EVALUATING THE SAFETY RISK OF NARROW MEDIANS AND RESTRICTED SIGHT DISTANCE

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ABSTRACT

In British Columbia, many highways are located in mountainous terrain and the costs of highway construction in these areas are high. One method of controlling construction costs is narrow medians but the safety consequences of using narrow medians were never determined. Recent research on safety in geometric design has focused on establishing quantitative relationships between collisions and cross sectional elements using collision prediction models (CMPs) and collision modification factors (CMFs). In some situations, such as the use of narrow medians, it is difficult to find CMPs and CMFs that adequately describe the design scenario. In other instances, it is difficult to measure the safety in terms of collision reduction because of a lack of data or difficulty isolating the impact of a single design element on collision frequency. In these situations, reliability analysis can be used to evaluate the risk associated with a particular design feature. In this paper, reliability analysis was completed on a series of horizontal curves with varying horizontal sight distance restrictions and the probability of being unable to stop within the available sight distance was calculated. The results of the study found that narrow medians combined with tight horizontal curves did not provide enough sight distance that vehicles would be expected to stop if an object was in a vehicle's path. The analysis was applied to a case study of two constrained alignments in mountainous terrain of British Columbia.

INTRODUCTION

In British Columbia, the standard median width to accommodate a median barrier is 2.6 m. On horizontal curves, the median barrier can restrict the sight distance and wider medians are usually required to meet the required sight distance. In mountainous terrain or areas with limited rights of ways, construction of wide medians can be prohibitively expensive. Narrow medians are frequently used in British Columbia on horizontal curves where construction is considered too expensive. The narrow median used in British Columbia is 4.0 m wide. No justification is normally required to use the narrow median. A modified narrow median has been developed for corridors that are particularly constrained; it is 0.7 m narrower than the narrow median.

Figure 1 shows each of the three types of medians commonly used in British Columbia. The object shown in the narrow median and modified narrow median details is shorter than the barrier, although the actual height of the object is not specified in the BC Ministry of Transportation (BC MoT) details. The width of the object can be calculated to be 0.3 m wide. It is not known how this object size was determined, although there might be sufficient distance for some vehicles to swerve around the object without hitting the median barrier or driving off the road.

The safety performance of narrow medians on horizontal curves compared with areas with no sight restrictions is unknown. Recent research on safety in geometric design has focused on establishing quantitative relationships between collisions and various cross sectional elements using collision prediction models (CMPs) and collision modification factors (CMFs). There have also been efforts to quantify the safety benefits of improving design consistency. These efforts have been successful in promoting safety based geometric design. However, it is difficult to use these methods in some situations. For example, in complicated design situations it may be difficult to find CMPs and CMFs that adequately describe the design scenario. In other instances, it is difficult to measure the safety in terms of collision reduction because of lack of data or difficulty isolating the impact of a single design element on collision frequency. Both CMPs and CMFs require sufficient data for their development.

In these situations, reliability analysis can be used to evaluate the risk associated with a particular design feature. It has been applied to other areas of civil engineering to help engineers evaluate the risks associated with the uncertainties of their engineering systems. Reliability analysis is not intended as an alternative to quantify safety using collision frequency but represents a complimentary method of measuring safety in terms of risk. Reliability analysis can be used to assess the level of risk associated with the use of narrow medians. The principles used in this analysis follow the limit states design approach used by structural and geotechnical engineers. In this approach, the variables in the design equations are treated as random variables, which are expressed as probability distributions rather than single values. In limit states design, when the demand exceeds the supply, the system is considered to have failed or not complied with the design parameter. There is a chance that supply by the highway design will be exceeded by the demand from the driver/vehicle combination. This event may occur but not only because of human error but because of a particular combination of random events.

BACKGROUND

Sight Distance on Horizontal Curves

On a horizontal curve, sight distance may be restricted if there is an object located on the inside of the curve. Formulae have been developed to calculate the minimum distance an object can be offset from the centerline of the inside travel lane without restricting the required sight distance (1). Figure 2 shows, in plan, how an object restricts horizontal sight distance. The distance M, shown in Figure 2, is calculated using Equation 1 (1). Equation 1 is valid when the alignment contains a simple horizontal curve and when the length of circular curve is greater than the sight distance. Otherwise, Equation 1 is an approximation.

$$M = R\left(1 - \cos\left(\frac{28.65SD}{R}\right)\right) \quad (1)$$

Where:

M = Middle ordinate, in m
R = Radius of the centerline of the inside lane, in m
SD = Required sight distance, in m
L = Length of curve, in m

When the horizontal curve radius is near the minimum permitted for a given highway design speed, the middle ordinate distance can be quite large, sometimes near 10 m or more to accommodate the minimum stopping sight distance found in the design manuals. Construction costs for wide medians can be extremely high, especially in mountainous terrain. It is usually not cost-effective in the mountainous areas of British Columbia to construct a full width median and consequently medians are frequently constructed with restricted horizontal sight distance.

Stopping Sight Distance

In this research, the stopping sight distance was used for the required sight distance. The minimum stopping sight distance can be calculated using Equation 2 (1). This equation is suitable for calculating the braking distance of passenger vehicles but the braking distance for heavy trucks is more complex. Thus, Equation 2 is not applicable for heavy trucks.

$$SSD = VT + \frac{V^2}{2gf_x} \quad (2)$$

Where:

SSD = Stopping sight distance, in m
V = Initial vehicle velocity, in m/s
T = Perception reaction time, in s
g = Gravitational constant, in m/s^2
f_x = Longitudinal braking force also known as the coefficient of friction in the longitudinal direction.

Horizontal Curves

In current geometric design practice, the design of horizontal curves involves the design engineer selecting the horizontal radius or degree of curve based on a minimum radius and maximum superelevation for a given design speed (2). Passenger vehicles may skid or rollover on horizontal curves. It is estimated that the rollover threshold for passenger vehicles is approximately 1.2g whereas the skidding-out threshold is approximately 0.8g (3; 4). In contrast, heavy trucks tend to rollover before they skid out. The rollover threshold for a heavy truck is around 0.3g (5). Similar to the stopping sight distance equation, this analysis is applicable to passenger vehicles only. Design equations found in the highway design guides for horizontal curves assume that passenger vehicles will skid out rather than roll over. Typically, design guides do not explicitly consider heavy trucks, which have rollover thresholds much lower than passenger vehicles. Equation 3 defines the minimum radius for a passenger vehicle for a given speed in most design guidelines (1; 6).

$$R_{\min} = \frac{V^2}{127(e_{\max} + f_{\max})} \tag{3}$$

Where:

V^2 = Design speed in km/h
e_{max} = Maximum allowable superelevation in m/m
f_{max} = Maximum allowable lateral coefficient of friction

When a vehicle is braking on a tangent section of road, it is assumed that the entire frictional force is available to be used for braking. On a horizontal curve, some of the frictional force is used to supply the lateral force required for centrifugal acceleration. Therefore, a reduced amount of frictional force is available for braking.

Reliability Theory

Ang and Cornell developed the use of probabilistic tools in structural design in the 1960's (7). Over time, reliability analysis was developed to the point that it could be included in structural design codes. In the simplest system, there is one variable for supply and one for demand. If it is assumed that the *S* is the supply and *D* is the demand, then Equation 4 denotes the performance function. When the performance function is negative, failure has occurred.

$$g = S - D \tag{4}$$

Reliability theory can be applied to Equation 4 to calculate a probability of failure for a two variable system. The simplest measure of safety is the central factor of safety. It uses the average demand, $\overline{D}$ and the average supply, $\overline{S}$ as noted in Equation 5. This equation is rarely used.

$$SF_{central} = \frac{\overline{S}}{\overline{D}} \tag{5}$$

A more common measure of safety is the conventional factor of safety where the average demand is increased by some multiple of the standard deviation for demand. The average supply is decreased by some multiple of the standard deviation for supply. By using this equation, designers are implying that there is a level of uncertainty in the values for supply and demand. To be conservative the supply is decreased and the demand is increased. In Equation 6, k is some multiple of the standard deviation σ.

$$SF_{conventional} = \frac{\overline{S} - k\sigma_S}{\overline{D} + k\sigma_D} \tag{6}$$

Reliability theory can be used to develop factors of safety that incorporate this uncertainty of the supply and demand variables. The resulting factor of safety is called the reliability index, β. Ang and Tang (8) described a process that can be used to derive the expected value and variance of a design parameter. This process can also be used to derive the measure of safety, *MS* given in Equation 7. Equation 8 shows the reliability index β.

$$MS = E(S) - E(D) \tag{7}$$

$$\beta = \frac{MS}{\sqrt{Var(S) + Var(D)}} \tag{8}$$

Where:

E(S) and E(D) = Expected value for S and D
Var(S) and Var(D) = Variance for S and D

In a two variable system, exact methods have been developed to solve for the probability of failure or non-compliance and beta. However, exact methods to solve reliability equations are not used when there are more than two variables in the performance function (7). These problems are solved using simulations, or approximate methods. The approximate methods include first order methods (FORM) and second order methods (SORM). FORM and SORM problems can be solved using a number of different commercially available, academic software programs (9) or using MATLAB subroutines (10). For this research, Reliability Analysis Software (RELAN) that was developed at the University of British Columbia (11) was used. It can use different techniques to calculate the probability of failure: FORM / SORM, response surface approaches and several simulations, including Monte Carlo simulation and adaptive sampling. In this research, all of the reliability calculations were completed using Monte Carlo simulations.

METHODOLOGY

The probability of being unable to stop or the probability of non-compliance was computed for a variety of available sight distances for a given horizontal radius. The horizontal radii investigated ranged from 200 m to 650 m calculated in 50 m intervals. Each horizontal radius was tested using wet and dry pavement conditions and for a variety of sight distances, (i.e. the median width varied). All of the analyses were completed on flat surfaces, i.e. the longitudinal grade assumed was zero.

Performance Functions

Performance functions were defined to calculate the probability of being unable to stop on a horizontal curve. There were two modes of failure for this problem. The first failure mode occurs when a vehicle is traveling too fast to negotiate the curve. The second mode of failure occurs when the vehicle is traveling slow enough to remain on the pavement but too fast to stop within the available sight distance. The first mode of failure is not related to restricted sight distance but is included in the problem for completeness of the analysis. Each failure mode has its own performance function based on the stopping sight distance, horizontal curve and middle ordinate equations. The performance functions are shown in Equations 9 and 10.

$$GXP = 0.92 f_{\sup ply} - \left(\left(\frac{V_0^{\,2}}{127R} \right) - e \right) \tag{9}$$

Where:

f_{supply} = Random variable representing coefficient of friction supplied by the road
V_0 = Random variable representing initial vehicle speed in km/h
R = Radius in meters, input by the user
e = Superelevation in m/m, input by the user

$$GXP = SSD_{\sup ply} - \left(V_0 T + \frac{V_0^2}{2 g f_{rem}} \right) \tag{10}$$

Where:

SSD_{supply} = Stopping sight distance supplied by the road in meters, input by the user
V_0 = Random variable for initial vehicle speed in m/s
T = Random variable for perception reaction time in seconds
g = Gravitational constant in m/s^2
f_{rem} = Random variable calculated using Equation 11, based on Lamm et. al. (4) from the random variable the coefficient of friction.

$$1.0 \le \left(\frac{f_R}{f_{R\max}} \right)^2 + \left(\frac{f_{rem}}{f_{T\max}} \right)^2 \tag{11}$$

Where:

f_R = Available lateral friction calculated using Equation 9
f_{Tmax} = Random variable representing friction supplied by the road

Where:

$$f_{R\max} = 0.92 f_{T\max} \tag{12}$$

Variables

One of the limitations of reliability analysis is that much of the information published on the variables used in the design equations provides only a single value for each of the variables. Thus, information on how the data are distributed (i.e. the variance and the mean values) is often not found in the published literature. In addition, some variables such speed are highly dependant on local conditions. This section outlines the values given to the distributions of the random variables used in the analysis.

Perception Reaction Time

Perception reaction time can be divided into four elements: perception or the time it takes to see an object; intellection which is the time to understand the implication of the object's presence; emotion which is the time to decide how to react and volition which is the time to initiate the action (12). The perception reaction time used in design calculations in Canada and the United States is 2.5 s, which represents a driver that is slow to react to changing conditions on the road. Roughly, 90% of the driving population has a faster perception reaction time than the value used in design calculations (1). Table 1 summarizes the results of some of the perception reaction studies.

Perception reaction time was assumed to be log-normally distributed for this study. The data in Olsen's studies on perception reaction time were found to be normally distributed (13). However, newer research suggests that perception reaction time may have a lognormal distribution (14). The literature indicates that other probability distributions have been used to describe perception reaction time (15). The average value used in this study for the perception reaction time for passenger vehicles was 1.5 s with a standard deviation of 0.4 s. These values are based on a study conducted by Lerner. The 85th percentile perception reaction time was reported to be 1.9 s (16).

Coefficient of Friction

Vehicle deceleration for a given speed is based on the coefficient of friction between the tires and road. The average coefficient of friction used in highway design represents poor tires on a wet road surface. In Canada, the design value varies with the initial speed of the vehicle and ranges from a high of 0.40 for 30 km/h design speeds to 0.28 for design speeds up to 130 km/h (1). In the United States, however, a deceleration rate is used rather than a coefficient of friction multiplied by the acceleration due to gravity. Before the publication of the 2001 AASHTO Green Book, a coefficient of friction that changed with the design speed was used. Rather than using a coefficient of friction that changed depending on the design speed, a constant deceleration rate of 3.4 m/s^2 is now specified in the 2001 AASHTO Green Book for all design speeds. This results in a coefficient of friction of 0.34, which most braking systems exceed on wet pavement (6).

The wet friction distribution developed using research conducted with newer tires (the PIARC standard European tire) (4). The mean value of the wet friction distribution changed with the mean speed. It was assumed that the wet friction distribution was normally distributed, although this assumption was not statistically tested.

Vehicle deceleration on dry pavements was based on data found in a driver braking performance study (17). A normal distribution was also assumed for the coefficient of friction on dry pavements. This assumption was tested and found to be valid using a Chi-Squared test. Table 2 shows the mean and standard deviation values used for the friction distributions on wet pavement and dry pavement. Lateral friction can be calculated from the longitudinal friction using Equation 12.

Vehicle Speed

Operating speeds on roadways are highly variable, with the desired operating speed changing from one road element, such as a horizontal curve, to the next. Drivers will select their desired operating speed for that element and will change their desired speed from element to element. The 85^{th} percentile speed of the individual drivers is called the operating speed. Vehicle speed was assumed to be normally distributed because most 85^{th} percentile speeds can be estimated by adding the mean speed to the standard deviation of the speed (18). Using a normal distribution, one standard deviation above the mean represents about 85% of the area under the normal distribution curve. Although a normal distribution for the speed was assumed, it is not known whether this assumption has been tested.

Research on speed prediction models has found that the 85th percentile speeds on rural two-lane highways with design speeds less than 100 km/h tended to be greater than the design speed. It also found that when the design speed is greater than 100 km/h the 85th percentile speed tended to be less than the design speed (19). On two-lane rural highways, operating speeds on long tangents tended to range between 95.5 km/h and 102.4 km/h with an overall average of 97.9 km/h. The desired speeds on tangents were found to be representative for two-lane highways with posted speeds that ranged from 80 km/h to 100 km/h (20). FHWA research has found that few drivers slow down on wet pavement when compared to travel speeds on dry pavement (19).

The values for the mean and standard deviation of speed distribution are shown in Table 3. These values were calculated using eleven speed prediction models. The average operating speed was determined by averaging the calculated speed for each radius. The standard deviation for each radius was also calculated. The speed prediction models used in this study are shown in Table 4.

Results

The probability of non-compliance found in this analysis is the probability that a vehicle-driver combination will be unable to stop in the available sight distance. This is not equal to the probability that a collision will occur. A collision would only occur if a vehicle-driver combination was traveling too fast to stop within the available sight distance and there was an object on the road at the same time.

Figure 3 summarizes the probability of non-compliance on a horizontal curve on wet pavement for a variety of curve radii. As shown in the figure, the probability of non-compliance decreases as the radius increases for a given median width. The vertical line with the number one pointing to it indicates the probability of non-compliance associated with the modified narrow median. The vertical lines with the number two pointing to them indicate the probability of non-compliance for a narrow median. The range associated with the narrow median is because the median barrier could be moved from the centre of the median to one side to maximize the available sight distance. The vertical line with the number three pointing to it indicates the probability of non-compliance for a median width providing 140 m of sight distance on a 250 m radius horizontal curve. The vertical line with the number four pointing to it indicates the probability of non-compliance for a median width providing 140 m of sight distance on a 650 m radius horizontal curve.

Case Study

A case study was completed on two proposed alignments in mountainous terrain in British Columbia. The first alignment was severely constrained with many of the horizontal curve radii well below those found in the design manuals for a design speed of 80 km/h. The superelevation was not consistently applied along the alignment, thus curves that had the same radius did not always have the same superelevation. The vertical grades ranged from over 7% to flat. The template used for the majority of this alignment was two travel lanes and 1.5 m wide shoulders.

The second alignment was also considered a constrained alignment, however the horizontal radii found on this design would generally meet the minimum horizontal radius for a design speed of 80 km/h. Unlike the first alignment, the superelevation was applied consistently throughout the alignment. The vertical grades ranged from over 6% to flat. The template for this design was three lanes wide and the shoulder widths varied from 1.0 m to 2.5 m wide. Table 5 shows the geometric characteristics of each alignment.

Modification for Downgrades

The performance functions used in the initial analysis were developed for grade of 0% (i.e. flat grades). The decrease in frictional forces on downgrades increases with grade. Thus, on downgrades the braking distance increases and on upgrades the braking distance decreases. Equation 13 can be used to account for the frictional changes due to grade (1). This equation was used to modify the performance functions used in the case study.

$$d = \frac{V^2}{2g\left(f \pm G\right)} \tag{13}$$

Where:

d = Braking distance, m
V = Initial velocity, m/s
f = Frictional force
g = Gravitational constant, m/s^2
G = Road grade in percent divided by 100 (positive values represent uphill grades while negative values represent downgrades)

Available Sight Distance

The probability of non-compliance was calculated using three different clearance scenarios for each horizontal curve on the alignments. The first scenario assumed that a roadside barrier was located at the edge of the shoulder. The second scenario assumed that a narrow rock ditch, 3.0 m wide, was located at the edge of the road for a tall rock cut.

The final scenario assumed that a wide rock ditch, 5.0 m wide, was located at the edge of a tall rock cut. Both ditch configurations and road shoulder with a barrier are shown in Figure 4. The shoulder width changes along the alignment and the applicable shoulder width was used in the analysis to calculate the available sight distance. For the case study, only wet pavement conditions were investigated as they are used for design purposes.

Case Study Results

The results for each clearance scenario analyzed for Alignment 1 and 2 are illustrated in Figure 5. The blue horizontal line in Figures 5 represents the probability of non-compliance for a vehicle traveling 80 km/h on a 250 m radius curve with a sight distance of about 140 m. The minimum radius curve for an 80 km/h design speed is 250 m and the minimum stopping sight distance is 140 m for a passenger vehicle on a flat grade (1). The probability of non-compliance is the highest for the roadside barrier scenario and the lowest for the large rock ditch scenario. Horizontal curves with the smallest radii tended to have the shortest sight distance and consequently the highest probability of non-compliance for a given sight distance restriction scenario. Conversely, curves that had very large horizontal radii were able to provide sight distances that were close the minimum stopping sight distances required by TAC (1) even in the roadside barrier scenario.

The tight horizontal curves in combination with the narrow 1.5 m wide shoulders restrict the sight distance to very short distances. The available sight distance for a 250 m radius curve, the minimum noted in BC design guidelines using 6% maximum superelevation, is about 84 m in the roadside barrier scenario. This distance is about 60% of the minimum stopping sight distance shown in TAC for an 80 km/h design speed on a tangent. The available sight distance for the horizontal curves with radii, less than 250 m would be less than 60% of the stopping sight distance found in TAC for an 80 km/h design speed. The probability non-compliance within the available sight distance for curves with radii less than 300 m in the roadside barrier scenario ranged from 80% to 95%. Only four of the horizontal curves in the roadside barrier scenario had sight distances that were very close to or exceeded the minimum TAC stopping sight distance.

The probability of non-compliance was lower in the rock ditch scenarios when compared with the roadside barrier scenario. For the horizontal curves with radii 250 m or less in the small rock ditch scenario, the probability of non-compliance was between 45% and 65%. With the roadside barrier, many of the objects found on a road, such as a vehicle, are taller than the barrier. In the case of the rock cut, it is unlikely that drivers will be able to see over the rock cut, thus the provision of adequate distance for stopping is more important. With the use of the large rock ditch, the probability of non-compliance decreased. For horizontal curves with a radius of 250 m or less, the probability of non-compliance within the available sight distance was between 17% and 51%. The probability of non-compliance tends to increases as the radius decreases. The available sight distance did not always increase when the radius increased because the shoulder widths varied. Thus, the sight distance from a smaller radius curve with a wide shoulder could be similar to the sight distance calculated from a larger radius curve with a smaller shoulder.

CONCLUSIONS

The results of the horizontal curve analysis and the case study are summarized below. As noted in the previous section, the probabilities are conditional. If a driver is traveling too fast to stop within the available sight distance, a collision may not occur. A collision would only occur if there were an object on the road and a driver was traveling too fast to stop.

1. The probability of non-compliance decreases for a given horizontal radius as sight distance increases. In the results, increased median width was used as a surrogate for sight distance.
2. Very narrow medians combined with tight or minimum radius horizontal curves produce limited sight distance, which leads to very high probabilities of being unable to stop for objects on the road. It may be good design practice to limit the combination of narrow medians and minimum radius horizontal curves.
3. The probability of non-compliance increases as the distance to the horizontal sight obstruction decreases. For example, the roadside barrier scenario had a greater probability of non-compliance than the large rock ditch scenario for the same horizontal curve.
4. The combination effects of grade and superelevation with the restricted sight distance influences the probability of non-compliance. Downgrades will increase the probability of non-compliance and upgrades will decrease the probability of non-compliance for a given horizontal radius, sight distance and superelevation rate when

compared with flat grades. Higher superelevation rates for a given radius and speed increase the remaining friction that can be used for braking purposes.

5. The combination of minimum radius horizontal curves with the short horizontal clearances to the roadside barrier leads to very high probabilities that a driver cannot stop if an object is on the road and hidden by the barrier.

Based on the findings above, some implications for geometric design can be made. Risk or the probability of non-compliance has never been used as tool in the geometric design process. As a result, there are no target values for the maximum probability of non-compliance. However, the use of reliability theory may assists engineers trying to make difficult design decisions when design guidelines are silent and collision prediction models are not sufficiently detailed to evaluate design alternatives. Currently, there is no link between the probability of non-compliance and the quantification of safety using collision frequency. Future research may be able to provide this link.

As shown by the analysis of restricted sight distance on horizontal curves and the case study, restricted sight distance caused by horizontal sight distance obstructions leads to drivers having a significant probability of non-compliance if there is an object on the road. Designers may wish to consider that sight distance restricted by a roadside barrier may not be as significant as a similar sight distance restriction caused by a tall object that drivers cannot see over. Thus, highway design with a vertical split in the median may require restrictions on the combination of curve radii and distance to the sight distance obstruction that would not be required on a median with a concrete median barrier.

Median and roadside barriers are available in several designs, some of which may provide drivers with a partial view through the barrier, unlike the concrete barriers. Some examples would be the box beam designs or the three-strand cable designs. While these types of median barriers require a minimum median width to accommodate deflections when they are struck by a vehicle, the required median width maybe less than required for a concrete median barrier.

One application for the results is to use figures similar to Figure 3. When a design parameter falls on the steep part of the curve, changing the value of that parameter slightly can change probability of non-compliance considerably. In contrast, on the flat portions of the curve, a change in the value of the design parameter will have very little change on the probability of non-compliance. The use of these curves may be an additional tool for designers to use to make better decisions. For example, if Figure 3 were to be used for design purposes and it was known that short objects were frequently on the road then increasing the median width would decrease the probability that a vehicle would be unable to stop within the available sight distance. The design team may also want to investigate methods to decrease the mean vehicle speed as well as increase the skid resistance of the pavement in wet conditions.

The use of reliability theory to determine the probability of non-compliance can allow the designer some flexibility in the application of standards by allowing designers to quantify a risk. In this example, designers could quantify a risk by determining the values of variables performing the reliability analysis. If desired, a sensitivity analysis could be completed to determine how the probability of non-compliance reacts to changes in the variables. This is important since many of the variables associated with highway geometric design do not have definite probability distributions and agreed upon values.

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TABLE 1 Summary of Perception Reaction Time

Researcher	85th Percentile (s)	95th Percentile (s)
Gazis et. al. (12)	1.48	1.75
Wortman et. al. (12)	1.80	2.35
Chang et. al. (12)	1.90	2.50
Sivak et. al. (12)	1.78	2.40
Lerner (16)	1.9	2.3*

Note: *Calculated using the mean and standard deviation

TABLE 2 Coefficient of Friction

Pavement Condition	Mean Speed	Mean	Standard Deviation
Wet distribution	80.4 km/h	0.4192	0.0913
Wet distribution	85 km/h	0.4013	0.0913
Wet distribution	90 km/h	0.3826	0.0913
Wet distribution	95 km/h	0.3571	0.0913
Wet distribution	99.8 km/h	0.3498	0.0913
Dry distribution	All speeds	0.8852	0.0949

Sources: Wet conditions (4)
Dry conditions (17)

TABLE 3 Vehicle Speed Distributions

Radius	Mean (km/h)	Standard Deviation (km/h)
200	80.38	8.119
250	84.21	6.537
300	86.78	5.623
350	88.61	5.094
400	90.00	4.803
450	91.09	4.659
500	91.96	4.598
550	92.68	4.598
600	93.28	4.630
650	93.80	4.687
700	94.23	4.751
750	94.62	4.825
800	94.95	4.898
900	95.52	5.051
Tangent	100.66	9.443

TABLE 4 Speed Prediction Models Used in the Study

Model	Model Equation	R^2
Lamm et. al., (21)	$V_{85} = 94.398 - \frac{3188.656}{R}$	R^2 = 0.79
Lamm & Choueiri (22)	$V_{85} = 95.6 - 0.0438CCR_S$	R^2 = 0.82
Lamm et. al., (4)	$V_{85} = \exp(4.561 - 0.000527CCR_S)$	R^2 = 0.63
Lamm et. al.– 3.6m Lane Width (4)	$V_{85} = 95.594 - 1.597DC$	R^2 = 0.787
Morrall & Talarico, (23)	$V_{85} = \exp(4.561 - 0.0058D)$ Where $D = \frac{5729.58}{R}$	R^2 = 0.631
TAC, (1)	$V_{85} = 102.45 + 0.0037LC - \frac{(8995 + 5.73LC)}{R}$	Unknown
Ottesen & Krammes 1, (20)	$V_{85} = 103.66 - 1.95DC$	R^2 = 0.80
Ottesen & Krammes 2, (20)	$V_{85} = 102.44 - 1.57DC + 0.012LC - 0.01DC * LC$	R^2 = 0.81
Voigt, 1996 (19)	$V_{85} = 99.61 - \frac{2951.37}{R} + 0.014LC - 0.13I + 71.82e$	R^2 = 0.84
Kanellaidis et al. (24)	$V_{85} = 129.88 - \frac{623.1}{\sqrt{R}}$	R^2 = 0.78
Islam & Seneviratne PC (25)	$V_{85} = 95.41 - 1.48DC - 0.012DC^2$	R^2 = 0.99
Islam & Seneviratne MC (25)	$V_{85} = 103.03 - 2.41DC - 0.029DC^2$	R^2 = 0.98
Islam & Seneviratne PT (25)	$V_{85} = 96.11 - 1.07DC$	R^2 = 0.90

Where:

DC = Degree of curve in degrees $DC = \frac{1746.38}{R}$ (19)

CCR_S = curve change rate in gons

$$CCR_S = \frac{63700 * \left(\frac{L_{CL1}}{2R} + \frac{L_{CR}}{R} + \frac{L_{CL2}}{2R} \right)}{L} \qquad (4)$$

Where:

R = Radius of horizontal curve in m
L_{CL1} = Length of spiral curve in m
L_{CR} = Length of circular curve in m
L_{CL2} = Length of spiral curve out in m
$L = L_{CL1} + L_{CR} + L_{CL2}$ in m
LC = Length of horizontal curve in m
I = Deflection angle of horizontal curve in degrees
e = Superelevation rate in m/m

TABLE 5 Geometric Characteristics for Each Alignment

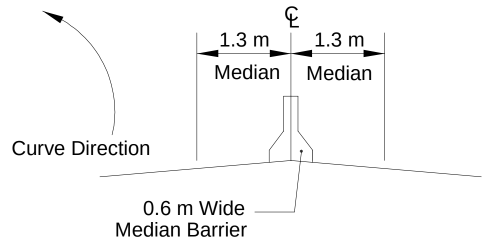
Alignment 1

Curve Segment	Length (m)	ADT (veh/day)	Lane Width (m)	Shoulder Width (m)	Shoulder Type	Curve Length (m)	Curve Radius (m)	Spiral Length In (m)	Spiral Length Out (m)	Grade (%)	Superelevation (%)
C-2	127.2	11,407	3.6	1.5	paved	17.2	143	40	70	-7.15	8.0
C-4	327.6	11,407	3.6	1.5	paved	177.6	144	80	70	-7.15	8.0
C-6	141.6	11,407	3.6	1.5	paved	41.6	165	60	40	-7.87	7.1
C-7	104.7	11,407	3.6	1.5	paved	104.7	750	0	0	-3.00	4.4
C-8	125.8	11,407	3.6	1.5	paved	15.8	130	50	60	0.50	7.8
C-10	164.5	11,407	3.6	1.5	paved	44.5	210	60	60	-0.50	6.4
C-12	180.4	11,407	3.6	1.5	paved	60.4	180	60	60	-0.50	6.9
C-14	174.2	11,407	3.6	1.5	paved	124.2	280	50	0	0.00	3.8
C-16	139.2	11,407	3.6	1.5	paved	139.2	250	0	0	0.10	4.0
C-18	36.0	11,407	3.6	1.5	paved	36.0	250	0	0	-0.50	4.0
C-20	85.5	11,407	3.6	1.5	paved	85.5	250	0	0	-0.50	4.0
C-22	28.9	11,407	3.6	1.5	paved	28.9	250	0	0	0.59	4.0
C-24	80.0	11,407	3.6	1.5	paved	0.005	250	40	40	0.60	4.1
C-26	121.1	11,407	3.6	1.5	paved	1.1	250	60	60	4.50	6.0
C-28	231.5	11,407	3.6	1.5	paved	91.5	290	70	70	7.16	8.0
C-30	63.1	11,407	3.6	1.5	paved	-36.9	800	50	50	7.14	2.0
C-32	186.5	11,407	3.6	1.5	paved	46.5	286	70	70	5.00	6.0
C-34	178.6	11,407	3.6	1.5	paved	68. 6	320	60	50	3.50	6.0
C-35	127.2	11,407	3.6	1.5	paved	127.2	1200	0	0	2.50	2.0
C-36	127.4	11,407	3.6	1.5	paved	127.4	1200	0	0	3.00	2.0
C-37	489.6	11,407	3.6	1.5	paved	369.6	292	50	70	6.00	6.0
C-39	288.6	11,407	3.6	1.5	paved	158.6	290	70	60	4.00	8.0
	4870.1	Total Length						Measured from drawings			

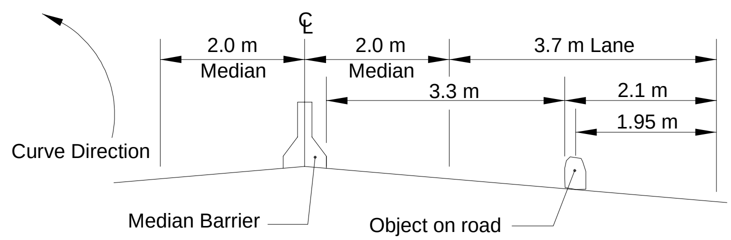
Alignment 2

Curve Segment.	Curve Dir (L or R)	Segment Length (m)	ADT (Veh/d)	Lane Width (m)	Shoulder Width (m)		Shoulder Type	Curve Length (m)	Curve Radius (m)	Spiral Length In (m)	Spiral Length Out (m)	Super Elev. (%)	Avg. Grade (%)
					SB	NB							
C-1	L	442	11,102	3.5	1.5	2.5	paved	342	1400	50	50	-2.5	0.4
C-3	R	377	11,102	3.5	1.5	2.5	paved	257	295	60	60	5.7	-6.401
C-5	L	394	11,102	3.5	1.0	1.0	paved	234	445	90	70	-4.9	-6.401
C-7	R	250	11,102	3.5	1.5	2.5	paved	130	360	60	60	5.3	-5.2
C-9	L	238	11,102	3.5	1.5	2.5	paved	118	350	60	60	-5.35	-1.6
C-11	R	219	11,102	3.5	1.5	2.5	paved	99	280	60	60	5.7	-3.307
C-12	L	215	11,102	3.5	1.5	2.5	paved	95	245	60	60	-5.7	1.6
C-14	L	189	11,102	3.5	1.5	2.5	paved	69	280	60	60	-5.8	-2.4
C-16	R	258	11,102	3.5	1.5	2.5	paved	188	390	70	0	5.2	-1.207
C-18	R	162	11,102	3.5	1.5	2.5	paved	92	840	0	70	3.53	1.2
C-20	L	267	11,102	3.5	1.5	2.5	paved	147	360	60	60	-5.3	0.118
C-22	R	185	11,102	3.5	1.2	1.2	paved	185	1500	0	0	2.4	2.552
C-24	R	263	11,102	3.5	1.2	1.2	paved	123	600	70	70	4.2	-4.8
C-25	L	183	11,102	3.5	1.2	1.2	paved	63	360	60	60	-5.3	0.0
		6870	Total Length					Indicates direction used in analysis to calculate available SD					

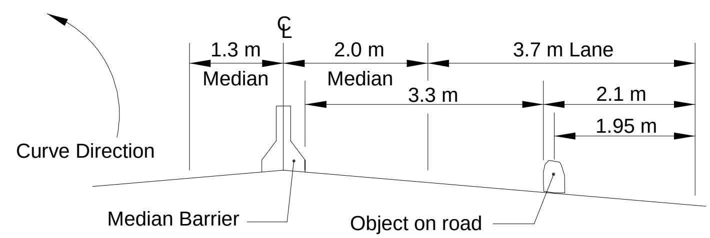
FIGURE 1 Medians Used in British Columbia



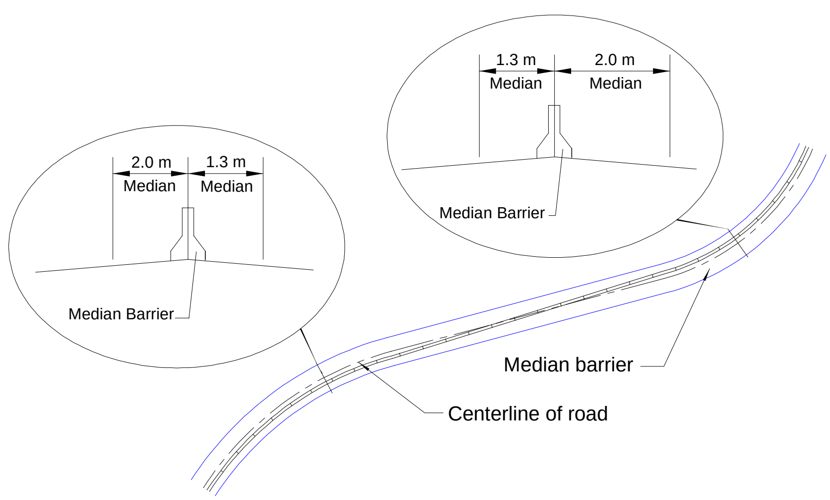
STANDARD 2.6 m WIDE MEDIAN



4.0 m WIDE NARROW MEDIAN



MODIFIED NARROW MEDIAN



BARRIER IN NARROW MEDIAN TO MAXIMIZE SIGHT DISTANCE

FIGURE 2 Horizontal Sight Distance Restriction

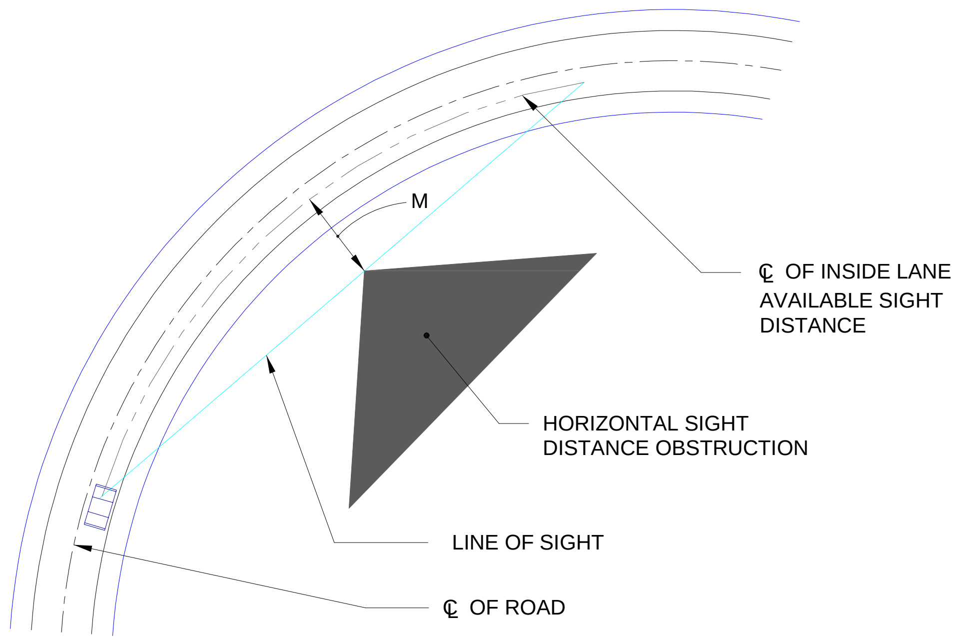
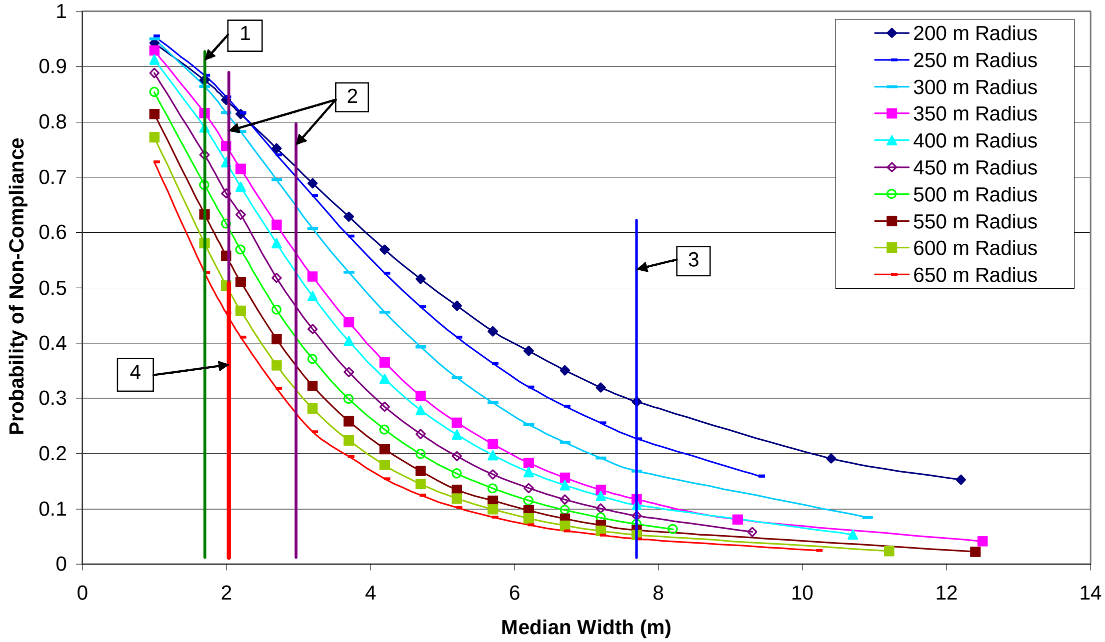


FIGURE 3 Probability of Non-Compliance Horizontal Curve with Restricted Sight Distance and Wet Pavement



Notes:

1. Indicates the probability of non-compliance associated with the modified narrow median
2. Indicates the probability of non-compliance for the narrow median
3. Indicates the probability of non-compliance for a median width providing 140 m of sight distance on a 250 m radius horizontal curve
4. Indicates the probability of non-compliance for a median width providing 140 m of sight distance on a 650 m radius horizontal curve

FIGURE 4 Typical Cross Sectional Elements

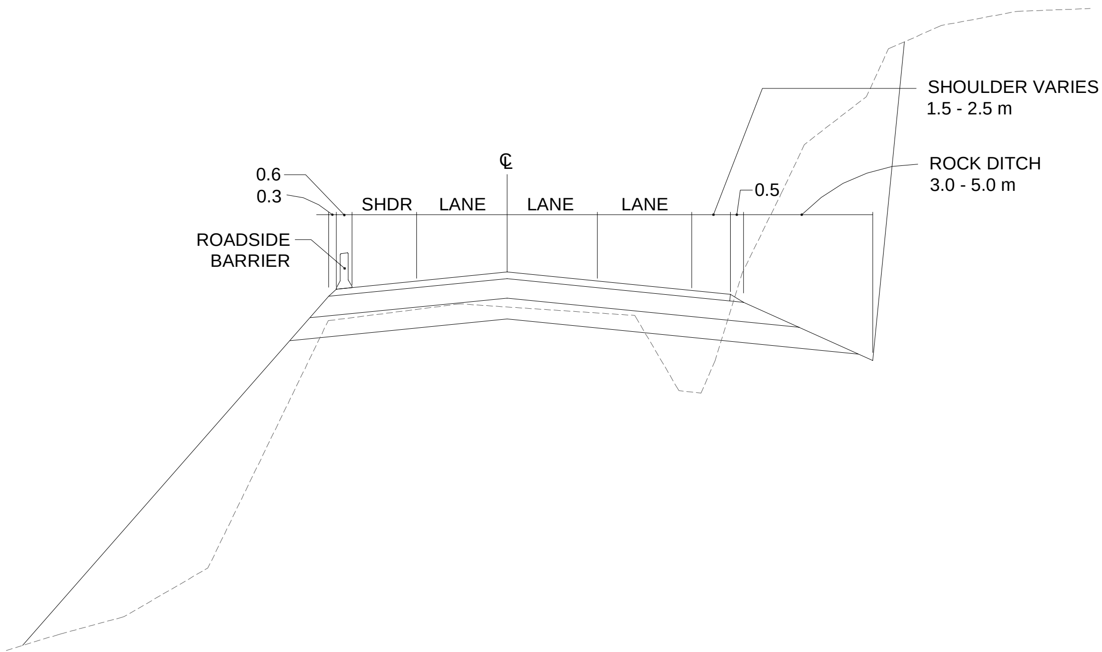
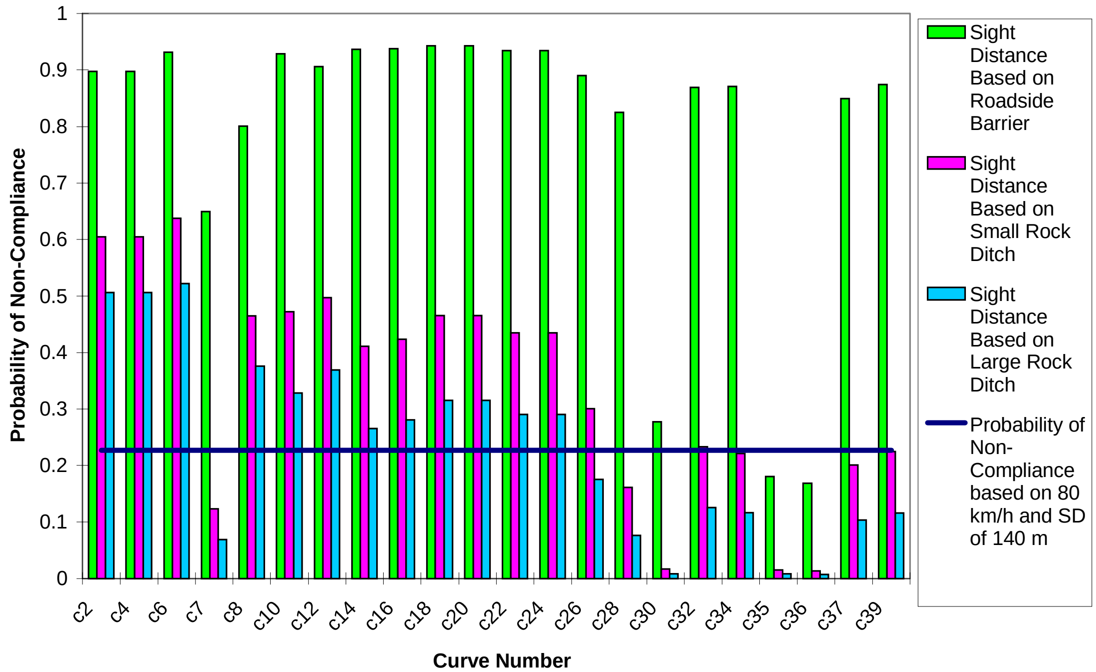


FIGURE 5 Probability of Non-Compliance on Alignments 1 and 2

Alignment 1



Alignment 2