assumptions	ContributionToVariance	RankCorrelation
Clearance Gap Gc (ft)	0.607983023	0.74668025
Speed Differential, m (ft/sec)	0.224117962	-0.453343372
Brake Deceleration Rate, d (ft/sec <sup>2</sup> )	0.124736882	0.33820995
Passing Vehicle Lengths, Lp (ft)	0.034756465	0.178528165
Perception Reaction Time, R (sec)	0.003415578	-0.055965588
Clearance Gap, Ga (ft)	0.002627644	-0.049087642
Passing Vehicle Speed, V (ft/sec)	0.001221324	-0.033466044
Impeding Vehicle Lengths, Li (ft)	0.000968376	-0.029799639
Head-on Clearance, C (ft)	0.000172746	0.012586134

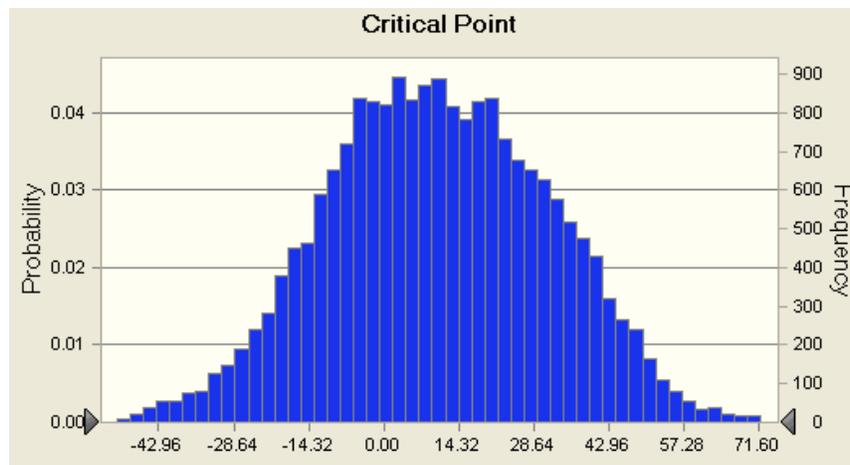
**Table 5.10 – Sensitivity of the PSD to the various parameters**

Assumptions	ContributionToVariance	RankCorrelation
Brake Deceleration Rate, d (ft/sec <sup>2</sup> )	0.555611432	-0.718526335
Passing Vehicle Speed, V (ft/sec)	0.227704997	0.459984698
Clearance Gap Gc (ft)	0.091918456	0.292252654
Head-on Clearance, C (ft)	0.081321661	0.274890813
Perception Reaction Time, R (sec)	0.021557782	0.141533467
Clearance Gap, Ga (ft)	0.008833562	0.09059934
Passing Vehicle Lengths, Lp (ft)	0.007082086	0.081121826
Impeding Vehicle Lengths, Li (ft)	0.005687904	0.0726998
Speed Differential, m (ft/sec)	0.000282119	0.016190983

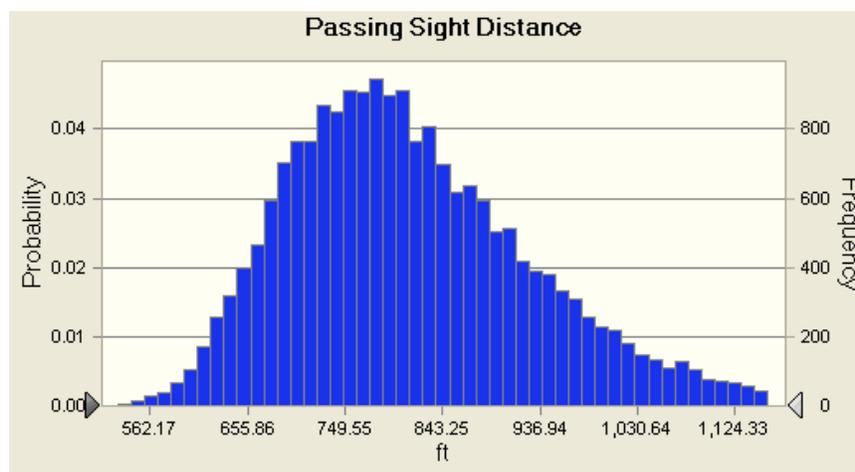
Tables 5.9 and 5.10 present the sensitivity of the critical point and the PSD values to the various contributing parameter, respectively. As mentioned earlier, positive coefficients indicate that an increase in the assumption (parameter) is associated with an increase in the forecast (PSD). Negative coefficients imply the opposite situation.

#### 5.3.3.3 Results for 60 mph Design Speed

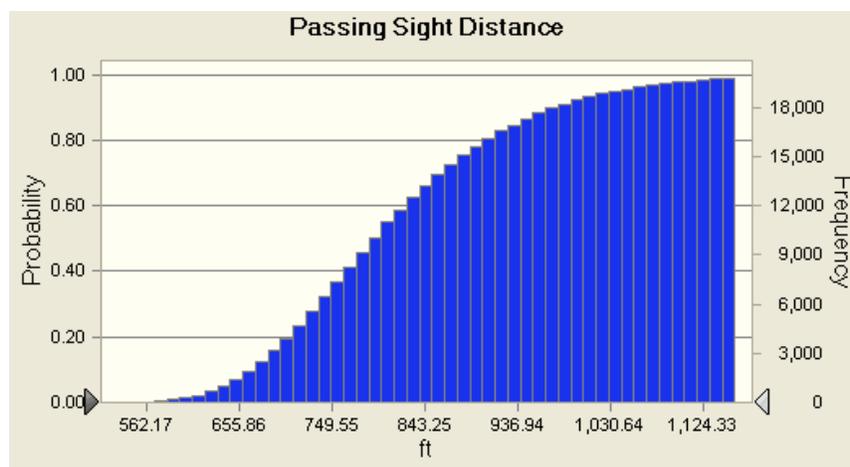
The third design speed considered is 60 mph. Twenty thousand runs are also performed for this setup. Separate speed profiles for the passing and impeding vehicles are input relative to this design speed, which varies the speed differential (m). Two probability distributions constitute the output of this simulation at a design speed of 60 mph similar to those obtained for the 40 and 50 mph design speed setups. The critical point output distribution is shown in Figure 5.12. The PSD distribution is shown in Figure 5.13. The cumulative distribution of the PSD is shown in Figure 5.14. The fit all statistics of the output data at 60 mph design speed is shown in Table 5.11. Gamma distribution is also found to be the best distribution to fit the output points. The same ‘goodness of fit’ tests are performed. Tables 5.12 and 5.13 respectively, presents the percentiles and statistics of the gamma distribution relative to the actual forecast values of the PSD.



**Figure 5.12 – Histogram of the Critical Point at 60 mph design speed**



**Figure 5.13 – Histogram of the PSD at 60 mph design speed**



**Figure 5.14 – Cumulative distribution of the PSD at 60 mph design speed**

The Gamma-fit curve of the PSD histogram is shown in Figure 5.15. The parameters of the Gamma probability density function are  $\beta = 45.68$ ;  $\gamma = 7.07$ ; &  $\mu = 492.39$ . Tables 5.14 and 5.15 present the sensitivity of the critical point and the PSD values to the various contributing

parameter, respectively. As mentioned earlier, positive coefficients indicate that an increase in the assumption (parameter) is associated with an increase in the forecast (PSD). Negative coefficients imply the opposite situation. Figure 5.16 is a sample sensitivity chart produced by the software to portray the results shown in Table 5.14.

**Table 5.11 – Goodness-of-fit tests of the PSD distribution**

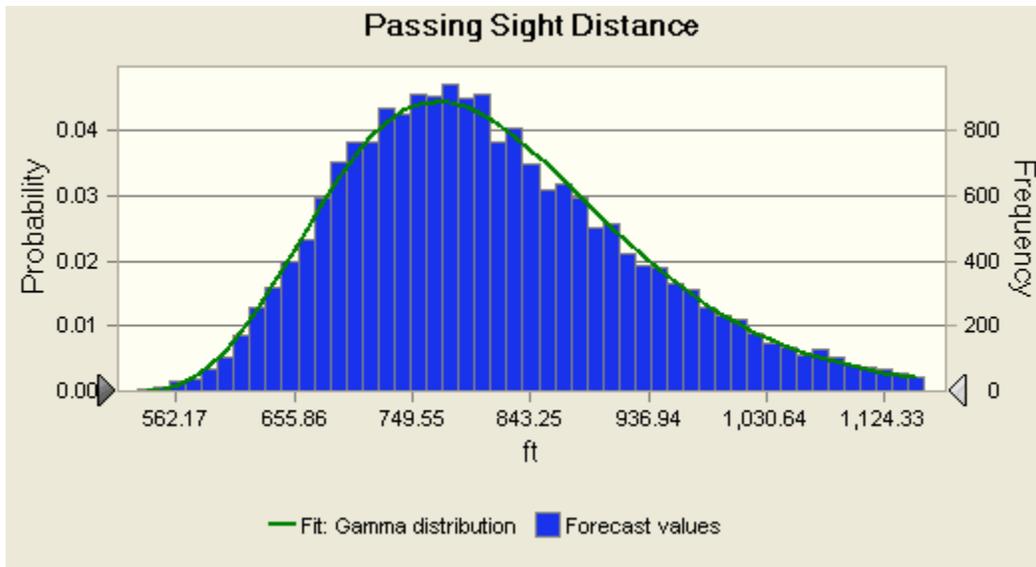
Distribution	A-D	Chi-Square	K-S	Parameters
Gamma	2.24	132.14	0.01	Location=492.39,Scale=45.68,Shape=7.06563
Beta	4.21	152.45	0.01	Minimum=553.36,Maximum=2,383.84,Alpha=3.82335,Beta=22.90961
Max Extreme	6.54	221.34	0.01	Likeliest=758.54,Scale=100.32
Lognormal	38.57	549.48	0.03	Mean=815.01,Std. Dev.=118.85
Weibull	68.86	872.55	0.04	Location=525.03,Scale=326.79,Shape=2.55633
Logistic	106.53	1621.92	0.04	Mean=806.16,Scale=68.70
Normal	153.82	1864.13	0.06	Mean=815.15,Std. Dev.=121.69
Student's t	210.06	3070.88	0.08	Midpoint=797.94,Scale=91.69,Deg. Freedom=1.79335
Triangular	1610.4	5942.09	0.2	Minimum=528.73,Likeliest=716.64,Maximum=1,399.78
Min Extreme	753.45	7229.8	0.13	Likeliest=879.76,Scale=142.19
Uniform	5042.1	22840.86	0.38	Minimum=531.22,Maximum=1,394.98
Pareto	3899.6	24208.7	0.34	Location=531.25,Shape=2.39548
Exponential	6765.9	86810.99	0.51	Rate=0.00

**Table 5.12 – Percentiles of the Gamma distribution**

Percentile	Fit: Gamma dist.	Forecast values
0%	492.39	531.26
10%	672.57	674.59
20%	711.14	711.02
30%	742.25	741.87
40%	771.12	770.26
50%	800.06	797.94
60%	830.95	828.57
70%	866.21	865.21
80%	910.44	910.29
90%	977.24	979.84
100%	Infinity	1,394.94

**Table 5.13 – Statistics of the Gamma distribution**

Statistic	Fit: Gamma dist.	Forecast values
Trials	---	20,000
Mean	815.15	815.15
Median	800.06	797.94
Mode	769.47	---
Standard Deviation	121.42	121.69
Variance	14,743.51	14,808.93
Skewness	0.75241	0.74749
Kurtosis	3.85	3.61
Coeff. of Variability	0.14896	0.14929
Minimum	492.39	531.26
Maximum	Infinity	1,394.94
Mean Std. Error	---	0.86



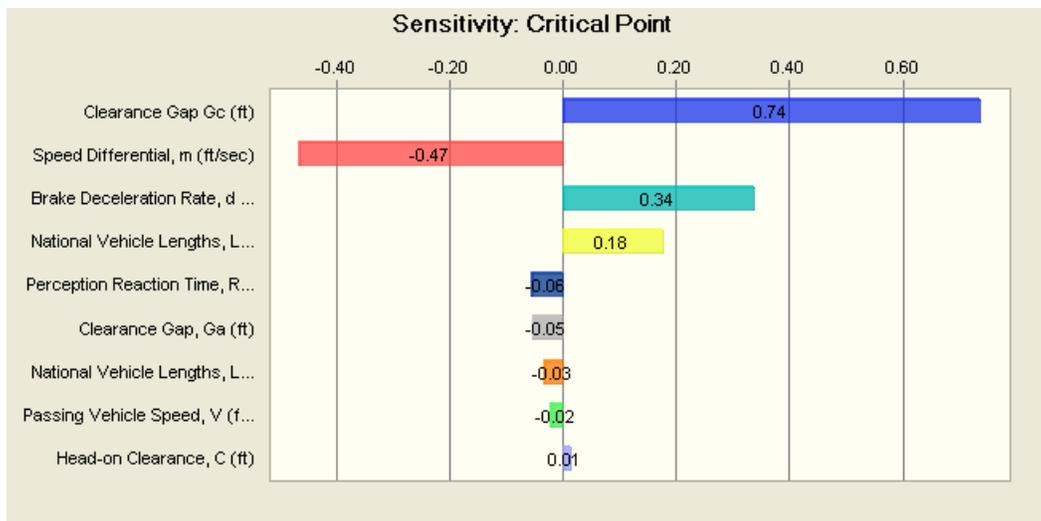
**Figure 5.15 – Gamma fit to the PSD distribution at 60 mph design speed**

**Table 5.14 – Sensitivity of the critical point to the various parameters**

Assumptions	ContributionToVariance	RankCorrelation
Clearance Gap $G_c$ (ft)	0.593290998	0.737094749
Speed Differential, $m$ (ft/sec)	0.23961771	-0.468434523
Brake Deceleration Rate, $d$ (ft/sec <sup>2</sup> )	0.124313507	0.337402734
National Vehicle Lengths, $L_p$ (ft)	0.034141133	0.176818786
Perception Reaction Time, $R$ (sec)	0.003459618	-0.056286408
Clearance Gap, $G_a$ (ft)	0.00313181	-0.053553416
National Vehicle Lengths, $L_i$ (ft)	0.001256212	-0.033917265
Passing Vehicle Speed, $V$ (ft/sec)	0.000611478	-0.023663543
Head-on Clearance, $C$ (ft)	0.000177534	0.012750575

**Table 5.15 – Sensitivity of the PSD to the various parameters**

Assumptions	ContributionToVariance	RankCorrelation
Brake Deceleration Rate, $d$ (ft/sec <sup>2</sup> )	0.587130156	-0.740355111
Passing Vehicle Speed, $V$ (ft/sec)	0.179407496	0.40925423
Clearance Gap $G_c$ (ft)	0.118984247	0.33328642
Head-on Clearance, $C$ (ft)	0.063583195	0.243637457
Perception Reaction Time, $R$ (sec)	0.023211638	0.147206092
Clearance Gap, $G_a$ (ft)	0.011258747	0.102522201
National Vehicle Lengths, $L_p$ (ft)	0.008902627	0.091165806
National Vehicle Lengths, $L_i$ (ft)	0.007280083	0.082440584
Speed Differential, $m$ (ft/sec)	0.000241811	0.015024871



**Figure 5.16 – Rank correlation of the critical point to the various parameters**

Table 5.16 below presents a summary of the output statistics of the PSD for the various design speeds considered in the research. It also tabulates the shape, location, and scale parameters of the Gamma distribution curve that has been fit to the PSD histograms.

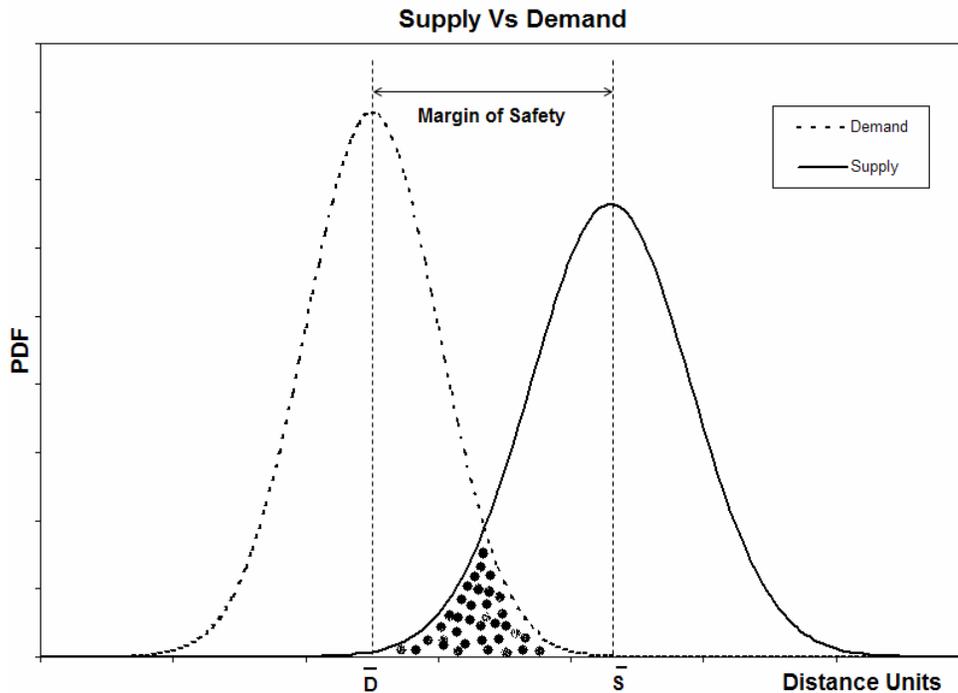
**Table 5.16 – Descriptive statistics of the PSD design values**

PSD Design Values (ft)	Speed Level		
	40 mph	50 mph	60 mph
<b>Trials</b>	20,000	20,000	20,000
<b>Mean</b>	580.95	704.04	815.15
<b>Median</b>	569.93	689.58	797.93
<b>Standard Deviation</b>	93.26	107.53	121.69
<b>Variance</b>	8,696.83	11,561.87	14,808.73
<b>Skewness</b>	0.65684	0.70079	0.74749
<b>Kurtosis</b>	3.5	3.57	3.61
<b>Coeff. of Variability</b>	0.16052	0.15273	0.14929
<b>Minimum</b>	353.36	418.07	531.26
<b>Maximum</b>	1,024.07	1,208.54	1,394.94
<b>Mean Std. Error</b>	0.66	0.76	0.86
<b>Gamma Distribution Parameters</b>			
<b><math>\gamma</math> ( Shape parameter)</b>	9.11	8.79	7.07
<b><math>\mu</math> (Location parameter)</b>	299.71	386.46	492.39
<b><math>\beta</math> (Scale parameter)</b>	30.88	36.14	45.68

#### 5.4. Reliability of current PSD standards

Knowing the PSD distribution for a specific road design speed, a preliminary reliability study of the current design values can be conducted. Navin presented a method to calculate the reliability index ( $\beta$ ), also known as the safety index, of design values with respect to the driver-vehicle demand distributions (Navin 1990). The discussion about the method to calculate the reliability index or margin of safety is found in chapter 3 of the dissertation, and is demonstrated

in Figure 5.17. The margin of safety represents the distance between the mean of the supply and that of the demand PSD distributions. The demand distribution in this case is demonstrated the forecasted histogram of PSDs obtained from the Monte Carlo simulation. The supply PSD is represented by the current design values. The latter values are single design values with no distributions. So, they exhibit zero variance or standard deviation. Thus, the margin of safety of the current design standards will represent the distance between these values and the mean of the forecasted (demand) PSD distribution for each design speed considered.



**Figure 5.17 – Sight distance supply versus demand**

The current PSD criteria are represented by three design standards, which are, the AASHTO “Green Book”, the MUTCD, and the Glennon design values. The equation used to calculate the safety index is as follows:

$$\beta_R = \frac{\bar{S} - \bar{D}}{\sqrt{\sigma_S^2 + \sigma_D^2}} \quad (5.2)$$

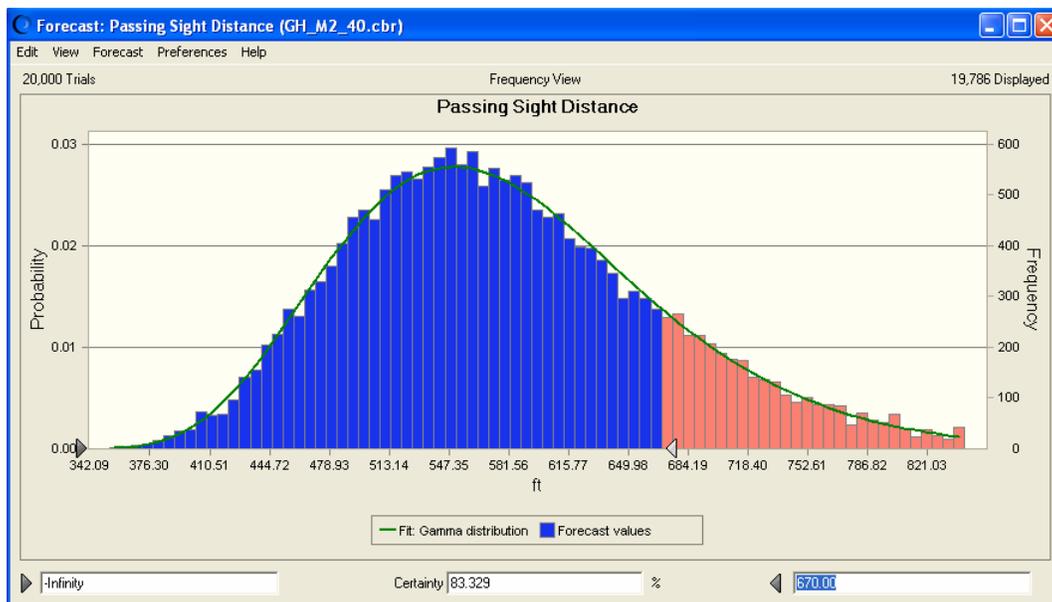
Where, S is the supply design values (MUTCD, AASHTO, Glennon’s), D is the driver-vehicle demand values represented by the forecast PSD distribution, and  $\sigma^2$  are the corresponding variances. Since MUTCD, AASHTO, and Glennon design values are constant values, their variances are zeros. Table 5.17 presents the various safety indices that the current PSD design

values exhibit with respect to the Monte-Carlo PSD distributions. Notice the conservative design values that the “Green Book” specify, which are demonstrated by the high safety index values.

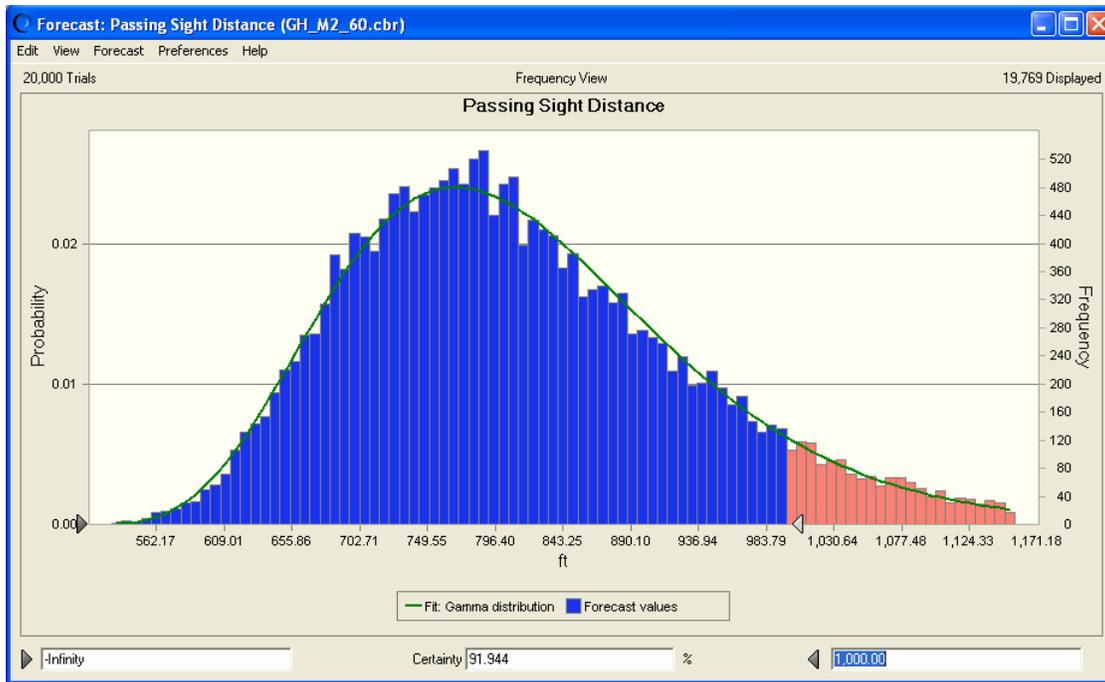
**Table 5.17 – Reliability index of different PSD design values**

	Monte-Carlo	AASHTO		MUTCD		Glennon	
	Mean PSD (ft)	PSD (ft)	$\beta$	PSD (ft)	$\beta$	PSD (ft)	$\beta$
40 mph	<b>580.95</b>	1470	9.53	600	0.2	670	0.95
50 mph	<b>704.04</b>	1832	10.41	800	0.87	830	1.15
60 mph	<b>815.15</b>	2133	10.83	1000	1.52	990	1.43

Another way of checking the reliability of the current design values against the obtained distribution is done automatically by the software. Crystal Ball enables the modeler to assess the certainty of a certain value in describing the forecasted distribution. In other words, the values of the current PSD design criteria are input into the forecast sheet to assess their certainty in describing the total distribution. The certainty level is an expression of the percentage of the distribution that these values cover. Figure 5.18 and 5.19 present two examples of the certainty level of current design standards with respect to the PSD distribution. The number in the lower right corner is the current design value. The number in the box in the lower center of the chart is the certainty level relative to the input design value. At 40 mpg design speed, Glennon’s design value is descriptive of 83.3% of the PSD distribution values. At 60 mph design speed, the current MUTCD design value covers 91.9% of the PSD distribution. The rest of the values are presented in Table 5.18.



**Figure 5.18 – Certainty level of Glennon’s PSD design value at 40 mph**



**Figure 5.19 – Certainty level of MUTCD PSD design value at 60 mph**

**Table 5.18 – Certainty level of the different PSD design values**

Design Speed	AASHTO		MUTCD		Glennon	
	PSD (ft)	% Certainty	PSD (ft)	% Certainty	PSD (ft)	% Certainty
40 mph	1470	100	600	62.68	670	83.33
50 mph	1832	100	800	82.21	830	87.35
60 mph	2133	100	1000	91.94	990	90.99

### 5.5 Analytical Model

This section of the dissertation discusses an analytical model that is devised to verify the results of the Monte-Carlo simulation model. The first and second moments are used to compare and contrast the results of both models. The first moment represents the mean of a given distribution. The second moment captures the variation in that distribution. Taking the “square root” of the second moment produces the standard deviation of the distribution. Thus, the analytical method is a parallel solution method used to obtain the first and second moments of the PSD distribution. Methods presented by Ang and Tang are utilized to derive the two moments (Ang and Tang 1984). The Taylor Series Expansion method is used for this purpose. It provides a numerical approximation of a function of random numbers, in this case, the PSD function. The revised version of Glennon’s PSD formulation, which is previously discussed in this dissertation, is used to develop the analytical model. It is a function of many random variables which are added, divided, and raised-to-a-power.

### 5.5.1 Theory

Taylor series approximations are extremely useful to linearize or otherwise reduce the analytical complexity of a function. Several methods exist for the calculation of Taylor series of a large number of functions. One can attempt to use the Taylor series as-is and generalize the form of the coefficients, or one can use manipulations such as substitution, multiplication or division, addition or subtraction of standard Taylor series to construct the Taylor series of a function, by virtue of Taylor series being power series. In some cases, one can also derive the Taylor series by repeatedly applying integration by parts. Taylor series may be generalized to functions of more than one variable, which is the case of the PSD function, with the following general expansion series formula:

$$\sum_{n_1=0}^{\infty} \dots \sum_{n_d=0}^{\infty} \frac{\partial^{n_1}}{\partial x_1^{n_1}} \dots \frac{\partial^{n_d}}{\partial x_d^{n_d}} \frac{f(a_1, \dots, a_d)}{n_1! \dots n_d!} (x_1 - a_1)^{n_1} \dots (x_d - a_d)^{n_d} \quad (5.3)$$

A simple derivation of the Taylor series approximation of a function of one random variable is shown first. Consider a function Y of one random variable X defined as:

$$Y = g(X) \quad (5.4)$$

The exact moments of Y may be obtained as the mathematical expectations of g(X) according to the following equation:

$$E(Y) = \int_{-\infty}^{\infty} g(x) f_X(x) dx \quad (5.5)$$

$$Var(Y) = \int_{-\infty}^{\infty} [g(x) - E(Y)]^2 f_X(x) dx \quad (5.6)$$

Where, E(Y) is the expected value of Y and the Var(Y) is the corresponding variance of Y. In order to be able to evaluate equations 5.5 and 5.6,  $f_X(x)$  needs to be known, which is not in most cases. Thus, a numerical approximation of the mean and variance of Y are needed. Expanding g(X) in a Taylor series about the mean value  $\mu_X$  gives (Ang and Tang 1984):

$$Y = g(\mu_X) + (X - \mu_X) \frac{dg}{dX} + \frac{1}{2} (X - \mu_X)^2 \frac{d^2g}{dX^2} + \dots \quad (5.7)$$

The first order approximation of the mean and variance of Y are given as:

$$E(Y) \approx g(\mu_X) \quad (5.8)$$

$$Var(Y) \approx Var(X - \mu_X) \left( \frac{dg}{dX} \right)^2 \quad (5.9)$$

The second order approximations of the first and second moments are given as:

$$E(Y) \approx g(\mu_x) + \frac{1}{2} \text{Var}(X) \frac{d^2 g}{dX^2} \quad (5.10)$$

$$\text{Var}(Y) \approx \text{Var}(X) \left( \frac{dg}{dX} \right)^2 - \frac{1}{4} \text{Var}^2(X) \left( \frac{d^2 g}{dX^2} \right)^2 + E(X - \mu_x)^3 \frac{dg}{dX} \frac{d^2 g}{dX^2} + \frac{1}{4} (X - \mu_x)^4 \left( \frac{d^2 g}{dX^2} \right)^2 \quad (5.11)$$

Due to the complexity of equation 5.11, only the first order approximation will be considered for the  $\text{Var}(Y)$  while the second order approximation will be calculated for the  $E(Y)$ . Suppose now that  $Y$  is a function of more than one random number whose equation is:

$$Y = g(X_1, X_2, \dots, X_n) \quad (5.12)$$

Using the same Taylor series expansion principles about the mean values  $\mu_{X1}, \mu_{X2}, \dots, \mu_{Xn}$ , the second order approximation of the mean of  $Y$  ( $E(Y)$ ) and the first order approximation of the variance of  $Y$  are expressed in equations 5.13 and 5.14, respectively (Ang and Tang 1984).

$$E(Y) \approx g(\mu_{X1}, \mu_{X2}, \dots, \mu_{Xn}) + \frac{1}{2} \sum_{i=1}^n \left( \frac{\partial^2 g}{\partial X_i^2} \right) \text{Var}(X_i) \quad (5.13)$$

$$\text{Var}(Y) \approx \sum_{i=1}^n c_i^2 \text{Var}(X_i) \quad (5.14)$$

Assuming that the random numbers ( $X_1, X_2, \dots, X_n$ ) are independent of each other; where  $c_i$  is the value of the partial derivative  $\frac{\partial g}{\partial X_i}$  evaluated at  $\mu_{Xi}$ .

### 5.5.2 Analytical Modeling

Since the calculation involved in the aforementioned formula is very complex to do by hand, a statistical software package, Mathematica, is used to obtain the results. Mathematica is a powerful numeric and symbolic computational engine with a graphics system, programming language, and documentation system. It can handle complex symbolic calculations that often involve hundreds of thousands or millions of terms; load, analyze, and visualize data; solve equations, differential equations, and minimization problems numerically or symbolically. Mathematica has the capabilities of doing numerical modeling and simulations, ranging from simple control systems to galaxy collisions, financial derivatives, complex biological systems, chemical reactions, environmental impact studies, and magnetic fields in particle accelerators. It also has an advanced connectivity capability to other applications.

The analytical model is a straightforward utilization of the Mathematica software capabilities. The equations of the critical point and the PSD, presented earlier, are input into the Mathematica calculation sheet. The model automatically distinguishes the various parameters involved in the calculation and lists them in the order they appear in the formulation. It then asks

for the mean and standard deviation of each of the parameters. Only the lengths of the passing and impeding vehicles,  $L_P$  and  $L_I$ , respectively, are discrete random variables. So, both equations are calculated in terms of these two parameters. As discussed in chapter 4 of this dissertation, four categories of vehicle lengths are considered for each of the passing or impeding vehicles, along with their corresponding probabilities. This assumption brings forth sixteen different scenarios that explain all the combinations of passing and impeding vehicle types. That is, the passing vehicle length can take four different values. The impeding vehicle length can also take four different values. Combined, there could be sixteen different values of  $L_P$  and  $L_I$ . Once all the means and the standard deviations of the various parameters are stored into the sheet, the model derives and simplifies the formulation in terms of the discrete random variables. The simplified equation of the first moment of the PSD in terms of  $L_P$  and  $L_I$  is as follows:

$$\frac{1}{(67.5 + L_i + L_p)^{3/2}} \left( 207807. + 9570.95 \sqrt{67.5 + L_i + L_p} + L_i^2 (46.0717 - 1.1456 \sqrt{67.5 + L_i + L_p}) + L_p^2 (46.0717 - 1.1456 \sqrt{67.5 + L_i + L_p}) + L_p (6219.68 + 64.4638 \sqrt{67.5 + L_i + L_p}) + L_i (6219.68 + 64.4638 \sqrt{67.5 + L_i + L_p} + L_p (92.1434 - 2.2912 \sqrt{67.5 + L_i + L_p})) \right)$$

The above equation represents the second order Taylor approximation of the first moment of the PSD distribution. Similarly, the first order Taylor approximation of the second moment of the PSD distribution is presented in the following equation:

$$\frac{1}{67.5 + L_i + L_p} \left( 597066. + 0.00121206 L_i^3 + 0.00121206 L_p^3 - 14827.6 \sqrt{67.5 + L_i + L_p} + L_p^2 (67.5006 + 0.102448 \sqrt{67.5 + L_i + L_p}) + L_i^2 (67.5006 + 0.00363618 L_p + 0.102448 \sqrt{67.5 + L_i + L_p}) + L_p (10624.8 + 72.6146 \sqrt{67.5 + L_i + L_p}) + L_i (10624.8 + 0.00363618 L_p^2 + 72.6146 \sqrt{67.5 + L_i + L_p} + L_p (135.001 + 0.204896 \sqrt{67.5 + L_i + L_p})) \right)$$

After plugging in the four possible values of the vehicle lengths and their corresponding probabilities, the model automatically enumerates the sixteen possible combinations and calculates a weighted average of the first and second moments of the PSD. The results of the analytical model proved to be very close to those obtained using the Monte Carlo simulation model. Table 5.19 compares the means and standard deviations of the Monte-Carlo simulation to those of the analytical model. Note that the value of the second moment obtained from the analytical model represents the variance of the PSD. The standard deviation is thus obtained by taking the “square-root” of the variance. The difference in the mean between the two models is

less than 2 percent. The results of the standard deviations obtained using both models differ by less than 5 percent.

**Table 5.19 – Comparison of the Monte-Carlo and the Analytical model results**

<b>Speed</b>	<b>Mean (ft)</b>			<b>Standard Deviation (ft)</b>		
	<b>Monte-Carlo</b>	<b>Analytical</b>	<b>% Dif.</b>	<b>Monte-Carlo</b>	<b>Analytical</b>	<b>% Dif.</b>
<b>40 mph</b>	580.95	573.34	1.31	93.26	97.57	4.41
<b>50 mph</b>	704.04	697.48	1.15	108.22	110.31	1.82
<b>60 mph</b>	815.15	804.21	1.34	121.69	126.07	3.47

## **5.6 Brief Summary**

This chapter presents a design procedure that accounts for the variations of all the contributing parameters in the PSD formulation. A Monte-Carlo simulation is developed for this purpose. The simulation utilizes the concept of random sampling from various probability distributions to represent a set of different conditions. An analytical model is also devised to verify the results of the Monte-Carlo simulation. The difference in the means between the two models is less than 2 percent. The difference in the standard deviations is less than 5 percent. Three PSD distributions are calculated for three design speeds. The obtained distributions are used to assess the reliability index of the current PSD standards. Also, the level of certainty of each of the current design criteria in describing the PSD distribution is assessed. Results indicate that while MUTCD and Glennon’s design values are close to the means of the PSD distributions, AASHTO “Green Book” values overestimate the PSD requirements.

## **CHAPTER 6:**

### **RISK INDEX OF PASSING SIGHT DISTANCES**

## 6.1 Introduction

Current design methods in transportation engineering do not address the level of risk associated with the design and use of various highway geometric elements. Passing sight distance (PSD) is an example of a geometric element designed with no risk measures. PSD is provided to ensure the safety of the passing drivers on two-lane roads. Its design requirements guarantee that the passing driver has enough clear view ahead. This minimizes the risk of collision with oncoming traffic (AASHTO 2004), but does not provide a measure for it. PSD is also designed to maintain a certain level of service on two-lane highways. Level of service is related to the ability to pass while driving on a two-lane stretch. It is captured by the delay time drivers incur due to slower vehicles driving ahead. Obviously, the capacity of a two-lane highway is inversely proportional to the percent time spent following (PTSF) factor, which is equivalent to the delay time. It is directly proportional to the ratio of passing to no-passing zones. As an example, higher PSD requirements would increase the safety level but it minimizes the number of passing zones and consequently reduces the level of service of the road. Adequate PSD design is a balanced compromise between the level of service and the safety level. The Highway Capacity Manual specifies the detailed method to calculate the Level-of-service (LOS) of a road stretch. In order to balance the trade-off equation, the level of safety needs to be calculated.

This chapter presents a direct method to calculate the level of safety for a specific PSD length at a certain design speed. A unique microscopic simulation is devised for this purpose. The simulation replicates passing maneuvers on two-lane two-way roads. It monitors the movement of three vehicles, the passing, the impeding, and the opposing vehicle. The level of safety is captured by the use of a Risk Index (RI) factor. The Risk Index is calculated for each PSD length at the end of the simulation runs. A trade-off analysis between the level of service (length of the PSD) and safety (risk index) is feasible using the simulation results.

## 6.2 Background

AASHTO's "Green Book" *A Policy on the Geometric Design of Highways and Streets* outlines the design criteria of the PSD. The Green Book formulation uses a simple but very conservative model to calculate the required PSD on a two-lane road. The assumptions and the detailed derivation of the distances are found in AASHTO's "Green Book" (2004). The FHWA *Manual on Uniform Traffic Devices for Streets and Highways* (MUTCD) states the operational criteria for marking passing and no-passing zones on two-way two lane highways (FHWA 2000).

The MUTCD standards define where no-passing zones are warranted. The rest of the road is then marked as a passing zone. These criteria are conflicting with each other and have remained virtually unchanged for more than five decades. Over the last three decades, many researchers have investigated alternative formulations for the PSD criteria.

Current PSD formulations provide limit values as design requirements for highway design. This practice constrains the flexibility of the design process and its interaction with prevailing conditions. Also, the current design practices replace the influencing parameters by single-value means in the calculation process. The previous chapter of this dissertation presented a method that accounts for the randomness of each of the parameters in the development of the PSD requirement. The method describes the procedures and the analysis for determining the probability distribution of the PSD. This chapter describes the method used to assess the risk associated with a particular PSD. A unique microscopic simulation that replicates passing maneuvers on two lane roads is devised for this purpose.

### **6.3 Literature Review**

Collecting field data on real passing maneuvers is the ideal way to estimate the PSD requirements. However, such a process needs huge amount of resources and a very long time to collect the needed data to assess the various parameters involved in the PSD design. As an alternative, a computer simulation is thus used to replicate real life passing maneuvers. Computer simulations have been extensively used to achieve the following three purposes:

1. To better understand the interactions among the different parameters that influence the system,
2. Get some estimation on how the system would perform under varying conditions, and
3. “What if” tests could be conducted to assess the sensitivity of the outcome due to some limited modifications of one or more parameters.

A computer simulation aims at predicting the behavior of a complex system by creating an approximate mathematical model of it. There are few computer packages already in the market which are capable of simulating passing maneuvers on two-lane roads. The known packages are the TWOPAS model and the TRARR model. TWOPASS was developed by Midwest Research Institute and others for the Federal Highway Administration. TRARR was developed by the Australian Road Research Board (now ARRB Transport Research, Ltd.). These two models are solely dedicated to two-lane highway configuration operations. The devised simulation package,

PARAMICS, is also capable of simulating passing maneuvers. A brief discussion of each model follows.

### **6.3.1 TWOPAS**

The TWOPAS model is a microscopic computer simulation model of traffic on two-lane highways. The predecessor of the TWOPAS model was originally developed by Midwest Research Institute (MRI) in NCHRP Project 3-19, “Grade Effects on Traffic Flow Stability and Capacity,” which resulted in the publication of *NCHRP Report 185* in 1978. The model was originally known as TWOWAF (for TWO Way Flow). MRI improved TWOWAF in 1981 in an FHWA study entitled, “Implications of Light-Weight, and Low-Powered Vehicles in the Traffic Stream.” Then, in 1983, Texas Transportation Institute (TTI) and KLD and Associates made further updates to TWOWAF, which resulted in the version of the model that was used in the development of Chapter 8 for the 1985 HCM (Harwood et al. 1999). TWOWAF had the capability of simulating traffic operations on normal two-lane highways, including both passing and no-passing zones, as well as the effects of horizontal curves, grades, vertical curves and sight distance. Subsequent to the publication of the 1985 HCM, MRI developed the TWOPAS model by adding to TWOWAF the capability to simulate passing lanes, climbing lanes, and short four-lane sections on two-lane highways. A modified version of TWOWAF known as ROADSIM was also developed and included in FHWA’s TRAF model facility. As a microscopic model, TWOPAS simulates the operation of each individual vehicle on the roadway. The operation of each vehicle as it advances along the road is influenced by the characteristics of the vehicle and its driver, by the geometrics of the roadway, and by the surrounding traffic situation. The following features are found in TWOPAS (Koorey 2002):

- Three general vehicle types—passenger cars, recreational vehicles, and trucks.
- Roadway geometrics specified by the user in input data, including horizontal curves, grades, vertical curves, sight distance, passing lanes, climbing lanes, and short four-lane sections.
- Traffic controls specified by the user, particularly passing and no-passing zones marked on the roadway.
- Entering traffic streams at each end of the simulated roadway generated in response to user-specified flow rate, traffic mix, and percent of traffic in platoon.
- Variations in driver performance and preferences based on field data.

- Driver speed choices in unimpeded traffic based on user-specified distribution of driver desired speeds.
- Driver speed choices in impeded traffic based on a car-following model that simulates driver preferences for following distances (headways), based on relative leader/follower speeds, driver desired speeds, and desire to pass the leader.
- Driver decisions concerning initiating passing maneuvers in the opposing lane, continuing/aborting passing maneuvers, and returning to normal lane, based on field data.
- Driver decision concerning behavior in passing/climbing/four-lane sections, including lane choice at beginning of added lane, lane changing/passing within added lanes and at lane drops, based on field data.
- Processing of traffic and updating of vehicle speeds, accelerations, and positions at intervals of 1 second of simulated time.

### **6.3.2 TRARR**

TRARR (“**TR**Affic on **R**ural **R**oads”) was developed in the 1970s and 1980s by the Australian Road Research Board. Originally run on mainframe computer systems, the program was ported to a PC version (3.0) in 1986. A recent version (4.0) was produced in 1994 and included a (DOS) graphical interface (albeit with reduced functionality) and the ability to import road geometry data for the creation of road sections. The latter greatly simplified the data creation requirements (Koorey 2002). TRARR is a microscopic simulation model. Each vehicle is randomly generated, placed at one end of the road and monitored as it travels to the other end. Various driver behavior and vehicle performance factors determine how the vehicle reacts to changes in alignment and other traffic. TRARR uses traffic flow, vehicle performance, and highway alignment data to establish, in detail, the speeds of vehicles along rural roads. This determines the driver demand for passing and whether or not passing maneuvers may be executed.

TRARR is designed for two-lane rural highways, with occasional passing lane sections. TRARR can be used to obtain a more precise calculation of travel time, frustration (via time spent following), and VOC benefits resulting from passing lanes or road realignments. For strategic assessment of road links, TRARR can also be used to evaluate the relative benefits of passing lanes at various spacing. TRARR uses four main input files to describe the situation to be simulated (Koorey 2002):

- **ROAD:** the section of highway to be studied, in 100m increments. It includes horizontal curvature, gradient, auxiliary (passing) lanes, and no-overtaking lines.
- **TRAF:** the traffic volume and vehicle mix to be simulated. Other information regarding the simulation time and vehicle speeds is also contained here.
- **VEHS:** the operating characteristics of the vehicle fleet. The relevant details relating to engine power, mass, fuel consumption, and so on are entered into this file.
- **OBS:** the points along the highway at which to record data on vehicle movements. TRARR can provide a range of values including mean speed, travel times, and fuel consumption.

A number of potential drawbacks to TRARR have been identified through practical experience that can be listed as follows:

- Inability to handle varying traffic flows down the highway, particularly due to major side roads.
- Inability to properly model the effects of restricted speed zones (such as small towns).
- Inability to model congested situations e.g. temporary lane closures or single-lane bridges.
- Difficulty in using field data for calibration, with no automatic calibration assistance built in.
- Difficulties creating and editing road data, particularly for planned new alignments.
- Limited ability to use the same tool to check for speed environment consistency and safety risks.

In recent work for the California Department of Transportation (Caltrans), the Institute of Transportation Studies (ITS) at the University of California-Berkeley (UCB) has developed a user interface, known as UCBRURAL, for use with the TRARR and TWOPAS models. The interface provides a convenient tool for users:

- To enter input data on traffic volumes, traffic characteristics, and geometric features of two-lane roads
- To run either the TRARR or TWOPAS model
- To display the output in a convenient graphical format

### **6.3.3 PARAMICS**

PARAMICS has a suite of microscopic simulation modules providing a powerful, integrated platform for modeling a complete range of real world traffic and transportation

problems. PARAMICS is fully scaleable and designed to handle scenarios as wide-ranging as a single intersection, through to a congested freeway or the modeling of an entire city's traffic system. PARAMICS has been particularly useful in modeling large-scale networks in California, New York City, and Sydney amongst others. The constituent modules of PARAMICS are: Modeler, Estimator, Processor, Analyzer, Programmer, Monitor, Designer, and Viewer (PARAMICS Website).

- The Modeler provides the three fundamental operations of model build, traffic simulation (with 3-D visualization) and statistical output. Every aspect of the transportation network can be investigated in Modeler including mixed urban and freeway networks, right hand and left hand drive capabilities, advanced signal control, roundabouts, public transportation, car parking, two lane activities, and truck lanes and high occupancy facilities among others.
- The Estimator is an OD Matrix estimation tool designed to integrate seamlessly with the core PARAMICS modules. Estimator is designed to make the OD estimation process as open, transparent and auditable as possible.
- Processor is a batch simulation productivity tool used for easy sensitivity and option testing. Processor can be used to automate simulation and analysis processes, reducing user down time, and speeding up the model development lifecycle.
- Analyzer is the powerful post-data-analysis tool used for custom analysis and reporting of model statistics.
- Programmer allows users to augment the core PARAMICS simulation with new functions, driver behaviors, and practical features. At the same time researchers can opt to override or replace sections of the core PARAMICS simulation with their own behavioral models.
- Monitor is a pollution evaluation framework module that can be used to collect pollution data at the vehicle level.
- Designer is a 3D model building and editing tool that can be used to prepare complex and life-like 3D models to aid the visualization of any traffic model for presentation and public exhibit.
- Viewer is a freely available network simulation / visualization tool. Viewer can provide full simulation and visualization of any PARAMICS network.

These packages have not been used to assess the risk of various PSDs for two reasons. First, they do not give the user control over the simulation clock. The author wishes to control the clock in order to monitor the movement of the three vehicles involved in the passing maneuver at short time-steps. Second, the main aim of simulating passing maneuvers is to assess the risk of the PSD length. So, the required simulation setup involves a stretch equal to the PSD length with no traffic flow. Three vehicles are only simulated in a passing maneuver in each run, and the result of the pass is recorded. Therefore, the authors devised a unique simulation to achieve higher fidelity of the passing maneuver by simulating every 0.1 second intervals. The unique simulation is only concerned with passing attempts to assess the risk of the selected PSD length. Thousands of passing attempts are simulated for every PSD length.

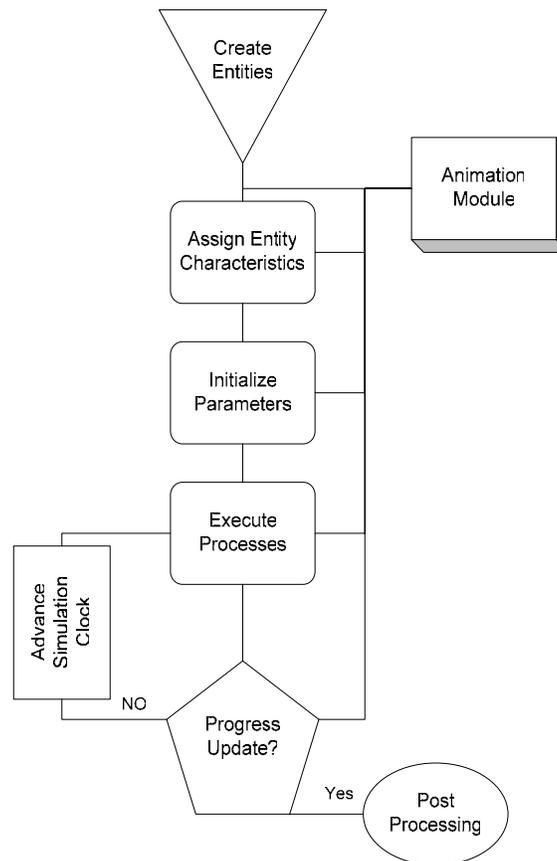
#### **6.4 The Simulation**

ARENA is a well established simulation package that is used worldwide. It is extremely user friendly and has a good graphical user interface (GUI). The authors used it to develop the PSD simulation runs. The computer simulation monitors the movement of three vehicles in a passing maneuver. The three vehicles are the impeding, the opposing, and the passing vehicle. ARENA uses ready-to-use code blocks to replace lines of code. Each code block has a certain general function. The modeler inputs the specific variables into the code blocks to perform the required task. Then by connecting the code blocks together in a definite sequence, the modeler is able to form a coherent simulation program. The simulation is basically comprised of four major components. The modules are:

- **Initiation Module:** is responsible for creating three vehicles per replication. Then, these vehicles are stochastically assigned certain characteristics along with the corresponding parameters. Vehicle related parameters include vehicle class, speed, location, acceleration, to mention a few. The user specifies the length of the PSD in this module. The module then generates a passing zone corresponding to the specified distance. The animation module moves the vehicles along the generated passing zone.
- **Main Analysis Module:** is the core of the simulation. It is responsible for dictating the three vehicles' actions based on the input parameters. It also maintains the logical interaction among them. The simulation clock is advanced in this module. Thus, the user has full control over the time step which can be as small as necessary. A 0.1 second time-step is selected for higher accuracy.

- **Animation Module:** is responsible for portraying the flow of the three vehicles to the user. The animation module converts the logic of the simulation into visual scenes. The modeler can use the animation to debug certain hidden flaws in the code. Nevertheless, the user can better understand the simulation logic without going deep into the code by looking at the visualization of the passing maneuver. Input parameters can be changed using an external User Interface module which is part of the animation module.
- **Post-Processing Module:** is responsible for collecting the data and performing the necessary analysis. The risk index is computed in this module for every PSD length tested. ARENA adds a lot to the post processing module capabilities. The modeler can specify which variables are of interest and ARENA will automatically calculate the corresponding statistics to obtain the significance/confidence intervals.

The overall simulation architecture is shown in figure 6.1.



**Figure 6.1 – Overall architecture of the simulation**

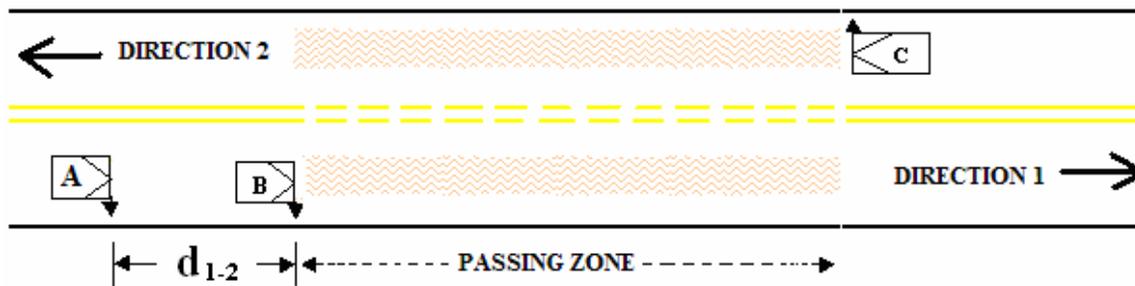
### **6.4.1 Input Parameters**

The simulation generates the characteristics of the three vehicles in the order they are created. The first vehicle is the impeding vehicle (veh 1), the second is the passing vehicle (veh

2), and the third is the opposing vehicle (veh 3). Most of the input parameters have been discussed in chapter 4 of the dissertation. However, some parameters are only related to this simulation setup, such as the vehicles locations.

#### 6.4.1.1 Vehicles location

The PSD length is specified before assigning the location of the three vehicles. A virtual passing zone is created based on the PSD length. At the beginning of the simulation clock, the locations of the three vehicles are assigned along the passing zone. Vehicles 1 and 2 (impeding and passing, respectively) are assigned locations relative to each other. Vehicle 3 is located at the end of the passing zone. Thus, its horizontal coordinate is equal to the PSD length. The passing vehicle (veh 2) is located at the beginning of the passing zone. The impeding vehicle is placed ahead of the passing vehicle along the passing zone. The locations of the three vehicles are shown in Figure 6.2.



**Figure 6.2 – Vehicles' locations along the passing zone**

Pitts car following model is used to calculate the minimum headway between the passing and the impeding vehicles (Halati et al. 1997). Minimum separation is logical to assume since the passing vehicle is seeking to narrow the distance with the impeding vehicle before initiating a passing maneuver. The formula used to calculate the distance headway between the two vehicles is the following:

$$d_{1-2} = L_1 + 10 + ku_2 + bk(u_1 - u_2)^2 \quad (6.1)$$

Where,

$d_{1-2}$  = Space headway between the impeding and the passing vehicle from front bumper to front bumper.

$L_1$  = Length of passed vehicle.

$k$  = Driver sensitivity factor of the passing vehicle.

$m$  = Speed difference between passing and passed vehicle.

$b$  = Calibration constant defined as 0.1 (when  $u_2 > u_1$ ) or 0 otherwise.

$u_1, u_2$  = Speeds of the impeding and passing vehicles, respectively.

The sensitivity of the passing driver is captured in this equation through the variable  $k$ . Values of  $k$  are randomly selected from a uniform distribution. Low values of  $k$  are favored in order to represent aggressive drivers.

#### *6.4.1.2 Vehicle Following Gap*

The vehicles following gap parameters have been discussed in the input parameter chapter of this dissertation. Two parameters have been identified;  $G_C$  and  $G_A$ . In this simulation, the two parameters have been renamed as ‘reentry’ and ‘setback’, respectively. But they retained the same values presented earlier. The selected values for the ‘reentry’ and ‘setback’ gaps are used in the simulation to assess whether a crash has occurred at the end of the pass/abort maneuver. That is, the ‘reentry’ gap dictates the minimum distance needed for a passing vehicle to merge ahead of the impeding vehicle. The ‘setback’ gap specifies the minimum distance needed for the passing vehicle to be able to setback behind the impeding vehicle in an abort maneuver.

#### *6.4.1.3 Clearance Gap (C)*

This is the clearance distance between the passing and opposing vehicles at the end of the passing maneuver. It has been discussed in chapter 4 of the dissertation as an input parameter. Glennon (1988) assumed a minimum head-on clearance of 1 second time headway. AASHTO (2004) considered values in excess of 3 seconds for speeds over 40 mph. Head-on clearance values ranging from 1.5 to 2.5 seconds are shown in the literature (Polus et al. 2000). In this simulation, this parameter is actually calculated for every passing/abort attempt, and is no more an input parameter. Its value influences the risk index of every run. Passing or abort attempts with final clearance time less than 2 seconds are considered risky, even though the result showed no crash. Two seconds of clearance time is selected as an average of the values presented in the literature.

### **6.4.2 Simulation Logic**

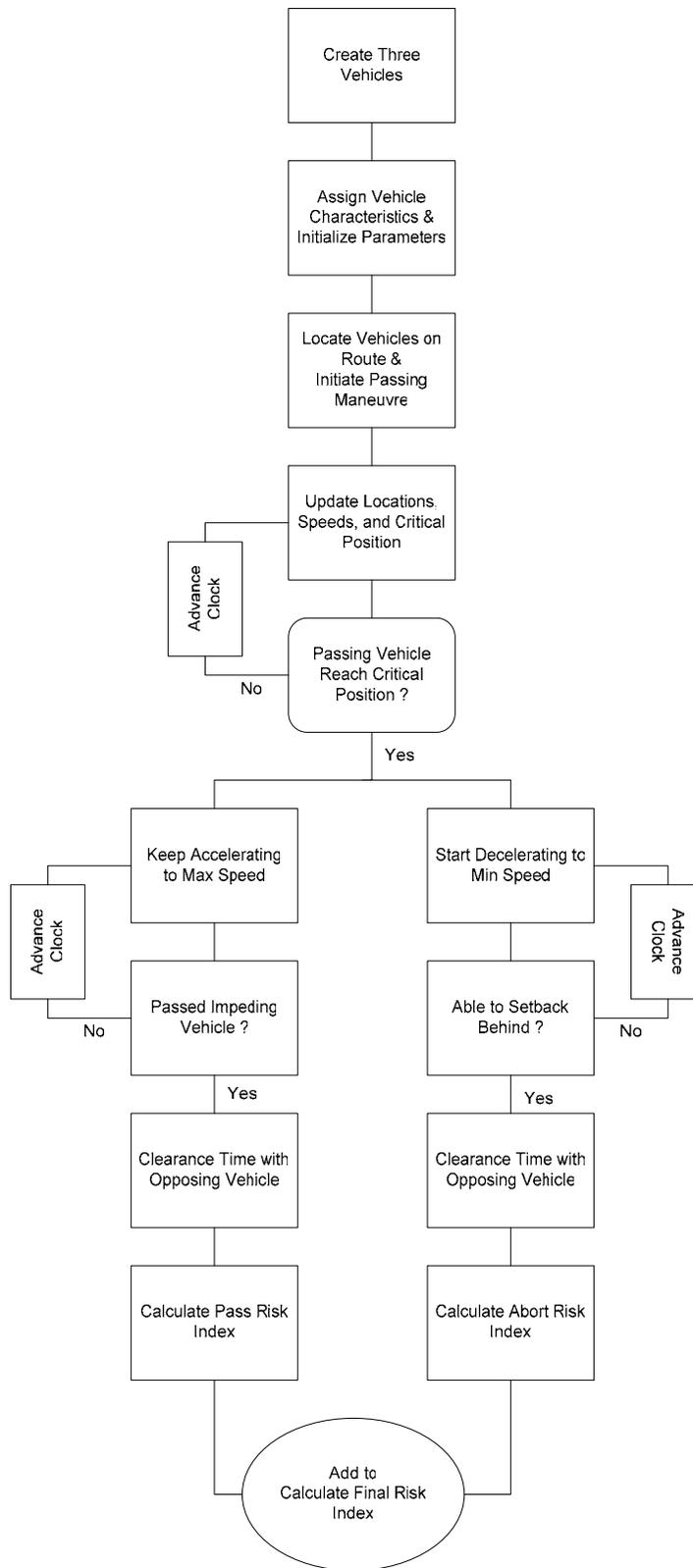
Once the input data is created and the initial locations of the vehicles are determined, the three vehicles are set to initiate the maneuver. The simulation starts by initializing the time clock. It then advances by a time step of  $1/10^{\text{th}}$  of a second. Few assumptions are made to govern the movement of the three vehicles, such as:

- The impeding and opposing vehicles maintain their originally assigned speeds all through the passing maneuver. The two vehicles are assumed to take no action toward the passing attempt (such as, to decelerate), which is the worst case scenario.

- The passing vehicle starts the maneuver at the same speed of the impeding vehicle (accelerative pass).
- The passing vehicle accelerates at the maximum rate until it reaches the ‘critical position’ relative to the impeding vehicle.
- At the ‘critical point’, the passing vehicle takes two actions. In the first scenario, the passing driver continues to accelerate to the maximum speed trying to complete the passing maneuver. The second decision is to decelerate in an attempt to setback behind the impeding vehicle.

The ‘critical position’ is selected to be the point of decision since it is the point where the distance needed to complete the passing maneuver is equal to the distance needed to abort it (Glennon 1988). Beyond the critical position, the driver is committed to complete the pass. For this reason, the critical point is considered in the simulation as the decision point for the passing driver. Aborting the pass at the critical position is the worst case scenario. The revised version of Glennon’s formulation is used to calculate the critical position. The value of the critical point is updated in every time step. It is dependent on the passing vehicle speed which varies with time.

The simulation of the three vehicles follows a simple logic that is summarized as follows. The impeding vehicle is the first vehicle released at the beginning of the zone. The passing vehicle is the second vehicle released on the route. The passing vehicle is released when the impeding vehicle reaches the necessary headway from the beginning of the passing zone. The opposing vehicle is released at the same time as that of the passing vehicle. Since the passing vehicle is supposed to overtake the slower vehicle; it starts to accelerate after the perception reaction time. All through the maneuver, the impeding and opposing vehicles maintain a constant speed. The locations of the three vehicles and the speed of the passing vehicle are updated at every time step. Two scenarios are simultaneously tested in each run, a ‘pass’ scenario and an ‘abort’ scenario. The two scenarios start when the passing vehicle reaches the ‘critical point’ relative to the impeding vehicle. The two scenarios are simulated in order to assess the risk of the passing situation for that specific PSD length. The pass scenario assesses the risk of completing the pass. The abort scenario assesses the risk of aborting the pass and setting back behind the slower vehicle. This way, the final risk of the passing maneuver can be assessed by summing both risk measures. The flowchart of the simulation logic is shown in Figure 6.3.



**Figure 6.3 – Flowchart of the simulation logic**

#### 6.4.2.1 Pass Scenario

In the pass scenario, the passing vehicle continues to accelerate to a maximum allowable speed. The maximum speed is one of the parameters initialized at the beginning of each run. It is randomly selected to be 5 to 10 mph higher than the posted speed. The model then decides whether the passing vehicle has passed the impeding vehicle based on the following equation (numbers 1, 2, and 3 stand for the impeding, passing, and opposing vehicles, respectively):

$$\text{Location}(2) \geq \text{Location}(1) + \text{Reentry\_Distance} + \text{Vehicle\_Length}(2) \quad (6.2)$$

If this condition is true, then the clearance time between the passing and opposing vehicles is computed. The latter variable will decide the risk index of the pass maneuver (PassRI).

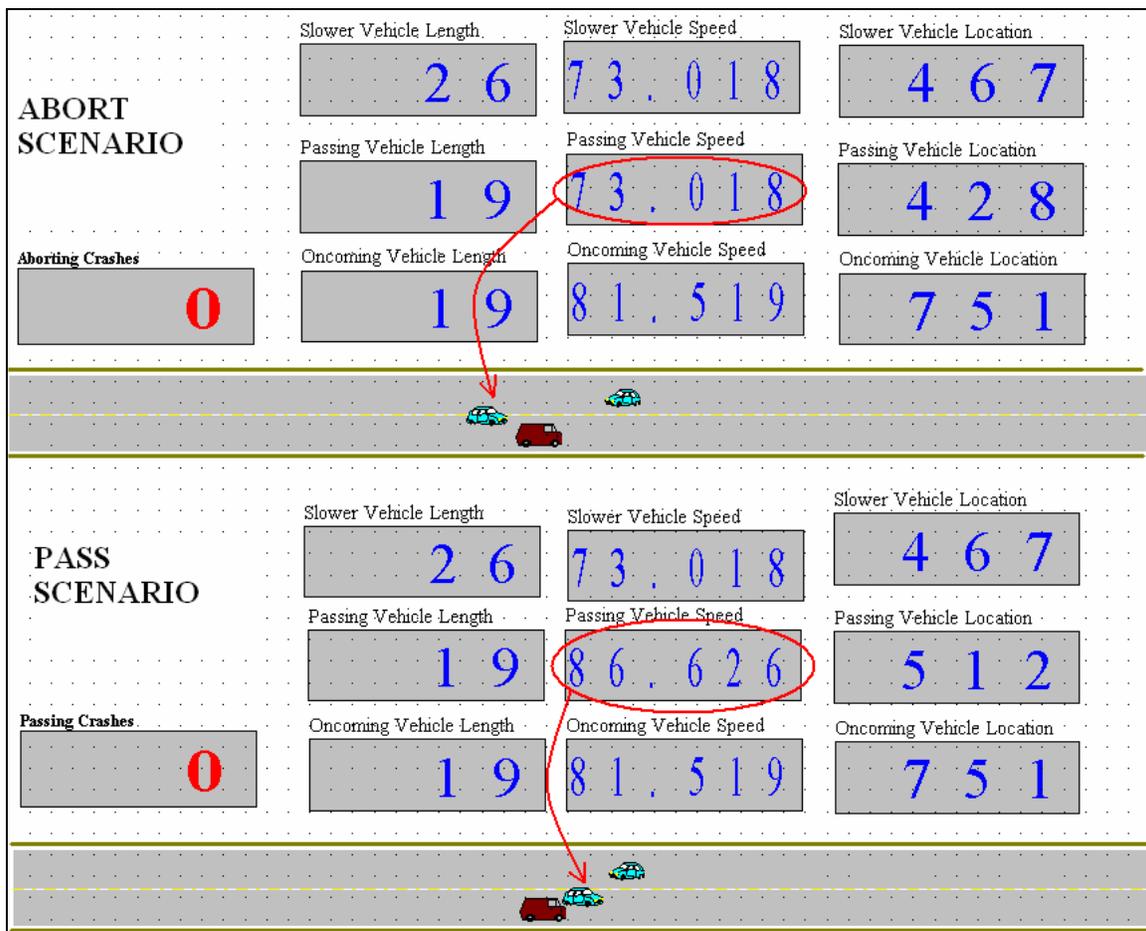
#### 6.4.2.2 Abort Scenario

In the abort scenario, the passing vehicle decelerates till it achieves a safe setback distance behind the impeding vehicle. This is checked in the simulation by computing the following equation:

$$\text{Location}(2) \leq \text{Location}(1) - \text{Setback\_Distance} - \text{Vehicle\_Length}(1) \quad (6.3)$$

If this condition is true, then the clearance time between the passing and opposing vehicles is computed. The latter variable will decide the risk index of the abort maneuver (AbortRI).

A snapshot of the animation of the two simulated scenarios is shown in Figure 6.4. The top part of the figure presents the Abort scenario, while the bottom section shows the Pass scenario. It is obvious in the Abort scenario that the passing vehicle is trying to setback behind the impeding vehicle. This is demonstrated by the reduction in the passing vehicle speed as highlighted in the figure. Also, the location of the passing vehicle is less than the impeding vehicle at this instant by 39 ft. As to the Pass scenario, the passing vehicle has managed to overtake the impeding vehicle. Both its location and speed are larger than those of the impeding vehicle. In both of these cases, the opposing vehicle is avoided, and the pass maneuver resulted in no collision. However, the clearance time seems to be much less in the case of the Pass scenario, which results in a higher risk level and thus a higher risk index value.



**Figure 6.4 – Snapshot of the simulation progress for the two scenarios**

### 6.4.3 Post Processing and Results

The post processing module focuses on collecting data and statistics about the final risk index. The final risk index is basically computed by adding the passing risk index (PassRI) and the abort risk index (AbortRI) at the end of each trial. The value of the PassRI or AbortRI depends mainly on the computed clearance time. Clearance time is the time gap between the passing vehicle and the opposing vehicle at the end of the maneuver; be it a pass or an abort maneuver. Three levels of risk are assumed based on the final conditions of the maneuver. The first level is when the maneuver exhibits no risk. That is when the maneuver is completed with more than 2 seconds of clearance time between the passing and opposing vehicle. By using clearance time as the decision parameter, the clearance distance varies with different design speeds, which is logical. The second risk level is declared when the maneuver is completed with less than 2 seconds of clearance time. Thus, the maneuver is somehow risky. The third and highest risk level is reached when the maneuver ends up in a crash. A crash is recorded when the pass/abort maneuver is executed with zero or less clearance time. Three risk indices are assigned,

one for each situation. For the first risk level, the PassRI/AbortRI takes a value of zero, since the maneuver is safely completed. The PassRI/AbortRI is 0.5 when the maneuver ends with less than 2 seconds of clearance time, as mentioned earlier. The risk index is equal to 2 when the passing and opposing vehicles collide before the later is able to merge ahead of the impeding vehicle at the end of the passing maneuver. A value of 2 is assigned for the third risk level in order to give it more weight than the second and first situations. At the end of each run, the PassRI and the AbortRI are summed to compute the final risk index, RI. RI can thus take one of six values, (0, 0.5, 1, 2, 2.5, and 4). A final risk index of 4 indicates that the selected PSD length is insufficient for safe passing maneuvers. That is the maneuver will end up in a crash for both the pass and abort scenarios. The selected risk scale is not intended to actually measure the risk value. It is a relative scale that contrasts risk levels between the different outcomes. The “0, 0.5, and 2” scale could be replaced by any scale, for example, “0, 1, and 5”, respectively. But the idea will still be the same and the risk measure will still be a relative measure between two or more scenarios.

The simulation is carried out for three design speeds. Ten PSD lengths are tested for each design speed. The values of the PSDs are selected from the distribution curves obtained using the Monte Carlo simulation which has been discussed in chapter 5 of the dissertation. The minimum and maximum values are based on the PSD distribution statistics. The authors tried various numbers of simulation runs for each PSD length. Two, ten and twenty thousand runs have been simulated for one selected PSD length. The results of the three setups were statistically similar. Thus, a two-thousand-runs setup is used to test all the other PSD lengths. The post processing is done for three design speeds, separately. Note that the results of the simulation represent passing maneuvers under the worst case conditions. For example, all the passing attempts are assumed to be accelerative rather than flying. A flying pass requires shorter PSD than an accelerative pass. In a flying pass, the passing driver maintains the speed difference and does not slow down to the speed of the impeding vehicle. The simulation is setup in a way to conduct the passing maneuver in the presence of an opposing vehicle. In addition, both the opposing vehicle and the impeding vehicle are assumed to be neutral all through the passing attempt. Real life situations exhibit less dramatic conditions. The drivers of both the opposing and the impeding vehicles normally decelerate in critical situations in order to avoid high risk situations. Also, the passing vehicle is not allowed to abort until it reaches the critical position. So, the model is basically simulating worst case passing situations.

#### 6.4.3.1 Results for 40 mph Design Speed

The PSD lengths have been selected from the statistics of the PSD distribution presented in Table 5.3. The selected values increase from a minimum of 350 ft to a maximum of 1050 ft at 100 ft intervals. One extra value is considered, 581 ft, which is the mean of the distribution. Two thousand simulation runs are performed for each value. At the end of each run, the clearance time is calculated and a corresponding risk index is recorded. The percentages of each risk index out of the 2000 runs per PSD length are presented in Table 6.1. At the end of the simulation setup, a weighted RI is computed for each PSD length. The last row of Table 6.1 presents the weighted-average RI for a design speed of 40 mph. The final Risk Index is weighted by the assigned risk scale and its percentage. The results presented in this table are also shown in Figure 6.5. The chart portrays the regions that each risk index covers relative to each PSD length. The weighted risk index is plotted against the PSD length, as shown in Figure 6.6.

**Table 6.1 – Percent of each RI category at 40 mph design speed**

PSD (ft)	350	450	550	581	650	750	850	950	1050
RI									
0	0.0%	0.0%	0.0%	0.0%	0.0%	12.4%	58.9%	85.0%	98.8%
0.5	0.0%	0.4%	21.7%	43.2%	74.0%	78.0%	36.4%	11.3%	0.0%
1	0.0%	1.8%	19.0%	11.7%	2.4%	0.3%	0.6%	0.6%	0.0%
2	6.8%	15.5%	14.9%	15.0%	12.8%	3.9%	1.3%	0.9%	1.2%
2.5	19.2%	64.7%	39.8%	26.6%	8.3%	2.9%	0.8%	0.8%	0.0%
4	74.0%	17.6%	4.6%	3.5%	2.6%	2.4%	2.1%	1.5%	0.0%
Sum	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%
<b>Weighted RI</b>	<b>3.57</b>	<b>2.65</b>	<b>1.77</b>	<b>1.43</b>	<b>0.95</b>	<b>0.64</b>	<b>0.31</b>	<b>0.15</b>	<b>0.02</b>

A curve is fit to the data points presented in Figure 6.6 in order to obtain a continuous function for the RI. This way, the RI for any PSD length can be estimated. A software package (CurveExpert) is utilized to obtain the best fit curve along with the corresponding statistics. The Gaussian function is found to best fit the data points using the following equation:

$$RI = a \times e^{-\frac{(PSD-b)^2}{2c^2}} \quad (6.4)$$

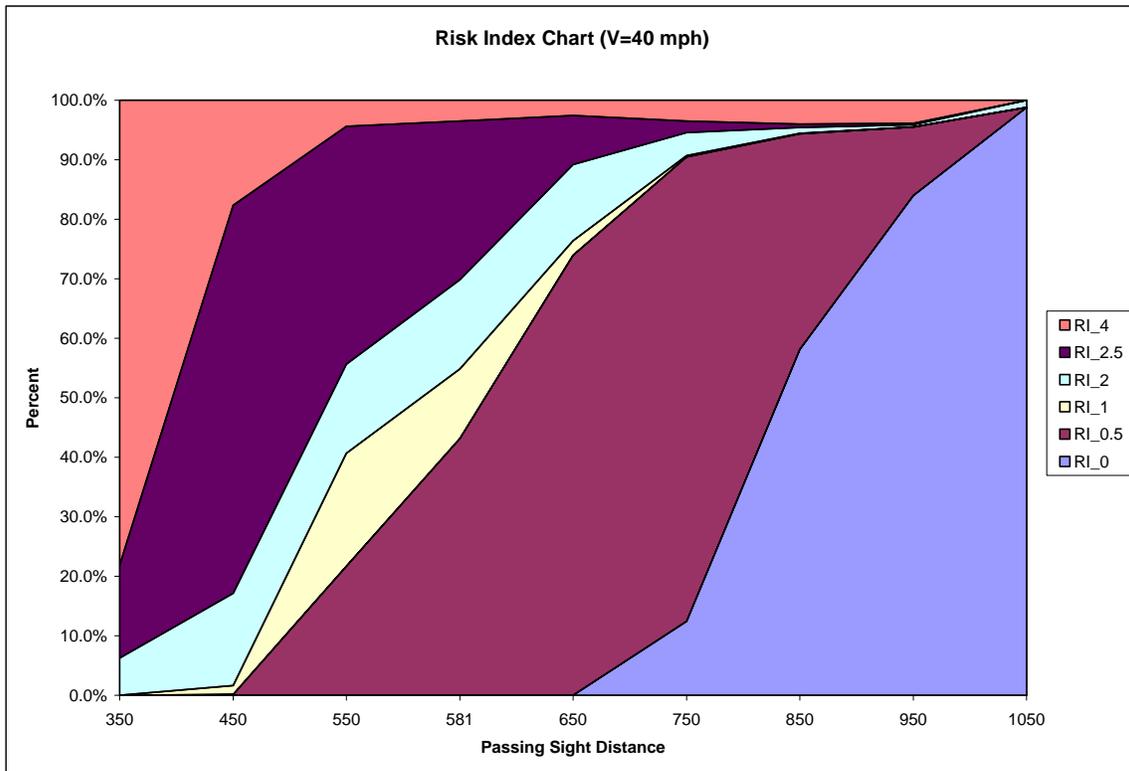
Where,

RI = Risk Index (using the Gaussian equation)

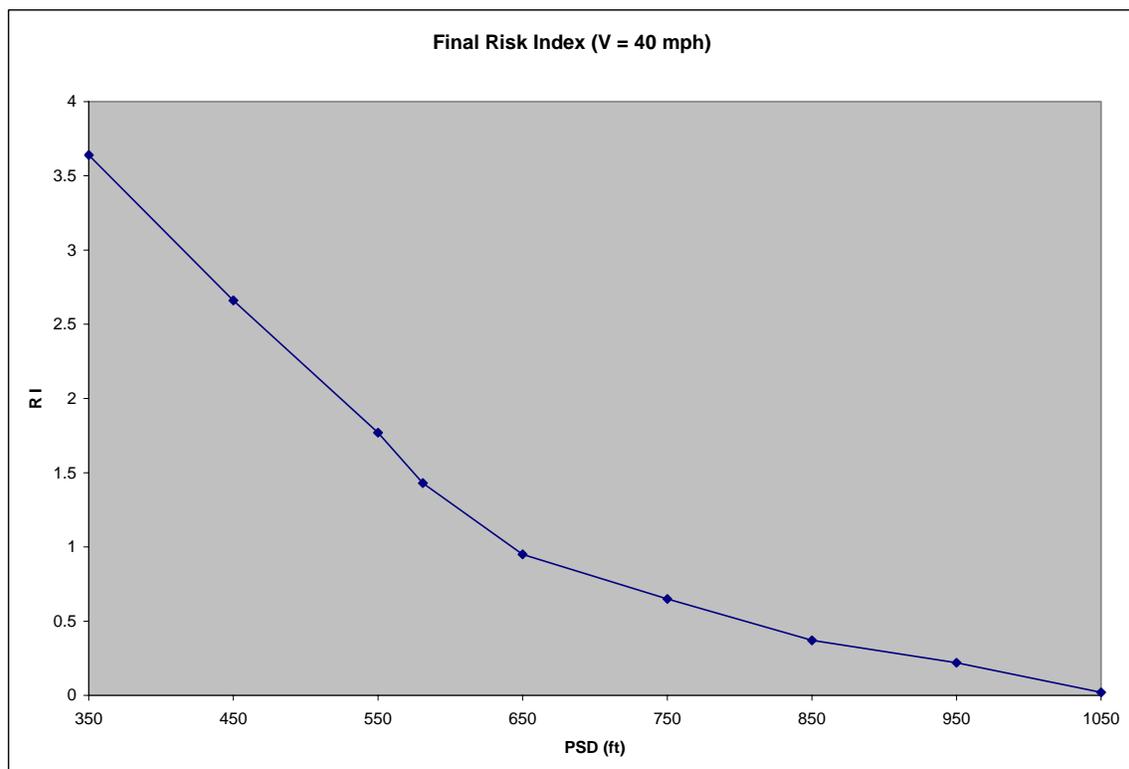
PSD = Desired PSD

a,b,&c = Coefficients of the Gaussian curve.

The coefficients and statistics of the curve fitting process are shown in Table 6.2. The derived coefficients of the Gaussian fit curve are applicable for PSDs from 350 to 1050 ft. A view of the curve fit process is shown in Figure 6.7.



**Figure 6.5 – Variation of the final RI at 40 mph**

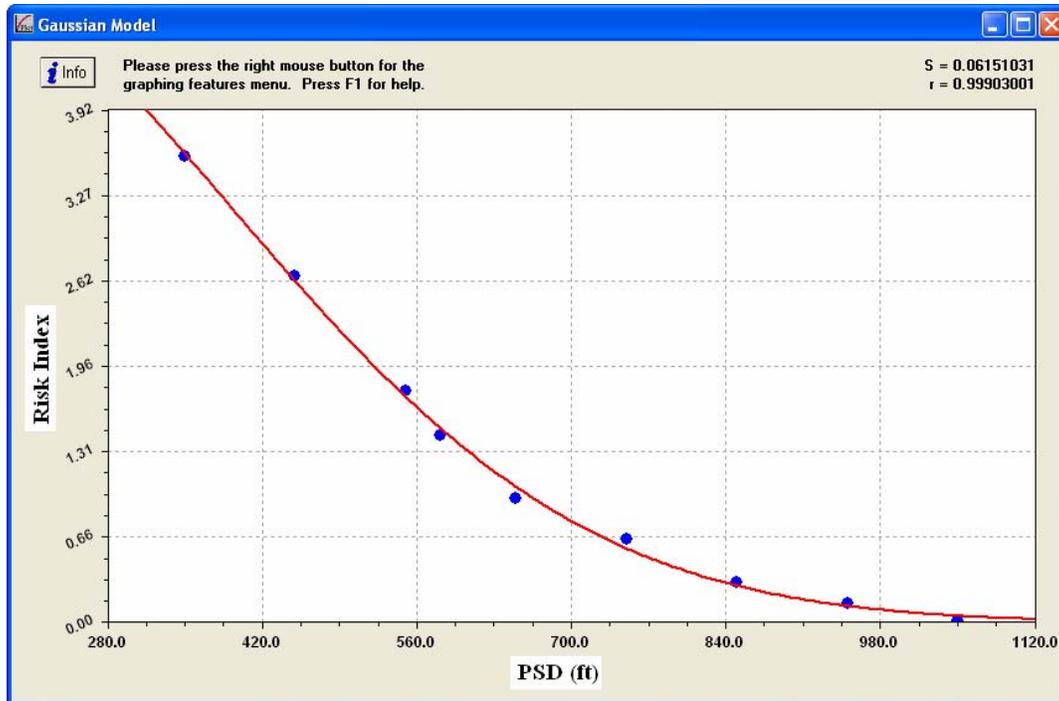


**Figure 6.6 – Weighted average of the final Risk Index at 40 mph**

**Table 6.2 – Coefficients and statistics of the Gaussian curve fit**

Design Speed (mph)	Coefficients			Stats			
	a	b	c	s	r	t	n
40	5.30	62.315	325.4	0.0615	0.9990	10 <sup>-6</sup>	5

s=Standard error; r=Correlation coefficient; t=tolerance; n=number of iterations



**Figure 6.7 – Sample curve fit of the RI using CurveExpert**

#### 6.4.3.2 Results for 50 mph Design Speed

Similarly, the PSD lengths have been selected from the statistics of the PSD distribution presented in Table 5.8. The selected values increase from a minimum of 450 ft to a maximum of 1250 ft at 100 ft intervals. One extra value is considered, 705 ft, which is the mean of the distribution. At the end of each run, the clearance time is similarly calculated and a corresponding risk index is recorded. The percentages of each risk index are presented in Table 6.3. The last row of Table 6.3 presents the weighted-average RI for a design speed of 50 mph. The results presented in this table are also shown in Figure 6.8. The chart portrays the regions that each risk index covers relative to each PSD length. The weighted risk index is plotted against the PSD length, as shown in Figure 6.9.

**Table 6.3 – Percent of each RI category at 50 mph design speed**

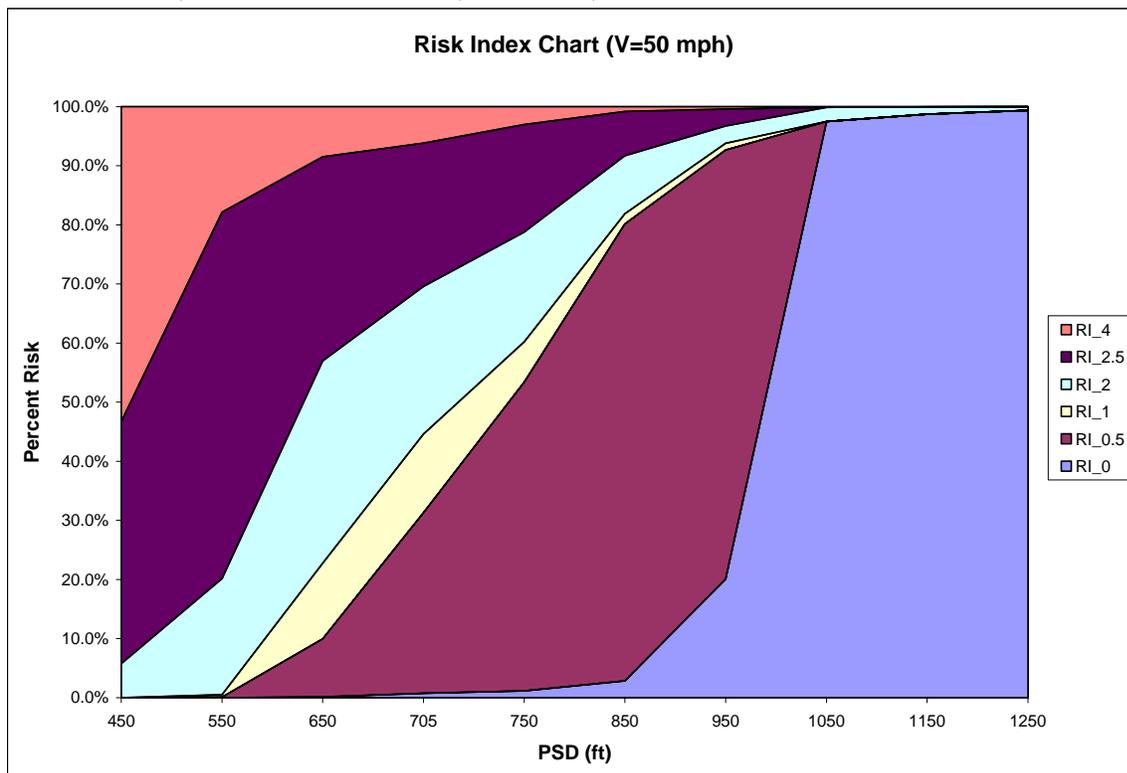
PSD (ft)	450	550	650	705	750	850	950	1050	1150	1250
RI										
0	0.0%	0.0%	0.2%	0.8%	1.2%	2.9%	20.1%	97.5%	98.7%	99.4%
0.5	0.0%	0.1%	9.8%	30.5%	52.2%	77.3%	72.6%	0.0%	0.0%	0.0%
1	0.0%	0.4%	12.8%	13.3%	6.8%	1.7%	1.1%	0.0%	0.0%	0.0%
2	5.7%	19.7%	34.2%	25.0%	18.6%	9.8%	2.9%	2.4%	1.2%	0.6%
2.5	41.0%	62.0%	34.6%	24.3%	18.2%	7.5%	2.9%	0.0%	0.0%	0.0%
4	53.3%	17.9%	8.5%	6.1%	3.0%	0.8%	0.4%	0.1%	0.1%	0.0%
Sum	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%
<b>Weighted RI</b>	<b>3.27</b>	<b>2.66</b>	<b>2.06</b>	<b>1.63</b>	<b>1.27</b>	<b>0.81</b>	<b>0.52</b>	<b>0.05</b>	<b>0.02</b>	<b>0.01</b>

Also, a curve is fit to the data points presented in Figure 6.9 in order to obtain a continuous function for the RI. Using the same software package (CurveExpert), the Gaussian function, which is demonstrated in equation 6.4, is found to best fit the data points. The coefficients and statistics of the curve fitting process are shown in Table 6.4. Note that the derived coefficients of the Gaussian fit curve are applicable for PSDs between 450 ft and 1250 ft. A view of the curve fit process is shown in Figure 6.10.

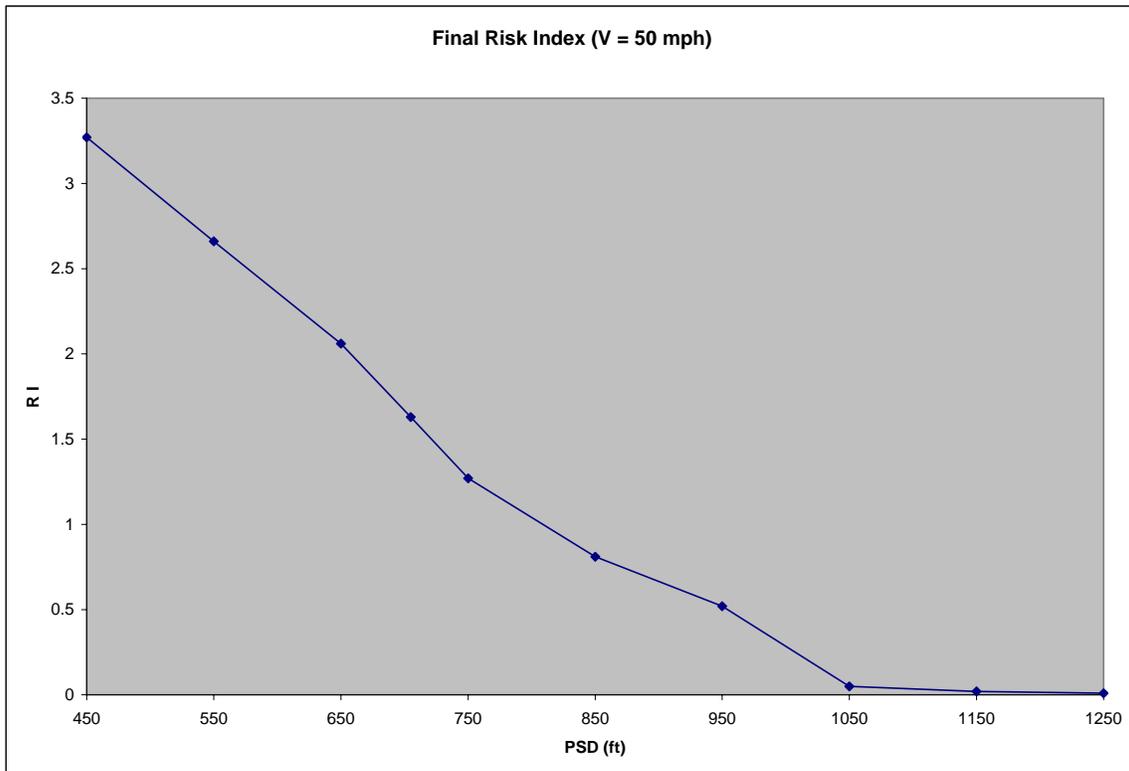
**Table 6.4 – Coefficients and statistics of the Gaussian curve fit**

Design Speed (mph)	Coefficients			Stats			
	a	b	c	s	r	t	n
50	3.469	344.6	292.4	0.0805	0.9983	10 <sup>-6</sup>	4

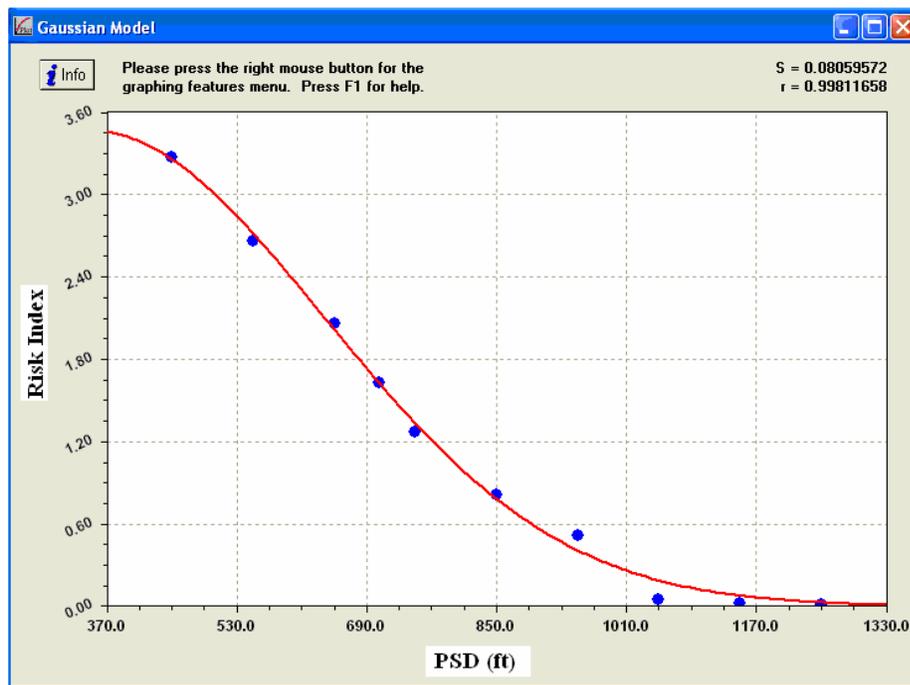
s=Standard error; r=Correlation coefficient; t=tolerance; n=number of iterations



**Figure 6.8 – Variation of the final RI at 50 mph**



**Figure 6.9 – Weighted average of the final Risk Index at 50 mph**



**Figure 6.10 – Sample curve fit of the RI at 50 mph (CurveExpert)**

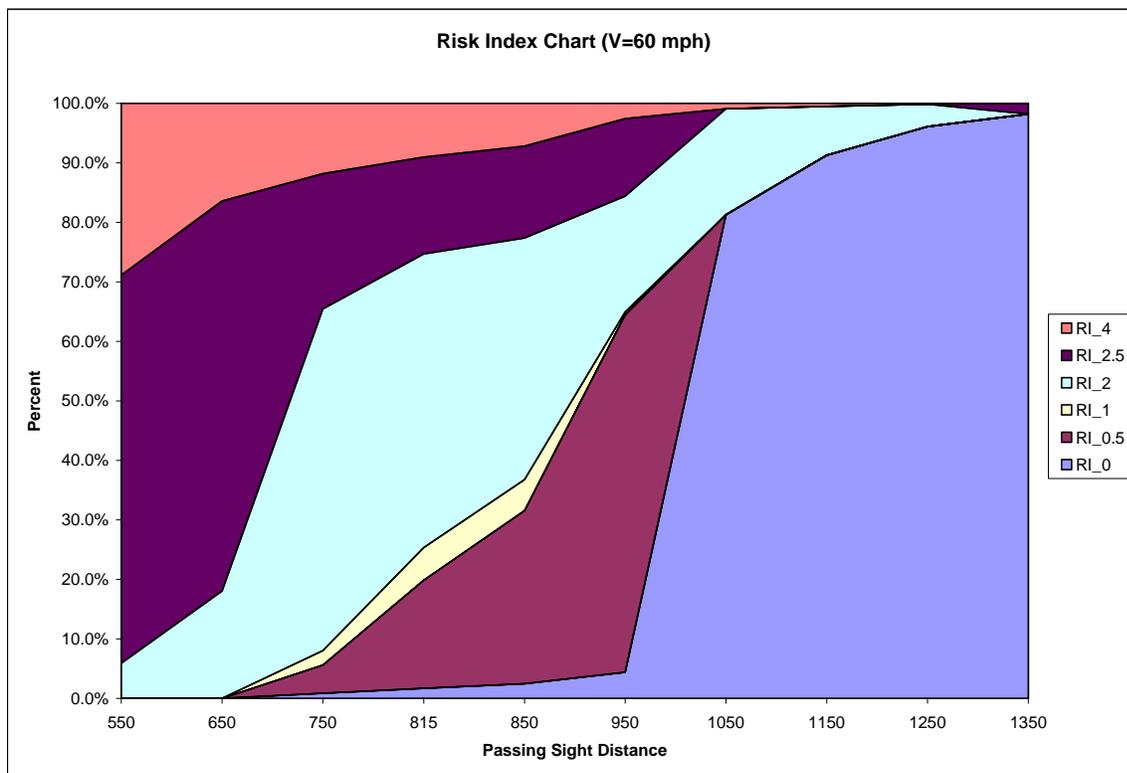
**6.4.3.3 Results for 60 mph Design Speed**

Statistics of the PSD distribution presented in Table 5.13 are used to select the PSD lengths for this design speed. The selected values increase from a minimum of 550 ft to a

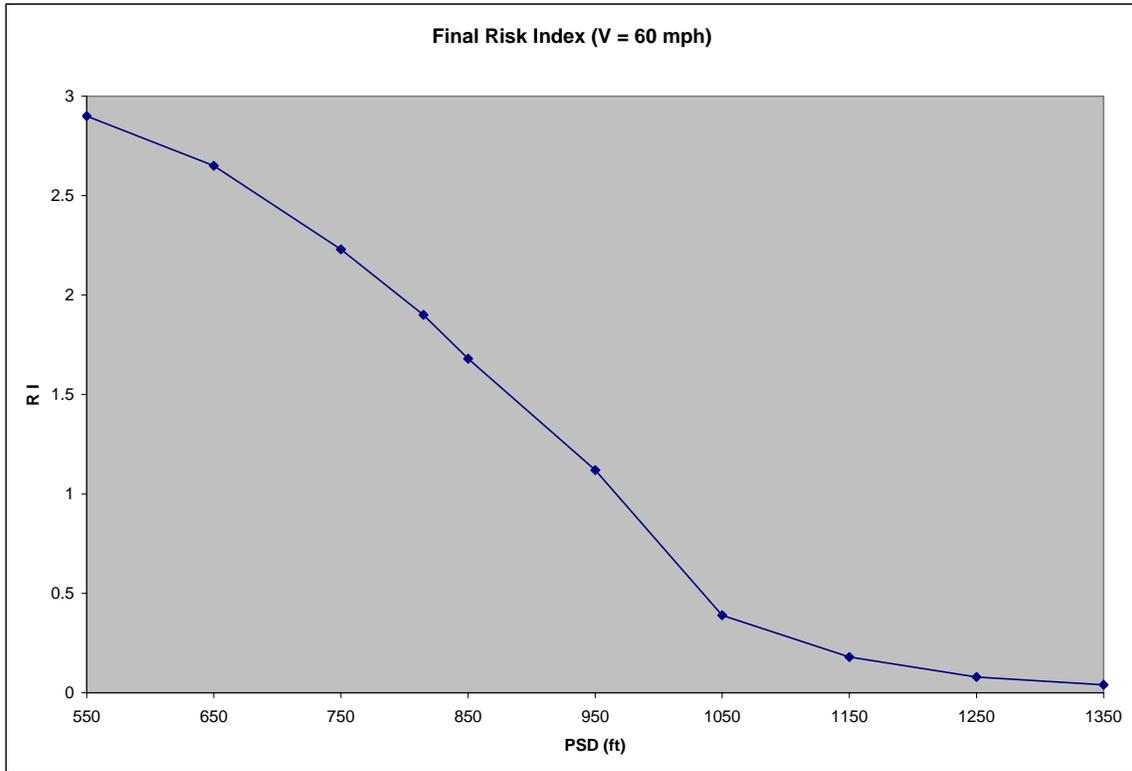
maximum of 1350 ft at 100 ft intervals. One extra value is considered, 815 ft, which is the mean of the distribution. The percentages of each risk index are presented in Table 6.5. The results presented in this table are also shown in Figure 6.11. The weighted risk index, shown in the last row of Table 6.5, is plotted against the PSD length, as shown in Figure 6.12.

**Table 6.5 – Percent of each RI category at 60 mph design speed**

PSD (ft)	550	650	750	815	850	950	1050	1150	1250	1350
RI										
0	0.0%	0.0%	0.9%	1.7%	2.5%	4.4%	81.3%	91.3%	96.1%	98.2%
0.5	0.0%	0.0%	4.7%	18.2%	29.1%	60.0%	0.0%	0.0%	0.0%	0.0%
1	0.0%	0.0%	2.5%	5.5%	5.2%	0.5%	0.0%	0.0%	0.0%	0.0%
2	5.9%	18.0%	57.4%	49.4%	40.6%	19.4%	17.8%	8.2%	3.8%	0.0%
2.5	65.2%	65.5%	22.7%	16.2%	15.4%	13.1%	0.0%	0.0%	0.0%	1.8%
4	28.9%	16.4%	11.8%	9.1%	7.2%	2.6%	0.9%	0.5%	0.2%	0.0%
Sum	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%
<b>Weighted RI</b>	<b>2.9</b>	<b>2.65</b>	<b>2.23</b>	<b>1.9</b>	<b>1.68</b>	<b>1.12</b>	<b>0.39</b>	<b>0.18</b>	<b>0.08</b>	<b>0.04</b>



**Figure 6.11 – Variation of the final RI at 60 mph**



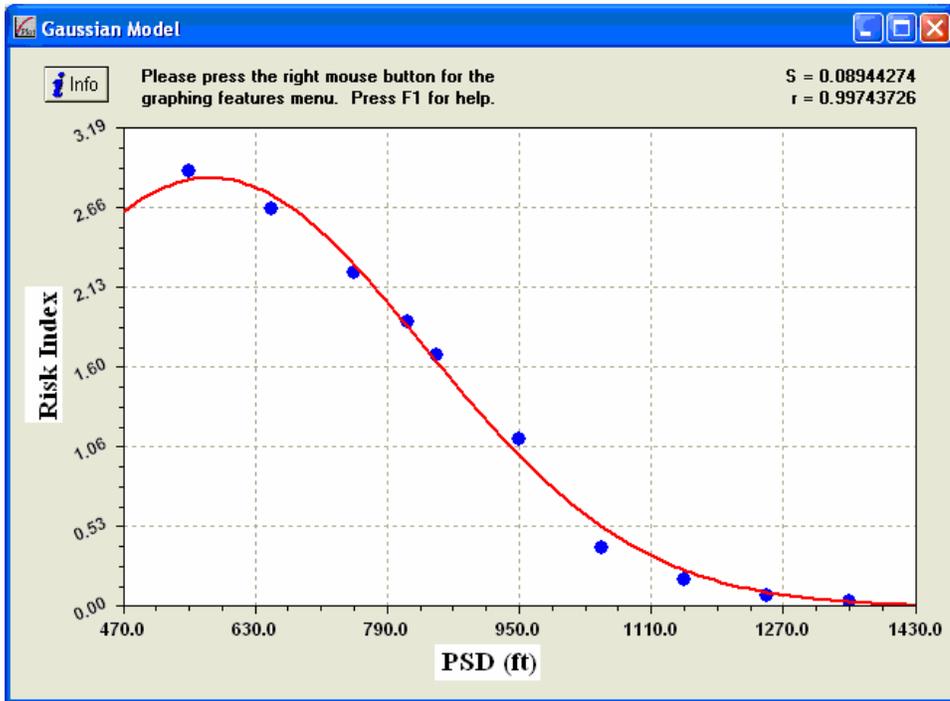
**Figure 6.12 – Weighted average of the final Risk Index at 60 mph**

A curve is also fit to the data points presented in Figure 6.12 in order to obtain a continuous function for the RI. CurveExpert is again used for this purpose. The Gaussian function, which is demonstrated in equation 6.4, is found to best fit the data points. The coefficients and statistics of the curve fitting process are shown in Table 6.6. Note that the derived coefficients of the Gaussian fit curve are applicable for PSDs between 550 ft and 1350 ft. A view of the curve fit process is shown in Figure 6.13.

**Table 6.6 – Coefficients and statistics of the Gaussian curve fit**

Design Speed (mph)	Coefficients			Stats			
	<b>a</b>	<b>b</b>	<b>c</b>	s	r	t	n
50	2.857	574.5	259.5	0.089	0.9974	10 <sup>-6</sup>	4

s=Standard error; r=Correlation coefficient; t=tolerance; n=number of iterations



**Figure 6.13 – Sample curve fit of the RI at 60 mph (CurveExpert)**

### 6.5 Risk Index of Current PSD standards

Using the devised microscopic simulation, the risk of the current PSD design values is assessed. Three PSD design values are presented in the literature, which are AASHTO’s, MUTCD’s, and Glennon’s. These values are used as input into the simulation. Two thousand simulation runs are performed for each design value. The risk indices are calculated similar to the previous setups. Table 6.7 presents the various risk indices of the current PSD design values.

**Table 6.7 – Risk Index of current PSD design standards**

Standard	AASHTO			MUTCD			Glennon(1988)		
Speed (mph)	40	50	60	40	50	60	40	50	60
PSD (ft)	1470	1835	2135	600	800	1000	670	830	990
Simulated RI	0	0	0	1.28	1.01	0.75	0.9	0.9	0.92
Estimated RI	$1.2 \times 10^{-3}$	$8 \times 10^{-6}$	$4 \times 10^{-8}$	1.35	1.03	0.74	0.95	0.88	0.8
% Diff. in RI	---	---	---	5%	2%	1%	5%	2%	12%

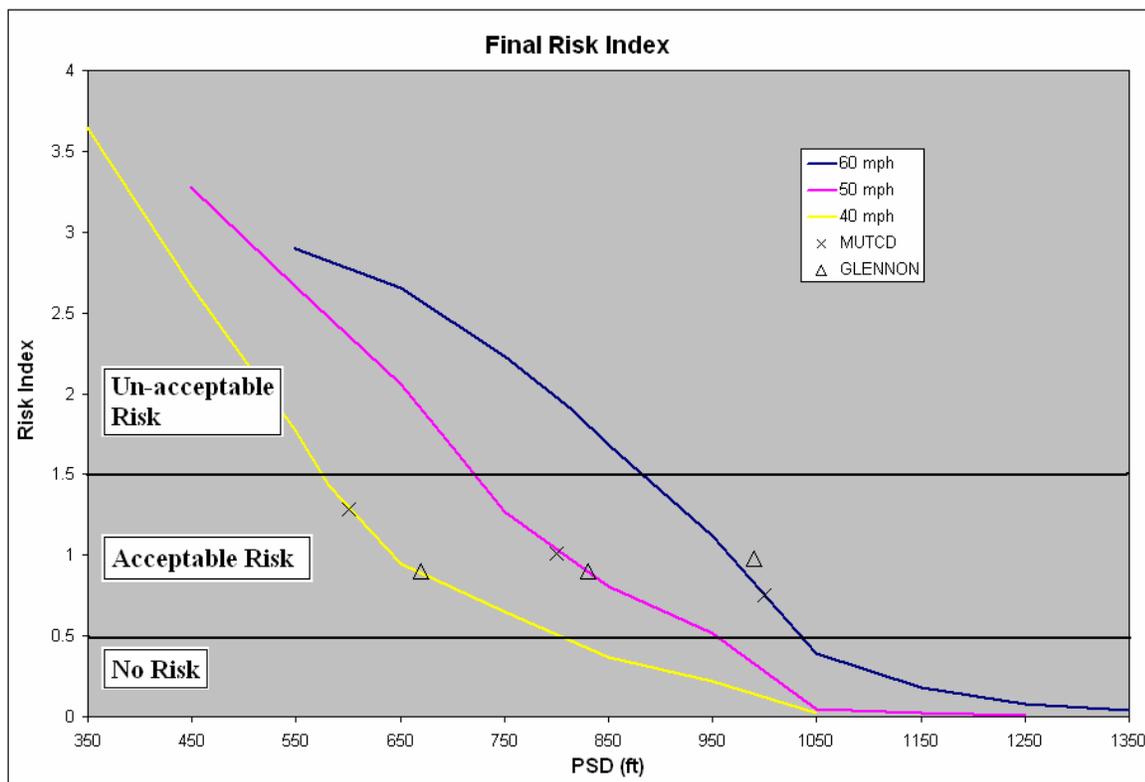
Notice the conservative design values that AASHTO “Green Book” specifies which are demonstrated by a zero risk index. Glennon’s values appear to be the most logical since they exhibit similar risk indices for different speed levels. Thus, the values seem to maintain a premeditated level of risk. The Gaussian curves are also utilized to obtain an estimated RI of the current PSD design requirements, as shown in the last row of Table 6.7. The RI index values obtained using the simulation and those using the curve fit are within 5 percent difference, except

for the last value in the table. This assures that the best-fit curves can fairly predict the RI for any PSD length.

## 6.6 Brief Summary and Discussion

This chapter presents a direct method to assess the risk of various PSD lengths. A unique microscopic simulation is devised to replicate passing maneuvers on two-lane roads. The simulation is built using the ARENA software. The movement of three vehicles is monitored for every 0.1 seconds time step. Two scenarios are tested in each run, the pass and abort scenarios. A separate risk index is computed for each scenario. Then, a final risk index is calculated by summing up the risk indices from the two scenarios. The PSD lengths are selected from the PSD distributions obtained in the previous chapter of this dissertation.

Using the simulation results, the author has attached risk measures to the values of the PSD distributions. In addition, the author has assessed the risk of the current PSD standards. Three levels of risk are identified, which are: No Risk, Acceptable Risk, and Un-acceptable Risk, as shown in Figure 6.14.



**Figure 6.14 – Weighted average of final RI**

As mentioned earlier, a final risk index equal to 2 means that a crash occurred in one of the scenarios, the abort or the pass. So, the author set the limit of the acceptable risk level at 1.5 to ensure that the weighted RI is well less than 2. The risk levels of the current PSD design criteria

fall within the acceptable risk range. But they are less than the limit value of that range. Thus, minimum PSD criteria can be reduced from the current values and still be in the acceptable risk level. The PSD criteria at a risk index of 1.5 are calculated using the Gamma-fit equations. The estimated limit values are **580 ft**, **725 ft**, and **875 ft** for design speeds of 40, 50, and 60 mph, respectively. The simulation is used to obtain the probability of being involved in an accident for a 50 mph design speed and a PSD of 725 ft and 1000 ft. Note that the probabilities of being involved in a collision are subject to the simulation constraints and conditions. Most of these conditions are selected to represent worst case passing scenarios. Thus, they do not reflect normal passing maneuvers. Actual passing drivers enjoy much more choices than the simulation can handle, and thus will encounter crash probabilities less than those presented by the simulation. However, the simulation results are used as a guide to compare the PSD lengths and their risk indices. The 725 ft PSD length is the limit value recommended to decrease the PSD criteria. The 1000 ft PSD length is the current PSD value applied to the US Route which is used as an application in Chapter 7 of the dissertation. The results are incorporated into the trade-off analysis that is conducted in the last task of the research. The results of the simulation run are presented in Table 6.8. The results show that the probability of a crash is 0.004 and 0.04 for PSD lengths of 1000 and 725 ft, respectively. Recall that the simulation is replicating passing maneuvers where the passer conducts an accelerative pass while there is an oncoming vehicle in the opposite direction. The opposing vehicle is only a PSD length away. The chances of such scenarios occurring on an actual road are minimal. More information is needed to actually quantify the real probability of a crash, such as the percent of total drivers passing on two-lane roads, percent of those passing drivers attempting risky maneuvers, and percent of passing maneuvers ending up in crashes. All these data need to be surveyed before the simulation could be calibrated. Once the model is calibrated, the results will then portray the actual crash probabilities.

**Table 6.8 – Percent of each RI category at 50 mph design speed**

<b>PSD (ft)</b>	<b>725</b>	<b>1000</b>
Risk Index		
0	1.0%	52.9%
0.5	50.1%	41.3%
1	1.9%	1.0%
2	36.7%	2.7%
2.5	6.3%	1.7%
4	4.0%	0.4%
Sum	100.0%	100.0%
<b>Weighted RI</b>	<b>1.5</b>	<b>0.35</b>

The recommendations to decrease the current criteria will depend on the results of the trade-off analysis. It will mainly depend on the amount of time savings (reduction in delay) that is accrued from decreasing the minimum PSD criteria. The variation in the PSD criteria has varying effects on the service measures of different roads. A simple example that illustrates the argument is the following: by reducing/increasing the PSD criteria, the percentage of no-passing zones on a rolling terrain two lane road varies accordingly. This variation is dependent on the topology and geometry of the road. Thus, any trade-off analysis within these lines is site specific. It is dependent on the designer’s safety and service preferences, as well as, on the prevailing conditions of the project.

Finally, the selected risk scale is not intended to actually measure the risk value, but to present a relative scale to contrast risk levels between the different outcomes. Relative scales are usually subjective in nature. As the author is aware of that fact, a different risk scale is utilized to verify that the results of the simulation are independent of the risk scale adopted. The “0, 0.5, and 2” scale is replaced by “0, 1, and 6”, respectively. Only one trial of simulation is reproduced to test the second risk scale for a design speed of 50 mph.

#### **6.6.1 New Risk Scale at 50 mph**

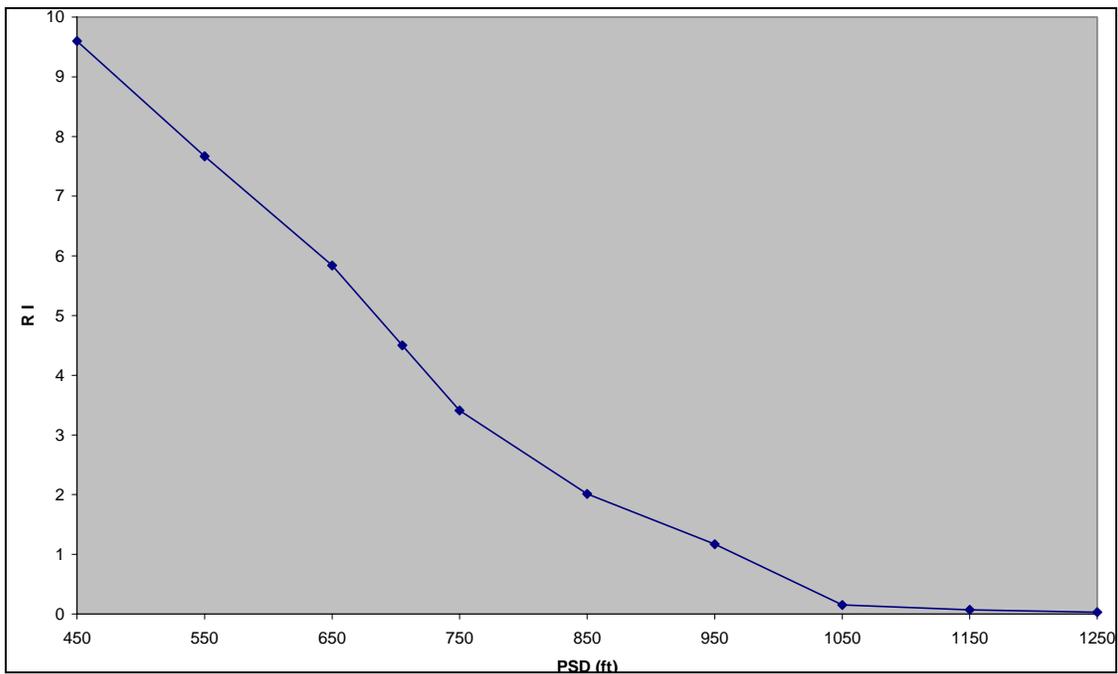
The same procedure defined in previous sections of this chapter is utilized to obtain the results. The selected values also increase from a minimum of 450 ft to a maximum of 1250 ft at 100 ft intervals. The percentages of each risk index are presented in Table 6.9. The last row of Table 6.9 presents the weighted-average RI for the trial risk scale. The weighted risk index is plotted against the PSD length, as shown in Figure 6.15.

**Table 6.9 – Percent of each RI category at 50 mph design speed**

PSD (ft)	450	550	650	705	750	850	950	1050	1150	1250
RI										
0	0.0%	0.0%	0.2%	0.8%	1.2%	2.9%	20.1%	97.5%	98.7%	99.4%
1	0.0%	0.1%	9.8%	30.5%	52.2%	77.3%	72.6%	0.0%	0.0%	0.0%
2	0.0%	0.4%	12.8%	13.3%	6.8%	1.7%	1.1%	0.0%	0.0%	0.0%
6	5.7%	19.7%	34.2%	25.0%	18.6%	9.8%	2.9%	2.4%	1.2%	0.6%
7	41.0%	62.0%	34.6%	24.3%	18.2%	7.5%	2.9%	0.0%	0.0%	0.0%
12	53.3%	17.9%	8.5%	6.1%	3.0%	0.8%	0.4%	0.1%	0.1%	0.0%
Sum	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%
<b>Weighted RI</b>	<b>9.6</b>	<b>7.67</b>	<b>5.84</b>	<b>4.5</b>	<b>3.41</b>	<b>2.01</b>	<b>1.17</b>	<b>0.15</b>	<b>0.07</b>	<b>0.03</b>

The same results were observed but with a different risk scale. The graph of the weighted average of the final risk index, shown in Figure 6.15, is slightly different from the previous one, shown in Figure 6.9. The difference is due to the different weights attributed to each risk level. In

the new risk scale, the crash situation is assigned a weight that is six times more than a risky maneuver with clearance time less than 2 seconds (6 vs 1, as opposed to 2 vs 0.5). That is, the graph is stretched up and flattened in the middle, but the results exhibit the same trend.



**Figure 6.15 – Weighted average of the final Risk Index at 50 mph (New risk scale)**

## **CHAPTER 7:**

### **OPERATIONAL EFFECTS OF PASSING SIGHT DISTANCES**

## 7.1 Introduction

The focus of this research is to combine the service and safety measures of passing sight distances (PSD) in one design process. In this case, trade-off analysis is a compromise between the corresponding safety and service measures of the highway PSD criteria. The safety measures of PSDs have been assessed and presented in the previous chapters. This chapter is dedicated to obtaining the service and operational impacts of varying the PSD requirements on two-lane highways. The Highway Capacity Manual concepts are used to quantify the effects of the PSD on the free flow speed (FFS) on a specific road. Then, the effects of the speed difference are converted into time difference, or otherwise stated, delay savings/incurred. The IHSDM software package is used to apply these concepts to a specific two lane section. Various PSDs are tested to assess their service measures and quantify their operational impacts on traffic. Then, a detailed methodology for applying these concepts is presented to assist the designer in the trade-off analysis process.

## 7.2 Background

The idea behind the research addresses the main intent of providing the PSD, which is to ensure the *safety* of the passing driver, and provide an adequate *level of service* to the traveling public. Driver safety is measured by the risk level encountered in conducting the passing maneuver. It is characterized by the probability of being involved in a collision or a crash with the opposing vehicle. This portion of the problem has been addressed and its results presented in chapter 6 of the dissertation. The level of service concept could be measured by the delay time incurred following slower vehicles on a particular two-lane road. This parameter is based on the difference between the desired free flow speed of drivers and their actual speed on the route due to the surrounding traffic. The reduction/gain in free flow speed on two way roads is related to many factors, as discussed in chapter 20 of the Highway Capacity Manual 2000. One of these parameters is the percentage of no-passing zone, and thus, to the passing opportunity. But the no-passing zone percentage is directly correlated to the PSD criteria on a specific two lane road. In fact, the PSD criteria are used to delineate those sections of the road where passing is prohibited (no-passing zone). The effect of passing is thoroughly discussed in the Highway Capacity Manual (HCM), AASHTO Green Book, and the MUTCD. It reads in AASHTO Green Book (2004):

“Sight distance adequate for passing should be encountered frequently on two-lane highways.....Frequency and length of passing sections for highways depend principally

on the topography, the design speed...and the cost.....The importance of passing sections is illustrated by their effect on the service volume of a two-lane, two-way highway....”

The MUTCD also emphasizes the importance of providing adequate/enough passing zones on two lane highways (FHWA 2000). Recently, the effects of the time drivers spend following slower traffic are being researched to quantify their impacts on the traffic stream. Moshe Pollatschek and Abishai Polus, from the department of Industrial and Civil engineering respectively at Technion, Israel Institute of Technology, are trying to model the impatience of drivers in passing maneuvers on two lane highways (Polus et al. 2000). In short, they have tried to quantify the impatience level of drivers based on the amount of time they are delayed while following slower traffic. Drivers become more aggressive, and thus take higher risks, in conducting a passing maneuver had they been trailing for a considerable amount of time. The idea projects a clear view of the importance of providing enough passing zones, especially on two lane highways that stretch for miles.

Generally, highway level of service concepts are widely discussed in the literature. They are mainly specified by the HCM. The HCM provides methods to calculate the level of service of any road section. These methods account for most of the traffic conditions and their effects on the level of service of the road under study. This chapter is merely presenting an application of some of the HCM concepts on two lane road sections. The aim is to calculate the variation in the performance measures of a specific two lane road section relative to the change in the PSD criteria.

### ***7.2.1 HCM Concepts***

Two lane highways are undivided roads with two lanes, one for use in each direction. Drivers use the opposing lane when passing a slower vehicle ahead, as sight distance and gaps in the opposing traffic stream permit. Most countries have two lane highways as a key element in their highway system. They perform a variety of functions, are located in all geographic areas, and serve a wide range of traffic (HCM 2000). Traffic operations on two-lane roads differ from those on other uninterrupted-flow facilities. Lane changing and passing are possible only in the face of oncoming traffic in the opposing lane. Passing demand increases rapidly as traffic volumes increase. Also, passing capacity decreases as volumes increase. Therefore, on two lane highways, normal traffic flow in one direction influences flow in the other direction. Motorists must adjust their travel speeds as volume increases and the ability to pass declines (HCM 2000). The following paragraph is explicitly stated in the Highway Capacity Manual 2000:

“... Efficient mobility is the principal function of major two lane highways connecting major traffic generators. These routes tend to serve long-distance commercial and recreational travelers. It is desirable to have consistent high-speed operations and infrequent passing delays on these facilities...Cost-effective access are the dominant consideration. Although beneficial, high speed is not the principal concern. Delay, as indicated by the formation of platoons, is more relevant as a measure of service quality. Two-lane roads also serve scenic and recreational areas in which the vista and environment are meant to be experienced and enjoyed without traffic interruption or delay. A safe roadway is desired, but high-speed operation is neither expected nor desired...”

#### *7.2.1.1 HCM Definitions*

##### *Percentage of no-passing zones*

The frequency of no-passing zones is used to characterize roadway design and to analyze expected traffic conditions along a two-lane highway. A no-passing zone is any zone marked for no passing or any section of road with a PSD of 1,000 ft or less. The average percentage of no-passing zones in both directions along a section is used for the analysis of two-way flow (HCM 2000).

##### *Average Travel Speed (ATS)*

On the other hand, average travel speed reflects the mobility on a two-lane highway. It is the length of the highway segment divided by the average travel time of all vehicles traversing the segment in both directions during a designated interval (HCM 2000).

##### *Percent-time-spent-following (PTSF)*

Percent-time-spent-following, as stated by the HCM 2000, is the percentage of time that drivers must travel in platoons following slower traffic due to their inability to pass on a certain two lane road. It represents the freedom to maneuver and the comfort and convenience of travel. Percent time-spent following is difficult to measure in the field. However, the percentage of vehicles traveling with headways of less than 3 s at a representative location can be used as a surrogate measure (HCM 2000).

##### *Two Lane Highway Categories*

Two-lane highways are categorized into two classes for analysis, as presented in the HCM 2000:

1. Class I: Motorists expect to travel at relatively high speeds on these types of facilities. Two lane highways that are major intercity routes, primary arterials connecting major

traffic generators, daily commuter routes, or primary links in state or national highway networks generally are assigned to Class I. Most arterials are considered Class I.

2. Class II: These are two lane highways on which motorists do not necessarily expect to travel at high speeds. They usually function as access routes to Class I facilities. Most collectors and local roads are considered Class II.

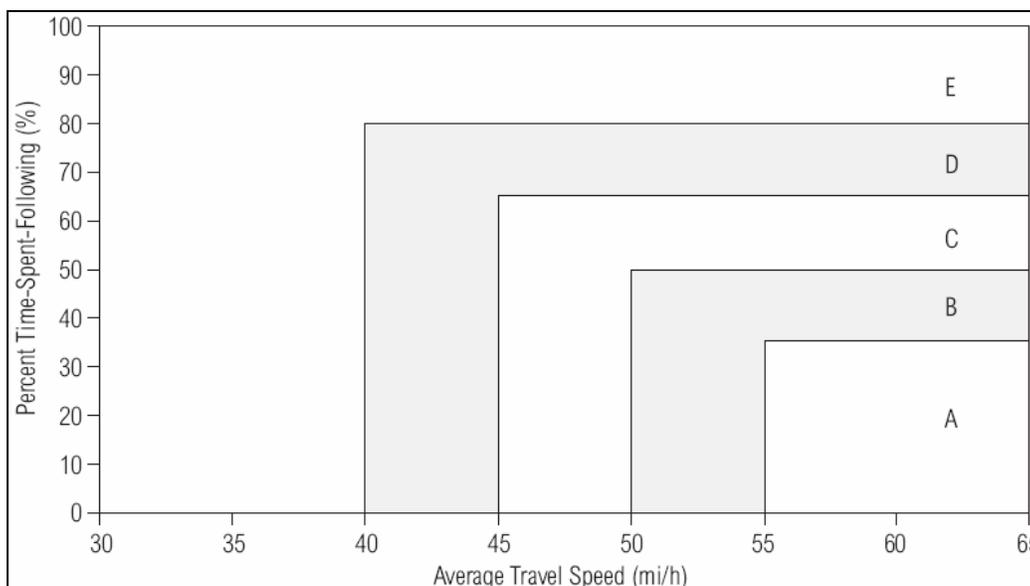
*LOS Categories (after HCM 2000)*

- A. LOS A is the highest quality of traffic service. Drivers are able to travel at their desired speed. On two lane Class I highways, the average speed is 55 mph or higher. The passing demand is well below the passing capacity, and platoons of three or more vehicles are rare. Drivers are delayed no more than 35 percent of their travel time by slow moving vehicles. A maximum flow rate of 490 pc/h total in both directions may be achieved with base conditions. On Class II highways, speeds may fall below 55 mi/h, but motorists will not be delayed in platoons for more than 40 percent of their travel time.
- B. LOS B characterizes traffic flow with speeds of 50 mph or slightly higher on level terrain Class I highways. The demand for passing to maintain desired speeds becomes significant. Drivers are delayed in platoons up to 50 percent of the time. Service flow rates of 780 pc/h total in both directions can be achieved under base conditions. Above this flow rate, the number of platoons increases dramatically. On Class II highways, speeds may fall below 50 mph, but motorists will not be delayed in platoons for more than 55 percent of their travel time.
- C. LOS C occurs when the flow further increases resulting in noticeable increases in platoon formation, platoon size, and frequency of passing impediments. The average speed still exceeds 45 mph on level terrain Class I highways, even though unrestricted passing demand exceeds passing capacity. At higher volumes the chaining of platoons and significant reductions in passing capacity occur. Percent-time-spent-following may reach 65 percent. A service flow rate of up to 1,190 pc/h total in both directions can be accommodated under base conditions. On Class II highways, speeds may fall below 45 mi/h, but motorists will not be delayed in platoons for more than 70 percent of their travel time.
- D. LOS D describes unstable traffic flow. The two opposing traffic streams begin to operate at higher volumes rendering passing extremely difficult. Passing demand is high, but passing capacity approaches zero. Mean platoon sizes of 5 to 10 vehicles are common. Speeds of 40 mph still can be maintained under base conditions on Class I highways. The

proportion of no-passing zones along the roadway section usually has little influence on passing. Motorists are delayed in platoons for nearly 80 percent of their travel time. Maximum service flow rates of 1,830 pc/h total in both directions can be maintained under base conditions. On Class II highways, speeds may fall below 40 mi/h, but in no case will motorists be delayed in platoons for more than 85 percent of their travel time.

- E. At LOS E, traffic flow conditions have a percent time-spent-following greater than 80 percent on Class I highways and greater than 85 percent on Class II. Even under base conditions, speeds may drop below 40 mph. Average travel speeds on highways with less than base conditions will be slower, even down to 25 mph on sustained upgrades. Passing is virtually impossible at LOS E. The highest volume attainable under LOS E is the capacity of the highway, usually 3,200 pc/h total in both directions.
- F. LOS F represents heavily congested flow with traffic demand exceeding capacity. Volumes are lower than capacity and speeds are highly variable.

Figure 7.1 summarizes the level of service criteria for Class I two lane highways in a graphical form. Table 7.1 presents the same criteria for Class II two lane highways. The primary measures of service for Class I two lane highways are the average travel speed and the PTSF. As for Class II two lane highways, the PTSF is the only measure of service. Note that LOS F is not mentioned since the traffic demand at that stage has exceeded the capacity. These concepts will be utilized to assess the service measures of the road under study relative to the operational impacts of varying the PSD lengths. The methodology for calculating the level of service on two lane highways is presented in Figure 7.2. Chapter 20 of the Highway Capacity Manual presents the detailed calculation process involved in obtaining the Level of service and other performance measures on two lane highways. It presents the equations needed for the calculation process, as well as, the exhibits for all the adjustment factors which are required in the equations. The methodology is not used explicitly in this chapter of the dissertation since only the effects of PSD are being assessed.



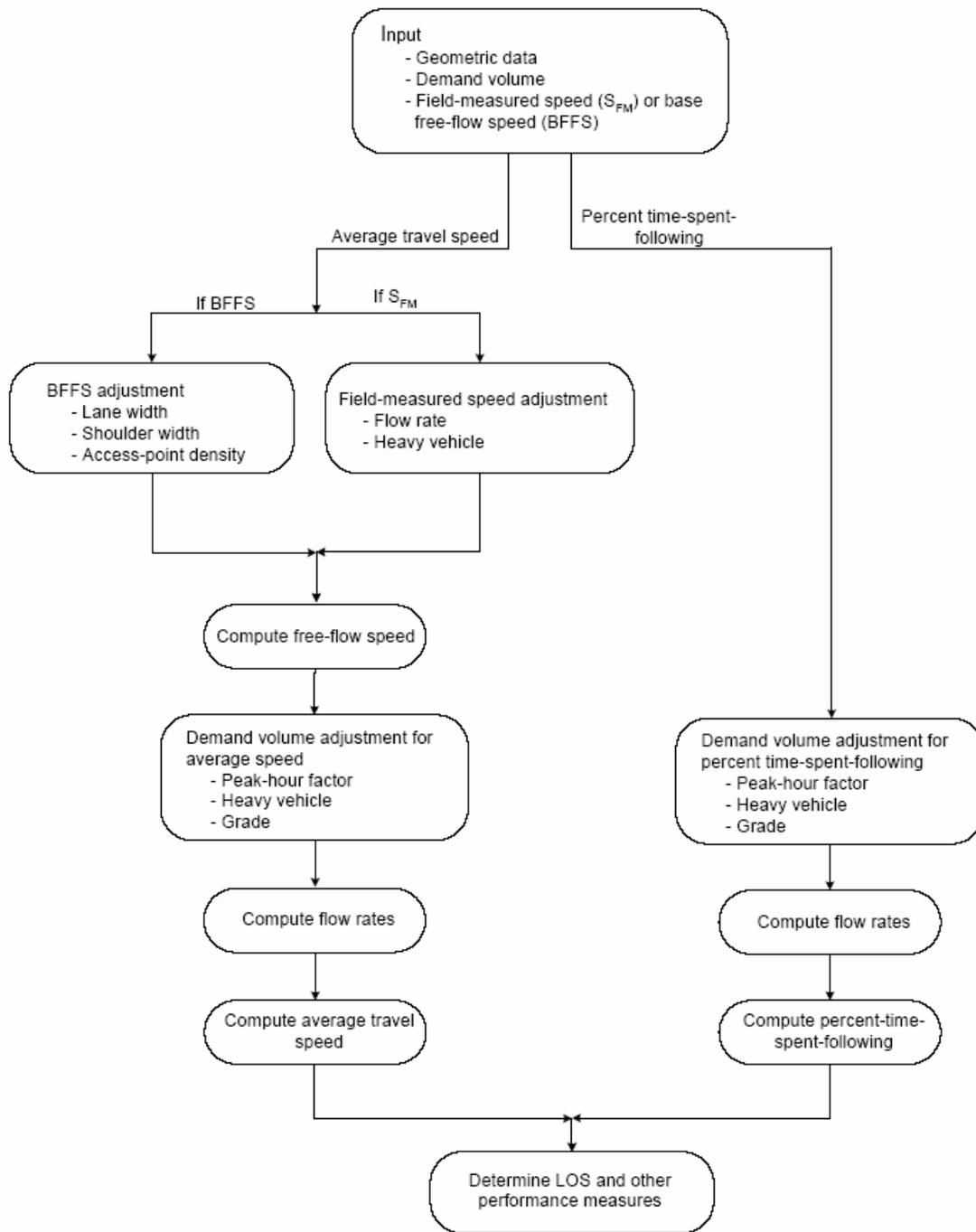
**Figure 7.1 – Level of service criteria for Class I two lane highways (HCM 2000)**

**Table 7.1 – Level of service criteria for Class II two lane highways (HCM 2000)**

LOS	Percent Time-Spent-Following
A	≤ 40
B	> 40–55
C	> 55–70
D	> 70–85
E	> 85

### 7.3 HCM Methodology for Determining Level of Service

Level of service on two lane highways is affected by the ability to pass impeding vehicles. It is captured by the percent-time-spent-following due to slower vehicles driving ahead. Obviously, the capacity of a two-lane highway is inversely proportional to the PTSF. It is directly proportional to the ratio of passing zones to no-passing zones. As an example, higher PSD requirements would increase the safety level but it minimizes the number of passing zones and consequently reduces the level of service of the road. Adequate PSD design is a balanced compromise between the level of service and the safety level. The Interactive Highway Safety Design Model (IHSDM) software package and the Highway Capacity Manual (2000) concepts are utilized to calculate the service measures of the road section relative to the variation in the PSD lengths. The IHSDM was selected for many reasons. First, it is based on the same concepts presented in the Highway Capacity Manual. Second, the IHSDM is distributed free of charge to everyone, it is fairly user friendly, and have been validated for two lane roads in the US.



**Figure 7.2 – Methodology for level of service on two lane highways (HCM 2000)**

### 7.3.1 Overview of the IHSDM

IHSDM is a suite of software analysis tools for evaluating operational effects of geometric design in the highway development process. The scope of the current release of IHSDM is mainly, but not limited to, two lane rural highways (FHWA 2004). IHSDM suite is intended to predict the functionality of proposed or existing designs by applying chosen design guidelines and generalized data to predict performance of the design. The suite of IHSDM tools

includes the following evaluation modules. Each module of IHSDM evaluates an existing or proposed geometric design from a different perspective and estimates measures of the expected safety and operational performance of the design (FHWA 2004).

1. Policy Review Module (PRM): The Policy Review Module checks a design relative to the range of values for critical dimensions recommended in AASHTO design policy.
2. Crash Prediction Module (CPM): The Crash Prediction Module provides estimates of expected crash frequency and severity.
3. Design Consistency Module (DCM): The Design Consistency Module estimates expected operating speeds and measures of operating-speed consistency.
4. Intersection Review Module (IRM): The Intersection Review Module leads users through a systematic review of intersection design elements relative to their likely safety and operational performance.
5. Traffic Analysis Module (TAM): The Traffic Analysis Module estimates measures of traffic operations used in highway capacity and quality of service evaluations.

The measures of operational performance estimated by IHSDM are intended as inputs to the decision making process. They are mainly provided by the Traffic Analysis Module (TAM). Other aforementioned modules, such as the PRM and the DCM, are used for checking the design adequacy of the road relative to the AASHTO's Green Book recommended values. The rest of the modules are not utilized in this research. The importance of the IHSDM lie in providing quantitative estimates of effects that previously could be considered only in more general, qualitative terms. The advantage of these quantitative estimates is that they permit for more informed decision-making. Estimates from IHSDM are expected values, in the statistical sense (FHWA 2004). That is, they represent the estimated average performance over a time period and among a large number of sites with similar characteristics. Actual performance may vary over time and among sites. For this reason, a specific site study is used as an application of the research methodology in assessing the service measures relative to the PSD criteria.

### ***7.3.2 Overview of the Traffic Analysis Module (TAM)***

The Traffic Analysis Module (TAM) may be used to evaluate the operational effects of existing and projected future traffic on a highway section. It can be also utilized to asses the effects of alternative road improvements such as realignment, cross-sectional improvements, and addition of passing lanes, climbing lanes, or changing the PSD lengths (FHWA 2004). Most aspects of the model have been validated against traffic operational field data. This module was exclusively used to develop the capacity and quality of service procedures for two-lane highways

contained in the Transportation Research Board Highway Capacity Manual since 1985 (HCM 2000). Both the TAM and the PRM utilities are used in this research in order to obtain the variation in the percentage of no-passing zones relative to the variation in the PSD requirements. The topography and the length of a two lane road are the main factors that affect the variation in the no-passing zone percentage. For this reason, an actual road section is utilized to demonstrate the research objectives. A general method of application is demonstrated and detailed for use by highway designers.

## **7.4 Input and Road Setup**

### **7.4.1 Input Features**

The PRM, DCM, and TAM modules of the IHSDM are consecutively applied to assess the adequacy of the road which is to be analyzed. In order to achieve realistic results, the program incorporates the following input features and their corresponding parameters (remarks in parenthesis indicate whether the parameter is an input, output, or Not applicable (NA) in the analysis):

1. Highway Geometry
  - a. Grades (input)
  - b. Horizontal curves (input)
  - c. Lane and shoulder width (input)
  - d. Passing sight distance (input – varied)
  - e. Passing and climbing lanes (input)
2. Traffic Control
  - a. Passing and no-passing zones (output)
  - b. Reduced speed zones (NA)
3. Vehicle Characteristics
  - a. Vehicle acceleration and speed capabilities (NA)
  - b. Vehicle lengths (NA)
4. Driver Characteristics and Preferences
  - a. Desired speeds (NA)
  - b. Preferred acceleration levels (NA)
  - c. Limitations on sustained use of maximum power (NA)
  - d. Passing and pass-abort decisions (NA)
  - e. Realistic behavior in passing and climbing lanes (NA)

5. Entering Traffic
  - a. Flow rates (input – varied)
  - b. Vehicle mix (NA)
  - c. Platoon-ing (NA)
  - d. Immediate upstream alignment (input)

#### **7.4.2 Road Setup**

The base conditions for a two lane highway defined in the Highway Capacity Manual summarize situations with no restrictive geometric, traffic, or environmental factors. Base conditions are not the same as typical or default conditions. These conditions include

- Lane widths greater than or equal to 12 ft
- Clear shoulders wider than or equal to 6 ft
- All passenger cars
- No impediments to through traffic, such as traffic control or turning vehicles
- Level terrain
- 50/50 directional traffic split

The methodology in Chapter 20 of the Highway Capacity Manual (2000) accounts for the effects of geometric, traffic, or environmental conditions that are more restrictive than the base conditions. For example, it reads in the Highway Capacity Manual (2000):

“.....Traffic can operate ideally only if lanes and shoulders are wide enough not to constrain speeds. Lane and shoulder widths less than the base values of 12 ft and 6 ft, respectively, are likely to reduce speeds and may increase percent time-spent-following...”

Meaning that, any deviation from the base conditions in any of the parameters affects the service and performance measures of the road section under study. But the main aim of the analysis is to assess the operational impacts of varying only the PSD criteria on a specific road section. All other factors that affect the level of service of the road are not discussed and are out of the scope of the research. Thus, a base configuration of the actual road is set up. Then, the performance and service measures of the road section under study are computed relative to the variation in the PSD criteria. A typical two lane road section from the US road network is analyzed. The author has data for only one road section with a design speed of 50 mph. Since this research task is an application of the HCM concepts, the analysis is carried for only this design speed.

### 7.4.2.1 Overview of the US Route

The total length of the road section considered in the analysis is 3.625 miles (5.834 Km). The road is classified as an arterial with a rolling type of terrain. Extensive traffic data has been collected on that route, including the 85<sup>th</sup> percentile speed, traffic volumes, type of pavement, etc. The posted speed limit on the road is 50 mph. The average speed, standard deviation, and 85<sup>th</sup> percentile speed have been noted as 51.45 mph, 5.95 mph, and 60.85 mph, respectively. In addition, a peak hour volume of 600 vph in both directions has been collected at a representative section of the road, where the directional split was 50/50. The pavement is high-type all through the length of the analyzed section. Figure 7.3 presents the profile view of the road section being studied. The road is made up of thirteen vertical curves, of which seven are sag curves, and the rest are crest curves. It also consists of seventeen horizontal curves (13 simple curves & 4 spiral curves) and twelve tangent sections, as shown in Figure 7.4.

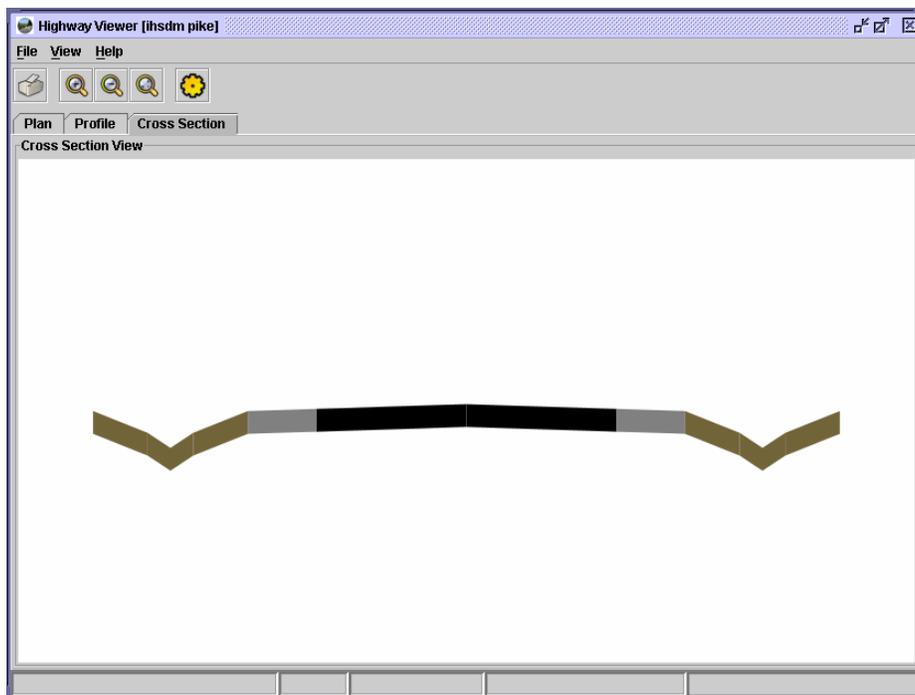


**Figure 7.3 – Profile view of the two lane US route**

The road is made up of two undivided lanes, one for each direction. The width of each traffic lane is 12 ft. In addition, the road has 4 ft paved shoulders in both directions. At the end of the shoulders, there are rounded V shape ditches for drainage on both sides. A typical cross sectional view of the road is shown in Figure 7.5.



**Figure 7.4 – Plan view of the two lane US route**



**Figure 7.5 – Typical cross section of the two lane US road**

#### 7.4.2.2 No-passing Zone Percentage Variation

After entering the highway data into the IHSDM software, few routine checks are conducted to verify the adequacy of the road design. The PRM module is run to check the road design relative to the range of values for critical dimensions recommended in AASHTO design policy. The DCM has been run to check the operating speeds and measures of operating-speed

consistency. The results revealed that no critical design problems exist within the current road stretch. Then, using one of the utilities of the TAM, the road is then marked for passing or no-passing automatically. The model checks the sight distances available on both directions in order to specify where passing/no-passing zones shall be marked. The process is based on the following rules:

- Use the PRM routine for calculating the available PSD requirement (this requirement can be changed by the modeler). Create no-passing zones for sections of the highway where the available PSD is less than the required sight distance.
- If the modeler has chosen to prohibit passing in reduced speed zones, expand the no-passing zones to include these zones in both directions.
- For both passing and climbing lanes, expand the no-passing zones for the direction in which these auxiliary lanes appear.
- For passing lanes, if passing is prohibited in the opposite direction, expand the no-passing zones accordingly.
- For both passing and climbing lane tapers, expand the no-passing zones for both directions.
- Make all short segments of passing zones (less than the minimum length for passing zones – can also be specified by the modeler) into no-passing zones.

Some of these rules were not applied for this road analysis since the road has no passing, climbing, or auxiliary lanes, for example. The PRM module uses the MUTCD marking criteria in delineating no-passing zones. Chapter 2 of this dissertation presents the MUTCD marking criteria, which is based on the 85<sup>th</sup> percentile speed. So, the speed data collected at the road is also inputted into the model. The PRM specified the PSD criteria to be 1000 ft, which is logical since the 85<sup>th</sup> percentile speed is 61 mph. The PSD criteria can be easily changed by the modeler. By changing the aforementioned criteria, the variation in the no-passing zone percentage is computed and tabulated using the TAM utility model.

The results are obtained for every 100 ft increment/reduction in the PSD criteria, in most cases. The selection of the PSD lengths is based on the results obtained in the previous two chapters of the dissertation, mainly from Table 5.8. The selected values increase from a minimum of 450 ft to a maximum of 1250 ft. Two additional representative values were also studied, which are the MUTCD criterion and the limit PSD value at 1.5 risk index (refer to the end of chapter 6). The analysis of the road stretch using the PRM module revealed different no-

passing zone percentages for each direction of travel. Thus, the results are categorized by direction of travel, be it Direction 1 or Direction 2. Table 7.2 presents the results of the PRM no-passing zone calculation for Direction 1 along that road stretch. The values highlighted in orange represent the marking scheme that is actually used on the road. A PSD requirement of 1000 ft is used to mark the road for no-passing/passing zones, as was mentioned earlier. With that criterion, 93 percent of the road has been delineated as no-passing. The values highlighted in grey represent the results of the TAM marking had the PSD been set to 725 ft, which is the PSD value at 1.5 risk index. The tabulated results demonstrate the variation in the no-passing zone percentage corresponding to the variation in the PSD criteria.

**Table 7.2 – Percentage variation in the no-passing zone length in Direction 1**

PSD (ft)	Passing Zones		No-passing Zones	
	Length (ft)	Percentage (%)	Length (ft)	Percentage (%)
450	13448	70	5693	30
550	11558	60	7583	40
650	8435	44	10705	56
<b>725</b>	<b>8435</b>	<b>44</b>	<b>10705</b>	<b>56</b>
850	4542	24	14599	76
950	4542	24	14599	76
<b>1000</b>	<b>1306</b>	<b>7</b>	<b>17835</b>	<b>93</b>
1050	1306	7	17835	93
1150	1306	7	17835	93
1250	227	1	18914	99

**Table 7.3 – Percentage variation in the no-passing zone length in Direction 2**

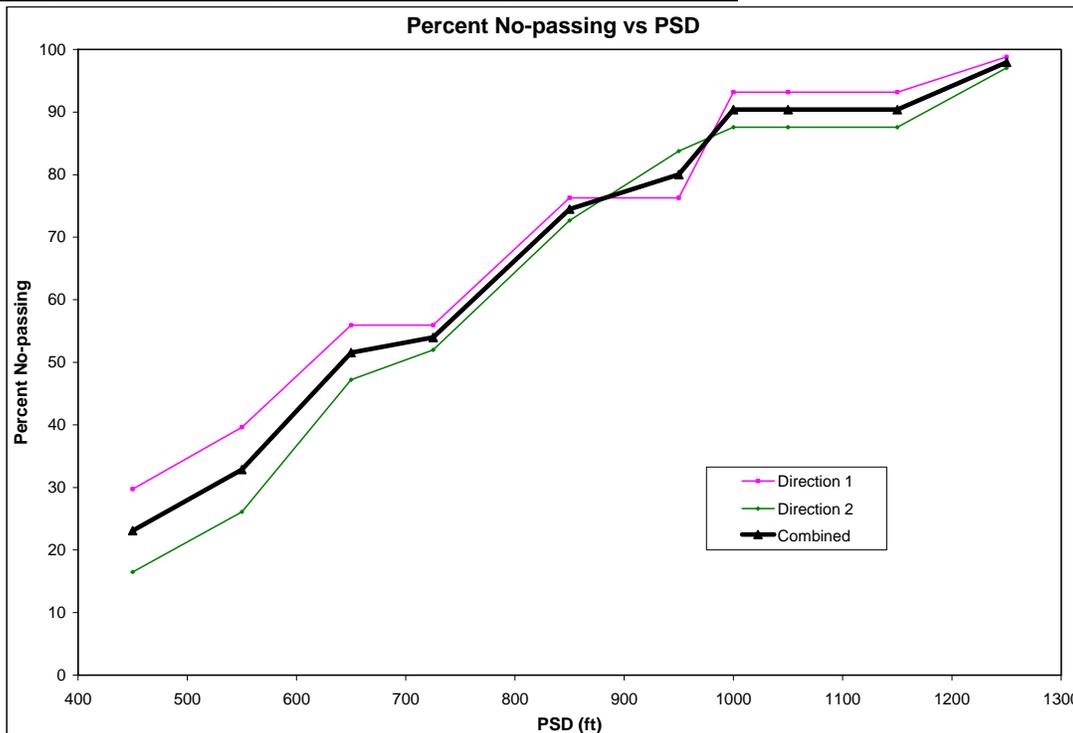
PSD (ft)	Passing Zones		No-passing Zones	
	Length (ft)	Percentage (%)	Length (ft)	Percentage (%)
450	15986	84	3154	16
550	14146	74	4995	26
650	10103	53	9037	47
<b>725</b>	<b>9187</b>	<b>48</b>	<b>9953</b>	<b>52</b>
850	5231	27	13910	73
950	3109	16	16031	84
<b>1000</b>	<b>2375</b>	<b>12</b>	<b>16765</b>	<b>88</b>
1050	2375	12	16765	88
1150	2375	12	16765	88
1250	562	3	18578	97

Similarly, Table 7.3 presents the results of the no-passing/passing zone analysis for Direction 2 along the road. It is clear that the two directions have different no-passing zone percentages. In this table, the results highlighted in orange also represent the base marking case using a 1000 ft PSD criterion. The variation in the no-passing zone percentage seems to be

smoother for this direction of travel. This fact is attributed to the road geometry, mainly the horizontal and vertical alignments. Section 7.3.2.1 of this chapter presented the definition of the no-passing zone percentage provided by the HCM. It specifies that the average percentage of no-passing zones in both directions along a section is used for the analysis of two-way flow (HCM 2000). Thus, the results are averaged as shown in Table 7.4. The results of the three tables are portrayed in a graphical form in Figure 7.6.

**Table 7.4 – Variation in the average percentage of no-passing zones**

PSD (ft)	Direction 1	Direction 2	Combined
	No-passing (%)	No-passing (%)	No-passing (%)
450	30	16	23
550	40	26	33
650	56	47	52
<b>725</b>	<b>56</b>	<b>52</b>	<b>54</b>
850	76	73	74
950	76	84	80
<b>1000</b>	<b>93</b>	<b>88</b>	<b>90</b>
1050	93	88	90
1150	93	88	90
1250	99	97	98



**Figure 7.6 – Variation in the percentage of no-passing zones with various PSDs**

### 7.4.3 Measures of Service Calculation

Now that the percentage of no-passing zones has been computed for the various PSD criteria, the variation in the measures of service of the road can be assessed. The HCM 2000

provides the methods to account for the effects of geometric, traffic, or environmental conditions that are restrictive to the free traffic flow on a specific road section. Some of the restrictive conditions on a two lane highway are heavy vehicles, lane widths, shoulder widths, access points, grades, no-passing zone percentages, directional distribution, and traffic flows. Two measures of performance are derived for each variation in the no-passing zone percentage, and these are the average travel speed and the PTSF.

#### 7.4.3.1 Average Travel Speed

The Highway Capacity Manual (2000) provides adjustment factors to account for the effects of the variation in the no-passing zone percentages on the average travel speed. The factor is called  $f_{np}$  and is presented in Table 7.5. Notice that the reduction in average travel speed increases as the percent of no-passing zones increases. That demonstrates the fact that drivers incur more delays on roads with less passing zones/opportunities. Also, the reduction factor and its variation decrease substantially at higher flow rates. That is, with higher volumes, the effect of no-passing zones on the average travel speed diminishes since there are scarce passing opportunities.

**Table 7.5 – Adjustment factor ( $f_{np}$ ) in average speed (HCM 2000)**

Two-Way Demand Flow Rate, $v_p$ (pc/h)	Reduction in Average Travel Speed (mi/h)					
	No-Passing Zones (%)					
	0	20	40	60	80	100
0	0.0	0.0	0.0	0.0	0.0	0.0
200	0.0	0.6	1.4	2.4	2.6	3.5
400	0.0	1.7	2.7	3.5	3.9	4.5
600	0.0	1.6	2.4	3.0	3.4	3.9
800	0.0	1.4	1.9	2.4	2.7	3.0
1000	0.0	1.1	1.6	2.0	2.2	2.6
1200	0.0	0.8	1.2	1.6	1.9	2.1
1400	0.0	0.6	0.9	1.2	1.4	1.7
1600	0.0	0.6	0.8	1.1	1.3	1.5
1800	0.0	0.5	0.7	1.0	1.1	1.3
2000	0.0	0.5	0.6	0.9	1.0	1.1
2200	0.0	0.5	0.6	0.9	0.9	1.1
2400	0.0	0.5	0.6	0.8	0.9	1.1
2600	0.0	0.5	0.6	0.8	0.9	1.0
2800	0.0	0.5	0.6	0.7	0.8	0.9
3000	0.0	0.5	0.6	0.7	0.7	0.8
3200	0.0	0.5	0.6	0.6	0.6	0.7

The values presented in Table 7.5 are converted into continuous curves for selected traffic flow rates. Using a continuous fit-curve, the reduction in speed could be obtained for any value of no-passing zone percentage. CurveExpert is used to fit the data points. The reduction factor in average speed is plotted against the percent of no-passing zones for a traffic flow rate of 600 vph (the rate observed on the road). The best continuous function to fit the data points is a 3<sup>rd</sup> degree polynomial function with the following equation:

$$f_{np} = a + bx + cx^2 + dx^3 \quad (7-1)$$

Where,  $x$  represents the no-passing zone percentage; { $a$ ,  $b$ ,  $c$ ,  $d$ } are the coefficients of the fit-function. The fitting curve is shown in Figure 7.7. The fitting process is carried out for few other flow rates, and the coefficients and statistics are presented in Table 7.6.

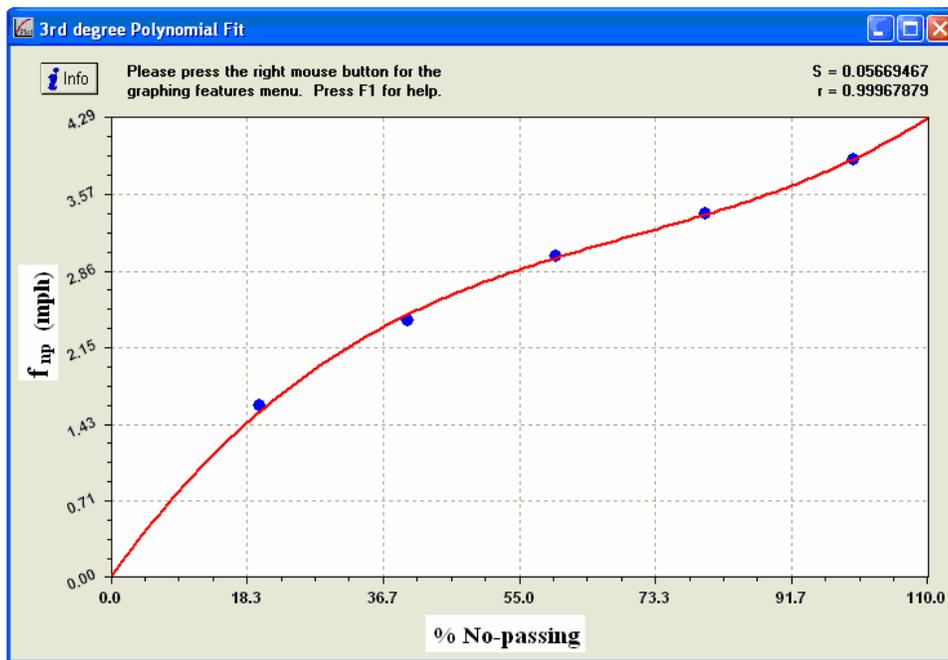


Figure 7.7 – 3<sup>rd</sup> degree polynomial function for  $f_{np}$

Table 7.6 – Coefficients of the 3<sup>rd</sup> degree polynomial function for  $f_{np}$

Flow Rate (vph)	a	b	c	d	R	S
400	0.00238	0.1027	$-1.03 \times 10^{-3}$	$4.51 \times 10^{-6}$	0.9997	0.0681
600	0.01429	0.0967	$-1.09 \times 10^{-3}$	$5.21 \times 10^{-6}$	0.9997	0.0566
800	0.03095	0.0806	$-9.58 \times 10^{-4}$	$4.51 \times 10^{-6}$	0.9978	0.1149
1000	0.00794	0.0683	$-8.52 \times 10^{-4}$	$4.28 \times 10^{-6}$	0.9994	0.0507
2400	0.01508	0.0286	$-3.97 \times 10^{-4}$	$2.20 \times 10^{-6}$	0.9945	0.0633

S=Standard error; R=Correlation coefficient;

The reduction in the average travel speed for each value of the average no-passing zone percentage presented in Table 7.4 is estimated using the 3<sup>rd</sup> degree polynomial function. The coefficients obtained for the 600 vph flow rate are utilized in the analysis. The results are

presented in Table 7.7. The variation in the ATS reduction factor is calculated relative to the base case, for which the PSD is 1000 ft (marked in orange). The 4<sup>th</sup> column in Table 7.7 presents the values of the relative reduction factors  $f_{np}$ , which are obtained by subtracting the base case (where  $f_{np}=3.62$ ) from the other values. As mentioned earlier, the current average speed on the road is 51.45 mph. This is the base case of the analysis. The last column in Table 7.7 presents the estimated values of the average speed relative to the each PSD criteria. The values are obtained by subtracting the reduction factor from the average base speed (51.45 mph). It basically demonstrates what the average speed on the road would be had the sight distance criteria reduced/increased. Based, on these speeds, the delay times are calculated.

**Table 7.7 – Reduction in ATS due to the percentage of no-passing zones**

PSD (ft)	Combined	$f_{np}$	Relative $f_{np}$	ATS
	No-passing (%)	(mph)	(mph)	(mph)
450	23	1.72	-1.90	53.35
550	33	2.20	-1.42	52.87
650	52	2.80	-0.82	52.27
<b>725</b>	<b>54</b>	<b>2.85</b>	<b>-0.77</b>	<b>52.22</b>
850	74	3.26	-0.36	51.81
950	80	3.40	-0.22	51.67
<b>1000</b>	<b>90</b>	<b>3.62</b>	<b>0</b>	<b>51.45</b>
1050	90	3.62	0	51.45
1150	90	3.62	0	51.45
1250	98	3.84	0.22	51.23

#### 7.4.3.2 Percent Time Spent Following

The Highway Capacity Manual (2000) also provides adjustment factors to account for the effects of the variation in the no-passing zone percentages on the PTSF. The factor is called  $f_{d/np}$  and is presented in Table 7.8. It is also quite obvious in the table that the PTSF increases as the percent of no-passing zones increases. Meaning that, more no-passing zones implies less passing zones and less chances to overtake impeding vehicles, thus, more time spent following. Also, the sensitivity in the PTSF decreases substantially at higher flow rates. With higher volumes, the effect of no-passing zones on the following time diminishes since there are scarce passing opportunities.

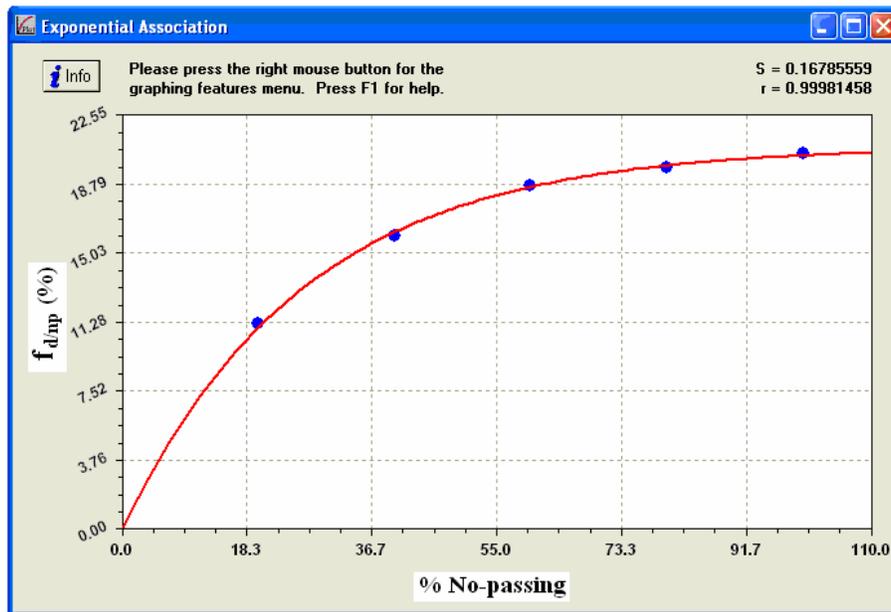
The factors presented in Table 7.8 are also converted into continuous curves for selected traffic flow rates. Using a continuous fit-curve, the increase in the PTSF could be obtained for any value of no-passing zone percentage. Again, CurveExpert is used to fit the data points. The increment factor in the PTSF is plotted against the percent of no-passing zones for a traffic flow rate of 600 vph (the rate observed on the road). The best continuous function to fit the data points is an exponential association function with the following equation:

$$f_{d/np} = a(1 - e^{-bx}) \quad (7-2)$$

Where,  $x$  represents the no-passing zone percentage;  $\{a, b\}$  are the coefficients of the fit-function. The fitting curve is shown in Figure 7.8. The fitting process is carried out for few other flow rates, and the coefficients and statistics are presented in Table 7.9.

**Table 7.8 – Adjustment factor ( $f_{d/np}$ ) in PTSF (HCM 2000)**

Two-Way Flow Rate, $v_p$ (pc/h)	Increase in Percent Time-Spent-Following (%)					
	No-Passing Zones (%)					
	0	20	40	60	80	100
Directional Split = 50/50						
≤ 200	0.0	10.1	17.2	20.2	21.0	21.8
400	0.0	12.4	19.0	22.7	23.8	24.8
600	0.0	11.2	16.0	18.7	19.7	20.5
800	0.0	9.0	12.3	14.1	14.5	15.4
1400	0.0	3.6	5.5	6.7	7.3	7.9
2000	0.0	1.8	2.9	3.7	4.1	4.4
2600	0.0	1.1	1.6	2.0	2.3	2.4
3200	0.0	0.7	0.9	1.1	1.2	1.4



**Figure 7.8 – Exponential function for  $f_{d/np}$**

**Table 7.9 – Coefficients of the Exponential function for  $f_{d/np}$**

Flow Rate (vph)	a	b	R	S
400	25.777	0.033433	0.9998	0.2376
600	20.844	0.037591	0.9998	0.1679
800	15.281	0.042744	0.9992	0.2663
2000	4.9095	0.022745	0.9999	0.0288

S=Standard error; R=Correlation coefficient;

The increase in the base PTSF for each value of the average no-passing zone percentage presented in Table 7.4 is estimated using the exponential function. The coefficients obtained for the 600 vph flow rate are utilized in the analysis. The corresponding results are presented in Table 7.10.

**Table 7.10 – Increase in PTSF due percentage of no-passing**

PSD (ft)	Combined	$f_{d/np}$	PTSF
	No-passing (%)	(%)	(%)
450	23	12.1	53.1
550	33	14.8	55.8
650	52	17.8	58.8
<b>725</b>	<b>54</b>	<b>18.1</b>	<b>59.1</b>
850	74	19.6	60.6
950	80	19.8	60.8
<b>1000</b>	<b>90</b>	<b>20.1</b>	<b>61.1</b>
1050	90	20.1	61.1
1150	90	20.1	61.1
1250	98	20.3	61.3

In addition, the Highway Capacity Manual also provides the equation to compute the base percent time spent following (BPTSF) on a two lane road. The formulation is as follows:

$$BPTSF = 100(1 - e^{-0.000879 \times v_p}) \quad (7-3)$$

Where,  $v_p$  is the flow rate for the peak 15-min period (vph). Thus, for a 600 vph flow rate, the BPTSF is calculated to be 41 percent. Then, the actual PTSF on the road is computed by summing the BPTSF and  $f_{d/np}$ , as shown in the last column of table 7.10.

#### 7.4.3.3 Delay Time

The delay time incurred by drivers in response to the variation in the no-passing zone percentage is calculated based on the reduction factor in the average travel speed. The calculation of the delay time is performed relative to the base case, where the PSD is 1000ft, the ATS is 51.45 mph, and the PTSF is 61.1 percent. Recall that the length of the road section is 3.625 miles (19,140 ft). In addition, the average daily traffic on the road has also been surveyed and the results are presented in Table 7.11. The traffic volume highlighted in orange in Table 7.11 is not actually collected on the road, but linearly extrapolated using the previous traffic data.

**Table 7.11 – Flow rates in vehicles per day on the US road**

Year	Volume (vpd)	Description
1993	3,000	Field survey
1994	3,200	Field survey
1995	3,500	Field survey
2001	5,000	Field survey
<b>2005</b>	<b>6,000</b>	<b>Extrapolated</b>

The CIA world fact sheet declares that the GDP per capita in the USA is approximately \$40,100. Then, based on a 260 working days per year (52 weeks x 5 working days /week = 260 days) and 40 hours of work per 5 days (business week), the average wage per person per hour is approximately \$19.3. Using these parameters, the delay time relative to the various PSD requirements is calculated, as shown in Table 7.12. The delay time per year is then converted into money equivalent in the last column of the aforementioned table. Note that negative values in the table indicate time savings by the users. Notice that the savings in time and money increase with the decrease in the PSD requirements.

**Table 7.12 – Time saved/incurred due to the PSD variation**

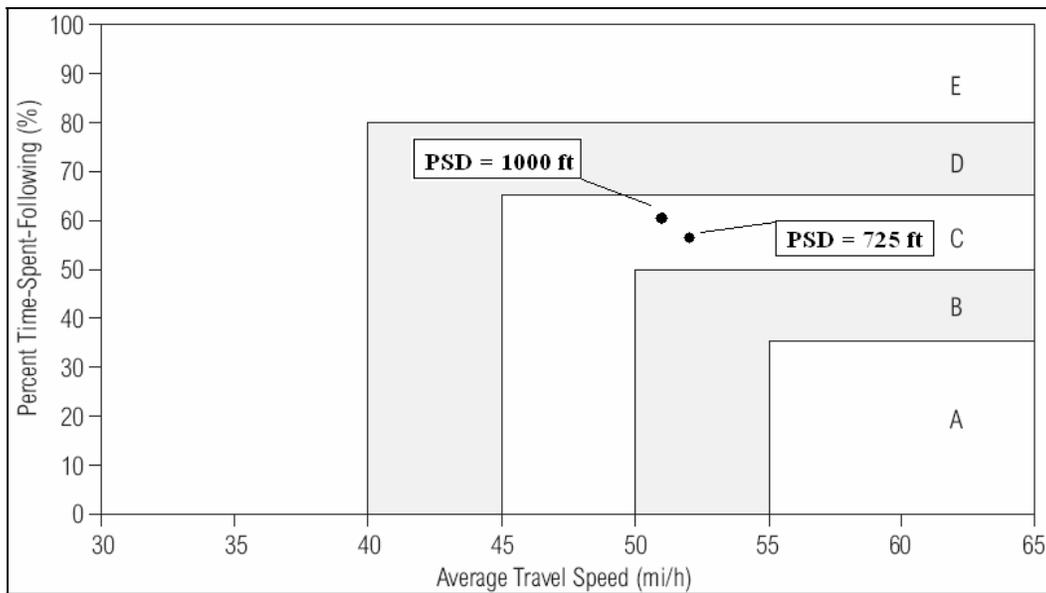
<b>PSD</b>	<b>No-passing</b>	<b>Relative <math>f_{np}</math></b>	<b>Relative <math>f_{np}</math></b>	<b>Delay / Trip</b>	<b>Delay / Day</b>	<b>Delay / Year</b>	<b>Savings / Year</b>
<b>(ft)</b>	<b>(%)</b>	<b>(mph)</b>	<b>(ft/sec)</b>	<b>(sec)</b>	<b>(hrs)</b>	<b>(hrs)</b>	<b>(\$)</b>
<b>450</b>	23	-1.9	-2.79	-9.39	-15.65	-5712	110,234
<b>550</b>	33	-1.42	-2.08	-7.17	-11.95	-4361	84,160
<b>650</b>	52	-0.82	-1.20	-4.33	-7.22	-2637	50,894
<b>725</b>	<b>54</b>	<b>-0.77</b>	<b>-1.13</b>	<b>-4.10</b>	<b>-6.83</b>	<b>-2492</b>	<b>48,088</b>
<b>850</b>	74	-0.36	-0.53	-2.12	-3.53	-1289	24,869
<b>950</b>	80	-0.22	-0.32	-1.44	-2.39	-873	16,856
<b>1000</b>	<b>90</b>	<b>0</b>	<b>0</b>	<b>0</b>	<b>0</b>	<b>0</b>	<b>0</b>
<b>1050</b>	90	0	0	0	0	0	0
<b>1150</b>	90	0	0	0	0	0	0
<b>1250</b>	98	0.22	0.32	0.73	1.22	446	-8613

\*\*Note: negative values mean savings in time.

#### **7.4.4 Discussion and Brief Summary**

This chapter demonstrates the methods used to calculate the performance and service measures of a road section relative to the variation in the PSD criteria. These methods are based on the concepts provided by the Highway Capacity Manual 2000. As mentioned earlier, the PSD criteria are used to mark road sections as passing or no-passing zones. The effects of the PSD criteria on the marking of any road are dependant on that road’s topography and geometry. In other words, it is a site specific situation. Thus, one road section is analyzed to demonstrate the methods of calculating the service measures. Two service measures are derived, which are the average travel speed reduction factor ( $f_{np}$ ) and the percent time spent following increase factor ( $f_{d/np}$ ). The obtained results highlighted in the tables correspond to two key PSD values. The first value, which is 1000 ft, represents the base PSD criterion used to mark the current road. The second value, which is 725 ft, represents the PSD criterion obtained in chapter 6 for a risk index

of 1.5. Based on the service measures of these two PSD criteria, the LOS of the road varies as shown in Figure 7.9. The LOS is shown to be approaching LOS B, and this is only due to the effects of minimizing the PSD criteria.



**Figure 7.9 – Variation in the LOS of the road**

Also, the average speed reduction factor is used to compute the delay time users saved/incurred due to the decrease/increase in the PSD criteria. Then, the time savings are converted into monetary values in US dollars. Finally, the last chapter of the dissertation focuses on combining the research results into one trade-off analysis. The analysis includes the PSD variation, the risk index of the PSD values, and their corresponding service measures.

## **CHAPTER 8:**

# **RESULTS, CONCLUSIONS, AND RECOMMENDATIONS**

## **8.1 Introduction**

The previous seven chapters of the dissertation have elaborated on the concept of passing sight distance (PSD) and quantified its safety and operational effects. Many random parameters affect the required PSD length, such as drivers' speeds, deceleration rate, and headway, to mention a few. Currently, the methods used to compute the aforementioned distance do not account for the variations of all the contributing parameters. In addition, current design procedures fail to accommodate for the concept of safety in the design process. Although highway elements are ideally intended to ensure a minimum level of safety on the road, no risk or safety measures are attached to the designed highway element, as was discussed in the second chapter of the dissertation. The main aim of the research is determine the level of risk in selecting a certain PSD and associate the impact of this distance on the level of service of the road. This chapter focuses on combining the results and conducting the trade-off analysis between the level of service and that of risk. The analysis is performed for a design speed of 50 mph.

## **8.2 Summary and Discussions**

1. Chapter 5 presents a design procedure that accounts for the variations of all the contributing parameters in the PSD formulation. A Monte-Carlo simulation is developed for this purpose. The simulation utilizes the concept of random sampling from various probability distributions to represent a set of different conditions. An analytical model is also devised to verify the results of the Monte-Carlo simulation. Three PSD distributions are calculated for three design speeds. The statistics of the PSD distribution for a 50 mph design speed are presented in section 5.3.3.2.
2. Chapter 6 presents a direct method to assess the risk of various PSD lengths. A unique microscopic simulation is devised to replicate passing maneuvers on two-lane roads. Two scenarios are tested in each run, the pass and abort scenarios. A separate risk index is computed for each scenario. Then, a final risk index is calculated by summing up the risk indices from the two scenarios. The PSD lengths are selected from the PSD distributions obtained in chapter 5 of this dissertation. Using the simulation results, the author has attached risk measures to the values of the PSD distributions. The risk indexes of the PSD values for a design speed of 50 mph are presented in section 6.4.3.2. The author set the limit of the acceptable risk level at 1.5 to ensure that the weighted risk index is well less than 2. The PSD criteria at a risk index of 1.5 are

calculated using the Gamma-fit equations. The estimated limit value at 50 mph design speed is **725 ft**.

3. Chapter 7 demonstrates the methods used (HCM 2000) to calculate the performance and service measures of a road section relative to the variation in the PSD criteria. The effects of the PSD criteria on the marking of any road are dependant on that road's topography and geometry. Thus, one road section is analyzed to demonstrate the methods of calculating the service measures. Two service measures are derived, which are the average travel speed reduction factor ( $f_{np}$ ) and the percent time spent following increase factor ( $f_{d/np}$ ). The obtained results for a design speed of 50 mph are presented in section 7.4.3. The average speed reduction factor is also used to compute the delay time users saved/incurred due to the decrease/increase in the PSD criteria. Then, the time savings are converted into money worth in US dollars, as shown in Table 7.12.

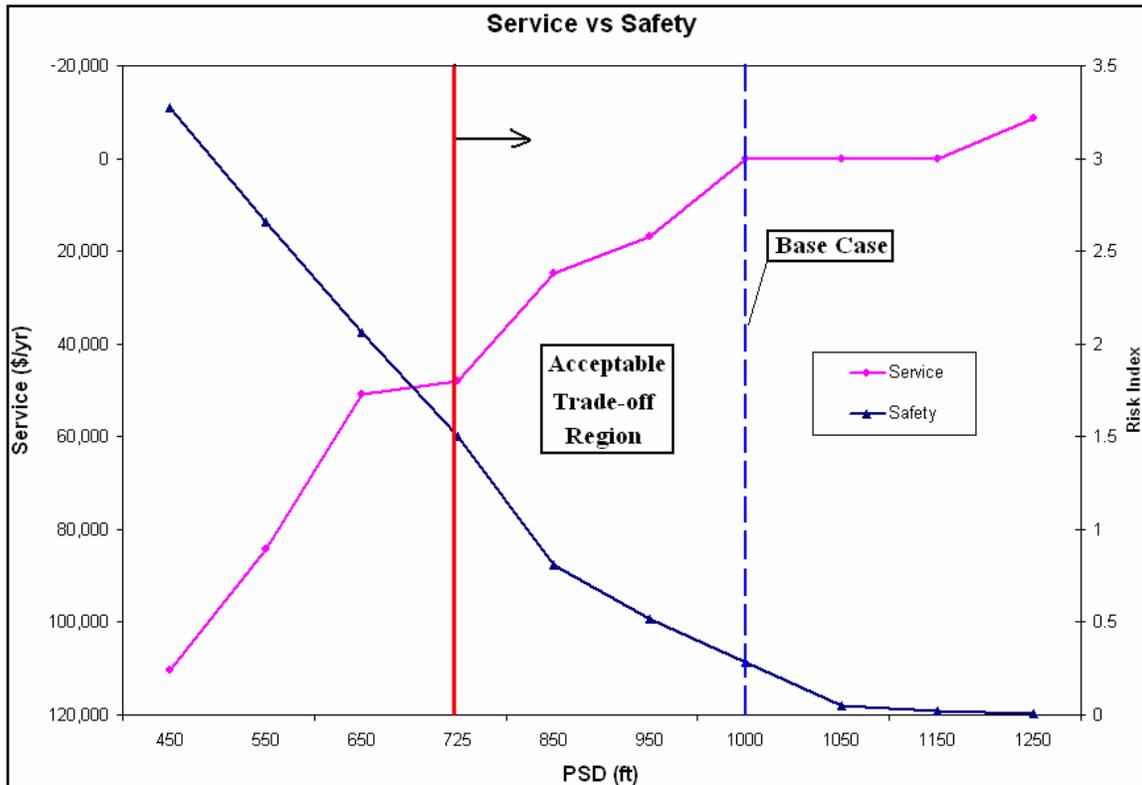
The results of the three accomplished research tasks are combined to conduct a design trade-off analysis relative to the PSD criteria. When obtaining the PSD distribution, actual speed data is used for the 50 mph design speed. The variation in the service measures relative to the variation in the PSD requirements are also derived for a road section with design speed of 50 mph. Thus, the design trade-off is conducted for the latter design speed only. The results are presented in Table 8.1. The results in the table show that an increase in the PSD criteria is associated with a decrease in the risk index (higher safety level and lower crash probability) but a decrease in the service and performance measures (less saving and more delay). Figures 8.1 and 8.2 portray the results in the table in a graphical form. The vertical dotted line highlights the current situation of the road, where a 1000 ft PSD criteria is used to mark the two lane road. Zero service measures are attributed to this case since it is the base case of the analysis. The vertical solid line depicts the situation had the PSD criteria been reduced to 725 ft. This latter value is inversely derived by setting the risk index to 1.5 and then computing the corresponding PSD value (refer to chapter 6). Values between these two lines are acceptable from both, the safety and service perspectives. For such a road, the author would recommend that a PSD of 850 ft be used since the probability of a crash is way too small yet the drivers will save about 1289 hours of delay per year. A better description of the safety level would be to include the actual number of expected crashes with the variation of the PSD lengths. Since the simulation model is not calibrated, the crash probability and the risk index were not representative of actual collisions but of a relative safety measure between the various PSD lengths. Depending on the specific project scope and the designer's

notion of acceptable risk, the value of the PSD requirement can be reduced to achieve a better road performance.

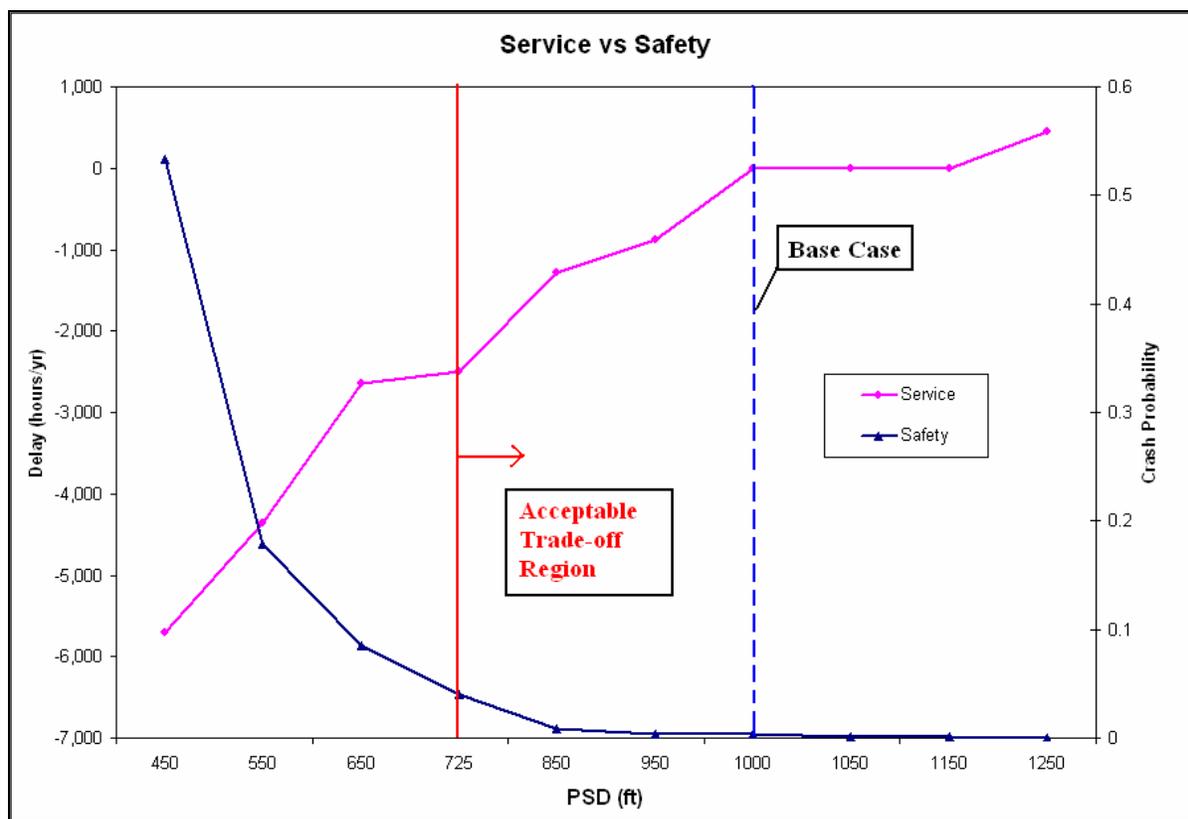
**Table 8.1 – Trade-off analysis between safety and service measures**

PSD Lengths	No-passing Zones	Safety		Service Measures		
		Risk Index	Crash Probability	PTSF	Delay	Savings
(ft)	(%)	(average)		(%)	(hrs/yr)	(\$/yr)
450	23	3.27	0.533	53.1	-5712	110,234
550	33	2.66	0.179	55.8	-4361	84,160
650	52	2.06	0.085	58.8	-2637	50,894
725	54	1.5	0.040	59.1	-2492	48,088
850	74	0.81	0.008	60.6	-1289	24,869
950	80	0.52	0.004	60.8	-873	16,856
1000	90	0.35	0.004	61.1	0	0
1050	90	0.05	0.001	61.1	0	0
1150	90	0.02	0.001	61.1	0	0
1250	98	0.01	0.000	61.3	446	-8613

\*\*Note: negative values mean savings in time.



**Figure 8.1 – Service measures versus safety index**



**Figure 8.2 – Delay time versus crash probability**

For example, close to \$17,000 in user costs are saved per year if the PSD criterion is reduced by 50 ft (to become 950 ft) and yet the crash probability is still the same. This is only for a road section stretching 3.625 miles, on which the average flow rate is 6000 vpd. The savings in time, and consequently money, are proportional to the road length. For common Class I two lane highways, the length of the road is considerably more than the analyzed length. Thus, more savings are accrued on such types of road sections by decreasing the PSD requirements. Reducing further the PSD criterion to 725 ft increases the savings in money by approximately 3 times from the case where the PSD is equal to 950 ft.

### 8.3 Methodology for Practitioners

The dissertation has presented a new approach for the design of passing sight distances. The new methodology accounts for the risk and service measures of using the PSD criteria in the design of two lane highways. The following section captures the adopted methodology by listing the required steps to achieve it:

1. Collect the required input data concerning the road, traffic, and drivers.
2. Input the data into the Monte-Carlo model presented in chapter 5.
3. Derive the PSD distribution and statistics.

4. Select the desired PSD lengths from the obtained distribution.
5. Use the microscopic simulation model, provided in chapter 6, to assess the risk of the desired PSD lengths. Or, use the derived Gaussian functions to estimate the corresponding risk index.
6. Input the road geometry and topography into the IHSDM and obtain the no-passing zone percentages for each desired PSD length.
7. Use the Highway Capacity Manual concepts to compute the service measures relative to the various PSD values. Or, use the polynomial and exponential curves to obtain the two service measures factor,  $f_{np}$  and  $f_{d/np}$ .
8. Conduct a trade-off analysis between the performance and the safety levels on the road. Set minimum desired safety level. Also, set a desired performance level.
9. Select the optimal PSD length that produces the maximum service measures, yet maintains a minimum desired safety level.

## 8.4 Conclusions

The objectives of the research are to conduct a comprehensive assessment of the safety and operational impacts of trade-offs in PSD design elements. Such trade-offs are needed to guide designers in weighing compromises in design elements against safety and operational concerns for two-lane road design. The methodologies and results presented through out the dissertation summarize the accomplishment of the research objectives. Based on the results obtained throughout the research, the author concludes the following:

1. The PSD criteria provided by AASHTO's Green book have been proven too conservative. The Green Book PSD lengths are greater than the maximum values of the derived PSD distributions in chapter 5. Also, when tested for risk in the microscopic simulation, they produced zero risk levels, and extremely high reliability index. Thus, the author recommends that the PSD criteria specified by the Green Book need to be revised in the new release of the standard.
2. The PSD criteria provided by the MUTCD are very close to the mean of the derived PSD distribution, as shown in chapter 5. However, the MUTCD criteria exhibited varying risk levels for different design speeds, as discussed in chapter 6. They are not based on a premeditated level of safety. They are rather a subjective compromise between distances needed for flying and delayed passes. The MUTCD PSD criteria also need to be revised to account for a specific safety level. By using the 85<sup>th</sup> percentile speed in determining

the PSD lengths, the MUTCD are flawed and consequently overestimating the required PSD lengths.

3. The author recommends that the current PSD criteria be reduced to improve two lane road performances while still maintaining acceptable safety levels. The author provokes designers to conduct trade-off analysis in the design of PSDs. The proposed methodology is discussed in section 8.3.
4. The author believes that the aforementioned methodology could be similarly applied to the design of other highway elements, examples of which could be stopping sight distances or crest curves. Trade-off analysis is a well established procedure that contributes to the informed policy development. It is time to be applied to the transportation engineering profession to enhance the design process.

### **8.5 Future Research Recommendations**

In addition, the author came up with many recommendations related to the input data, adopted methodologies, and research results.

#### *Recommendations for input data*

Actual data about some of the input parameters is lacking. These input parameters contribute to the PSD variation. Thus, few assumptions about the characteristics of these parameters were made, to the author's best knowledge, in order to achieve the research objectives. Consequently, most of the recommendations pertain to these assumptions, and are as follows:

1. Collect actual speed data on roads with design speeds varying from 30 to 70 mph. Then, use the real data to input the corresponding speed distribution for each case of design speed.
2. Collect actual data on the clearance distances adopted at the end of a completed/aborted pass maneuver and for various speed ranges. Use curve fitting tools to obtain the real distribution of this parameter.
3. Collect actual data on the gap distances that drivers maintain at the end of a completed/aborted passing maneuver. Use curve fitting tools to obtain the real distribution of this parameter, as well.
4. Obtain detailed data on drivers' deceleration rates at each speed level and fit the appropriate distributions to the data points.
5. Use actual data specific to the road under study when accounting for the vehicle lengths and percentages distributions.

### *Recommendations about the used methodologies*

The author has few recommendations about two steps in the methodology used to obtain the final results. The comments are as follows:

1. A uniquely devised microscopic simulation is used to obtain the risk/safety level corresponding to each PSD length. The model is used to derive risk indexes using a relatively subjective risk scale. An enhancement to this procedure could be to obtain actual field data on the risks involved in passing maneuvers. Then, using the surveyed data, the model could be calibrated accordingly. As a result, the risk index will be an actual indicator of the probability of a collision, for example, or a risky overtaking situation.
2. The Highway Capacity Manual and the IHSDM utilities are utilized to quantify the performance and service measures of a road section relative to the variation in the PSD criteria. The author recommends that a software package, well established in the market, be used to obtain the service measures. Sensitivity analysis could then be easily conducted to vary the speeds, flow rates, PSD criteria, vehicle composition, and all other factors affecting the level of service measures. In fact, that has been the plan in this research. Though it is intended primarily for two lane operations, TWOPASS was actually used in this research but failed to capture the variation in the service measures with the different PSD criteria. Testing a trial setup using another software package could be a future research enhancement.

### *Recommendations for final results*

Depending on future endeavors in this research effort, the author has few extra recommendations related to the presentation of the final results in two areas, which are the risk index and the service measures. The comments are as follows:

1. With a calibrated microscopic simulation model, the risk could be assessed and quantified differently. The results could be in the form of collision probabilities per year, or even twenty years, of facility operation. The risk indexes results could then be actual ratings of the road safety performance with respect to passing maneuvers.
2. By using a well established market package which is capable of capturing the effects of the PSD criteria, the results could then be obtained for numerous scenarios.

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## VITA

**John El Khoury** was born in Lebanon on October 14<sup>th</sup>, 1979. In 2000, he graduated from the University of Balamand with distinction and received the Engineering School Student Excellence Award granted for best academic achievement and good character among the graduates. Two years later, in 2002, John graduated from the Lebanese American University (LAU) with high distinction. At the end of that year, John was granted full scholarship to pursue graduate studies in the Civil Engineering department at Virginia Polytechnic Institute and State University (Virginia Tech). By the end of 2003, he received his Master's degree in Civil Engineering. In 2003, John was elected president of the Virginia Tech Institute of Transportation Engineers (ITE) Student Chapter. He also passed the Virginia state Engineer in Training (EIT) exam. John won the Virginia Section of ITE student scholarship award in 2004. In 2005, he represented the state of Virginia in the Southern District of ITE Student Traffic Bowl. John worked under Professor Antoine Hobeika's supervision for three and a half years, and graduated with a PhD degree in Civil Engineering in December 2005. He helped in preparing proposals and managed an ongoing ITS project. John's other research interests include ITS modeling and applications, road safety analysis, and planning.



### **Professional Affiliations**

- President, Institute of Transportation Engineers, Virginia Tech Chapter (2003-2005)
- Member, American Society of Civil Engineers (ASCE-2002-current)
- Member, Institute of Transportation Engineers (ITE-2002-current)
- Member, Chi Epsilon, National Civil Engineering Honor Society (2003-current)
- Member, Transportation Research Board (TRB-AFB10)

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