QUANTIFYING THE RELATIONSHIP BETWEEN GEOMETRIC DESIGN CONSISTENCY AND ROAD SAFETY

by

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ABSTRACT

Geometric design consistency is emerging as an important rule in highway design. Identifying and treating any inconsistency on a highway can significantly improve its safety performance. Considerable research has been undertaken to explore this concept, including identifying potential consistency measures and developing models to estimate them. However, little work has been carried out to quantify the safety benefits of geometric design consistency. The objectives of this study are to investigate and quantify the relationship between design consistency and road safety. A comprehensive collision and geometric design database of two-lane rural highways has been used to investigate the effect of several design consistency measures on road safety. Several collision prediction models which incorporate design consistency measures have been developed. The generalized linear regression approach has been used for model development. The models can be used as a quantitative tool to evaluate the impact of design consistency on road safety. An application is presented where the effectiveness of collision prediction models which incorporate design consistency measures is compared with those which rely on geometric design characteristics. It has been found that models which explicitly consider design consistency can identify the inconsistencies more effectively and reflect the resulting impacts on safety more accurately than those which do not. Finally, a systematic approach to identify geometrically inconsistent locations using the safety-consistency factor has been proposed.

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1.0 INTRODUCTION

This chapter gives the necessary background information to understand why a quantitative relationship between design consistency and road safety needs to be investigated. It provides a brief historical background of the development of geometric design to explain how the importance of design consistency gradually emerges. In addition, it describes the sources of design inconsistency in current geometric design practice. Finally, it presents the objectives and the structure of this thesis.

1.1 Development of Geometric Design

Until the turn of the twentieth century, rail and waterways had been the main means of transport while road system was only a supplementary network. As motorized road vehicles became available and more affordable, the demand for roads increased. At that time, the "trafficability" of roads was the main focus of design, the considerations of which included the structural adequacy of pavements, drainage, grades, and widths. After World War II, a rapid growth of motorization and road usage was experienced by the developed countries. A highway system which could support efficient and safe transportation soon became imperative. The focus of highway geometric design shifted from "trafficability" to safety and efficiency. Design standards became necessary to ensure uniformity across jurisdictions. They were available in the 1950s and were developed based on a combination of empirical research, professional experience and judgment. In the 1970s, several factors led to a re-assessment of the role of geometric standards. First, the use of geometric standards gradually moved from road building to upgrading, the design options of which might be limited. Second, due to budgetary constraints, the high cost of upgrading one section of a highway to a rigorous standard might affect other improvement projects. Third, advances in technology and richer experience with geometric design allowed highway design to become more of an engineering procedure with optimization. Thus, design by objectives rather than by standards arose as a new approach to geometric design in the 1980s. The new approach called for an assessment of objectives, one of which was design consistency (*1*). In fact, as early as 1975, Oglesby (*2*) contended that "possibly the most important single rule in highway design is consistency. Only by making every element conform to driver's expectation and by avoiding abrupt changes in standards can a smooth-flowing, collision-free facility be produced." Because of the emerging importance of design consistency in geometric design, research has been conducted to further our understanding on this concept. The following explains what constitutes a consistent design and further discusses its significance to road safety.

1.2 Importance of Geometric Design Consistency

Design consistency is the conformance of a highway's geometry with driver expectancy. A consistent design avoids abrupt changes in operating speed over a short period of time and in geometric feature of adjacent highway elements. Its successive elements act in coordinated way to produce harmonized driver performance. It ensures that the expectancy or ability of the motorist to guide and control a vehicle in a safe manner is not violated (*3*, *4*, *5*).

The importance of design consistency and its significant contribution to road safety can be justified with an understanding of the driver-vehicle-roadway interaction. Roadway geometry, traffic conditions, and roadside environment are the primary inputs to the driving task and determine the workload requirement on the driver. How quickly and how well these inputs are handled depend on driver expectancy and other human factors. Once these inputs are processed, they are translated into vehicle operations. When an inconsistency exists which violates driver's expectation, the driver may adopt an inappropriate speed or inappropriate maneuver, leading to collisions. In fact, Lamm et al. (*6*) have reported that half of all collisions on two-lane rural highways may be indirectly attributed to inadequate speed adaptation, indicating that design consistency is related to safety. Yet, despite the importance of geometric design consistency to road safety, it is not always ensured in current design practice.

1.3 Sources of Geometric Design Inconsistency

There are a number of sources of design inconsistency in current geometric design practice. One of the main sources is inherited in design standards, which is discussed below along with other sources.

1.3.1 Inadequacy of the Design Speed Concept

One of the main sources of design inconsistency is inherent in geometric design standards which are developed based on the design speed concept. The concept has been in use since the 1930s and is still in use today. The rationale of the concept is based on the dynamics of a vehicle when a driver negotiates a circular horizontal curve at a constant speed. The vehicle experiences centrifugal acceleration acting away from the center of the curve, which is counteracted by the centripetal acceleration provided by the side friction between the tires and pavement and by a component of gravity if the road is superelevated. The relationship is expressed as

$$e + f = \frac{V^2}{127.5R} \tag{1}$$

where:

e = superelevation rate,
f = coefficient of side friction,
V = vehicle speed (km/h), and
R = radius of horizontal curve (m).

Before establishing the e, f, and R required to provide sufficient centripetal acceleration, the design speed must be determined first. Design speed is loosely defined as the speed selected to establish appropriate geometric design elements for a particular section of a highway, including but not limited to horizontal and vertical alignment, superelevation, sight distance, lane width, shoulder width, side slope ratio, and clearance from obstacles. The choice of design speed depends on the type of highway, terrain, and maximum superelevation rate permitted in the jurisdiction (*7*).

The problem with the design speed concept is that the design speed, as expressed in the above formula, is not the maximum permissible safe speed. First, there is no quantitative guidance on the choice of design speed, making it difficult to ensure compliance and in turn design consistency. Second, the use of above minimum values for various design elements is encouraged. However, without a pre-defined upper limit on these values, design consistency between sections may not be established. Third, the design speed concept assumes that the motorist would choose an operating speed which is less than or equal to the design speed. This assumption may not be in accordance with reality. Empirical data have shown that drivers adjust their speed according to a number of factors, including their desired speed, the posted speed, traffic volume, and their perceived alignment risks (*7*). In addition, empirical data have shown that operating speed may exceed the design speed when the latter is less than or equal to 100 km/h (*8*). Thus, adherence to the design speed may not be guaranteed. Fourth, the design speed can only be applied to horizontal curves and has no practical meaning to tangents. The design speed is applicable only when physical highway characteristics limit the speed of travel. Consequently, drivers can reach an operating speed on a tangent which is substantially higher than the design speeds of the horizontal curves at its either ends. Driver's expectation may be violated due to geometric design inconsistency at such transitions. Unfortunately, current design standards do not consider proper coordination among individual geometric features along a highway to ensure design consistency. For example, neither the American nor the Canadian design standards (*7*, *9*) have any provision of maximum tangent lengths to control the maximum operating speeds attainable. These are but a few limitations associated with the design speed concept which may generate geometric design inconsistencies (*10*).

1.3.2 Other Sources Related to Practical Application

Progressive changes to the geometric design standards over the last few decades to address increasing traffic volumes, speeds, and safety concerns have led to sections along the same highway to have inconsistent design speeds and cross-sections. This is especially true for two-lane rural highways. Also, other factors such as budgetary constraints, impacts on the environment and on adjacent land use can take precedence over compliance to design guidelines, resulting in inconsistencies. The alteration of existing design features without total redesign is yet another source of inconsistency (*11*, *12*).

With a relatively high number of sources of design inconsistency in current geometric design practice, the impact of inconsistencies of existing alignments on road safety must be investigated. A quantitative relationship which allows for a comprehensive evaluation of the impact of design inconsistency on road safety is essential to improve road safety. Yet research on design consistency is still in the early stage. Several measures of design consistency have been identified and models to estimate these measures have been developed. Design consistency evaluation criteria based on these measures have also been established. However, little work has been undertaken to quantify the safety benefits of geometric design consistency, which is the topic of this research. The objectives are formulated in the following section.

1.4 Thesis Objectives

This research is conducted with the following objectives:

1. To investigate and to quantify the relationship between design consistency and road safety in terms of expected collision frequency.
2. To determine whether models which explicitly consider design consistency are more effective in identifying inconsistencies on an alignment and reflecting the impact on collision frequency than existing models which rely on geometric design characteristics to predict collision frequency.
3. To develop a systematic approach to identify geometric design inconsistencies using collision prediction models.

Because of the overrepresentation in collision and fatality occurrences on horizontal curves of two-lane rural highways in most highway networks, research on design consistency has been focusing on this classification of highway. Indeed, over 82% of the

Canadian highway network is made up of two-lane rural highways, where 227500 injuries and 2917 fatalities are reported in 2000 alone (*13*). Therefore, the scope of this research is also limited to two-lane rural highways.

1.5 Thesis Structure

This thesis consists of seven chapters. Chapter One presents an introduction outlining the historical background of the development of geometric design and how the importance of design consistency gradually emerges. Chapter Two provides an extensive literature review on design consistency and its relationship to safety. Chapter Three describes the data and the methodology used to develop quantitative relationships between design consistency and safety. Chapter Four shows the modeling results along with a detailed discussion. Chapter Five includes three applications of the developed models. Finally Chapter Six brings forward the conclusions, and Chapter Seven gives some recommendations for future research. The references are included in the end of this thesis.

2.0 GEOMETRIC DESIGN CONSISTENCY AND ITS RELATIONSHIP TO ROAD SAFETY

This chapter provides an extensive literature review on geometric design consistency and its relationship to road safety. The potential measures of design consistency, the corresponding prediction models and evaluation criteria, and the latest evaluation software are presented. Also, the current understanding on the relationship between design consistency and safety is described, as well as that between geometric design consistency and highway capacity.

2.1 Potential Measures of Geometric Design Consistency

Most of the research on design consistency has focused on identifying quantitative measures for design consistency evaluation and developing models to estimate them. The measures can be classified into four main categories: operating speed, vehicle stability, alignment indices, and driver workload. Each of these categories is described below along with the corresponding models and evaluation criteria.

2.1.1 Operating Speed

Operating speed is a common and simple measure of design consistency. It is defined as the speed selected by the driver when not restricted by other users, i.e., under free-flow conditions, and is normally represented by the 85th percentile speed and denoted as V_{85} (*14*). It has been found that the speed-curvature relationship can better describe driver behavior than the side friction-speed relationship, as the operating speed of drivers may not always be equal to or less than the design speed as assumed in the design speed concept (*8*). The difference between operating speed and design speed (V_{85}-V_d) is a good indicator of any inconsistency at a single element, while the speed reduction between two successive elements (ΔV_{85}) can identify any inconsistency experienced by drivers when traveling from one element to the next. Subsequent discussion on operating speed is divided into two sub-sections: single elements and successive elements.

2.1.1.1 Single Elements

The following presents some models which estimate operating speed on single elements and some suggested criteria for design consistency evaluation based on V_{85}-V_d.

2.1.1.1.1 Predicting Operating Speed

Lamm et al. (*6*) have argued that although the degree of curve (*DC*) is a successful parameter in estimating operating speeds on horizontal curves, it is limited to the circular curve itself and does not consider the preceding and/or succeeding transition curves. Therefore, they have suggested another measure, the curvature change rate (CCR_s), which takes transition curves into consideration as shown below:

$$CCR_s = \frac{63700\left(\frac{L_{cl1}}{2R} + \frac{L_{cr}}{R} + \frac{L_{cl2}}{2R}\right)}{L} \tag{2}$$

where:

CCR_s = curvature change rate of a single circular curve with transition curves (gon/km) [gon is a designation of the angular unit (1 gon = 0.9°)],

L_{cr} = length of circular curve (m),

L_{cl1}, L_{cl2} = length of spirals preceding and succeeding the circular curve (m),

R = radius of circular curve (m), and

$L = L_{cr} + L_{cl1} + L_{cl2}$ = total length of curve and spirals (m).

CCR_s can account for more of the variability in operating speeds on horizontal curves than the degree of curve. Yet, most models developed to determine operating speed incorporate the degree of curve (or the radius) instead of CCR_s. Lamm et al. (*15*) have expressed a model developed by Morrall and Talarico (*16*) in terms of CCR_s and is shown in Table 1. Other models which are applicable to horizontal curves only are also presented in this table.

Table 1 Operating Speed Prediction Models

#	85th Percentile Speed Prediction Model	R^2	Source
1	$V_{85} = 59.515 - 0.038CCR_s$ *for lane width = 12 ft; V_{85} expressed in mph, and CCR_s in degree per half-mile. Models also available for other lane widths.*	0.84	(*17*)
2	$V_{85} = 59.746 - 0.998DC$ *for lane width = 12 ft; V_{85} expressed in mph, and DC in degree per 100 ft. Models also available for other lane widths.*	0.82	
3	$V_{85} = 94.398 - \dfrac{3188.656}{R}$	0.79	(*18*) as quoted by (*5*)
4	$V_{85} = 129.88 - \dfrac{623.1}{\sqrt{R}}$	0.78	(*19*) as quoted by (*5*)
5	$V_{85} = 95.41 - 1.48DC - 0.012DC^2$ *at point of curvature (PC); DC expressed in degree per 30 m.*	0.99	
6	$V_{85} = 103.03 - 2.41DC - 0.029DC^2$ *at middle of curve (MC); DC expressed in degree per 30 m.*	0.98	(*20*)
7	$V_{85} = 96.11 - 1.07DC$ *at point of tangent (PT); DC expressed in degree per 30 m.*	0.98	
8	$V_{85} = \exp(4.561 - 0.0058DC)$ or $V_{85} = \exp(4.561 - 0.000527CCR_s)$	0.631	(*16*) (*15*)
9	$V_{85} = 103.66 - 1.95DC$ *DC expressed in degree per 30 m.*	0.80	
10	$V_{85} = 102.45 - 1.57DC + 0.012L_{cr} - 0.10DF$ *DC expressed in degree per 30 m.*	0.82	(*21*)
11	$V_{85} = 41.62 - 1.29DC + 0.0049L_{cr} - 0.12DF + 0.95V_{85T}$ *DC expressed in degree per 30 m.*	0.90	

Note: Unless indicated otherwise, V_{85} = 85th percentile speed on horizontal curve (km/h); CCR_s = curvature change rate of a single circular curve with transition curves (gon/km); R = radius of horizontal curve (m); DC = degree of curve (central angle subtended by an arc of 100 m, (degree per 100m)); DF = deflection angle (degree); L_{cr} = length of horizontal curve (m); V_{85T} = 85th percentile speed on approach tangent (km/h).

According to Gibreel et al. (*5*), the maximum tangent operating speed for model 1 to 3 in Table 1 ranges from 94 to 96 km/h, while that for model 4 is 129 km/h. For models 5 to 7, the difference between the 85^{th} percentile operating speed at the point of curvature (PC), middle of curve (MC), and point of tangent (PT) increases gradually with an increase in the degree of curve above 8° (or with a decrease in radius smaller than 218 m). Models 9 to 11 may be statistically deficient due to the use of correlated variables. These models assume that the operating speed is constant along horizontal curves, and acceleration and deceleration occur on tangents only at a rate of 0.85 m/s^2. Model 10 has been validated (*22*), and it is found that they provide reasonable but simplified representation of speed profiles. They do not account for the effect of nearby intersections on operating speeds. The assumed 0.85 m/s^2 value is found to be reasonable for deceleration when approaching horizontal curves but it may overestimate acceleration when departing from curves.

Unlike the models in Table 1 which are applicable to horizontal curves only, some operating speed prediction models have been developed which consider the combination of horizontal curves with vertical grade and/or vertical curves. These models have been developed by Fitzpatrick and Collins (*23*) and are shown in Table 2 below.

Table 2 Operating Speed Prediction Models for Passenger Vehicles on Two-Lane Highways (Fitzpatrick and Collins 2000)

#	Alignment Condition	Model	R^2
1	Horizontal Curve on Grade: -9% ≤ G < -4%	$V_{85} = 102.10 - \frac{3077.13}{R}$	0.58
2	Horizontal Curve on Grade: -4% ≤ G < 0%	$V_{85} = 105.98 - \frac{3709.90}{R}$	0.76
3	Horizontal Curve on Grade: 0% ≤ G < 4%	$V_{85} = 104.82 - \frac{3574.51}{R}$	0.76
4	Horizontal Curve on Grade: 4% ≤ G < 9%	$V_{85} = 96.61 - \frac{2752.19}{R}$	0.53
5	Horizontal Curve Combined with Sag Vertical Curve	$V_{85} = 105.32 - \frac{3438.19}{R}$	0.92
6	Horizontal Curve Combined with Non Limited Sight Distance Crest Vertical Curve	(see below)	N/A
7	Horizontal Curve Combined with Limited Sight Distance Crest Vertical Curve (i.e. K ≤ 43m / %)	$V_{85} = 103.24 - \frac{3576.51}{R}$ (see below)	0.74
8	Sag Vertical Curve on Horizontal Tangent	V_{85} = assumed desired speed	N/A
9	Vertical Crest Curve with Non Limited Sight Distance (i.e., K > 43 m/%) on Horizontal Tangent	V_{85} = assumed desired speed	N/A
10	Vertical Crest Curve with Limited Sight Distance (i.e. K ≤ 43m/%) on Horizontal Tangent	$V_{85} = 105.08 - \frac{149.69}{K}$	0.60

Note: V_{85} = 85th percentile speed of passenger cars at the midpoint of the curve (km/h); R = radius of horizontal curve (m); K = rate of vertical curvature (m); G = grade (%).

The models in Table 2 assume that the operating speed is constant throughout the horizontal or vertical curve. For alignment condition 6, the lowest speed among those predicted by the models for alignment condition 1 or 2 (for the downgrade) and alignment condition 3 or 4 (for the upgrade) should be chosen. Also, for alignment condition 7, in addition to the model listed, the speeds predicted by the models for alignment conditions 1 to 4 should be computed and only the lowest among the five predicted speeds should be chosen. This can ensure that the predicted speed on combined curves will be lower than that on flat horizontal curves.

Although the models in Table 2 have been developed with the consideration of the presence of grade and/or vertical curves, the length of either horizontal or vertical curve is not included. Gibreel et al. (*24*) have developed a set of models which consider the three-dimensional nature of highways and have shown that the accuracy of prediction have improved significantly (R^2 ranges from 0.79 to 0.98). This set of models is based on speed data collected in Ontario and can predict operating speed on horizontal curves in sag or crest combinations. The models predict operating speed at five different points along the combined curve: (1) on the approach tangent, (2) at the beginning of the curve, (3) at the middle of the curve, (4) at the end of the curve, and (5) on the departure tangent, as illustrated in Figure 1. The models are presented in Table 3.

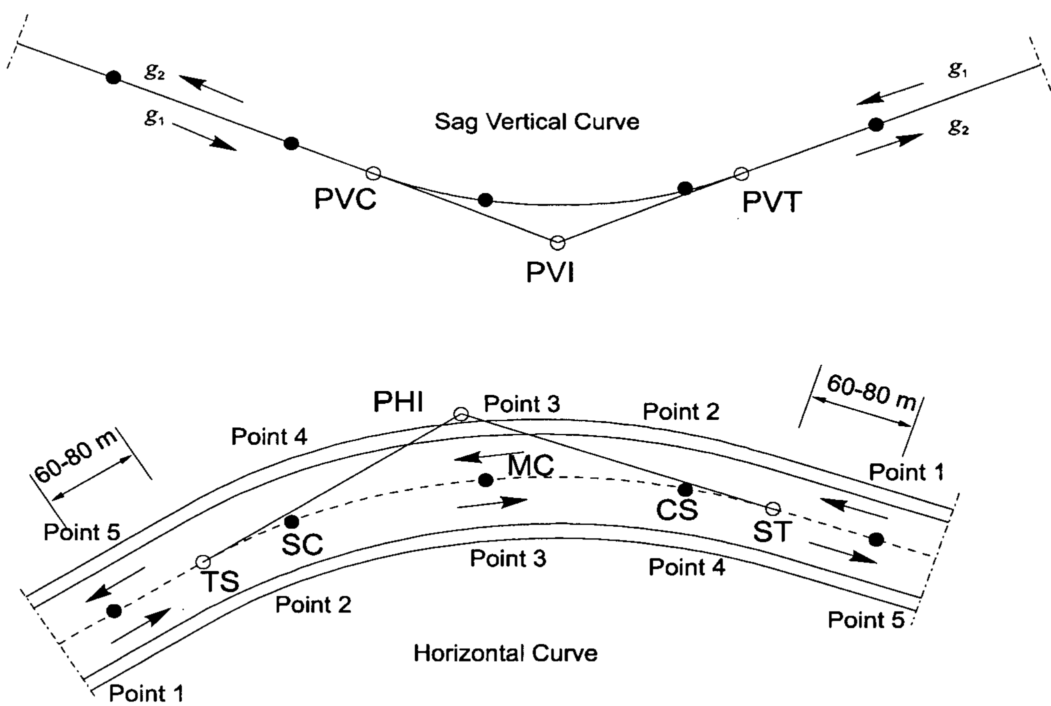


Figure 1 Setup of Speed Measurement Points on Combined Alignments for Three-Dimensional Modeling (Gibreel et al. 2001)

Table 3 Operating Speed Prediction Models on Horizontal Curves in Sag or Crest Combinations (Gibreel et al. 2001)

Sag Combinations

$$V_{S1} = 91.81 + 0.010\,R + 0.468\,\sqrt{L_V} - 0.006\,G_1^3 - 0.878\,\ln(A) - 0.826\,\ln(L_0)$$

$$V_{S2} = 47.96 + 7.217\,\ln(R) + 1.534\,\ln(L_V) - 0.258\,G_1 - 0.653\,A + 0.02\,\exp(e) - 0.008\,L_0$$

$$V_{S3} = 76.42 + 0.023\,R + 0.00023\,K^2 - 0.008\,\exp(A) + 0.062\,\exp(e) - 0.000123\,L_0^2$$

$$V_{S4} = 82.78 + 0.011\,R + 2.067\,\ln(K) - 0.361\,G_2 + 0.036\,\exp(e) - 0.0001091\,L_0^2$$

$$V_{S5} = 109.45 - 1.257\,G_2 - 1.586\,\ln(L_0)$$

Crest Combinations

$$V_{C1} = 82.29 + 0.003\,R - 0.05\,DF + 3.441\ln(L_V) - 0.533\,G_1 + 0.017\exp(e) - 0.000097\,L_0^2$$

$$V_{C2} = 33.69 + 0.002\,R + 10.418\ln(L_V) - 0.544\,G_1 + \frac{8.699}{\ln(1+A)} + 0.032\,\exp(e) - 0.011\,L_0$$

$$V_{C3} = 26.44 + 0.251\sqrt{R} + 10.381\ln(L_V) - 0.423\,G_1 + \frac{6.462}{\ln(1+A)} + 0.051\,\exp(e) - 0.028\,L_0$$

$$V_{C4} = 74.97 + 0.292\,\sqrt{R} + 3.105\,\ln(K) - 0.85\,G_2 + 0.026\,\exp(e) - 0.00017\,L_0^2$$

$$V_{C5} = 105.32 - 0.418\,G_2 - 0.123\,\sqrt{L_0}$$

Note: V_{S1} to V_{S5} and V_{C1} to V_{C5} = 85^{th} percentile operating speeds at point 1 to point 5 on sag and crest combinations respectively (km/h); R = radius of horizontal curve (m); L_V = length of vertical curve (m); e = the superelevation rate (percent), A = algebraic difference in grades (percent); K = rate of vertical curvature (m), G_1 and G_2 = first and second grades in the direction of travel (percent); DF = deflection angle of horizontal curve (degree); L_0 = horizontal distance between point of vertical intersection and point of horizontal intersection (m).

2.1.1.1.2 Geometric Design Consistency Evaluation Criteria Based on Design Speed and Operating Speed

Two sets of design consistency evaluation criterion based on design speed and operating speed have been proposed (*15*, *25*). They are summarized in Table 4 and Table 5 below.

Table 4 Design Consistency Evaluation Criteria Based on Operating Speed (Leisch and Leisch 1977)

Design Speed Consistency
The difference in the design speed of successive elements should be no greater than 15 km/h.
Operating Speed Consistency
1. The speed variations of passenger cars should be no greater than 15 km/h within a given design speed along a highway. 2. The difference between average truck speeds and average passenger cars speeds should be no greater than 15 km/h on common lanes.

Table 5 Design Consistency Evaluation Criteria Based on Operating Speed (Lamm et al. 1999)

Criterion
Good design: $V_{85} - V_d \leq 10$ km/h (consistency)
Fair design: 10 km/h $< V_{85} - V_d \leq 20$ km/h (minor inconsistency; traffic warning devices required)
Poor design: $V_{85} - V_d > 20$ km/h (strong inconsistency; redesign recommended)
Note: $V_{85} = 85^{th}$ percentile operating speed (km/h); V_d = design speed of the roadway (km/h).

Islam and Seneviratne (*20*) have found that the free-flowing operating speed at the point of curvature (PC) and at the point of tangent (PT) differ significantly from that at the middle of the curve (MC). Therefore, they have recommended that design consistency should be evaluated based on the difference between the operating speed at the point of tangent (PT) and the design speed of the horizontal curve.

2.1.1.2 Successive Elements

Some models have been developed to predict the speed reduction of successive elements, and some design consistency evaluation criteria based on speed reduction have also been established. The following discussion presents these findings.

2.1.1.2.1 Predicting Speed Reduction

Speed reduction is usually expressed as the difference in the 85th percentile operating speeds between approach tangent and curve. Speed reduction may be the most visible and effective indicator of inconsistencies since drivers usually reduce their operating speed when the design of a roadway violates driver expectancy. Models which have been developed to predict speed reduction (*4*, *26*) are shown in Table 6 below in chronological order.

Table 6 Speed Reduction Models (Al-Masaeid et al. 1995 and Abdelwahab et al. 1998)

#	Model	R^2	Variables
Simple Curves (Circular Curves Preceded by a Straight Section with a Length of at least 800m). Transition curves may or may not be present.			
1	$\Delta V_{85} = 3.30 + 1.58DC$	0.62	ΔV_{85} = speed reduction between tangent and curve for all vehicles (km/h); DC = degree of curve (degree per 30 m);
2	$\Delta V_{85} = 1.84 + 1.39DC + 4.09PC + 0.07G^2$	0.77	PC = pavement condition (for $PSR \geq 3$, $PC = 0$, otherwise $PC = 1$), where PSR = Present Serviceability Rating;
3	$\Delta V_{85} = 1.45 + 1.55DC + 4.00PC + 0.00004{L_{VC}}^2$	0.76	G = gradient (average slope between the points of speed measurements on the tangent and the curve center, (%));
4	$\Delta V_{85} = 0.9433DC + 0.0847DF$	0.92	L_{VC} = length of vertical curve within the horizontal curve (m); DF = deflection angle (degree).
Continuous Curves Consisting of Two Successive Horizontal Curves Separated by a Short Tangent with a Maximum Length of 300 m			
5	$\Delta V_{85} = \frac{5081}{R_2} - \frac{5081}{R_1}$	0.81	R_1, R_2 = radius of preceding and succeeding curves respectively (m).

All the models in Table 6 have been developed based on data collected in Jordan by Al-Masaeid et al. (*4*) except model 4. ΔV_{85} is the speed reduction between the approach tangent and the curve (km/hr), expressed as the difference between the 85^{th} percentile speed on tangent and that on curve. The Present Serviceability Rating (*PSR*) for each highway section was determined by a panel of raters. Models 1 to 3 are recommended for horizontal curves on a flat gradient, a specific gradient, and vertical curves, respectively. Model 4 has been developed by Abdelwahab et al. (*26*), who have argued that despite the statistical correlation that may exist between degree of curvature and deflection angle, the inclusion of these two basic variables in a speed reduction model is expected to improve its performance.

Another approach has been proposed to examine speed reduction for design consistency evaluation by McFadden and Elefteriadou (*27*). They have suggested the use of the 85^{th} percentile speed reduction experienced by a driver population on an approach tangent-horizontal curve combination. They have investigated the 85^{th} percentile maximum speed reduction (*85MSR*), which have been determined based on the maximum speed reduction experienced by each driver when traveling from an approach tangent to a horizontal curve. The maximum speed reductions from a group of drivers are sorted, and the 85^{th} percentile value is termed *85MSR*. It is found that on average, *85MSR* is approximately two times larger than the commonly used 85^{th} percentile speed reduction, which is computed as the difference between the 85^{th} percentile speed on tangent and that on horizontal curve. Two models have been developed using the least squares linear regression method to predict the *85MSR* as a function of some geometric design features, as shown in Table 7. The models are based on data collected at 21 sites in Pennsylvania and Texas. The first model includes the approach tangent speed as an independent variable while the second model does not, which can be used when the approach tangent speed is not available. They have concluded that these models can complement existing operating speed models.

Table 7 Speed Reduction Models Based on 85MSR (McFadden and Elefteriadou 2000)

#	Model	R^2
1	$\Delta V_{85} = -14.9 + (0.144 \times V_{85_{PC200}}) + (0.0153 \times L_T) + (\frac{954.55}{R})$	0.712
2	$\Delta V_{85} = -0.812 + (0.017 \times L_T) + (\frac{998.19}{R})$	0.603

Note: ΔV_{85} *= estimated 85th percentile speed reduction into curve (km/h);* $V_{85_{PC200}}$ *= 85th percentile speed at 200 m prior to the point of curvature (km/h);* L_T *= length of approach tangent (m);* R *= radius of horizontal curve (m).*

2.1.1.2.2 Geometric Design Consistency Evaluation Criteria Based on Speed Reduction

Several design consistency evaluation criteria based on the speed reduction on successive elements of highway have been proposed. The criteria are summarized in Table 8 in chronological order.

Table 8 Design Consistency Evaluation Criteria Based on Speed Reduction

Criterion	Source
A consistent and safe design is one where the difference between the operating speed on two successive elements must be less than 15% of the speed on the preceding element.	(*28*) as quoted by (*5*)
Good design: speed reduction from tangent to the following curve does not exceed 10 km/h.	(*19*)
Good design: $\Delta V_{85} \leq 10$ km/h (consistency) Fair design: 10 km/h < $\Delta V_{85} \leq 20$ km/h (minor inconsistency; traffic warning devices required) Poor design: $\Delta V_{85} > 20$ km/h (strong inconsistency; redesign recommended) *Note: This criterion can check the effect which the driver experiences when traveling from an approach tangent to a horizontal curve, but not the effect when traveling from a sharp curve following a flat curve.*	(*15*)
The maximum difference in the operating speed of successive curves is 10 km/h, and this limit is lowered to 5 km/h if the curve is isolated or first in a series.	(*29*)
10 km/h is the most appropriate value as the threshold speed reduction. Neither 5 km/h nor 15 km/h are suitable because the former is too stringent while the latter is too liberal.	(*26*)
A good design is one where the degree of curves is less than 4.24° on flat grades. If the horizontal curve is combined with a vertical gradient, then the value of the maximum degree of curve will depend on the gradient. Similarly, if the horizontal curve is combined with a vertical curve, the maximum degree of curve will depend on the length of the vertical curve.	(*5*)

Note: ΔV_{85} = speed reduction (km/h).

2.1.1.3 Operating Speed on Tangents

So far, the discussion on operating speed has focused on horizontal curves. Nonetheless tangents which connect horizontal curves are also important for design consistency evaluation. The length of these connecting tangents is one of the factors which determine the necessary speed reduction when entering a horizontal curve. An independent tangent is defined as one that is long enough which allows drivers to reach their desired operating speed. Consequently, a speed reduction of greater than 20 km/h (corresponding to a poor design) is required when they enter the following curve (*30*). In contrast, a non-independent tangent is one that is not long enough, therefore the necessary speed reduction is less than or equal to 20 km/h. The necessary speed reduction when entering a horizontal curve is also found to be affected by the radii of the preceding and succeeding curves. A good consistent design is one where the two curves have identical radii (*4*). Based on this finding, the maximum tangent length above which the ratio of curve radii is no longer a sufficient criterion for safety in design can be determined using the following equation:

$$L_T = \frac{(V_{85_1}{}^2 - V_{85_2}{}^2)}{22.03} \qquad (3)$$

where:

L_T = length of tangent (m), and

V_{85_1} and V_{85_2} = 85th percentile speeds on preceding and succeeding curves respectively (km/h).

A constant acceleration and deceleration rate of 0.85 m/s^2 was assumed in the equation above. Several models have been developed to predict the operating speed on non-independent tangent based on data collected in Jordan, as shown in Table 9 below. It is found that the operating speed is affected by the length of the common tangent, the degree of successive horizontal curves, and the deflection angles of the two curves (*4*).

Table 9 Operating Speed Prediction Models for Non-Independent Tangents (Al-Masaeid 1995)

#	Model	R^2	Variables
1	$V_{85A} = 108.3 - \frac{3498}{L_T} - 0.71\left[\frac{(DF_1 \times DF_2)}{(DF_1 + DF_2)}\right]$	0.72	V_{85A} = 85th percentile operating speed of all vehicles (km/h); L_T = length of common tangent (m); DF_1 and DF_2 = deflection angles of first and second curve, respectively (degree); DC_1 and DC_2 = degree of successive curves for the preceding and succeeding curves respectively (degree per 30 m).
2	$V_{85A} = 105.47 - \frac{3792}{L_T} - 0.27(DC_1 \times DC_2)$	0.63	

The operating speed on independent tangents is more complex and depend on a whole array of roadway character, making it difficult to develop reasonably accurate prediction models. It is significantly influenced by the preceding and succeeding horizontal curves. Nonetheless, some models have been developed recently by Polus et al. (*31*) for predicting operating speeds on short and long tangents. The objective of their study was to analyze the variability of operating speeds on tangents of two-lane rural highways where volume is low enough and does not affect speed. Tangents found between horizontal curves have been classified into one of four groups, and the corresponding models are summarized in Table 10.

Table 10 Operating Speed Prediction Models on Tangents (Polus et al. 2000)

Description	**Model**	**R^2**
Group 1 Small radii (≤ 250 m) and small tangent length (< 150 m)	$V_{85} = 101.11 - \frac{3420}{GM_S}$	0.553
Group 2a Small radii (≤ 250 m) and intermediate tangent length (150 m to 1000 m). GM_L must be less than 1500.	$V_{85} = 98.405 - \frac{3184}{GM_L}$	0.684
Group 2b Small radii (≤ 250 m) and intermediate tangent length (150 m to 1000 m) and if the maximum 85^{th} percentile speed is established as 105 km/h. GM_L must be less than 1500.	$V_{85} = 105.00 - \frac{28.107}{\exp(0.00108 GM_L)}$	0.742
Group 3 Intermediate radii (> 250 m) and intermediate tangent length (between 150 m to 1000 m)	No successful models identified due to large variability in data	
Group 4 Large tangent length (> 1000 m) and any reasonable radius (i.e. no less than the minimum radius based on the design speed)	$V_{85} = 105.00 - \frac{22.953}{\exp(0.00012 GM_L)}$	0.838

V_{85} = 85^{th} percentile operating speed on tangent (km/h)

GM_L and GM_S are geometric measures of the tangent and the attached curves, and are formulated as

$$GM_L = \frac{[L_T \times \sqrt{R_1 \times R_2}]}{100}, \text{ for } L_T \geq t, \tag{4}$$

and

$$GM_s = \frac{(R_1 + R_2)}{2}, \text{ for } L_T < t, \tag{5}$$

where:

R_1, R_2 = radii of preceding and succeeding curves respectively (m),

L_T = length of tangent (m), and

t = selected threshold for length of tangent (m).

It is concluded that the models for Group1 and Group 2 provide good fit to the data and can be used during the planning process for new two-lane highways, while that for Group 4 is considered preliminary and additional data are needed to improve their usefulness.

2.1.1.4 Effects of Adverse Weather Conditions on Operating Speeds and Speed Reductions

Al-Masaeid et al. (*32*) have found that adverse weather conditions affect operating speed on tangents and horizontal curves of two-lane highways. Posted speed and rainfall intensity have significant impact on the operating speed on tangents, while the degree of curve, rainfall intensity, and night-time conditions have significant impact on horizontal curves. Night-time conditions can cause a drop in the operating speed of passenger cars at curve entries by about 4 km/h relative to day-time. This drop can be explained by the more limited sight distance on horizontal curves and on the vehicle's headlight condition at night.

Adverse weather conditions are also found to affect speed reductions between the approach tangent and the horizontal curves of two-lane highways. The degree of curve, rainfall intensity, and night-time conditions have significant impact on speed reduction between the approach tangent and the curve. In addition, greater reduction is observed in summer than in winter. A speed reduction of 0.57 km/h per degree of curve and 0.4 km/h per mm/h of rainfall intensity are found. Compared to day-time conditions, night-time conditions increase the speed reduction of passenger cars by nearly 6 km/h. Table 11 summarizes the models which predict operating speed or speed reduction as a function of adverse weather conditions, while Table 12 presents the established evaluation criterion which takes into consideration adverse weather conditions.

Table 11 Operating Speed and Speed Reduction Prediction Models Accounting for Adverse Weather Conditions (Al-Masaeid et al. 1999)

#	Model	R^2	Variables
Operating Speed Prediction Models			V_{85T} = operating speed of all vehicles on the tangent (km/h);
1	$V_{85T} = 48.55 + 0.31SL - 1.19PR + 11.18P - 9.36H$	0.82	V_{85C} = operating speed of all vehicles on the horizontal curve (km/h);
2	$V_{85C} = 56.46 + 0.18SL - 1.5PR + 0.59DC - 2.26T + 10.41P - 8.60H$	0.84	SL = posted speed limit (km/h); PR = rainfall intensity (mm/h);
3	$V_{85C} = 55.01 + 0.21SL - 1.48PR - 0.33DC - 0.075DF - 2.28T + 10.20P - 8.60H$	0.85	P = dummy variable to account for vehicle type (1 for passenger cars, and 0 otherwise); H = dummy variable to account for vehicle type (1 for trucks, and 0 otherwise);
4	$V_{85C} = 57.26 + 0.20SL - 1.48PR - 0.63DC - 0.013L_{cr} - 2.25T + 10.25P - 8.60H$	0.85	T = dummy variable to account for time condition (1 for night-time and 0 for day-time);
Speed Reduction Prediction Model			DC = degree of curve (unclear whether it is in degree per 30 m or degree per 100 m);
5	$\Delta V_{85} = 0.42DC + 0.30PR + 2.59P + 3.65T$	0.85	DF = deflection angle (degree); L_{cr} = length of horizontal curve (m); ΔV_{85} = speed reduction between tangent and middle of the curve (km/h).

It should be noted that the addition of the deflection angle or the length of curve to models 3 and 4 only improves the prediction accuracy by 1%. A design consistency evaluation criterion has been proposed as presented in Table 12.

Table 12 Design Consistency Evaluation Criterion Based on Degree of Curve and Rainfall Intensity (Al-Masaeid et al. 1999)

Criterion
Good design: degree of curve ≤ 4° and rainfall intensity ≤ 4 mm/h
Fair design: degree of curve ≤ 10° and rainfall intensity ≤ 18 mm/h

2.1.2 Vehicle Stability

Vehicle stability is an important measure of design consistency. When a horizontal curve lacks vehicle stability, meaning that its friction assumed is insufficient, vehicles may slide out or be involved in head-on collisions. Unfortunately, vehicle stability is not always present because it is inaccurately represented in geometric design. Current geometric standards established since the 1930s are primarily based on the road-vehicle interaction described previously, the mathematical representation of which is associated with a number of criticisms. First, the vehicle is represented by a point mass, which ignores the interaction between side and longitudinal friction as well as the distribution of friction on the vehicle's tires. Second, the assumption that vehicles will travel at a constant speed when negotiating a curve is invalid (*5*). The choice of speed is found to be a compromise of the driver's desired speed and his acceptable level of lateral acceleration. Third, the assumption that drivers will follow a path with a radius identical to the curve radius is also shown by empirical data to be invalid. Faster drivers accept side friction demanded which is in excess of the comfort limit (*33*). Thus, vehicle stability may not be guaranteed even if the design is made according to design standards.

When a roadway lacks vehicle stability, it violates drivers' expectation and their ability to guide and control the vehicle in a safe manner, thus can be considered as a geometric design inconsistency. As such, assessing vehicle stability can help identify inconsistent locations. The difference between side friction assumed and side friction demanded, which is denoted as Δf_R, is used to represent vehicle stability.

2.1.2.1 Predicting Vehicle Stability

Several models have been developed to predict friction assumed and side friction demanded separately. These models are presented below in chronological order.

Table 13 Vehicle Stability Prediction Models

#	Model	R^2	Source	Variables
	Side Friction Assumed			
1	$f_{RA} = 0.082 + 4.692\times10^{-3} V_{85} - 7\times10^{-5} V_{85}{}^2$	0.74	(*34*) as quoted by (*5*)	f_{RA} = side friction assumed; V_{85} = 85[th] percentile operating speed (km/h); V_d = design speed of the roadway (km/h); R = radius of horizontal curve (m); e = superelevation rate (percent); f_{RD} = side friction demanded; V_a = 85[th] percentile approach speed (km/h); V_c = 85[th] percentile curve speed (km/h); I_{TR} = indicator variable (=1.0 for turning roadways; 0.0 otherwise).
2	$f_{RA} = 0.25 - 2.04\times10^{-3} V_d - 0.63\times10^{-5} V_d{}^2$ *(rural) for flat topography and implementing e_{max}; e_{max} = 0.08 assumed.*	N/A		
3	$f_{RA} = 0.22 - 1.79\times10^{-3} V_d + 0.56\times10^{-5} V_d{}^2$ *(rural) for hilly and mountainous topography and implementing e_{max}; e_{max} = 0.07 assumed.*	N/A	(*35*)	
4	$f_{RA} = 0.05 - 0.45\times10^{-3} V_d + 0.14\times10^{-5} V_d{}^2$ *(rural) for all topography and implementing e_{min}; e = 0.025 assumed.*	N/A		
	Side Friction Demanded			
5	$f_{RD} = 0.253 + 2.330\times10^{-3} V_{85} - 9\times10^{-5} V_{85}{}^2$	0.56	(*34*)	
6	$f_{RD} = \dfrac{V_{85}{}^2}{127R} - e$		(*9*)	
7	$f_D = 0.256 - 0.0022V_a + B\times(V_a - V_c)$ where $V_c = 63.5R\times\left(-B+\sqrt{B^2+\dfrac{4c}{127R}}\right)\le V_a$ with $c = \dfrac{e}{100} + 0.256 + (B-0.0022)\times V_a$ and $B = 0.0133 - 0.00741 I_{TR}$	0.88	(*36*)	

Models 2 to 4 which have been developed by Lamm et al. (*35*) consider the combined effect of side friction demanded and curve geometry on the operating speed on horizontal curves. These models offer a human behavior based explanation for driver's choice of speed on curves, and are indirectly developed based on an overall regression relationship between the tangential friction factor and the design speed. Some assumptions were made with respect to the utilization ratio (the percentage of side friction factor utilized out of the maximum permissible side friction factor) and the maximum superelevation rate in different topography.

2.1.2.2 Geometric Design Consistency Evaluation Criteria Based on Vehicle Stability

Some design consistency evaluation criteria based on vehicle stability have also been developed. They are summarized in Table 14 below.

Table 14 Design Consistency Evaluation Criteria Based on Vehicle Stability

Criterion	Source
Good design: $\Delta f_R \geq +0.01$ Fair design: $+0.01 > \Delta f_R \geq -0.04$ Poor design: $\Delta f_R < -0.04$ For highways evaluated as good design, no improvement is required. For those evaluated as fair design, the superelevation must be related to the operating speed to ensure that the side friction assumed will accommodate the side friction demanded. For highways evaluated as bad design, redesign is recommended.	*(15)*
A margin of safety between the safe and the design speed has been suggested. The safe speed is defined as the speed at which the side friction demanded is equal to the maximum value of side friction, and the design speed is defined as the 85th percentile speed according to the Australian design guide. It should be noted that having a margin of safety is not enough to achieve design consistency. Nonetheless, the margin should be consistent by keeping its standard deviation small.	*(37)*
The difference between the operating speed and the safe (limiting) speed, which depends on the sight distance, vehicle stability, and driver comfort, can be used for design consistency evaluation. This approach is also capable of 3D modeling.	*(38)*

$\Delta f_R = f_{RA} - f_{RD}$ = difference between side friction assumed and demand; f_{RA} = side friction assumed; f_{RD} = side friction demanded.

2.1.3 Alignment Indices

Alignment indices are quantitative measures of the general character of an alignment. They reveal the geometric inconsistencies where the characteristics of the alignment change significantly. While speed reduction and vehicle stability are good measures of design consistency, they are symptoms rather than causes. It is the geometric design itself, specifically the geometric characteristics and the combinations of tangents and horizontal curves, that create inconsistencies. Some of the indicators of geometric inconsistency include a large increase or decrease in the value of an alignment index for successive sections, a high rate of change in an alignment index over some length of the highway, and a large difference between the value of an alignment index of an individual feature and the average value of the alignment (*3*).

2.1.3.1 Proposed Alignment Indices

A number of potential alignment indices have been studied, some of which are not recommended by researchers. The alignment indices are summarized in Table 15.

Table 15 Proposed Alignment Indices

Measure	Definition	Source
Indices Related to Horizontal Curvature Only		
Curvature Change Rate (CCR_S)	Absolute sum of the angular changes in the horizontal alignment divided by the length of the highway section (the formula is shown in equation (2))	*(6)*
Average radius $R_{avg} = \frac{\sum_i^n R_i}{n}$	R_{avg} = average radius of a set of horizontal curves in a specific highway section (m), R_i = radius of horizontal curve i (m), and n = total number of curves on the highway section.	*(3)*
R_{max} / R_{min}	The maximum radius divided by the minimum radius of a highway alignment	*(39)*
Ratio of individual radius to average radius $CRR_i = \frac{R_i}{R_{avg}}$	R_i = radius of horizontal curve i on the highway section (m), and R_{avg} = average radius (m).	*(39)*
Indices Related to Both Horizontal and Vertical Curvature		
Length Ratio	Sum of horizontal and vertical curve lengths on a specific highway section divided by the length of the section	*(3)*
Indices related to vertical curvature only		
Average Rate of Vertical Curvature $AVC = \frac{\sum_i^n \frac{L_i}{\lvert A_i \rvert}}{n}$	AVC = average rate of vertical curvature (m/grade), L_i = length of vertical curve i (m), A_i = algebraic difference in grade for vertical curve i on the highway section (percent), and n = number of vertical curves on the highway section.	*(3)*
Average Hilliness	Sum of the distances between each crest vertical curve and the following sag vertical curve in a specific highway section divided by the length of this section	*(5)*
Indices Related to Tangent Length		
L/R	Ratio of the length of the approach tangent to the radius of the horizontal curve	*(40)*

Anderson et al. (*39*) have stated that the ratio of the maximum radius to the minimum radius is not recommended as a design consistency measure due to its relatively low sensitivity to collision frequency compared to other alignment indices studied. Also, statistical measures such as the standard deviation, coefficient of variation, and weighted averages are not recommended as alignment indices. These indices provide information related to the general character of the entire alignment without indicating where the individual inconsistent sections are located (*3*).

2.1.3.2 Geometric Design Consistency Evaluation Criteria Based on Alignment Indices

Design consistency evaluation criterion based on alignment indices is not as well established as those based on operating speed or vehicle stability. Nonetheless, there are rules pertaining to geometric design features which are included in some European design standards and are summarized in Table 16.

Table 16 Design Consistency Evaluation Criteria Based on Alignment Indices

Criterion	Source
Good design: $\Delta DC \leq 5°$ (consistency exists) Fair design: $5° < \Delta DC \leq 10°$ (minor inconsistencies, traffic warning devices warranted.) Poor designs $\Delta DC > 10°$ (strong inconsistencies; redesign recommended.)	(*17*)
Good design: $\lvert CCR_i - CCR_{avg} \rvert \leq 180$ gon/km Fair design: $180 < \lvert CCR_i - CCR_{avg} \rvert \leq 360$ gon/km Poor design: $\lvert CCR_i - CCR_{avg} \rvert > 360$ gon/km	(*15*)
Good design if $DC < 4.24°$ and if radii of successive curves (separated by a short tangent) are equal	(*4*)
The ratio of the flatter radius to the sharper radius should be $\leq$ 3:2 for two successive horizontal curves without a connecting tangent.	(*9*)

ΔDC = change in degree of curve between successive design elements; CCR_i, CCR_{avg} = value of design element i and the average value, respectively.

Figure 2 below also presents a criterion for evaluating design consistency based on the radii of two successive horizontal curves. A design may be considered good when the two radii are about equal (*41*).

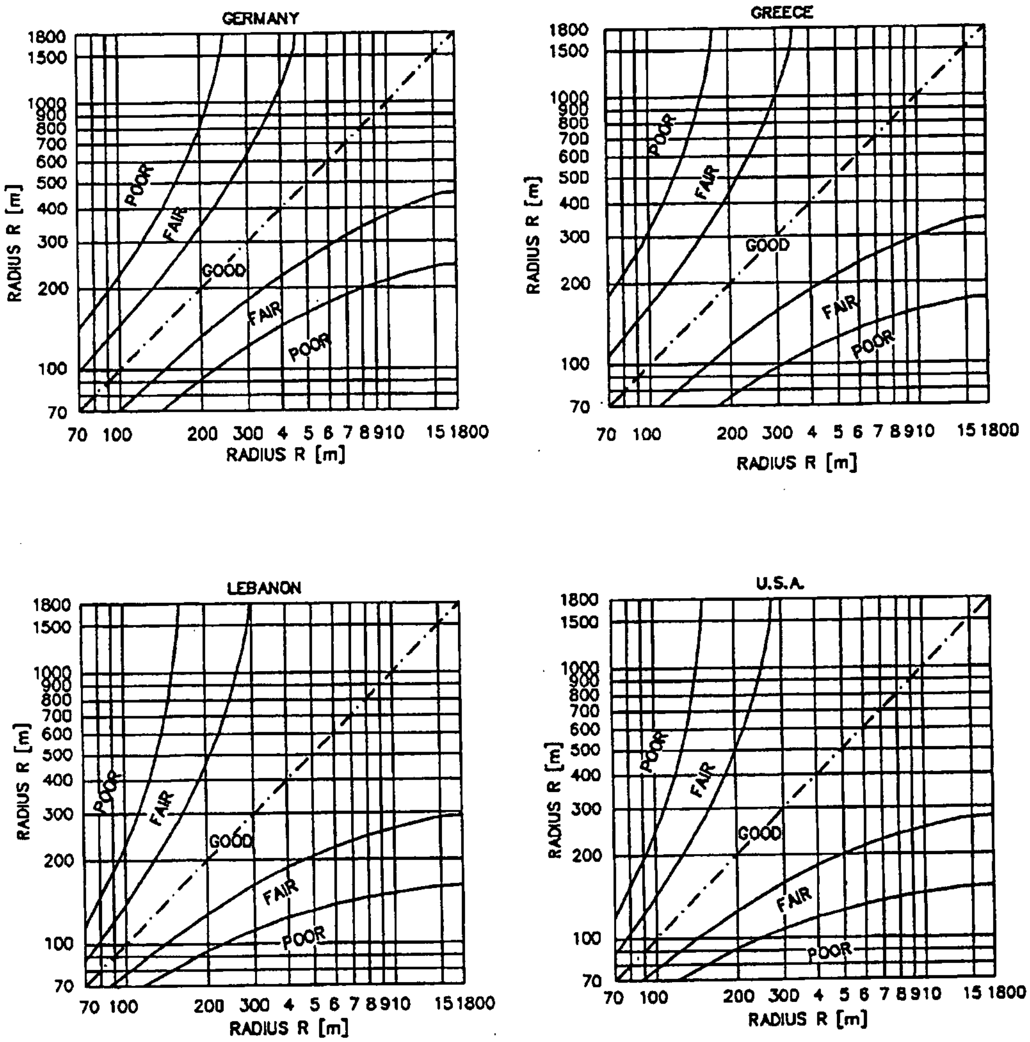


Figure 2 Criterion for Safety Evaluation Based on Radii of Successive Horizontal Curves (Lamm et al. 1995)

2.1.3.3 Discussion on Alignment Indices as a Geometric Design Consistency Measure

Alignment indices are supposedly the most direct measures of design consistency; however, they are not the most effective due to a number of reasons. First, most alignment indices such as average radius or average curvature are indices for an alignment rather than for an individual section. Often, it is the abrupt transition between successive sections such as tangent to curve or curve to curve that violates driver's expectation. The sharpness of abrupt transitions may not be accurately depicted when an alignment index represents an alignment rather than an individual section. Second, it is difficult to determine the degree of inconsistency from an alignment index. Only one value is obtained for each alignment and evaluation can only be made by comparing to other similar alignments. Third, even if some alignment indices represent individual sections such as *CRR*, the difference in the indices' values of successive sections is difficult to interpret. It is hard to justify when the difference is unacceptable.

2.1.4 Driver Workload

As explained previously, the roadway, the vehicle, and the driver interact in an interrelated manner. Therefore, it is logical to include driver workload as a measure of design consistency. Driver workload can be defined as the time rate at which drivers must perform the driving task which changes continuously until it is completed (*42*). Both the time available to perform the task and the complexity of the driving environment considerably affect the mental effort required. Conceptually, driver workload can be a more appealing approach for identifying inconsistencies than operating speed because it represents the demands placed on the driver by the roadway, while operating speed is only one of the observable outputs of the driving task. However, the use of driver workload is much more limited than operating speed due to its subjective nature (*43*).

Driver's expectancy is an important component of driver workload. It is defined as the driver's readiness to respond to the driving situation predictably and perform the driving task successfully (*11*). There are two types of expectancy: *a priori* expectancy and *ad hoc* expectancy. The *a priori* expectancy is the long-term expectancy developed cumulatively from previous driving experience, while the *ad hoc* expectancy is the short-term expectancy which is acquired during the present driving task. If either type of expectancy is not met, collisions may result (*44*).

2.1.4.1 Proposed Evaluation Methods of Driver Workload

There are four different methods to objectively measure driver workload: the primary task method, the secondary task method, the direct measurement of psycho-physiologic variables, and the information storage method. Most recent studies have been conducted using the information storage method. This method assumes that the intensity of the driver's attention varies depending on the driving environment and the perceived risks (*45*). Vision occlusion is an example of this method, where the driver's vision of the roadway is blanked out. The amount of time the driver needs to view the roadway is measured to calculate the visual demand, which is a proposed measure of driver workload. Visual demand is defined as the amount of visual information needed by the driver to maintain an acceptable path on the roadway (*46*).

2.1.4.2 Proposed Measures of Driver Workload

A subjective rating scale has been developed by Messer et al. (*42*) to estimate the average workload and the level of consistency of nine basic geometric features with a scale from 0 (no problem) to 6 (critical problem). 21 highway design engineers and researchers rated the features according to the feature type, design attributes, sight distance, separation distance, operating speed, and driver familiarity. The ratings are summarized in Table 17. An expression to estimate the driver workload of a geometric feature is developed based on these ratings and is included in Table 18. The rating provides a good basis for *a priori* expectancy evaluation.

Table 17 Summary of Geometric Feature Ratings for Average Conditions on Two-Lane Rural Highways (Messer 1980)

Geometric Feature	**Two-Lane**	
	High	**Mediocre** [a]
Bridge		
Narrow Width, No Shoulder	5.4	5.4
Full Width, No Shoulder	2.5	2.5
Full Width, With Shoulders[b]	1.0	1.0
Intersection		
Unchannelized	3.7	2.8
Channelized	3.3	2.5
Railway Grade Crossing	3.7	3.7
Shoulder Width Change		
Full Drop	3.2	2.4
Reduction	1.6	1.2
Alignment		
Reverse Horizontal Curve	3.1	2.3
Horizontal Curve	2.3	1.7
Crest Vertical Curve	1.9	1.4
Lane-Width Reduction	3.1	2.3
Crossroad Overpass	1.3	1.0
Level Tangent Section	0.0	0.0

a: surface treatment pavement without paved shoulders

b: assumed

Other than the subjective rating, two measures have been proposed to measure driver workload. They are sight distance and visual demand. Limited sight distance increases driver workload as the driver needs to update his information more frequently and process it more quickly. However, little research has been conducted to investigate the relationship between driver workload and sight distance. In contrast, models have been developed to estimate the visual demand of drivers familiar and unfamiliar with the highway. The models are summarized in Table 18.

Table 18 Driver Workload Predicting Models on Two-Lane Rural Highways

#	Model	R^2	Variables	Source
1	$VD_{LU} = 0.173 + \frac{43.0}{R}$	N/A	VD_{LU} = visual demand of unfamiliar drivers, VD_{LF} = visual demand of familiar drivers, and R = radius of horizontal curve (m).	(*46*)
2	$VD_{LF} = 0.198 + \frac{29.2}{R}$	N/A		
3	$WL = 0.193 + 0.016DC$	0.90	WL = average workload over the first half of the curve	(*21*)
4	$WL_n = U \cdot S \cdot E \cdot R_f + C \cdot WL_{n-1}$	N/A	WL_n = workload at feature, U = driver unfamiliarity factor ($0.4 < U < 1$), S = sight distance factor ($0.6 < S < 1.8$), E = feature expectation factor ($E = 1$ if feature is not similar to n-1 feature, otherwise $E = 1 - C$), R_f = workload rating (from table below), C = feature carryover factor ($0 < C < 1$, depending on the distance between features), and WL_{n-1} = workload value for the preceding feature n-1.	(*42*)

Models 1 and 2 are based on on-road test results and limited to horizontal curves with radius less than 552 m. Model 4 is based on test track results.

It has been found that driver workload is inversely proportional to horizontal radius, meaning that it increases with a decrease in radius. It should be noted that a driver workload criterion as represented by visual demand should not be based on radius alone because radius is directly affected by the design speed. Such a criterion will be biased against designs with low design speeds. Instead, a criterion which also includes design speed is more appropriate (*47*).

Yaw, which has been defined by Wooldridge (*48*) as the difference between the moving average workload and a specific feature's workload, is useful for *ad hoc* expectancy evaluation. Also, a driver expectancy checklist has been created which outlines some reminders when examining various design features. However, the checklist does not discuss the principles behind the reminders nor provide any explicit measures of driver workload for systematic applications (*49*).

2.1.4.3 Geometric Design Consistency Evaluation Criteria Based on Driver Workload

Some of the proposed indicators of design inconsistency based on driver workload include a high workload and a large positive change in workload. Subjective level of consistency criteria as found in Table 19, which are based on the workload evaluation developed by Messer et al. (*42*), can also be used to identify inconsistent geometric features. Still, an acceptance limits to changes in visual demand should be developed to help facilitate evaluation (*46*, *48*).

Table 19 Driver Workload-Based Level of Consistency Criteria (Messer 1980)

Level of Consistency	Workload Value (WL_n)	Driver Expectation
A	≤ 1	No problem expected
B	≤ 2	
C	≤ 3	Small surprises possible
D	≤ 4	
E	≤ 5	
F	≤ 6	Big problem possible

2.1.4.4 Discussion on Design Consistency and Driver Workload

Designing highway sections with very low or very high driver workload should be avoided. Errors are likely to occur on underloaded highway sections, where the driver's attention is being lowered for an extended period of time and his ability to handle surprising features is weakened. Similarly, sections with high workload should also be avoided as the increased complexity of the features and the limited time available for decision and maneuver may violate driver's expectation and lead to collisions (*48*).

2.2 Geometric Design Consistency Evaluation Software

The interactive highway safety design model (IHSDM) is an integrated design process tool which focuses on the safety implications and evaluates the cost-effectiveness of various highway design alternatives. Its design consistency module assesses the consistency of a geometric design in terms of operating speed, driver workload, and driver expectation. It evaluates the coherence of the entire design by analyzing the interaction of the various design features. The module produces a profile of the 85th percentile operating speed and a workload rating for each highway section, and identifies the inconsistent sections of the highway. It should be noted that the driver workload is determined based on the subjective rating method, the parameters of which cannot be estimated until the detail design is complete (*50*).

2.3 Road Safety Performance Evaluation

Since one of the objectives of this research is to quantify the relationship between geometric design consistency and road safety in terms of the expected collision frequency, a literature review on road safety has been conducted. The latest approach to safety performance evaluation and a previously developed collision prediction model of interest is described below.

2.3.1 Collision Prediction Models

Collision prediction models are the latest approach to evaluate the safety performance of a location. They are statistical regression models which relate collision occurrence to traffic and geometric characteristics of a location, and are developed based on a group of locations of similar geometric make-up. The models can be used to predict future collision occurrence at other locations of similar characteristics. They can also be used to identify collision-prone locations, to set up critical collision frequency curves, to rank collision-prone locations, and to perform before-and-after studies to show the effectiveness of an implemented treatment (*51*).

2.3.1.1 Generalized Linear Regression Method (GLM)

To estimate the parameters of collision prediction models, the generalized linear regression method (GLM) is used. GLM has the advantage of overcoming the limitations associated with the use of conventional linear regression in modeling collision occurrence, which is random, discrete, and non-negative in nature. Since the conventional linear regression requires that the model must be a linear combination of the explanatory variables, the error terms of which must be normally distributed, uncorrelated, and have equal variance, it is not suitable for modeling collision occurrence (*52*, *53*, *54*). In contrast, GLM allows for the specification of a Poisson or negative binomial error structure which depicts the nature of collision occurrence more fittingly. The following provides the theoretical background of GLM, which is based on the work of Hauer et al. (*53*), Kulmala (*55*), and Sayed and Rodriguez (*51*).

Let Y be the random variable that represents the number of collisions at a location in a specific time period, and assume that it follows the Poisson distribution with parameter λ. Let Λ be the variable that represents the mean of the Poisson distribution, such that $\Lambda = \lambda$. Hauer et al. (*53*) have shown that for an imaginary group of locations of similar characteristics, Λ can be regarded as a random variable which follows the gamma distribution with parameters κ and κ/μ, the mean and the variance of which are as follow:

$$E(\Lambda) = \mu \tag{6}$$

and

$$Var(\Lambda) = \frac{\mu^2}{\kappa} \tag{7}.$$

Consequently, considering the collision occurrence characteristics of a specific location and the imaginary group to which the location belongs, Hauer et al. (*53*) and Kulmala (*55*) have shown that Y follows the negative binomial distribution instead, with the mean and variance being equal to

$$E(Y) = \mu \tag{8}$$

and

$$Var(Y) = \mu + \frac{\mu^2}{\kappa} \tag{9}.$$

As such, the variance is equal to the expected value only when κ approaches infinity. The exception is equivalent to assuming that Y follows the Poisson distribution (*55*). Assuming a Poisson error structure is computationally simple because the mean and variance are equal. However, the negative binomial error structure can more realistically depict the overdispersion of the data, as the variance of this distribution is greater than the mean (*56*).

2.3.1.2 Model Structure and Development

The model structure relates collisions to exposure and other explanatory variables such as geometric design features or design consistency measures. The following two model forms can be adopted when studying highway sections, the merits of which is discussed in detail in Sawalha and Sayed (*57*).

$$E(\Lambda) = a_0 \times MVK^{a_1} \times e^{\Sigma b_j x_j} \tag{10}$$

$$E(\Lambda) = a_0 \times L^{a_1} \times V^{a_2} \times e^{\Sigma b_j x_j} \tag{11}$$

where:

$E(\Lambda)$ = expected collision frequency,
MVK = exposure in million-vehicle-kilometer = $L \times V$,

L = length of section,
V = average annual traffic volume,
x_j = any additional variable,
a_o, a_1, a_2 = model parameters, and
b_j = model parameters of additional variables.

To estimate the model parameters, the error structure is first assumed to follow the Poisson distribution. The dispersion parameter (σ_d) is calculated to determine whether this assumption is valid, as shown below:

$$\sigma_d = \frac{Pearson\chi^2}{n-p} \tag{12}$$

where:

n = number of observations,
p = number of model parameters, and

$$Pearson\chi^2 = \sum_{i=1}^{n} \frac{[y_i - E(\Lambda_i)]^2}{Var(y_i)} \tag{13}$$

where:

y_i = observed number of collisions on section i,
$E(\Lambda_i)$ = predicted number of collisions on section i, and
$Var(y_i)$ = variance of the observed collisions on section i.

Pearson χ^2 follows the χ^2 distribution with n-p-1 degrees of freedom. This parameter has been noted by McCullagh and Nelder (*58*) to be a useful statistic for assessing the variability in the observed data. If σ_d is greater than 1.0, it signifies that the data have greater dispersion than the Poisson distribution can accurately model, thus the negative binomial error structure is required. The parameters of the negative binomial distribution are estimated by an iterative process based on the maximum-likelihood estimates (*53*).

The selection of independent variables to be included in collision prediction models for safety performance evaluation, the main concern is the model's prediction accuracy. Only sufficient number of independent variables is included to maintain the model's prediction accuracy. Sawalha and Sayed (*57*) have provided a detailed explanation on the selection of independent variables.

2.3.1.3 Goodness of Fit

Two statistical measures can be used to assess the goodness of fit of collision prediction models developed using GLM. These are the *Pearson* χ^2 statistic, defined in equation (13), and the scaled deviance (*SD*) (*58*). The scaled deviance is computed differently depending on whether the error structure follows the Poisson or negative binomial distribution. The scaled deviance can be obtained using equation (14) if the error structure follows the Poisson distribution, and equation (15) if the error structure follows the negative binomial distribution, as follow:

$$SD = 2\sum_{i=1}^{n}[y_i \ln\left(\frac{y_i}{E(\Lambda_i)}\right)] \tag{14}$$

$$SD = 2\sum_{i=1}^{n}[y_i \ln\left(\frac{y_i}{E(\Lambda_i)}\right) - (y_i + \kappa)\ln(\frac{y_i + \kappa}{E(\Lambda_i) + \kappa})] \tag{15}$$

where:

SD = scaled deviance if the error structure follows the Poisson distribution,
y_i = observed number of collisions on section *i*,
$E(\Lambda_i)$ = predicted number of collisions on section *i*, and
κ = shape parameter of the gamma distribution which the imaginary group follows.

2.3.2 Previously Developed Collision Prediction Models

Some collision prediction models have been developed using GLM. The following presents a model of interest, which includes both geometric design features and a design consistency measure as explanatory variables.

The multivariate Poisson regression model has been developed by Saccomanno et al. (*59*) to identify collision-prone locations along a two-lane state highway Strada Statale 107 in southern Italy. This highway has long been recognized as having overall safety problems such as poor geometry, high operating speeds, and adverse weather conditions. Two geometric features and speed reduction are incorporated as explanatory variables. The model and its statistics are summarized in Table 20. The response variable is the number of collisions on section *i* for seven years, and travel exposure is represented by the section length only as a uniform traffic volume is reported for the entire highway.

Table 20 Poisson Regression Model Results for Strada Statale 107 (Saccomanno et al. 2001)

Model Variables	**Coefficient Value**	**Standard Error**	**Significance**
Base Coefficient	-1.420	0.000	0.0001
Section length (m)	0.003	0.000	0.0001
Number of private driveways in section	0.056	0.014	0.0001
Number of major intersections in section	0.539	0.149	0.0003
Speed reduction (ΔV_{85} in km/h)	0.018	0.005	0.0005

2.3.3 Safety Performance Evaluation Software

The collision prediction module (*60*) of IHSDM can estimate the number and severity of collisions, identify geometric deficiencies, and suggest countermeasures. Currently the models are being validated and evaluated, and the software implementation of the models is underway. The model for predicting the safety performance of road sections on two-lane rural highways is developed by Vogt and Bared (*61*) using the negative binomial regression method.

To improve prediction accuracy, an algorithm has been developed using historical data, regression analysis, before-and-after studies, and expert judgment. The algorithm consists of a base model and some collision modification factors (AMFs). The base model predicts the total collision frequency of a two-lane rural highway section as a function of traffic volume, geometric design features, and traffic control features. Only non-intersection related collisions will be predicted, that is, collisions occurring within 76 m (250 ft) of the intersection or due to the presence of an intersection will not be predicted. The base model is shown below in English units:

$$
\begin{aligned}
N_{br} &= EXPO \\
&\times \exp(0.6409 + 0.1388 STATE - 0.0846 LW - 0.0591 SW + 0.0668 RHR + 0.0084 DD) \\
&\times \left[\sum WH_i \times \exp(0.0450 \times DC_i)\right] \\
&\times \left[\sum WV_j \times \exp(0.4652 \times V_j)\right] \\
&\times \left[\sum WG_k \times \exp(0.1048 \times GR_k)\right]
\end{aligned}
\tag{16}
$$

where:

N_{br} = predicted number of total collisions per year on a particular highway section (coll./yr),

$EXPO$ = exposure in million vehicle kilometers of travel per year = $ADT \times 365 \times L \times 10^{-6}$

ADT = average annual daily traffic (veh/day),

L = section length (mi),

$STATE$ = location of highway (0 for Minnesota, 1 for Washington),

LW = lane width (ft), or average lane width if the two directions of travel differ,

SW = shoulder width (ft), or average shoulder width if the two directions of travel differ,

RHR = roadside hazard rating, representing the average level of hazard in the roadside environment along the highway section (definitions of rating categories can be found in Harwood et al. (*60*)),

DD = driveway density (driveways per mi) on the section,

WH_i = weight factor of horizontal curve i, which is the length of that portion of the horizontal curve lying in a highway section divided by the section length,

DC_i = degree of curve of horizontal curve i in a highway section (degree per 100 ft),

WV_j = weight factor of vertical curve j, which is the length of that portion of the vertical curve lying in a highway section divided by the section length,

V_j = vertical curve grade rate of vertical curve j in a highway section = $V_j = \dfrac{|g_{j1} - g_{j2}|}{l_j}$, (percent per 100 ft),

g_{j1}, g_{j2} = roadway grades at the beginning and at the end of vertical curve i (percent),

l_j = length of vertical curve j in a highway section (in 100 ft),

WG_k = weight factor of grade k in a highway section (grade length/section length), which is the length of that portion of the straightaway – that is, constant grade – lying in a highway section divided by the section length, and

GR_k = absolute grade of straightaway k in a highway section = $|g_k|$, (percent), and

g_k = grade of a straightaway portion of the section (in percent).

The weight factors WH_i, WV_j, and WG_k are non-negative and the sum of each of these factors for a highway section is one. The base model has been reduced for a specified set of nominal base conditions as outlined in Table 21.

Table 21 Nominal Conditions for the Base Model of Two-Lane Rural Highway Sections (Harwood et al. 2000)

Geometric Design Element	**Nominal Value**
Lane Width (*LW*)	3.6 m (12 ft)
Shoulder Width (*SW*)	1.8 m (6 ft)
Roadway Hazard Rating (*RHR*)	3
Driving Density (*DD*)	3 driveways per km (5 driveways per mi)
Horizontal Curvature	None
Vertical Curvature	None
Grade	Level (0 percent)

The reduced base model is:

$$N_{br} = ADT \times 365 \times 10^{-6} \times L \times \exp(-0.4865) \tag{17}$$

where:

N_{br} = predicted number of total collisions for base case (coll./yr),
ADT = average annual daily traffic (veh/day), and
L = length of section (mi).

The base model must be adjusted by the collision modification factors (AMFs) to account for the effect of individual geometric design and traffic elements which deviate from the nominal conditions. Each AMF is formulated so that it equals to 1.00 for the corresponding nominal condition. Conditions leading to higher collision experience will have AMFs greater than 1.00. Each AMF is formulated based on the best research available as selected by an expert panel. As an illustration, the AMF for horizontal curve is shown below which is based on the work of Zegeer et al. (*62*):

$$AMF_h = \frac{(1.55L_c + \frac{80.2}{R} - 0.012S)}{1.55L_c} \quad (18)$$

where:

$AMFh$ = collision modification factor for horizontal curve,
L_c = length of horizontal curve (mi),
R = radius of horizontal curve (ft),
S = 1 with spiral, 0 without spiral,
ADT = average annual daily traffic (veh/day), and
L = length of section (mi).

The AMFs represent the incremental effects of individual geometric design. However, the disregard of the potential interactions among the collision modification factors is a weakness of the algorithm. Nonetheless, the collision prediction model can be summarized as:

$$N_{rs} = N_{br} \times (AMF_1 \cdots AMF_n) \quad (19)$$

where:

N_{rs} = predicted number of total collisions per year on a segment (coll./yr),
N_{br} = predicted number of total collisions per year for the base case (coll./yr),
$AMF_1, AMF_2, \ldots, AMF_n$ = collision modification factors for various geometric design and traffic control elements.

2.3.4 Limitations of Collision Prediction Models

Despite the growing popularity of collision prediction models, they are related to a number of limitations. First, the models do not necessarily reflect cause-and-effect relationships. Many factors contribute to collisions, but not all of them are well understood and quantifiable. Also, the development of regression models depends heavily on the availability and accuracy of the data. If data are unavailable or inaccurate, the ability of the models to reflect cause-and-effect relationships would be weakened. Second, practitioners may be tempted to interpret each coefficient in the model as the true effect of an incremental change in the associated location characteristics on collision occurrence. This interpretation is not necessarily true. If the independent variables are either correlated to other variables in the model or to some important variables which have not been included, it would be difficult to isolate their individual impact. Third, collision prediction models should reflect local conditions and be current. For example, models developed for one region may not be applicable for another region due to reasons such as differences in climate, driver populations, and collision reporting practices. Thus, different jurisdictions are required to develop their own sets of models, unless calibration procedures are available so that models developed for one region can be calibrated and applied in another region. Fourth, it should be noted that collision prediction models are reliable only within the range of independent variables of the original data used for model development.

2.4 Relationship Between Geometric Design Consistency and Road Safety

It has been explained earlier that geometric design inconsistency may violate driver's expectations and lead to collisions. A criterion has been suggested by Lamm et al. (*63*) to evaluate design consistency based on collision rates (Table 24). It conforms to the German design guidelines (*64*), Swedish standards (*65*), and Swiss standards (*29*), and is the basis behind other safety criteria developed by Lamm et al., as shown in Table 5, Table 8, and Table 14. The criterion has been confirmed by Anderson et al. (*39*) using collision data on more than five thousand horizontal curves in the United States (Table 23). They have concluded that average collision rate is highest on horizontal curves which are rated poor in terms of design consistency, and is lowest on horizontal curves which are rated good.

Table 22 Design Consistency Criterion Based on Collision Rate (Lamm et al. 1988)

Criterion
Good Design: collision / 10^6 veh-km ≤ 2.27
Fair Design: $2.27 <$ mean collision rate (collision / 10^6 veh-km) ≤ 5.00
Poor Design: collision / 10^6 veh-km > 5.00

Table 23 Collision Rates at Horizontal Curves by Design Safety Level (Anderson et al. 1999)

Design Safety Level	Number of Horizontal Curves	3-year Collision Frequency	Exposure (million veh-km)	Collision Rate (collisions/million veh-km)
Good	4518	1483	3206.06	0.46
Fair	622	217	150.46	1.44
Poor	147	47	17.05	2.76
Combined	5287	1747	3373.57	0.52

Although the criterion shows that geometric design consistency is related to road safety, a quantitative relationship between the two is lacking. Studies have been conducted to investigate the relationship between individual design consistency measures and road safety, including speed reduction, alignment indices, and driver workload (no research has been performed to relate vehicle stability and road safety). However, a more comprehensive understanding of the relationship between design consistency and road safety is still missing. Models which combine several consistency measures can be useful to predict the safety benefits of improving design consistency in terms of collision reduction. Nevertheless, the following provides the state-of-the-art knowledge on the relationships between individual design consistency measures and road safety.

2.4.1 Speed Reduction and Road Safety

Anderson et al. (*39*) have investigated the relationship between design consistency and safety using loglinear regression models. They have found that speed reduction is strongly related to collision frequency. Two models have been developed which relate collision frequency with traffic volume, curve length, and speed reduction. However, the low coefficients of determination of both models signify that a large proportion of the variation in the data have not been well accounted for.

$$Y = \exp(-7.1977) \cdot AADT^{0.9224} \cdot CL^{0.8419} \cdot \exp(0.0662 \Delta V_{85}) \text{ [R}^2 = 0.195\text{]} \quad (20)$$

$$Y = \exp(-0.8571) \cdot MVKT \cdot \exp(0.0780 \Delta V_{85}) \text{ [R}^2 = 0.156\text{]} \quad (21)$$

where:

Y = number of collisions that occurred on the horizontal curve during a 3-year period,

ΔV_{85} = speed reduction on the horizontal curve from the approach tangent or curve (km/h),

$AADT$ = average annual daily traffic (veh/day),

L_{cr} = length of horizontal curve (km), and

$MVKT$ = exposure (million veh-km of travel for a 3-year period).

In a different research, Anderson and Krammes (*66*) have investigated the relationship between the mean collision rate and the mean speed reduction using a database of 563 curves. A linear regression analysis has been conducted to relate speed-reduction intervals and the corresponding mean collision rates. They have concluded that horizontal curves which require speed reduction, that is, curves with degree of curve greater than 4° (corresponding to design speeds of less than 100 km/h and with 85th percentile speed less than that on tangents) have higher collision rates than those which do not. The relationship is presented below along with the corresponding figure shown in Figure 3, which has also been published in the Canadian design standards (*7*).

$$meanAR = 0.54 + 0.27(mean\Delta V_{85}) \quad (22)$$

where:

AR = collision rate (collisions per million vehicle-kilometers), and

ΔV_{85} = difference between the estimated maximum 85th percentile speed on the approach tangent and the estimated 85th percentile speed at the midpoint of the horizontal curve (km/h).

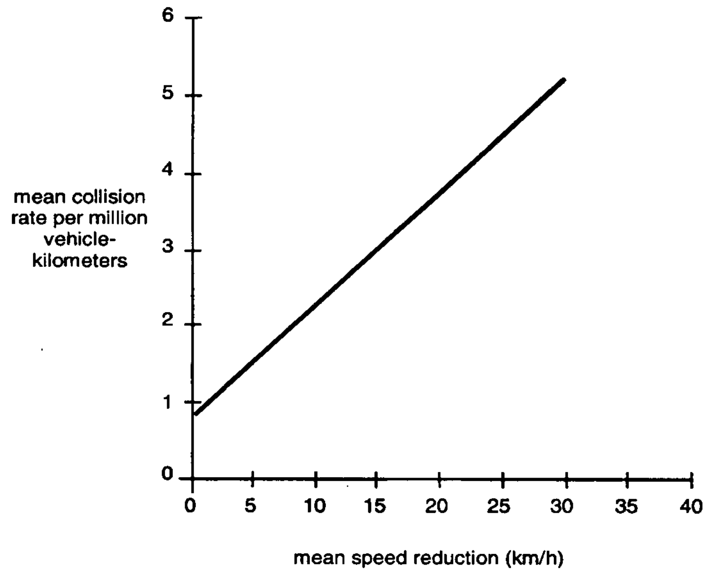


Figure 3 Mean Collision Rate Versus Mean Speed Reduction (Anderson and Krammes 2000)

2.4.2 Alignment Indices and Road Safety

Anderson et al. (*39*) have also investigated the relationship between collision occurrence and several alignment indices which are found to be sensitive to collision occurrence (*3*). Among these alignment indices is the ratio of the radius of an individual horizontal curve to the average radius of the roadway section (*CRR*). A model has been developed which relates collision frequency, curve length, and *CRR*.

$$Y = \exp(-5.932) AADT^{0.8265} CL^{0.7727} \exp(-0.3873 CRR) \; [\mathrm{R}^2 = 0.196] \tag{23}$$

where:

CRR = ratio of the radius of an individual horizontal curve to the average radius of the roadway section.

Lamm et al. (*15*) have compiled regression models developed in both Germany and US to relate collision rate and curvature change rate, CCR_S. However, most of these models are associated with low coefficients of determination, indicating that much of the variability in the data are not well accounted for. Nonetheless, it has been found that collision rate increases as CCR_s increases. The models are presented in Table 24 below. It should be noted that only run-off-the-roads collision data were used.

Table 24 Average Collision Rate Prediction Models (Lamm et al. 1999)

Lane Width	Model	R^2
Germany		
< 3.25 m	$CR = -0.31 + 9.4 \times 10^{-3}\, CCR_s$	0.35
≥ 3.25 m	$CR = -0.18 + 6.4 \times 10^{-3}\, CCR_s$	0.33
United States		
3.00 m	$CR = -0.639 + 0.0259 CCR_s$	0.30
3.60 m	$CR = -0.341 + 0.0185 CCR_s$	0.73
CR = average collision rate; CCR_s = curvature change rate.		

As quoted by Abdelwahab et al. (*26*), Glennon et al. (*67*) has studied the relationship between the reduction in collisions, ΔAF, and the reduction in degree of curve ΔDC, which is expressed in degree per 30 m. They have established the following relationship:

$$\Delta AF = 0.56 \times \Delta DC \qquad (24)$$

2.4.3 Driver Workload and Road Safety

Krammes and Glascock (*43*) have investigated the relationship between collision experience and driver workload on two-lane rural highways in Texas. Analyses have been performed at the microscopic level, which evaluates the relationship between collision frequency and the effective workload for individual geometric features, and the macroscopic level, which evaluates the relationship between the overall collision rate and the mean effective workload for an extended highway section. Effective workload is derived from the workload procedure developed by Messer et al. (*42*), and is the highest workload for the overlapped features within a uniform highway section. The mean effective workload quantifies the workload consistency along the alignment, and is computed as:

$$\mu_{EWL} = \frac{\sum (l_i \cdot EWL_i)}{\sum l_i} \tag{25}$$

where:

μ_{EWL} = mean effective workload value for the extended highway section,
l_i = length of feature i, and
EWL_i = effective workload value for feature i.

Results from the microscopic analysis shows that more collisions are associated with sections of greater effective workloads. Also, results from the macroscopic analysis indicate that collision rates are smallest on sections with moderate mean effective workloads (about 0.70 to 0.85) than sections with either low or high mean effective workloads (Figure 4). Considerable variability in collision rates is observed at sections with very low workloads. Thus, there can be a threshold workload value below which collision rates are not sensitive to driver workload, and above which collision rates will increase with driver workload. In conclusion, drivers need moderate workloads to stay attentive while not bored (due to low workload) nor exhausted (due to high workload) by the alignment.

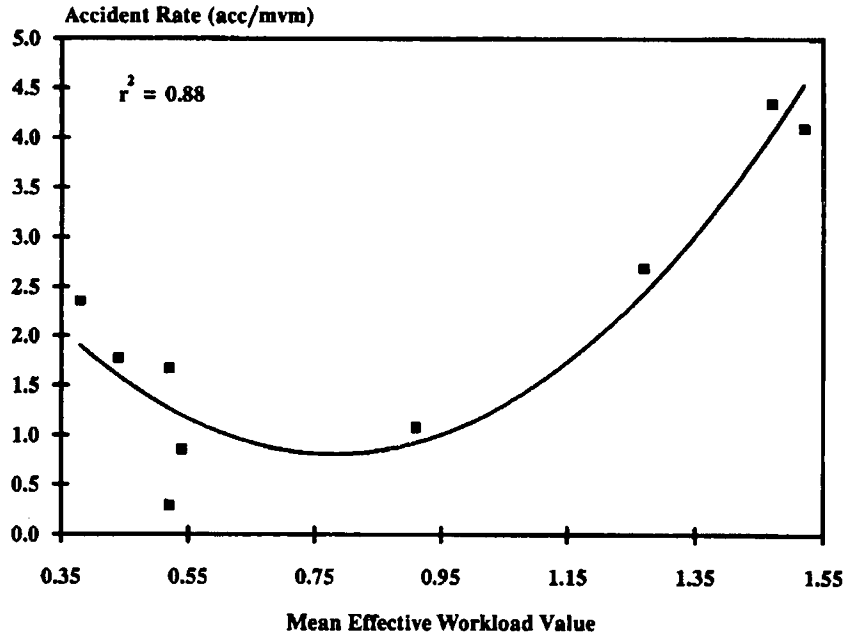


Figure 4 Collision Rate Versus Mean Effective Workload Value (Krammes and Glascock 1992)

2.5 Relationship Between Geometric Design Consistency and Highway Capacity

Gibreel et al. (*68*) have studied the relationship between geometric design consistency and highway capacity based on a three-dimensional analysis. Design consistency is a cost-effective way to maximize highway capacity utilization by improving the service flow rate and the level of service. They have compared the actual service flow rate as determined based on observed traffic volume data, and the theoretical flow rate as calculated based on highway capacity analysis. The conventional formula used for the latter is:

$$SF_{cal} = 2800 \cdot (v/c) \cdot F_d \cdot F_w \cdot F_{HV} \tag{26}$$

$$F_{HV} = \frac{1}{[1 + P_T(E_T - 1) + P_B(E_B - 1) + P_{RV}(E_{RV} - 1)]} \quad (27)$$

where:

SF_{cal} = calculated service flow rate,
v/c = volume to capacity ratio,
F_d = adjustment factor for directional distribution of traffic,
F_w = adjustment factor for narrow lane and restricted shoulder width,
F_{HV} = adjustment factor for presence of heavy vehicles,
P_T, P_B, P_{RV} = percentages of trucks, buses, and recreational vehicles in the traffic stream respectively, and
E_T, E_B, E_{RV} = passenger car equivalent for trucks, buses, and recreational vehicles respectively.

The level of service on each roadway section is determined based on the percent time delay on the section. Two types of 3D alignment combinations have been studied: the sag combination (a sag vertical curve combined with a horizontal curve), and the crest combination (a crest vertical curve combined with a horizontal curve). The results show that the actual service flow rate is always smaller than the theoretical one, with the ratio of *SF* of SF_{cal} ranging from 0.74 to 0.98. Gibreel et al. (*68*) argue that the difference is due to geometric design inconsistencies. Thus, a new adjustment factor called the consistency factor ($F_c \leq 1$) is developed to account for the difference. The theoretical service flow rate formula now becomes:

$$SF_{cal} = 2800 \cdot (v/c) \cdot F_d \cdot F_w \cdot F_{HV} \cdot F_c \quad (28)$$

Models have been developed for an accurate determination of this new consistency factor F_c based on geometric design parameters or based on speed variations on sag and crest combinations and are presented in Table 25.

Table 25 Consistency Factors Fc Prediction Models (Gibreel et al. 1999)

Consistency Factor Model	R^2	Independent Variables
Fc Based on Geometric Elements		
Sag Combinations $F_{c-sag} = 0.317 + 0.170\left[\ln(\frac{(R+1)(K+1)}{(L_0+1)})\right]^{0.7}$	0.59	R = radius of horizontal curve (m); K = rate of vertical curvature (m); L_0 = distance between point of vertical intersection and point of horizontal intersection (m).
Crest Combinations $F_{c-crest} = 0.383 + 0.302\left[\ln(\frac{(R+1)(K+1)}{(L_0+1)})\right]^{0.3}$	0.54	
Fc Based on Speed Variations		
Sag Combinations $F_{c-sag} = 1 - 0.002(V_{85} - V_d + 1)[\ln(\Delta V_{\max} + \exp(1))]^2$	0.52	V_{85} = maximum 85th percentile operating speed along the horizontal curve (km/h); V_d = design speed of horizontal curve (km/h); ΔV_{max} = maximum reduction in the 85th percentile operating speed along the 3D combination (km/h).
Crest Combinations $F_{c-crest} = 1 - 0.002[(\Delta V_{\max} + 1)^{0.6}][\ln(V_{85} - V_d + \exp(1))]^3$	0.52	

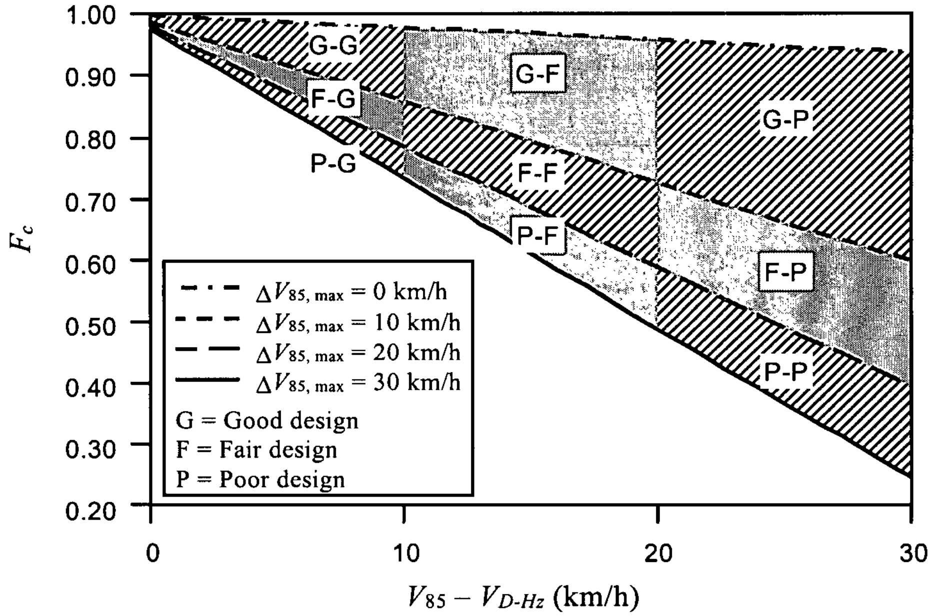
These models can be used to determine the expected loss in service flow rate due to geometric design inconsistencies. Typical values of F_c, as found on Table 26 and Figure 5, have been established and can be used to determine the approximate loss in service flow rate if the available speed data are not accurate. In addition, an overall consistency evaluation criterion of F_c has been developed based on the two operating speed criteria formulated by Lamm et al. (V_{85}-V_d and ΔV_{85} as shown in Table 5 and Table 8 respectively) and is included in Table 26.

Table 26 Typical Values of F_c for Sag and Crest Combinations (Gibreel et al. 1999)

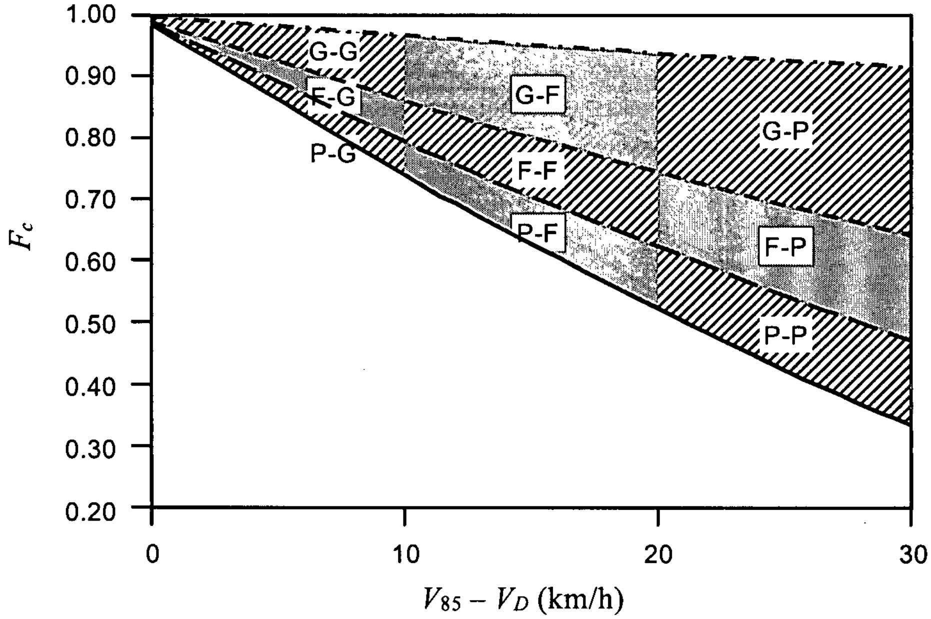
Design Consistency Evaluation Measure		**Operating Speed Reduction (ΔVmax) criterion**		
		Good	**Fair**	**Poor**
a) Sag Combinations				
	Good	$Fc \geq 0.85$	$0.78 \leq Fc < 0.97$ (0.86)	$0.74 \leq Fc < 0.97$ (0.82)
$(V_{85}\text{-}V_d)$	Fair	$0.74 \leq Fc < 0.98$ (0.88)	$0.58 \leq Fc < 0.85$ (0.76)	$0.49 \leq Fc < 0.78$ (0.64)
	Poor	$0.60 \leq Fc < 0.96$ (0.82)	$0.40 \leq Fc < 0.74$ (0.55)	$Fc < 0.58$
b) Crest Combinations				
	Good	$Fc \geq 0.86$	$0.80 \leq Fc < 0.98$ (0.87)	$0.73 \leq Fc < 0.98$ (0.82)
$(V_{85}\text{-}Vd)$	Fair	$0.75 \leq Fc < 0.96$ (0.87)	$0.62 \leq Fc < 0.86$ (0.74)	$0.53 \leq Fc < 0.80$ (0.65)
	Poor	$0.64 \leq Fc < 0.93$ (0.82)	$0.48 \leq Fc < 0.73$ (0.62)	$Fc < 0.62$

Table 27 Final Typical Values of Fc for Sag and Crest Combinations for Design Consistency Evaluation (Gibreel et al. 1999)

Criterion
Good Design: $0.80 \leq Fc < 1$
Fair Design: $0.67 \leq Fc < 0.80$
Poor Design: $Fc < 0.67$



(a) Sag combinations.



(b) Crest combinations.

Figure 5 Estimation of F_c Based on Design Consistency Evaluation (Gibreel et al. 1999)

2.6 Summary

Several measures of geometric design consistency have been identified in the literature and classified into four main categories: operating speed, vehicle stability, alignment indices, and driver workload. Models which can be used to estimate these measures and design consistency evaluation criteria have been presented. The latest approach to road safety evaluation, which is the application of collision prediction models, has also been discussed. Past studies investigating the relationship between geometric design consistency and road safety have been shown. In addition, the relationship between design consistency and highway capacity has also been described.

From the literature review, it can be observed that little work has been undertaken to quantify the safety benefits of geometric design consistency. Thus, the objectives of this study include investigating and quantifying the relationship between design consistency and road safety, as well as estimating the safety benefits of implementing a consistent design. Specifically, the relationships between various measures of design consistency and road safety in terms of expected collision occurrence are to be studied.

3.0 DATA DESCRIPTION AND MODEL DEVELOPMENT

This chapter provides a description of the data used to investigate the relationship between geometric design consistency and road safety. It also presents the methodology adopted and the design consistency measures selected for model development.

3.1 Data Description

Geometric design, collision, and traffic volume data of a two-lane rural highway located in the Okanagan and Kootenay regions of the province of British Columbia, Canada, are used. These data are obtained from the BC Ministry of Transportation. The geometric design data are extracted from "as-built" drawings prepared by the ministry which presents the data in many "strip maps" with the corresponding aerial photographs. The collision data are also extracted from these drawings which record the frequency and location of the collisions that occur from January 1991 to December 1995. The traffic volume data are obtained from the Traffic Information Management System (TIMS) maintained by the ministry. Collisions which may be related to the presence of a nearby intersection have been removed. Specifically, collisions which occur within 50 m of signalized intersections or within 20 m of all other types of intersections are eliminated. The data are classified into two groups: horizontal curves only, and horizontal curves and tangents combined. There are 319 horizontal curves in the first group, and 316 horizontal curves and 511 tangents in the second group. Table 28 provides some basic statistics of the relevant data used to develop models relating road safety and geometric design consistency.

Table 28 Summary Statistics of the Data Used for Model Development

Data Description	Minimum	Maximum	Average	Std. Dev.	Total
Horizontal Curves Data Only (319 Horizontal Curves)					
Section Length (km)	0.04	1.14	0.28	0.16	88.3
Radius of Horizontal Curve (m)	175	950	513	201	N/A
Average Annual Daily Traffic (veh/day)	3311	11396	5122	1883	N/A
Total Number of Collisions (Collisions per 5 years)	0	9	1.40	1.76	447
Horizontal Curves and Tangents Data Combined (316 Horizontal Curves and 511 Tangents)					
Section Length (km)	0.005	4.60	0.35	0.43	288.8
Average Annual Daily Traffic (veh/day)	3311	11396	5142	1856	N/A
Total Number of Collisions (Collisions per 5 years)	0	30	1.73	3.05	1429

3.2 Model Development

The methodology used in this study is based on the development of Collision Prediction Models incorporating design consistency measures. The generalized linear regression modeling (GLM) approach is adopted for model development, the theoretical basis and the advantages of which have been explained in section 2.3.1. Relationships between road safety and each of the four categories of geometric design consistency measures: operating speed, vehicle stability, alignment indices, and driver workload, are studied. The measures selected to represent each category and the corresponding models used are presented below.

3.2.1 Operating Speed

The difference between operating speed and design speed (V_{85}-V_d) and the speed reduction between two successive elements (ΔV_{85}) are used. Since the investigation of the relationship between geometric design consistency and road safety is limited to horizontal curves in this study, the operating speed model developed by Morrall and Talarico (*16*) is adopted, which relates V_{85} (km/h) on horizontal curves to the degree of curve (DC) using data on two-lane rural highways in Alberta. The model is limited to simple horizontal curves with constant lane and shoulder width and with vertical grade of less than 5%. It has a coefficient of determination (R^2) of 0.631 and is shown below:

$$V_{85} = \exp(4.561 - 0.0058DC) \tag{29}$$

and

$$DC = \frac{5729.58}{R} \tag{30}$$

where:

V_{85} = 85th percentile operating speed (km/h),
DC = degree of curve defined in metric units as the central angle subtended by an arc of 100 m, and
R = radius of horizontal curve (m).

The operating speed on tangents is computed using the model in equation (29) by assuming that the degree of curve is equal to zero, as currently available operating speed models on independent tangents are considered preliminary (*31*). This assumption results in a constant speed of 95.7 km/h for all tangents.

The 85th percentile maximum speed reduction (*85MSR*) experienced by a driver population on a tangent-horizontal curve combination as proposed by McFadden and Elefteriadou (*27*) is also investigated. The model is shown below:

$$\Delta V_{85} = -14.9 + (0.144 \times V_{85_{PC200}}) + (0.0153 \times L_T) + (\frac{954.55}{R}) \qquad (31)$$

where:

$V_{85_{PC200}}$ = 85th percentile speed at 200 m prior to the point of curvature (km/h),
L_T = length of the preceding tangent (m),
R = radius of the horizontal curve (m).

3.2.2 Vehicle Stability

The difference between side friction assumed and side friction demanded, denoted as Δf_R, is used to represent vehicle stability as shown in equation (32). The side friction assumed model developed by Lamm et al. (*35*) which is suitable for rural hilly and mountainous topography is adopted and presented in equation (33). It offers a human behavior based explanation for driver's choice of speed on curves. The side friction demanded model based on the laws of physics is adopted and presented in equation (34). Despite some criticisms associated with this model, it is widely used because of its relative simplicity.

$$\Delta f_R = f_{RA} - f_{RD} \qquad (32)$$

where

$$f_R = 0.22 - 1.79 \times 10^{-3} V_d + 0.56 \times 10^{-5} V_d^{\ 2} \qquad (33)$$

and

$$f_{RD} = \frac{V_{85}^{\ 2}}{127R} - e \qquad (34)$$

where:

f_{RA} = side friction assumed,
f_{RD} = side friction demanded,
V_d = design speed (km/h),
V_{85} = operating speed as represented by 85th percentile speed (km/h),
R = radius of horizontal curve (m), and
e = superelevation rate.

3.2.3 Alignment Indices

Two parameters are chosen to represent alignment indices. One of them is the ratio of the radius of an individual horizontal curve to the average radius of the alignment (*CRR*), as Anderson et al. (*39*) have found that safety is sensitive to this alignment index. It should be noted, however, that *CRR* does not recognize the effect of curve length and it can be significantly affected by the presence of a particularly sharp or flat curve. The other alignment index adopted is the length of the preceding tangent to the radius of the successive horizontal curve, denoted as *L/R*.

3.2.4 Driver Workload

The models which estimate visual demand of drivers unfamiliar and of drivers familiar with the road developed by Wooldridge et al. (*46*) are adopted and presented below:

$$VD_{LU} = 0.173 + \frac{43.0}{R} \tag{35}$$

$$VD_{LF} = 0.198 + \frac{29.2}{R} \tag{36}$$

where:

VD_{LU} = visual demand on unfamiliar drivers,
VD_{LF} = visual demand on familiar drivers, and
R = radius of horizontal curve (m).

All the design consistency measures mentioned above, namely V_{85}-V_d, ΔV_{85}, *85MSR*, Δf_R, *CRR*, *L/R*, VD_{LU}, and VD_{LF}, are computed for each section of the two-lane rural highway under study. However, design speed is not known. This value has been back solved using equation (1) given the radius of curvature. A table of minimum radii for limiting values of superelevation rate (*e*) and friction coefficient (*f*) for rural highways as found in the Canadian design guideline (*7*) is used to infer the design speed. The values of *e* and *f* and the corresponding design speed which yield a radius closest to the actual radius of a horizontal curve are used to compute the exact design speed. Since the design value is inferred, different sections are associated with different design speeds. For the same reason, friction assumed, which depends on the design speed, varies from section to section. Table 29 provides a summary of the design consistency measures as applied to the alignment under study.

Table 29 Summary Statistics of the Design Consistency Measures as Applied to the Alignment Under Study

Design Consistency Measure	Minimum	Maximum	Average	Std. Dev.
Horizontal Curves Data Only (319 Horizontal Curves)				
V_{85}-V_d	-43.02	11.56	-15.92	15.40
V_1-V_2	3.29	16.55	6.97	2.80
85MSR	0.50	68.05	6.86	7.42
Δf_R	-0.10	0.53	0.02	0.056
CRR	0.34	1.87	1.01	0.40
L/R	0.01	6.79	0.65	0.87
VD_{LU}	0.22	0.42	0.27	0.041
VD_{LF}	0.23	0.36	0.26	0.028
Horizontal Curves and Tangents Data Combined (316 Horizontal Curves and 511 Tangents)				
V_{85}-V_d	-43.0	11.6	-6.13	12.3
V_1-V_2	0	16.5	2.65	3.79
85MSR	0	68.1	2.63	5.68
Δf_R	-0.096	0.53	0.0089	0.037
CRR	0	1.87	0.39	0.55
L/R	0	6.79	0.25	0.62
VD_{LU}	0	0.42	0.10	0.13
VD_{LF}	0	0.36	0.10	0.13

4.0 MODELING RESULTS

This chapter presents the models developed in this study. Two groups of models have been developed with different objectives. The objectives of the first group are to investigate how design consistency as represented by each individual measure relates to road safety and to assess the direction of correlation. The objective of the second group is to develop quantitative relationships which can serve as evaluation tools to investigate the impact of design consistency on road safety. As many design consistency variables as are statistically significant are incorporated to improve the prediction accuracy of the models. In total, twenty-four models are presented which predict the safety performance of two-lane rural highway sections. The error structure of each model follows the negative binomial distribution, and the *Pearson* χ^2 and *SD* of each model are smaller than the corresponding critical χ^2 value. The developed models are presented with the degree of freedom, the t-ratio of each independent variable, the model parameter κ, the *Pearson* χ^2, the corresponding critical χ^2 value, and *SD*.

4.1 Models Relating Exposure to Road Safety

A model relating road safety and exposure has been developed to assess how strongly section length and traffic volume are related to collision data for this particular data set and is shown in Table 30. Both horizontal curves and tangents data have been used to develop this model. Exposure is represented by two separate terms: section length and traffic volume. Both variables are statistically significant at the 5% significance level and are positively correlated to collision frequency as expected, indicating that the longer the section length and the larger the traffic volume, the higher the collision frequency would be. Section length is much more strongly correlated to collision frequency than traffic volume. This can be due to the fact that traffic volume is constant for an extended length and in essence does not vary much, while section length changes considerably from one section to the next for this set of data.

Table 30 Model Relating Safety Performance to Exposure Only

Model Form		t-ratio	κ	Pearson χ^2 (χ^2 test) SD
	a_0	-1.93		825.15
$Coll./5yrs = \exp(-2.003) \times L^{1.065} \times V^{0.4265}$	a_1	22.99	2.454	(891.89)
	a_2	3.52		841.83

Note: Developed based on horizontal curves and tangents data combined; degree of freedom = 824.

4.2 Model Investigating Safety Performance of Tangents

Another model has been developed to predict the safety performance of tangents of two-lane rural highways and is shown in Table 31. Only the exposure variables (section length and traffic volume) are statistically significant. Again, the t-ratio of section length is much higher than that of traffic volume, meaning that section length is more strongly related to collision frequency than traffic volume for this set of data.

Table 31 Model for Predicting Safety Performance of Tangent Sections Only

Model Form		t-ratio	κ	Pearson χ^2 (χ^2 test) SD
	a_0	-0.829		563.49
$Coll./5yrs = \exp(-1.059) \times L^{1.102} \times V^{0.3194}$	a_1	21.98	2.916	(568.9)
	a_2	2.144		505.09

Note: Developed based on tangents data only; degree of freedom = 515.

4.3 Models Relating Only One Geometric Design Consistency Measure to Road Safety

Twenty models which relate each individual design consistency measure to safety are presented in Table 32. They have been developed using two different data sets:

horizontal curves data only, and horizontal curves and tangents data combined. Some models consider exposure with million vehicle kilometers (*MVK*) and others with section (*L*) and average annual daily traffic (*V*). It is found that models developed using horizontal curves and tangents data combined and which represent exposure with *L* and *V*, are associated with higher κ than those developed using horizontal curves data only and which represent exposure with *MVK*. A higher κ generally indicates a better fit of the model to the data based on which it is developed.

Table 32 Models Showing the Relationship Between Each Design Consistency Measure to Road Safety

#	Model Form		t-ratio	κ	Pearson χ^2 (χ^2 test) SD
DIFFERENCE BETWEEN OPERATING AND DESIGN SPEED (V_{85}-V_d)					
1a	$Coll./5yrs = \exp(-3.380) \times L^{0.8920} \times V^{0.5913}$ $\times \exp[0.009091 \times (V_{85} - V_d)]$	a_0 a_1 a_2 a_3	-1.904 6.511 2.784 1.985	1.539	293.24 (357.39) 341.26
1c	$Coll./5yrs = \exp(-5.415) \times MVK^{0.8199}$ $\times \exp[0.009322 \times (V_{85} - V_d)]$	a_0 a_1 a_2	-6.201 6.650 2.027	1.510	289.81 (358.46) 340.53
SPEED REDUCTION (ΔV_{85})					
2a	$Coll./5yrs = \exp(-3.796) \times L^{0.8874} \times V^{0.5847}$ $\times \exp(0.04828 \times \Delta V_{85})$	a_0 a_1 a_2 a_3	-2.072 6.482 2.742 2.043	1.533	289.19 (357.39) 341.26
2b	$Coll./5yrs = \exp(-2.281) \times L^{1.096} \times V^{0.455}$ $\times \exp(IC \times 0.02421 \times \Delta V_{85})$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0 a_1 a_2 a_3	-2.19 22.46 3.74 2.25	2.484	826.23 (890.9) 839.59
2c	$Coll./5yrs = \exp(-5.856) \times MVK^{0.8147}$ $\times \exp(0.04840 \times \Delta V_{85})$	a_0 a_1 a_2	-6.152 6.613 1.952*	1. 504	289.12 (358.46) 340.57
2d	$Coll./5yrs = \exp(-7.169) \times MVK^{1.019}$ $\times \exp(IC \times 0.02419 \times \Delta V_{85})$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0 a_1 a_2	-19.92 22.13 2.21	2.231	795.24 (891.89) 834.62

#	Model Form		t-ratio	κ	Pearson χ^2 (χ^2 test) SD
DIFFERENCE BETWEEN SIDE FRICTION ASSUMED AND DEMANDED (Δf_R)					
3a	$Coll./5yrs = \exp(-3.303)\times L^{0.8733}\times V^{0.5680}$ $\times\exp(-2.194\times\Delta f_R)$	a_0 a_1 a_2 a_3	-1.851 6.412 2.672 -1.986	1.521	294.01 (357.39) 341.27
3b	$Coll./5yrs = \exp(-2.086)\times L^{1.063}\times V^{0.4374}$ $\times\exp(IC\times -1.416\times\Delta f_R)$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0 a_1 a_2 a_3	-2.02 23.05 3.61 -1.64*	2.485	825.27 (890.9) 843.22
3c	$Coll./5yrs = \exp(-5.373)\times MVK^{0.8002}$ $\times\exp(-2.179\times\Delta f_R)$	a_0 a_1 a_2	-6.126 6.538 -1.949*	1.493	290.42 (358.46) 340.61
3d	$Coll./5yrs = \exp(-6.865)\times MVK^{0.9894}$ $\times\exp(IC\times -1.896\times\Delta f_R)$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0 a_1 a_2	-20.79 22.80 -1.70*	2.25	797.33 (891.89) 839.31
RATIO OF THE RADIUS OF AN INDIVIDUAL SECTION TO THE AVERAGE RADIUS OF THE ALIGNMENT (*CRR*)					
4a	$Coll./5yrs = \exp(-3.159)\times L^{0.8898}\times V^{0.5906}$ $\times\exp(-0.3606\times CRR)$	a_0 a_1 a_2 a_3	-1.791 6.514 2.785 -2.016	1.541	294.06 (357.39) 341.27
4c	$Coll./5yrs = \exp(-5.177)\times MVK^{0.8180}$ $\times\exp(-0.3700\times CRR)$	a_0 a_1 a_2	-6.034 6.661 -2.062	1.512	290.45 (358.46) 340.55
VISUAL DEMAND OF UNFAMILIAR DRIVERS (VD_{LU})					
5a	$Coll./5yrs = \exp(-4.297)\times L^{0.8866}\times V^{0.5831}$ $\times\exp(3.076\times VD_{LU})$	a_0 a_1 a_2 a_3	-2.231 6.476 2.735 2.040	1.531	295.34 (357.39) 341.23
5b	$Coll./5yrs = \exp(-2.184)\times L^{1.091}\times V^{0.4438}$ $\times\exp(IC\times 0.556\times VD_{LU})$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0 a_1 a_2 a_3	-2.10 22.36 3.66 1.96	2.462	825.25 (890.9) 839.24
5c	$Coll./5yrs = \exp(-6.375)\times MVK^{0.8136}$ $\times\exp(3.156\times VD_{LU})$	a_0 a_1 a_2	-5.780 6.604 2.074	1.503	292.23 (358.46) 340.54

#	Model Form		t-ratio	κ	Pearson χ^2 (χ^2 test) SD
5d	$Coll./5yrs = \exp(-7.106) \times MVK^{1.012}$ $\times \exp(IC \times 0.5223 \times VD_{LU})$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0 a_1 a_2	-19.78 22.06 1.96	2.211	793.88 (891.89) 834.48
VISUAL DEMAND OF FAMILIAR DRIVERS (VD_{LF})					
6a	$Coll./5yrs = \exp(-4.679) \times L^{0.8873} \times V^{0.5841}$ $\times \exp(4.566 \times VD_{LF})$	a_0 a_1 a_2 a_3	-2.323 6.481 2.740 2.027	1.533	295.58 (357.39) 341.26
6b	$Coll./5yrs = \exp(-2.164) \times L^{1.089} \times V^{0.4416}$ $\times \exp(IC \times 0.5419 \times VD_{LF})$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0 a_1 a_2 a_3	-2.08 22.34 3.64 1.85*	2.459	824.54 (890.9) 839.27
6c	$Coll./5yrs = \exp(-6.764) \times MVK^{0.8144}$ $\times \exp(4.684 \times VD_{LF})$	a_0 a_1 a_2	-5.464 6.610 1.985	1.504	292.08 (358.46) 340.57
6d	$Coll./5yrs = \exp(-7.089) \times MVK^{1.01}$ $\times \exp(IC \times 0.5008 \times VD_{LF})$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0 a_1 a_2	-19.78 22.06 1.68*	2.209	795.53 (891.89) 834.58

a: Developed based on horizontal curves data only; exposure represented by section length (L) and traffic volume (V); degree of freedom (DoF) = 315.

b: Developed based on horizontal curves and tangents data combined; exposure represented by L and V; DoF = 823.

c: Developed based on horizontal curves data only; exposure represented by million-vehicle-kilometer (MVK); DoF = 316.

d: Developed based on horizontal curves and tangents data combined; exposure represented by MVK; DoF = 824.

** Statistically significant at the 10% significance level.*

Collision frequency is shown to be positively correlated to V_{85}-V_d, ΔV_{85}, VD_{LU}, and VD_{LF}, and is negatively correlated to Δf_R and *CRR* in the models. The resulting models show the direction of correlation as expected. The larger the difference between the operating speed of drivers and the design speed of a section (V_{85}-V_d), the more collisions are expected to occur. Similarly, the larger the speed reduction required when moving from one section to the next (ΔV_{85}), the more collision are expected to occur. Also, the larger the difference between side friction assumed and side friction demanded (Δf_R), the less collisions are expected to occur as indicated by the negative parameter estimate. For alignment index *CRR*, collision frequency decreases when the radius of a given section is significantly higher than the average radius, and increases when the radius is significantly lower than the average. Finally, the higher the visual demand of a driver on a roadway (as represented by either VD_{LU} or VD_{LF}), the more collisions are expected to occur.

The two measures which consider the length of the preceding tangent in design consistency evaluation, namely *85MSR* and *L/R*, are statistically insignificant and therefore models with these measures are not possible. However, it does not necessarily indicate that the length of the preceding tangent does not affect design consistency or road safety. It only shows that the relationship between road safety and each of the two design consistency measures are not strong enough.

4.4 Quantitative Relationship Between Geometric Design Consistency and Road Safety

A quantitative relationship between design consistency and collision occurrence is an important tool in the evaluation of the impact of design consistency on road safety. The models presented above reveal the relationship between each design consistency measure to road safety, but they may not be very useful for a more comprehensive evaluation of the impact. Therefore, two models relating as many design consistency measures as are statistically significant to road safety to improve the model prediction accuracy are developed and are presented in Table 33. Exposure is represented with two separate terms (L and V) as this approach fits the data better. Model 7a is developed based on horizontal curves data only, and three design consistency measures are found to be statistically significant (V_{85}-V_d, ΔV_{85}, and Δf_R). On the other hand, model 7b is developed based on both horizontal curves and tangents data combined, and only two design consistency measures are statistically significant (ΔV_{85} and Δf_R). It should be noted that model 7a is applicable to horizontal curves only, and model 7b is applicable to both horizontal curves and tangents with the provision of the IC variable. Furthermore, from the κ parameter of these two models, it can be concluded that model 7b fits its data better than model 7a fits its horizontal curves data. In conclusion, because model 7b is applicable to both horizontal curves and tangents and because it demonstrates a relatively better fit to its data, model 7b is recommended for use in future evaluation of the impact of design consistency on road safety.

Table 33 Models for Evaluating the Impact of Design Consistency on Road Safety

#	Model Form		t-ratio	κ	Pearson χ^2 (χ^2 test) SD
7a	$Coll./5yrs = \exp(-3.369) \times L^{0.8858} \times V^{0.5841} \times \exp[0.0049 \times (V_{85} - V_d) + 0.0253\Delta V_{85} - 1.177\Delta f_R]$	a_0	-1.894	1.734	248.66 (355.26) 339.26
		a_1	6.480		
		a_2	2.749		
		a_3	2.085		
		a_4	2.022		
		a_5	-1.932*		
7b	$Coll./5yrs = \exp(-2.338) \times L^{1.092} \times V^{0.4629} \times \exp[IC \times (0.022 \times \Delta V_{85} - 1.189\Delta f_R)]$ *where IC = 0 for tangents or IC = 1 for horizontal curves.*	a_0	-2.25	2.511	828.68 (889.81) 841.06
		a_1	22.46		
		a_2	3.81		
		a_3	2.06		
		a_4	-1.64*		

a: Developed based on horizontal curves data only; degree of freedom = 313.

b: Developed based on horizontal curves and tangents data combined; degree of freedom = 822.

** Statistically significant at the 10% significance level.*

5.0 APPLICATIONS

The overall purpose of investigating the relationship between geometric design consistency and road safety is to identify inconsistent sections of an alignment so they may be treated to improve safety. The collision prediction models developed in this study can be used to identify inconsistent sections and to estimate the safety benefits of improving design consistency. Three applications which make use of the models are presented below. The first application illustrates how to evaluate the safety performance of two-lane rural highways. The second application investigates the effectiveness of collision prediction models which explicitly consider design consistency compared to those which do not. The third application presents a systematic approach to identify inconsistent locations.

All applications uses two fictitious alignments designed with intended inconsistencies by Sayed et al. (*69*). The alignments are denoted A-I and A-II with design speed of 70 and 100 km/h respectively. The geometric design data are shown in Table 34 and the profiles are shown in Figure 6. All alignments are assumed to have constant lane width, constant maximum superelevation rate, and no intersections. An average annual daily traffic of 25000 vehicles per day is assumed for all applications. Each alignment has eight horizontal curves (C1-C8) which are separated by tangents and are described below:

- C1 has a design speed of 100 km/h and C8 of 70 km/h; both allow for the testing of the effects of transition on successive highway sections with different design speeds.
- C2 and C3 are reverse curves separated by a short tangent.
- C4 is a compound curve made up of two curves.
- C5 is preceded by a long tangent T5.
- C6 is a long simple curve with a radius greater than the minimum value required.
- C7 has a higher design speed than C8 on alignment A-II but not on A-I.

Table 34 Horizontal Alignment Data of Two Fictitious Alignments (Sayed et al. 2000)

Element	Alignment I (V_d = 70 km/h)			Alignment II (V_d = 100 km/h)		
	L (m)	R (m)	L_s (m)	L (m)	R (m)	L_s (m)
T1	500.0	—	—	464.1	—	—
C1	424.8	600.0	60.0	391.2	600.0	60.0
T2	300.0	—	—	177.4	—	—
C2	174.5	190.0	70.0	394.1	440.0	80.0
T3	300.0	—	—	110.1	—	—
C3	137.9	190.0	70.0	334.0	440.0	80.0
T4	464.4	—	—	292.7	—	—
C4-1	150.0	200.0	—	300.0	440.0	—
C4-2	250.0	400.0	—	500.0	660.0	—
T5	1402.4	—	—	1000.2	—	—
C5	508.3	450.0	40.0	709.7	600.0	60.0
T6	611.4	—	—	350.3	—	—
C6	891.2	1000.0	—	765.2	1000.0	—
T7	380.8	—	—	374.0	—	—
C7	130.0	190.0	70.0	234.0	440.0	80.0
T8	277.4	—	—	253.9	—	—
C8	120.3	200.0	60.0	77.7	200.0	60.0
T9	500.0	—	—	502.4	—	—

Note: V_d = design speed; L = length of circular curve or tangent; R = radius of horizontal curve; and L_s = length of spiral curve.

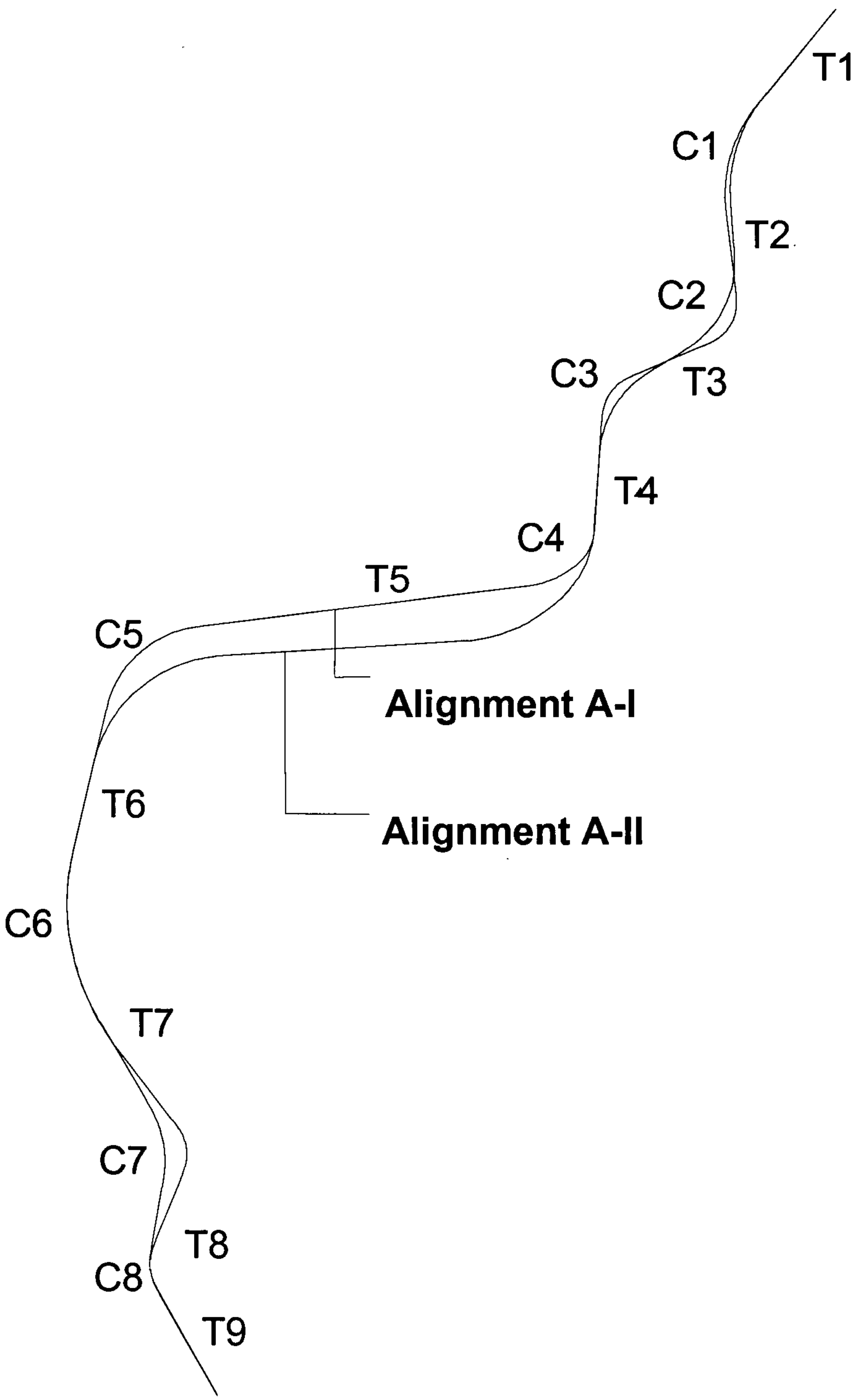


Figure 6 Profiles of the Two Fictitious Alignments (Sayed et al. 2000)

In addition, the applications use model 7b developed in this study to represent collision prediction models which incorporate design consistency measures as explanatory variables. The model has been modified slightly to predict collision frequency for a period of one year instead of five years, as shown below:

$$Coll./yr = \frac{\exp(-2.338) \times L^{1.092} \times V^{0.4629} \times \exp[IC \times (0.022 \times \Delta V_{85} - 1.189\Delta f_R)]}{5} \tag{37}$$

where:

- $Coll./yr$ = predicted collision frequency per year (coll./yr),
- L = length of section (km),
- V = average annual daily traffic (veh/day),
- IC = dummy variable ($IC = 0$ for tangents or $IC = 1$ for horizontal curves),
- ΔV_{85} = speed reduction between the approach tangent and the horizontal curve (km/h), and
- Δf_R = difference between side friction assumed and side friction demanded.

5.1 Evaluating the Safety Performance of Two-Lane Rural Highways

To illustrate how the collision prediction models developed in this study can be used to evaluate the safety performance of two-lane rural highways, model 7b has been applied to the two fictitious alignments. The results are shown in Figure 7 below.

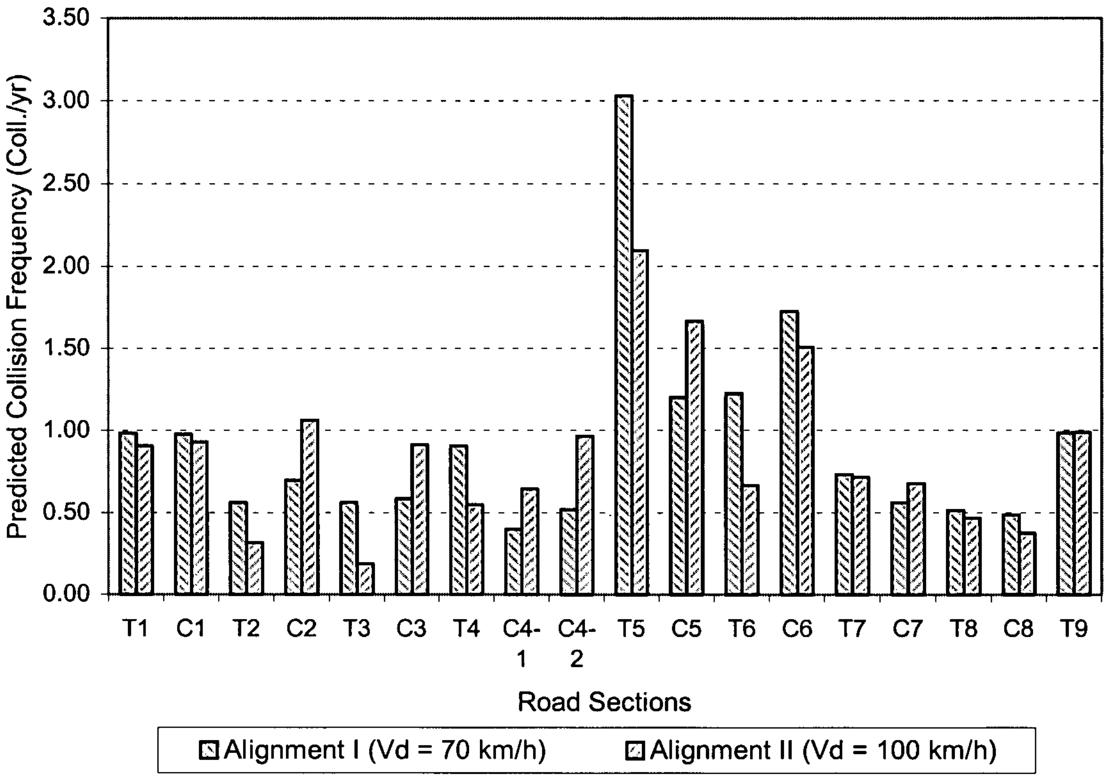


Figure 7 Predicted Collision Frequency of the Two Fictitious Alignments

The total predicted collision frequency of alignment A-I is 16.7 collisions/year, while that of alignment A-II is 15.6 collisions/year. Thus, alignment A-II is a safer alternative than alignment A-I. In spite of its superior safety performance, alignment A-II may be associated with higher construction costs and therefore may not be favored. With the total predicted collision frequency available for each alignment, designers can perform a benefit to cost analysis to determine whether the safety benefits of alignment A-II (lower total collision frequency) justify the economic costs.

5.2 Comparing the Effectiveness of Two Different Types of Collision Prediction Models in Evaluating Road Safety Based on Geometric Design Consistency

The objective of the second application is to determine whether models which explicitly consider design consistency are more effective in identifying inconsistent sections of a highway and reflecting the impact on collision frequency than existing models which rely on geometric design characteristics only. Model 7b represents the first type of model while the model incorporated in the crash prediction module of IHSDM represents the second type. Due to the simplicity of the fictitious alignments, the model is reduced to the following in English units:

$$N_{rs} = N_{br} \times AMF_h \tag{38}$$

where

$$AMF_h = \frac{(1.55L_c + \frac{80.2}{R} - 0.012S)}{1.55L_c} \tag{39}$$

and

$$N_{br} = AADT \times 365 \times 10^{-6} \times L \times \exp(-0.4865) \tag{40}$$

where:

N_{rs} = predicted number of total collisions per year on a section (coll./yr),
N_{br} = predicted number of total collisions per year for base case (coll./yr),
AMF_h = collision modification factor for horizontal curves,
L_c = length of horizontal curve (mi),
R = radius of horizontal curve (ft),
S = 1 if spiral transition curve is present, 0 if spiral transition curve is not present,
ADT = average annual daily traffic (veh/day), and
L = length of section (mi).

Both models have been applied to the two fictitious alignments, the results of which are discussed in the following sections.

5.2.1 Qualitative Analysis

The profile generated by model 7b and that by the model incorporated in the crash prediction module of IHSDM are shown in Figure 8 and Figure 9 respectively for an average annual daily traffic of 25000 vehicles per day. Although the general outlook of the two profiles are somewhat similar, there are differences between the two which distinguish their ability to identify geometrically inconsistent sections as reflected in the predicted collision potential of the sections. It should be noted that the latter model has not been calibrated for British Columbia conditions, thus causing differences in the magnitude of the predicted collision frequency by the two models. Nonetheless, the comparison is performed qualitatively as it is the difference in the predicted collision frequency between sections, rather than the value of an individual section, which indicates where the inconsistencies are. The results of the comparison are discussed below.

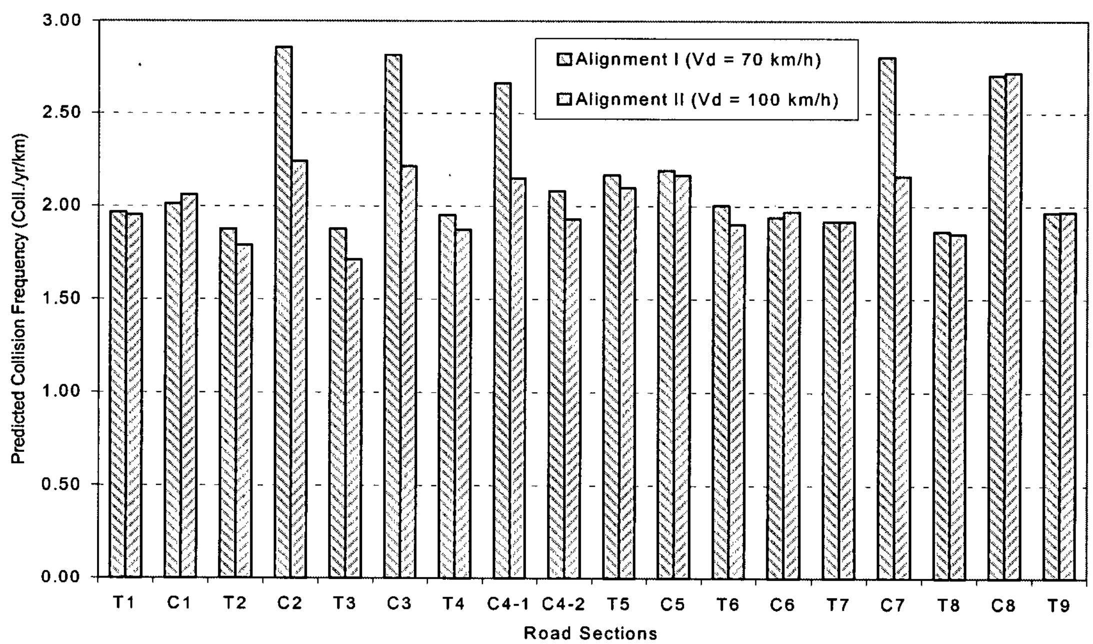


Figure 8 Safety Performance Evaluation Based on Model 7b

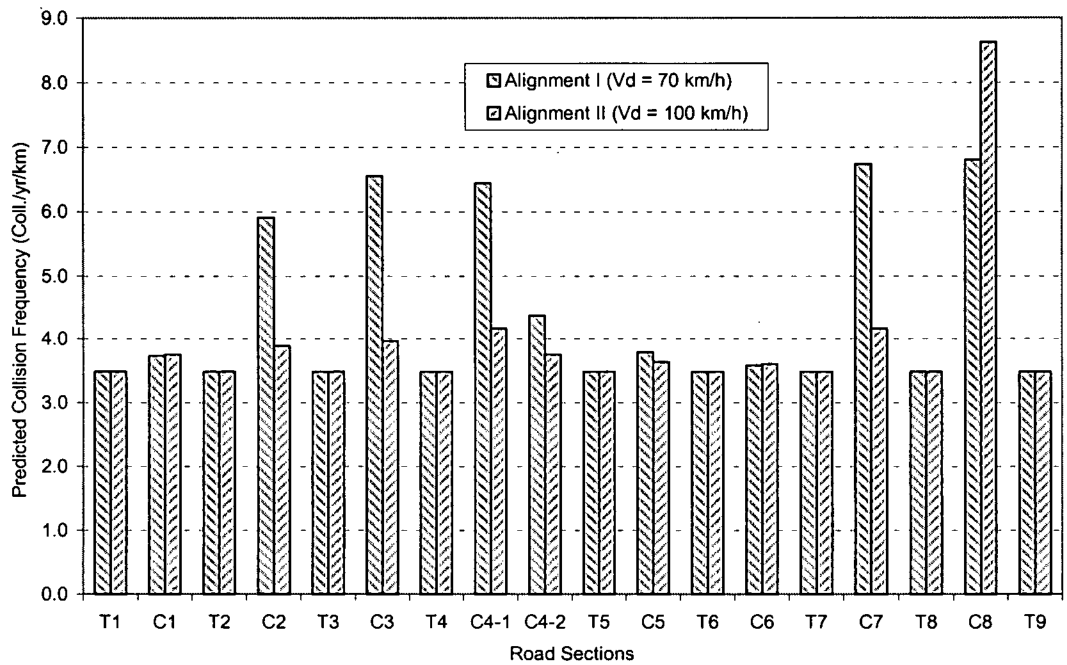


Figure 9 Safety Performance Evaluation Based on the Algorithm in IHSDM

Both profiles show a considerable increase from C1 to C2, with the increase being especially significant for alignment A-I where the design speed is reduced by 30 km/h from C1 to C2. Thus, both models can identify the inconsistency at C1 due to the difference in its design speed to that of C2. Model 7b predicts a slight decrease from C2 to C3 and similarly from C3 to C4-1, while the other model predicts an increase. It can be argued that the driver's level of attention should be maintained from C2 to C3, the two reverse curves with identical curvature separated by a short tangent section, after it has been raised when the driver travels from the flatter curve C1 to the tighter curve C2. The collision potential on C3 should be slightly lower than that at C2 because of the maintained level of attention. In addition, the steady decrease should extend to C4-1, the first part of the compound curve designed with a curvature similar to that of C2 and C3 for the same reason. Both models accurately predict a notable drop from C4-1 to C4-2 because of the larger radius of the second part of this compound curve. From C4-2 to C5, model 7b predicts an increase in collision potential while the other model predicts a decrease. The emergence of C5 with a moderate curvature at the end of a long tangent may violate the driver's expectation, therefore the collision potential at C5 is expected to increase. Both models show that C6 is the safest curve because it is designed with a radius greater than the minimum value required. Also, both models accurately predict that the collision potential at C7 rises remarkably after C6. The driver may expect another flat curve at C7, the element which follows C6 after a relatively short tangent. Although both C6 and C7 share the same design speed, the selection of radius greatly affects the consistency of the geometric design of an alignment. Finally, both models show an increase from C7 to C8 on A-II. Despite one's increased attention from C6 to C7, the model shows that the collision potential at C8 is greater than that at C7. Indeed, the inconsistency between C7 and C8 becomes more severe due to the design speed reduction of 30 km/h between these two elements.

5.2.2 Ability to Identify Geometrically Inconsistent Sections

Both models are capable of locating inconsistent sections which result from a difference in the design speed of successive elements, such as at C1 and C8. Also, they can predict a lower collision potential for sections of more generous curvatures. However, model 7b can show a steady decrease in collision potential on reverse curves and detect the effect of a long preceding tangent on the succeeding horizontal curve, while the other model fails to do so. Thus, collision prediction models which explicitly consider design consistency can locate more inconsistencies and reflect the resulting effect on collision potential more accurately than models which rely on geometric design characteristics to predict collision frequency.

5.3 A Systematic Approach to Identify Geometric Design Inconsistencies

Although inconsistent sections may be identified by their relatively higher collision frequency, the designer may find it difficult to decide how high is high enough. The safety-consistency factor proposed herein is a practical approach to systematically identify geometrically inconsistent sections using collision prediction models. It indicates how much the predicted collision frequency of a section differs from that of a tangent with identical section length and traffic volume. It is also easy to compute. First, estimate the collision frequency of the section using model 7b. Then, assuming that the section is converted to a tangent, predict its collision frequency. The ratio of the first collision frequency to the second is the safety-consistency factor. Thus, the greater the safety-consistency factor is, the greater the predicted collision frequency will be. A threshold value of the safety-consistency factor should be established for a systematic identification.

5.3.1 Establishing the Threshold Value of the Safety-Consistency Factor

To illustrate how the threshold value of the safety-consistency factor can be obtained, the two-lane rural highway used for model development is used again. The factor is computed for each of the 371 horizontal curves of this highway, and a cumulative distribution of the factor is plotted in Figure 10. The distribution has an average value of 1.13 and a variance of 0.022. To establish the threshold value, the 85th percentile value is selected. Thus, sections of the alignment with a safety-consistency factor greater than 1.33 can be identified as geometrically inconsistent and should be investigated. It should be noted that other percentile values may also be chosen, such as the 90th percentile and the 95th percentile value, depending on the policies of the jurisdictions.

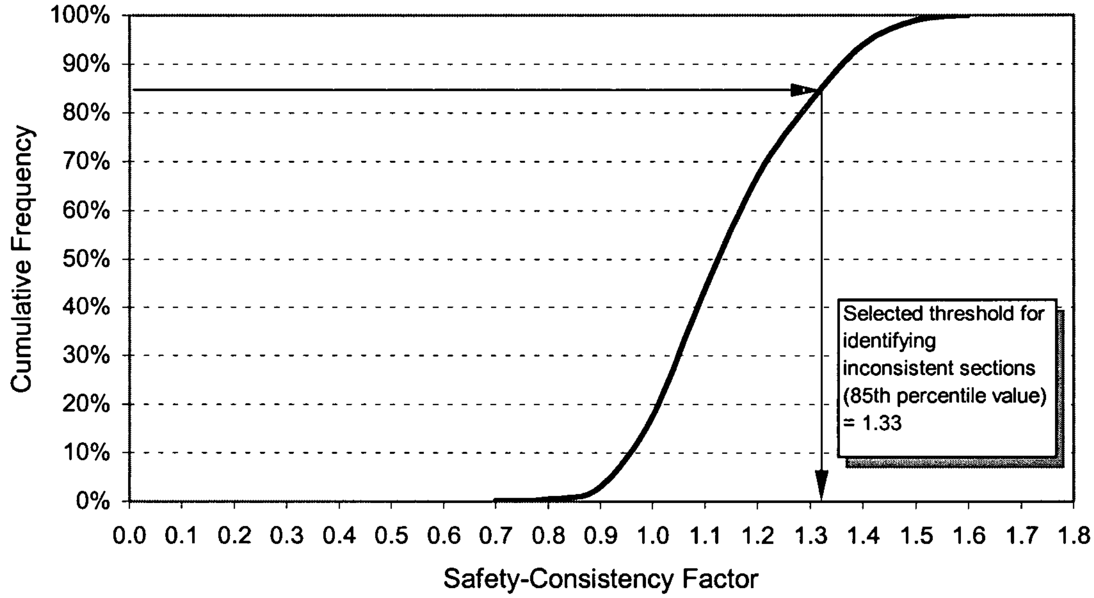


Figure 10 Cumulative Distribution of the Safety-Consistency Factors of Horizontal Curves of an Existing Alignment

5.3.2 Proposed Alignments

To systematically identify inconsistent sections of the two fictitious alignments, the safety-consistency factor has been computed for each section as shown in Figure 11. An average annual daily traffic of 25000 vehicles per day is assumed. Using the threshold value of 1.33 established above, horizontal curves C2, C3, C4-1, C7, and C8 of alignment A-I and C8 of alignment A-II can be considered inconsistent. The following discusses these sections in greater detail.

Horizontal curves C2, C3, C4-1, and C7 of alignment A-I are more inconsistent than the corresponding elements of alignment A-II. This may be due to the lower design speed of A-I (70 km/h) compared to that of A-II (100 km/h), which results in smaller radii and therefore lower predicted operating speeds. Since the operating speed on tangents is constant, significant speed reduction is observed from preceding tangents to these horizontal curves of alignment A-I. Moreover, the side friction assumed is insufficient to meet the demand on these horizontal curves of alignment A-I due to the lower design speed. Therefore, the larger speed reduction and the inadequate supply of side friction of alignment A-I result in higher values of the safety-consistency factor.

Similar level of inconsistency can be observed on horizontal curve C8 of both alignments. Since C8 is designed with a design speed of 70 km/h, significant speed reduction is observed for both alignments. The slight difference in the value of the safety-consistency factor of the two alignments is due to the lower side friction assumed of alignment A-II. In conclusion, the horizontal curves which have been identified as inconsistent should be modified, if possible, to improve the overall safety performance of the alignments.

On a separate note, although the two parts of the compound curve C4 is designed with identical design speed, C4-1 is classified as inconsistent for both alignments while C4-2 is not. The larger radius of C4-2 allows for a greater predicted operating speed and a lower side friction demanded, therefore the safety-consistency factor is lower for C4-2 than C4-1.

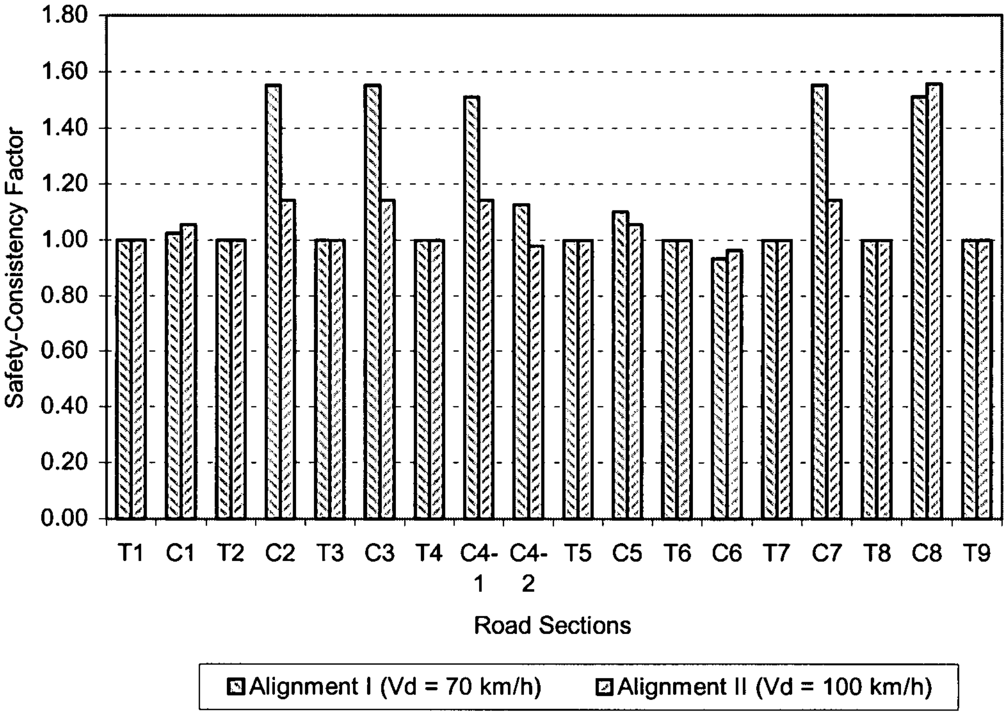


Figure 11 Safety-Consistency Factors of the Two Fictitious Alignments

6.0 CONCLUSIONS

Geometric design consistency is the conformance of a highway's geometry with driver expectancy. When an inconsistency exists which violates driver's expectation, the driver may adopt an inappropriate speed or inappropriate maneuver, leading to collisions. Despite its importance to road safety, geometric design consistency is not always ensured in current design practice. The inadequacy of the design speed concept, such as the fact that the design speed may not be the maximum permissible safe speed, and progressive changes to geometric design standards ,which result in sections along the same highway with inconsistent design speeds and cross-sections, are some of the sources of design inconsistency. Research on design consistency is still in the early stage and little work has been undertaken to quantify the safety benefits of geometric design consistency. This research investigates and quantifies the relationship between design consistency and road safety in terms of expected collision prediction frequency.

Twenty-four collision prediction models which relate design consistency to road safety have been developed. Twenty models investigate the relationship between individual design consistency measures to collision occurrence and show the direction of correlation as expected. For a more comprehensive evaluation of the impact of design consistency on road safety, two models which incorporate several design consistency measures to quantify the impact have been developed. The models show that when design consistency is considered, the safety performance of an alignment is improved. An example illustrating how the safety performance of two-lane rural highways can be evaluated has been shown. A qualitative comparison has also been made to compare collision prediction models which explicitly consider design consistency with those which rely on geometric design characteristics for predicting collision occurrence. It has been shown that the first type is superior as it can locate more inconsistencies and reflect the resulting effect on collision potential more accurately than the second. In addition, a systematic approach to identify geometrically inconsistent locations using the safety-consistency factor has been proposed.

7.0 FUTURE RESEARCH

The prediction accuracy of collision prediction models is limited by the quality of their independent variables. As such, the models developed in this study depend heavily on the design consistency measures used. Therefore, future research effort should be devoted to improving the prediction of these measures. For example, operating speed models should be developed which reflect local conditions. Models for tangents are also needed. In addition, alignment indices which include the length of the preceding tangent and are statistically significant to collision potential should be formulated. Furthermore, models which predict visual demand of drivers should include design speed as one of the independent variables.

The models developed in this study are limited to horizontal curves and tangents of two-lane rural highways only. More work is needed to expand the applicability to sections which are combined with vertical curves, as well as to those of other types of highways. The effect of changes in cross-section is another source of geometric design inconsistency which require further investigation.

Finally, design consistency evaluation criteria developed previously are derived from collision rates. New criteria based on predicted collision frequency may be developed using collision prediction models which explicitly consider design consistency. For example, new criteria may be developed using the safety-consistency factor. Sections may be classified into different levels of safety-consistency. In addition, a cumulative distribution similar to that of Figure 10 may be obtained for each design consistency measure, and evaluation criteria similar to that of the safety-consistency factor may be introduced.

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