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Modelling Operating Speed and Speed Differential for Design Consistency Evaluation

Submitted by

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A thesis submitted to the Faculty of Graduate Studies and Research in partial fulfillment
of the requirements for the degree of Master of Applied Science

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This thesis is humbly dedicated to:

The pure soul of my beloved mother,
Nahid Namdar Irani

My lovely wife, *Marjan Rohani*

My dear mother-in-law,
Dr. Mahindokht Khorramianpour

and

*All Baha'i students, across the globe, who are
deprived from higher educations*

Abstract

The geometric features of a highway network play a significant role considering the fact that a great percentage of collisions occur on horizontal curves. Based on extensive literature review, the problem mainly stems from the lack of *geometric design consistency* – conformance of highway geometric characteristics with drivers' expectations. In other words, drivers select their speeds according to their own perception of the road (referred to as the *operating speed*) rather than the designer's perception (referred to as the *design speed*). To address operating speed consistency evaluation in Canada, the development of two sets of models for operating speed and speed differential is discussed based on speed data collected on twenty curves on two-lane rural highways in Ontario. The results suggested that no statistically significant relationship was found to predict the operating speed while a good relationship was presented for speed differential.

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Nomenclature

<i>85MSR</i>	: 85 th percentile of maximum speed reduction from tangent to the middle of the curve (km/h)
<i>A</i>	: The algebraic difference between G_1 and G_2 (%)
<i>AADT</i>	: Average Annual Daily Traffic (vehicles/day)
<i>ADT</i>	: Average Daily Traffic (vehicles/day)
<i>CCR</i>	: Curvature Change Rate (deg/km)
<i>DC</i>	: Degree of Curvature (degree)
<i>DF</i>	: Curve Deflection Angle (degree)
<i>DFC</i>	: Circular Curve Deflection Angle (degree)
<i>DF₁, DF₂</i>	: Deflection Angles for curves 1 and 2 in case of a compound curve (degree)
<i>e</i>	: Superelevation rate (%)
<i>G₁, G₂</i>	: The first and the second grade in the direction of travel (upgrade has positive and downgrade is negative value) (%)
<i>HC</i>	: Horizontal Curve
<i>HT</i>	: Heavy Truck
<i>K</i>	: Rate of vertical curvature; required horizontal length for having 1 percent change in the grade (m)
<i>L₀</i>	: The horizontal distance between horizontal point of intersection and vertical point of intersection (m)
<i>L_C</i>	: Length of Horizontal Curve (m)
<i>LSD</i>	: Limited Sight Distance ($K \leq 43$)
<i>L_T</i>	: Length of tangent (m)
<i>LT</i>	: Light Truck
<i>L_{T1}</i>	: Length of preceding tangent (m)
<i>L_{T2}</i>	: Length of succeeding tangent (m)
<i>L_V</i>	: Length of vertical curve (m)
<i>LW</i>	: Lane width (m)
<i>NLSD</i>	: Non-Limited Sight Distance ($K > 43$)
<i>PC</i>	: Passenger Car
<i>Pcon</i>	: Pavement condition; $PSR \geq 3$: $Pcon=0$, otherwise: $Pcon=0$
<i>R</i>	: Radius of the curve (m)
<i>Ra, pre_radius</i>	: Radius of the preceding curve (m)
<i>Suc_radius</i>	: Radius of the succeeding curve (m)
<i>SW</i>	: Shoulder Width (m)

T	: Truck
$V_{85(1)} - V_{85(5)}$	: 85 th percentile speeds at for 60 m – 80 m from TS (tangent to spiral), SC (spiral to curve), MC (middle of the curve), CS (curve to spiral), and 60 m – 80 m after ST (spiral to tangent) respectively horizontal curves (km/h)
$V_{85(C1)} - V_{85(C5)}$	: 85 th percentile speeds at 60 m – 80 m from TS (tangent to spiral), SC (spiral to curve), MC (middle of the curve), CS (curve to spiral), and 60 m – 80 m after ST (spiral to tangent) respectively for the combination of horizontal curve and crest vertical curve (km/h)
$V_{85(S1)} - V_{85(S5)}$	: 85 th percentile speeds at 60 m – 80 m from TS (tangent to spiral), SC (spiral to curve), MC (middle of the curve), CS (curve to spiral), and 60 m – 80 m after ST (spiral to tangent) respectively for the combination of horizontal curve and sag vertical curve (km/h)
ΔV_{85A}	: The difference between the speed at the middle of the curve and 250 m before the beginning of the curve for all vehicles (km/h)
ΔV_{85LT}	: The difference between the speed at the middle of the curve and 250 m before the beginning of the curve for light trucks (km/h)
ΔV_{85PC}	: The difference between the speed at the middle of the curve and 250 m before the beginning of the curve for passenger cars (km/h)
V_{85T}	: 85 th percentile speed on tangents in the distance of 200 m from the beginning of the curve (km/h)
ΔV_{85T}	: The difference between the speed at the middle of the curve and 250 m before the beginning of the curve for trucks (km/h)
$V_a, V_F, V_{\text{tangent}}$	: Approach tangent speed (km/h)
VC	: Vertical Curve
V_d	: Desired speed (km/h)
V_P	: Posted speed limit (km/h)

Chapter 1: Introduction

1.1 Background

In today's modern world, the essentiality of an efficient, effective, safe, and environmentally friendly transport system is undeniable. In truth, the socio-economic growth of each country is, to a high degree, dependent on the accessibility, applicability, and practicality of its various transportation modes. In this regard, the developed countries such as U.S. and Canada are in a more critical situation due to the fact that, they form only 5% of the world's population while owning 43% of the motor vehicles (Oglesbey and Hicks, 1982).

Looking at the statistics given by the Transportation Association of Canada (TAC, 1990), it is realized that among the various components of a transportation system, highways are of an utmost importance in Canada. As stated in TAC (1999), 93% of all domestic travel by Canadians in 1990 plus the freight travel of more than 84 billion tone-kilometers in 1988 were served by highways. Therefore, the importance of highway engineering as an area which plays a significant role in the field of transportation engineering becomes more evident.

A highway engineer should logically follow the four stages mentioned in the literature as the rational highway design process. These stages include planning, route selection, geometric design, and structural design. In the planning stage, the goal is "the establishment of a highway network capable of accommodating all highway travel in an orderly, safe, efficient, and economical manner" (Michael, 1960). After passing the first stage, when the need for building a highway between two certain points is established, different possible routes are compared to find the best alternative.

Having a specific route between two control points, the engineer needs to design the road based on the geometric design guide of that country, e.g., TAC guide (1999). Using the mentioned guide, a designer is able to suggest specific values for the geometric features of the road such as cross section, curve radius, longitudinal grade, etc.

In the last stage, the materials required for the construction of the various pavement layers as well as the thickness of each layer are introduced by the designer. Such a pavement is to safely carry the traffic loads and resist the environmental conditions during its design life. Among the abovementioned phases, the third one, geometric design of highways, is the phase relevant to this study. In this study, the uniformity of the design elements along some portions of a highway and its conformance to the expectation of the majority of drivers is investigated in accordance with the concept of design consistency.

Design consistency is defined as the conformance of the geometric features of a road with drivers' expectations (Nicholson, 1998). According to Gibreel et al. (1999), a design can be referred to as consistent if it ensures that the successive elements are coordinated in a way to produce harmonized driver behaviour without surprising events. Al-Masaeid et al. (1995) defined the roadway consistency as the degree to which highway systems are designed and constructed to avoid critical driving manoeuvres that can lead to collision risk. Considering the abovementioned definitions of design consistency, its usefulness in making uniformly designed highways, and consequently improving safety, becomes clear.

There are several methods to evaluate the consistency of a design in a highway; namely, alignment indices, driver workload, and operating speed methods (Fitzpatrick et al. 2000b). Among these approaches, the operating speed approach has been known as the most efficient and quantified measure to date (Fitzpatrick et al. 2000b). It is more

accurate than the alignment indices method while being much less sophisticated and more quantifiable than the driver workload approach. The operating speed is defined as the speed at or below which, the majority of the drivers (e.g., 85%) are travelling, while not being impeded by any factors but the geometric features of the road (Lamm et al. 1986).

In the operating speed approach, the consistency of design is evaluated based on one of two different ways or a combination of both. The first way is evaluating the consistency based on the value of the speed differential between two successive elements of a road (two curves or curve-tangent). The second way, on the other hand, is based on the difference between the operating speed and design speed values. If the speed differential between two successive elements or the difference between the operating and design speed values is greater than a certain value, the feature is considered inconsistent. Therefore, the first step that should be followed to evaluate the design consistency is the development of regression models by the use of which designers can predict the operating speed of an element or the speed differential between the two elements. These models should be able to predict the speed values or speed differentials based on the geometric and traffic characteristics of the road elements such as radius, longitudinal grade, daily traffic, etc.

However, as a measure to assess the consistency of a design, the operating speed approach has faced some critical problems. The first problem is that there are several models, developed in various countries, whose coefficients and independent variables are, in most of the cases, substantially different from each other. This, in fact, shows that no uniformed model has been found yet to be accepted worldwide. The second problem is that there is no criterion for design consistency evaluation recommended in North American design guides.

The only existing design consistency criteria are those established by Swiss, German, and Australian researchers, and the one introduced by Leisch and Leisch (1977) in the U.S. These criteria are now being used in the design practice of those countries except for the one established by Leisch and Leisch (1977). All these criteria have been introduced through a collision analysis. They categorize the designs as good, fair, or poor according to the corresponding collision rate of the related vehicles.

1.2 Problem Definition

Current design practices in North America do not ensure the consistency of design. In fact, the traditional design process starts with the adoption of an arbitrary value for the design speed and then designing the individual elements based on that value provided that they meet the minimum requirements of the guides. The American Association of State Highway Officials (AASHTO) Green Book (2001), A Policy on Geometric Design of Highways and Streets, just implies the concept: "Consistent designs should be sought" or "Sharp curves should not be introduced at the ends of long tangents" or "Sudden changes from areas of flat curvature to areas of sharp curvature should be avoided" (AASHTO, 2001).

The TAC guide (1999), Geometric Design Guide for Canadian Roads, on the other hand, explicitly addresses the design consistency in a separate chapter. In that chapter, three different approaches to evaluate design consistency are introduced; namely cross section consistency, operating speed consistency, and driver workload consistency. Among these methods, the second one, operating speed consistency, has been introduced in more details and a model has been recommended to predict the vehicles' speed at the curved sections. However, the applicability of the mentioned formula should be verified for the specific conditions of Canadian roads. Moreover, speed reduction introduced in TAC (1999) calculated by the simple subtraction of

operating speeds at the middle of the curve and the tangent section, has an underlying assumption that the speed distributions at the two aforesaid points are the same; an assumption whose validity should be examined.

In the past 50 years, several models have been developed to predict the operating speed at the curved sections; however, only two of them have been developed using the speed data of the Canadian roads (Talarico and Morrall, 1994; Gibreel et al. 2001). In addition, few studies namely, Fitzpatrick et al. (2000), Gibreel et al. (2001), etc. focused on developing speed prediction models for tangent sections.

The necessity of models capable of direct calculation of speed differential from a tangent to a curve is another issue of concern that should be carefully considered. The importance of this matter becomes clear when we look at the existing criteria to evaluate the operating speed consistency. The existing criteria try to limit either the speed differential between the curved and straight sections or the difference between the operating speed and the design speed. Therefore, to mark a design as an acceptable one based on consistency criteria, the first step is to find a logical way to predict the speed differential.

The current approach to calculate the speed differential is based on calculating the operating speed of the drivers on the curved and the tangent sections and then, subtracting these two values and naming it as the speed differential value. However, this methodology does not seem rational (Hirsh, 1987; McFadden and Elefteriadou, 2001). According to the two mentioned studies, the speed distributions at the curved and tangent sections are not the same, and thus the simple subtraction of the operating speed values should not be performed. Also, even if the speed distributions are the same, the 85th percentile driver needs not to be the same in the two locations. Moreover, the result of the study by McFadden and Elefteriadou (2001) showed that the simple

subtraction of the operating speed values underestimates the real values of speed differential by an approximate factor of two.

1.3 Objectives

Considering the aforementioned statements, it is, first, essential to verify the validity of the previously developed speed prediction models for the specific situation of the Canadian roads. This can be done by choosing the same independent variables as employed in those studies, in the process of operating speed model development. Second, there should be an attempt to formulate the equations for the prediction of speed differential between the curved and straight sections. These are, in fact, the main objectives of this study. Specifically, this study has two main objectives:

1. Operating speed model development for passenger cars and heavy trucks.
2. Speed differential model development for passenger cars and heavy trucks.

As stated earlier, the mentioned models can be used for design consistency evaluation based on the operating speed methodology. The development of these models is in fact part of a broader research initiative at Carleton University for design consistency evaluation. Obviously, in order to relate the consistency evaluation to the different measures, reliable models that can predict operating speed and speed differential while accounting to local conditions must first be developed.

1.4 Scope of the Study

The scope of this study is limited to two-lane two-way rural highways for dry pavement conditions, daytime, and free-flow traffic. It means that unfavourable climate conditions such as rain or the effect of ambient light on drivers' performance is not considered in this study. In addition, the only acceptable traffic condition is the free-flow stream of traffic in which no driver is impeded by other vehicles. Hence, as will be

explained later, those parts of the speed database within nighttime or rain time will be excluded from the speed observations that are employed for the purpose of model development. Also, those vehicles that have a time gap of less than five seconds from the leading vehicles are excluded from the speed database. In other words, the focus of this study is on the circumstances in which only the geometric features of a road element are the predominant factors influencing drivers' reaction while traversing a curve.

1.5 Thesis Organization

In the second chapter of the thesis, a comprehensive literature review of the previous studies related to operating speed design consistency will be presented. In Chapter Three, the research methodology and data collection process will be described in detail. Chapter Four mainly deals with database preparation and preliminary analysis. In Chapters Five and Six the procedure for the development of operating speed and speed differential models will be fully explained, respectively. Finally, summary of the findings of the research and recommendations for future work will be presented in Chapter Seven.

Chapter 2: Literature Review

2.1 Introduction

As mentioned earlier, design consistency is defined as the conformance of roadway geometry with drivers' expectations (Nicholson, 1998). In other words, a consistent design does not violate the picture that the majority of drivers build in their minds of the upcoming features of the road ahead of them. There are several approaches that can be considered for design consistency evaluation namely, alignment indices, driver workload, vehicle stability or side friction, and operating speed (Fitzpatrick et al. 2000b; Gibreel et al. 1999). In view of comprehensiveness of the literature, several papers addressing the issue of design consistency from different points of view have been studied and their bottom lines will be spelled out in the following sections.

These approaches can be generally divided into two different categories (Fitzpatrick et al. 2000b): non-speed-based methods and speed-based methods. Non-speed-based methods contain alignment indices, driver workload, and vehicle stability or side friction as introduced by Gibreel et al. (1999). Speed-based methods, on the other hand, include speed distribution measures and operating speed methodology. Among the abovementioned design consistency evaluation measures, the operating speed methodology is of great significance in the literature due to its simplicity and quantifiability. In other words, this methodology is quite straightforward as it is dealing with the variations in the speed at which a driver is negotiating a specific portion of a highway. Having these speed variations in hand, a designer will be able to evaluate the consistency of that portion of highway using the related criteria. The question, then, would be about a methodology to predict the speed (or speed differential) values at some portions of highways with a relatively high possibility of speed variation. As an

example of such portions, a segment including both straight and curved sections (horizontal alignment) can be named.

Hence, more emphasis can be realized in the literature review on the operating speed methodology, as it is, also, the most relevant avenue to the objectives of this research. In this regard, the existing criteria for design consistency evaluation based on the operating speed methodology will first be explained. Then, the discussion will be narrowed down to the issue of developing the speed prediction models. In view of comprehensiveness of the literature, there would be a sub-section reflecting the design consistency issue in the North American design guides. In that section, the two design guides by AASHTO and TAC will be scrutinized in terms of their contribution to design consistency.

After reviewing the previous studies addressing the operating speed methodology, their shortcomings will be discussed in Section 2.6 of this chapter. In that section, especially, the common deficiencies of the existing speed prediction models will be highlighted. Addressing the pitfalls of the speed prediction models in the past 50 years will, in truth, pave the path for the present study.

2.2 Promising Aspects of Design Consistency

2.2.1 Safety Consideration

Gibreel et al. (1999) reviewed many studies that focused on the relationship between geometric features of the road (such as lane width, shoulder width, grade, radius, sight distance, and AADT) and the collision rate. They, then, concluded that the collision rate can be substantially diminished by widening the pavement width (e.g. between 22% to 47% decrease in collision rate together with the increase of pavement width from 5.5 m to 6.7 m).

Wilson (1968) stated that the radius of the curve plays a very significant role in the safety of the highways. He, for instance, reported that the collision rate on horizontal curves with $R < 170$ m was about five times greater than that of the curves with $R > 910$ m. Based on some series of safety analysis, Lamm et al. (1987; 1988) suggested the design consistency criterion as follows:

- Good Design: Collision rate < 2.27
- Fair Design: $2.27 < \text{Collision rate} < 5.00$
- Poor Design: Collision rate > 5.00

Another factor that was found to influence the collision rate was grade. Silyanove (1973), Choueiri et al. (1994), and Anderson et al. (1999) found that together with the increase of grade, the collision rate increases. Sight distance and traffic volume were recognized to have an impact on the accident rate. However, no criteria were established to quantify the relationship between these two factors, accident rate and design consistency (Gibreel et al. 1999).

Ng and Sayed (2003) developed eight accident prediction models to relate the individual design consistency measures to safety. The models showed improvement in the safety performance of the alignment as the consistency of the alignment was improved. Ng and Sayed (2003) also, compared the accident prediction models, which directly consider design consistency, with those that only use the geometric features of the road using artificial alignments. The comparison showed the superiority of the first group due to its potential in locating the inconsistencies.

2.2.2 Driver Anticipation

This approach introduced by Hirsh (1987), suggested that the design inconsistency occurs when the changes in highway alignment are not in accordance with driver's anticipation. Gibreel et al. (1999) stated that the driver anticipation is related to

the available sight distance. To their opinions, any unexpected hidden element such as sharp horizontal curve after a crest vertical curve might increase collisions, and therefore should be avoided.

2.2.3 Highway Aesthetics

According to Easa (1995), "Exact adherence to design standards does not guarantee obtaining a satisfactory and aesthetically pleasant design". Gibreel et al. (1999) stated that the consistent appearance of highway has a direct or indirect impact on driver's behaviour and performance. As examples of inconsistency based on highway aesthetics, short horizontal curves between tangents or long tangents with constant grades can be named (Gibreel et al. 1999).

2.3 Non-Speed-Based Methods

Non-speed-based methods refer to those approaches for design consistency evaluation considering features of the road and/or the driver other than speed. This category includes such avenues as alignment indices, driver workload, and vehicle stability or side friction (Fitzpatrick et al. 2000b; Gibreel et al. 1999). The detailed explanation of these avenues is presented in this section.

2.3.1 Alignment Indices

"Alignment indices are quantitative measures of a general character of a roadway segment's alignment that appear to have several conceptual advantages for use in design consistency evaluation" (Fitzpatrick et al. 2000b). According to Fitzpatrick et al. (2000b), there are at least two advantages for the utilization of alignment indices as a design consistency evaluation technique. First, the formulation is straightforward and easy to use. Second, the alignment indices provide a mechanism for quantitative evaluation of successive components from a system-wide perspective that is ideal in

design consistency evaluation. Design inconsistencies may occur when there is a large increase/decrease in the values of alignment index for successive road elements, a high change rate in the index over some parts of a roadway, or a large difference between individual characteristics of highway and the average value of alignment index.

Among the alignment indices proposed by the researchers from different countries, or those that are thought to be potentially useful, four were found to be promising in terms of providing information about the individual features of the road (Fitzpatrick et al. 2000b). They are generally introduced as horizontal alignment indices, vertical alignment indices, and combined alignment indices. The horizontal alignment indices are the average radius of the horizontal curves, the ratio of maximum to minimum radius of the curves, and average tangent length. Average vertical curvature rate, on the other hand, can be named as a vertical alignment index. This index is defined as the average value of the ratios of length of sections and absolute values of algebraic difference in grades (Fitzpatrick et al. 2000b).

The rationale behind recommending the aforementioned radii measures (the first two indices) is that they are believed to be the indicators of the sharpness of the curves and, as a result, have a good relationship to both speed and collision frequency. According to Fitzpatrick et al. (2000b), the speed of the motorists on two-lane roads is constrained by the curves they encounter. Also, a review of accident spot maps showed that accidents tend to cluster on curves, specially the sharp ones (Kayser et al. 1986).

Using the average radius of the individual curves, each curve can be evaluated in terms of having conflict with the general character of the road and consequently showing inconsistency. The ratio of maximum and minimum radii, also, seems promising due to its ability to represent the consistency of the design in terms of having curves with similar radius (i.e., when the ratio approaches one). However, one should be prudent of the problems associated with the use of this ratio at the extreme values. For instance, the

ratio of maximum to minimum radii is the same for the cases with $R_{\max} = 900$ m, $R_{\min} = 450$ m and $R_{\max} = 250$ m, $R_{\min} = 125$ m, while it is hypothesized that driver's reaction would be different in these two situations (Fitzpatrick et al. 2000b). As a result, the use of the other alignment indices is preferred over this one.

The average tangent length can be thought of as a measure to which the length of each tangent can be compared to understand how far it is from drivers' expected tangent length (average tangent length). The average vertical curvature rate can be thought of as a representative of hilliness value that might affect speed and collision data.

After preliminary evaluation of the recommended alignment indices, it was necessary to build regression models to see if there was a strong relationship between each alignment index and safety. Therefore, statistical models were developed by Fitzpatrick et al. (2000b) and Anderson et al. (1999) to predict accident frequency as a function of exposure and section length (both on the logarithmic scale), and each alignment index.

These models showed that among the recommended alignment indices, average radius, CRR (defined as the ratio of an individual curve radius to the average curve radius), and average vertical curvature rate have statistically significant relationship with collision frequency. However, none of them established strong relationships like the speed prediction models that will be introduced later in Section 2.4.2. As a result, it seems logical if the alignment indices are used as a supplementary, rather than the primary, measure of design consistency evaluation.

2.3.2 Driver Workload

Previous studies showed that "the driver is almost continuously processing visual and kinaesthetic information, making decisions, and carrying out control movements"

(Fitzpatrick et al. 2000b). In fact, “the task of driving involves, among other things, tracking the lane or path selected by the driver. It requires a driver to visually evaluate the path ahead, predict the steering and speed control inputs necessary for maintaining the desired path, make the control inputs, then using visual feedback, manipulate the controls to compensate for deviation” (Fitzpatrick et al. 2000b). This process continues until the vehicle reaches the destination or brought to a stop. In case of an experienced driver, little visual information processing capacity is being devoted and many tasks are done automatically provided that the free-flow condition dominates.

“A consistent roadway geometry allows a driver to accurately predict the correct path while devoting little visual information capacity, thus allowing attention or capacity to be dedicated to obstacle avoidance and navigation” (Fitzpatrick et al. 2000b). The rationale behind the driver workload method is that the more difficult an element of a road, the greater the information processing required, and consequently the less consistent the element would be with other less complicated sections of the road.

Among different measures to assess the driver workload, *Vision Occlusion* method is introduced here due to its validity in the literature. In vision occlusion method, a driver wears an LCD visor. This visor is opaque unless a button underneath driver's foot is pressed, and consequently a signal is transmitted to the visor that makes it transparent (see Figure 2.1). Therefore, whenever the driver needs visual information of the road, he/she presses the button, and subsequently he/she will be able to have a glimpse of the road (e.g. for 0.5 second) after which the visor will become opaque again. Using such a device, the visual demand of a driver for a specific section of a road can be measured as the ratio of the glance length (e.g. 0.5 second) and the interval between two successive glances. In practice, an average value of the mentioned ratio is taken by repeating this test for specific number of times at the same section of the road (Fitzpatrick et al. 2000b). The relatively high values of visual demand show those

sections of the road that need more visual demand and, as a result, are less consistent with the road geometry as a whole.

Figure 2.1: LCD visor

The procedure described above is carried out by the driver himself/herself. On the other hand, in order to be able to see when the driver leaves lane or fails to move the steering wheel in time, another person (e.g. the experimenter) should conduct this task. In this case, the experimenter gradually makes the intervals between the glances longer and longer until the vehicle leaves the lane. The difference between the average values of the workload measured by the driver (driver-controlled vision occlusion) and the threshold value measured by the experimenter (experimenter-controlled vision occlusion) gives a tolerance value. The greater the tolerance, the more consistent the feature is with its preceding feature. The reason is that in the expected situation the driver would be able to summon more than enough resources to cope with the actual needs of the situation.

Conducting driver-controlled and experimenter-controlled vision occlusion approach, researchers have been able to do a *subjective rating* for various features of the road in terms of the degree of difficulty imposed on the driver. This method that is

carried out in *test tracks*, considers the value of one for a very easy task and the value of ten for an impossible task. In other word, rating = 1 means: no mental effort needed to drive; rating = 10 means: curves are too sharp to stay on the road.

The findings of the study by Fitzpatrick et al. (2000b) regarding the workload methodology can be summarized as follows:

- Several driver workload studies proved that high amounts of workloads, or workload changes, lead to an increased collision potential.
- Based on drivers' responses, it was found that driver workload is significantly affected by the geometric properties of roadway curves.
- The visual demand of the drivers were found to increase about 90 m from the beginning of the curve, reaches its maximum value near the beginning, remain almost constant through the curve, and gradually return to the baseline value after the end of the curve.
- Although driver workload is known as a promising method for design consistency evaluation, it is not vastly used due to the complexity of the method and necessary improvements. In truth, additional investigations (e.g. defining critical values for subjective rating, etc.) seem to be essential to make this approach more quantifiable and practical.

2.3.3 Vehicle Stability or Side Friction

Vehicle stability is one of the important factors that ensure the safety of the vehicles. As stated in the AASHTO design guide (2001), the stability of a vehicle can be maintained by the counteraction of three forces: centrifugal force, the downward component of the weight, and the side friction. The relationship among these forces is formulated as:

$$r = \frac{V^2}{127(e + f_s)} \quad (2.1)$$

Where

r = radius of the curve (m)

V = speed of a vehicle (km/h)

e = superelevation rate

f_s = side friction coefficient

According to Gibreel, et al. (1999), the validity of this formula is under question because of two reasons. First, it ignores the effect of vertical alignment (the effect of longitudinal grade is not considered in the equation). Second, the model based on which this equation has been developed is a point mass model that lacks the appropriate distribution of friction forces between different tires.

Design consistency criteria were developed by Lamm et al. (1995b) by establishing two separate formulas for *side friction supply* and *side friction demand* (Equations 2.2 and 2.3).

$$F_R = 0.082 + 4.692 \times 10^{-3} \times (V_D) - 7 \times 10^{-5} \times (V_D)^2 \quad (2.2)$$

$$F_{RD} = 0.253 + 2.33 \times 10^{-3} \times (V_D) - 9 \times 10^{-5} \times (V_D)^2 \quad (2.3)$$

Where

F_R = side friction supply

F_{RD} = side friction demand.

V_D = Design speed

The consistency of the design can then be evaluated based on the following criteria:

- Good Design: $F_R - F_{RD} \geq 0.02$
- Fair Design: $-0.02 \leq F_R - F_{RD} < 0.02$
- Poor Design: $F_R - F_{RD} < -0.02$

2.4 Speed-Based Methods

Speed-based methods can be generally divided into two different groups, speed distribution measures and operating speed approach. In these approaches, the signs of design inconsistency are sought in the variation and distribution of vehicular speeds. In other words, the variations of vehicular speeds are investigated either between two successive elements of the road (e.g., a tangent and a curve or two successive curves), or between the design speed and operating speed. Speed distribution measures consist of descriptive statistics of speed such as variance, standard deviation, coefficient of variation, skewness, and kurtosis. Operating speed methodology, on the other hand, addresses the consistency of a design using two methods: operating speed consistency and design speed consistency.

2.4.1 Speed Distribution Measures

Each driver has a desired speed at which he/she drives when not being impeded by other vehicles. Since the desired speed depends on many factors including urgency of the trip, weather, roadway condition, environment, roadway geometry, etc., it is normally replaced with a more easily quantified term. That is the operating speed, which can be calculated using a sample of free-flow vehicle speeds. In addition, the speed that drivers adopt depends on their perceptions of the road. As a result, it would be more reasonable if the utilization of statistical measures such as mean, variance, standard deviation, etc. is considered together with free-flow speed calculation (Fitzpatrick et al. 2000b).

The rationale behind the use of speed distribution measures is that it is hypothesized that a speed sample measured under free-flow conditions, is normally distributed. Consequently, any deviation from a perfect normal curve (kurtosis=3,

skewness=0) or significant change in speed distribution may indicate the presence of design inconsistency in an alignment feature (Fitzpatrick et al. 2000b).

According to Fitzpatrick et al. (2000b), there are six descriptive speed statistics used as speed distribution measures: mean speed, variance, standard deviation, coefficient of variation, skewness, and kurtosis. Mean speed is defined as the arithmetic average spot speed of a sampling population (km/h). Variance is defined as a measure of variability of speeds about the mean (km/h)², while the standard deviation is defined as the square root of the variance (km/h). This measure is typically used to estimate the proportion of speeds that falls within a specific range about the mean. Coefficient of variation is defined as the standard deviation divided by the sample mean. It is used to normalize two different samples with different means and standard deviations.

Skewness is defined as the third moment of distribution, and it describes the symmetry of the distribution around the mean. In other words, the value of zero signifies a distribution with the equal number of values less than and greater than the mean, a negative value signifies a distribution with more values above the mean, and a positive value signifies a distribution with more values lower than the mean. Kurtosis is defined as the fourth moment of distribution. It is, in fact, a measure of the peakedness (or flatness) of the distribution. In other words, the value of three means the normal peak and any value below or above that signifies a sharp (leptokurtic) or flat (platykurtic) distribution, respectively.

As mentioned before, it is believed that high speed variances imply inconsistent design characteristics, while low speed variances show that the features are in harmony with driver expectancy (Fitzpatrick et al. 2000b). In order to investigate the veracity of this belief, the following four hypotheses were studied by Fitzpatrick et al. (2000b):

- The mean speeds are significantly different between the tangents and horizontal/vertical curves. However, preliminary analysis showed that no significant differences established between the mentioned elements of the road.
- There is a relationship between the design speed or posted speed and the speed distribution measures such as standard deviation, skewness, and kurtosis. As a preliminary analysis, design speed was compared against standard deviation, skewness, and kurtosis but resulted in no clear trends in the data set. However, the analysis showed that higher posted speed limits typically had higher standard deviations and mean speeds. The skewness was negative for higher posted speeds and kurtosis was higher for the higher posted speed locations. It should be noted that none of the mentioned relationships between speed distribution measures and the posted speed were found to be statistically significant.
- Speed distribution measures show the same behaviour at the curves and their preceding tangents. Therefore, if there is a high variability in the speed sample at the middle of the tangent, then there is a high variability at the middle of the following curve. However, no strong relationship was recognized between speed variance and the features of the successive elements.
- As the radius of a horizontal curve decreases, the standard deviation of the speed increases. This, in turn, shows some degree of inconsistency with the preceding features of the road. The study showed that the speed variance in the curve is 9.5% lower than that of preceding tangent. This finding is in clear conflict with this fourth hypothesis.

As a result, the hypothesis that speed distribution measures can be used as an alternative design consistency evaluation tool was rejected. Such measures can only be used as supplementary measures to the operating speed methodology. The findings of

the study by Fitzpatrick et al. (2000b) and Gibreel et al. (1999) explicitly encourage designers to use the operating speed methodology as the most efficient consistency measure to date. A detailed explanation of this methodology is presented in the next section.

2.4.2 Operating Speed Methodology

As a result of several previous studies, it has been widely accepted that the most practical consistency measure is the operating speed approach. The other aforementioned methodologies, on the other hand, might be implemented just as supplementary approaches to this avenue. Since this study focuses mainly on the operating speed approach, this part of the literature review is allocated to the detailed investigation of this methodology.

In this section, the existing criteria for evaluating design consistency, based on the operating speed approach, will first be explained. Among these criteria, those established by Leisch and Leisch (1977), and those reflected in the design guides of Switzerland, Germany, and Australia, can be named. The focus will then be narrowed down to address the main objectives of this study; namely, the development of operating speed and speed differential models. In this connection, the details of the development of the previous speed prediction models will be elucidated. In addition, the different models developed in previous studies, will be compiled in Table 2.1 at the end of this section.

Leisch and Leisch (1977) suggested, for the first time in the U.S., the concept of design consistency. They concluded that the design speed concept in the existing design guide did not guarantee the consistency of highway alignments. Leisch and Leisch (1977) believed that the problem manifests itself particularly when the design speed is lower than 90 km/h. They believed that a designer should try to design a road,

which conforms to drivers' expectations and complies with his/her inherent traits in order to achieve operational consistency and consequently improve comfort and safety.

According to Leisch and Leisch (1977), the main problem with the design speed is that it is sometimes lower than driver expectancies and a good judgment of what the reasonable speed is. Thus, they introduced the following criterion to ensure the consistency of design:

- Within a given design speed, the potential average automobile speed should not vary by more than 15 km/h.
- When a reduction in design speed is necessary, it should be normally not more than 15 km/h.
- On common lanes, the potential average truck speed should be generally not more than 15 km/h lower than the average automobile speeds.

The rationale behind the use of 15 km/h as a critical value stems from past experience that a driver would be able to cope with speed adjustment of 15 km/h in most circumstances (Leisch and Leisch 1977). The Leisch's method involves using a speed profile technique in order to be able to visualize the sections of the road that are in conflict with the general character of the road and consequently show inconsistency. To reach this goal, Leisch and Leisch (1977) established the following procedure:

- 1) The average running speed on tangents should be determined in accordance with the low-volume relations between the average running speed and design speed depending on the type of highway (interstate, rural, etc.) and its quality (favourable, moderate) from the AASHTO guide (1965).
- 2) The average running speed on the horizontal curves should be determined according to low-volume relations between the average running speed and design speed depending on the radius of the curve from the AASHTO guide (1965).

- 3) The results of the first two steps should be combined with the acceleration and deceleration rates for passenger cars or trucks as well as the required acceleration/deceleration distances taken from nomograms that were adapted and extrapolated from the AASHTO guide (1965).
- 4) The speed profile could be drawn using the information obtained from aforementioned steps and the consistency of the design can be evaluated for any stretch of the road using the 15 km/h rule.

The interesting point about the Leisch's method is that it considers both horizontal and vertical alignment in developing the speed profile. This happens while even the recently developed models, except for the two developed by Gibreel et al. (2001) and Fitzpatrick et al. (2000a), do not account for the 3D alignment effect.

According to Lamm et al. (1986), Switzerland was one of the first countries that applied the speed profile models in the geometric design of highways. In the Swiss method (*Highway Design Fundamentals*, 1981), the design speed is chosen based on the importance of the road, the traffic volume, and topography. In addition to the design speed, a theoretical design speed model, called *project speed*, is used to evaluate the consistency of the design. In fact, project speed in the Swiss method has the same meaning as the operating speed in the United States. Standard values for the project speed are determined through field research and tabulated for various curve radii.

According to the Swiss method, the speed is assumed to remain constant over the length of the curve. The formulas were, also, developed to calculate the acceleration/deceleration rates and their comparable required distances. Having this information and considering a 20 km/h cut-off value as a consistency criterion, designers would be able to spell out the portions of the road that show inconsistency. These portions are marked for more investigation of collision experience. If the studies show

significant collision experience, redesigning and realigning the curve may be considered (Lamm et al. 1986).

Lamm et al. (1986) also reported that in the German design practice, the design speed is chosen similar to the Swiss method and is used to design the elements of a road. Then, designers use the operating speed values to control the designed features. Since the operating speed is normally higher than the design speed, it can be used as a surrogate for the design speed to determine the adequate superelevation rate and necessary stopping sight distance. Operating speed is defined as the 85th percentile speed of the passenger cars under free-flow condition for a clean, wet road surface (*Guidelines for Design of Roads, 1984*).

The difference between the German and Swiss methods is that in the German method a parameter, called curvature change rate (CCR), is used to consider the overall highway homogeneity (the advantage of German method). This homogeneity is believed to prevent abrupt transitions in operating speeds between the features of a roadway. CCR is defined as the absolute sum of the angular changes in the horizontal alignment divided by the length of the highway section (Lamm et al. 1986).

$$CCR = \frac{57300}{L} \left(\sum \frac{L_{ci}}{r_i} + \sum \frac{L_{sj}}{2r_j} \right) \quad (\text{Deg/Km}) \quad (2.4)$$

Where

L = total length of the curve (m)

L_c = length of circular curve (m)

r = radius of the curve (m)

L_s = length of spiral curve (m)

German researchers found that the operating speed remains relatively constant over the length of a section with similar characteristics. They also learned that there is a strong relationship between the operating speed and CCR (Lamm et al. 1986).

Operating speed values can be obtained using nomograms relating 85th percentile speed to *CCR* and the pavement width. Having in hand these data, a designer would be able to draw the speed profile of the specific proportion of a highway and then point out the inconsistent locations using the following criteria:

1. Operating speed consistency criterion
 - Good Design: $V_{85(i+1)} - V_{85(i)} \leq 10 \text{ km/h}$
 - Fair Design : $10 \text{ km/h} < V_{85(i+1)} - V_{85(i)} \leq 20 \text{ km/h}$
 - Poor Design: $V_{85(i+1)} - V_{85(i)} > 20 \text{ km/h}$
2. Design speed consistency criterion
 - Good Design: $V_{85} - V_D \leq 10 \text{ km/h}$
 - Fair Design : $10 \text{ km/h} < V_{85} - V_D \leq 20 \text{ km/h}$
 - Poor Design: $V_{85} - V_D > 20 \text{ km/h}$

In Australia, McLean (1979) studied 120 horizontal curves on two-lane rural roads. He concluded that in case of having a design speed greater than 110 km/h in rolling terrain or 120 km/h in flat terrain, the 85th percentile speed tend to be lower than the design speed. For the design speed of 90 km/h or less, on the other hand, the 85th percentile speed is in excess of the design speed. Based on his findings, the design procedure for horizontal alignments on highways with lower design speeds ($V_D < 100 \text{ km/h}$) was changed and the design speed was replaced by the 85th percentile speed. The 85th percentile speed is estimated based on the curve radius and the desired speed on the highway (see Table 2.1). The desired speed is defined as the 85th percentile observed speed on the tangent sections (McLean, 1979) and can be obtained from the Australian design guide.

As a criterion to evaluate the consistency of the features, McLean (1979) recommended a maximum desirable difference of 10 km/h between the long tangent

(more than 250 m) and the curved sections. The value of 15 km/h, on the other hand, was stated as an absolute maximum difference between the speeds of the aforesaid elements.

Lamm and Choueiri (1987) modified the German method for use in the United States. Having the basic concepts from the German method, Lamm and Choueiri (1987) started collecting the speed data in the state of New York from June 1984 till May 1986. The data included spot speed data of passenger cars for 261 curves using an electronic stop watch and transverse pavement marking. They used these data for the purpose of establishing regression models that were capable of predicting the 85th percentile speeds at the curved sections (see Table 2.1). They also recommended the intercept of the speed model (94.4 km/h) as a proper value for the speed in tangents. The criteria used for site selection were as follows (Lamm and Choueiri, 1987):

- No influence of intersections.
- No physical features like narrow bridges that create abnormal hazard.
- Constant pavement and shoulder width.
- Delineated pavement with paved shoulder.
- Grades less than 5%.
- AADT: 400 – 5000 vehicles /day.

Krammes et al. (1995) tried to establish speed prediction models for the curves using spot speed data of passenger cars for 284 sites (50 to 100 observations for each site). These sites include 138 curves, 78 paired tangent-curves, and 68 tangents whose speed data were collected using a Radio Detecting And Ranging (RADAR) gun. The models were improved later on in 1997 in order to be used in the “design consistency module” of the Interactive Highway Safety Design Model (IHSDM) (Krammes, 1997). For that purpose, Krammes (1997) calibrated the previously developed models using the

data of 29 highways in five different states in the U.S. (see Table 2.1). It should be noted that the authors were not able to develop any model to predict the operating speed on tangents; therefore, a value of 97.9 km/h was introduced as the mean of the 85th percentile speeds on the tangents. The site selection criteria were the same as those of the study by Lamm and Choueiri (1987) plus the fact that the pavement condition should be "good".

Al-Masaeid et al. (1995) introduced another set of regression equations for predicting the speed of the vehicles. The equations presented in that study can be used to directly predict the speed reduction between two successive elements (curve-tangent or curve-curve) for different classes of vehicles (see Table 2.1). However, these models were developed using the observed values of $(V_{85(1)} - V_{85(2)})$. This methodology has the same disadvantage of assuming that the 85th percentile driver on the first element is the same 85th percentile driver on the second element. In addition, the speed distributions on two different features of the road are different from each other (Hirsh, 1987). The speed database included spot speed data of passenger cars, light trucks and heavy trucks for 93 sites in Jordan (57 simple curves + 36 continuous curves with non-independent tangents) collected by 40-m speed traps. The site selection criteria were the same as those of the study by Lamm and Choueiri (1987) plus the fact that the pavement condition should be the same between two successive elements.

Collins and Krammes (1996) validated the equations obtained by Krammes et al. (1995). The speed database included spot speed data of passenger cars, for ten curves collected by seven photoelectric sensors (speed traps) in Texas. In addition, they concluded that the assumption of having constant speed through the curve might not be realistic. The site selection criteria were the same as those of the study by Lamm and Choueiri (1987).

Abdelwahab et al. (1998) presented another regression equation for speed reduction from tangent to curve for passenger cars (see Table 2.1). The speed database included 46 curves (minimum 35 observations per site) from five different highways in Jordan. The speed data were collected using a stop watch. A maximum allowable speed reduction value of 10 km/h was suggested by the authors as a criterion for operating speed consistency evaluation. The same problem mentioned for the study by Al-Masaeid et al. (1995), the subtraction of the 85th percentile values for two points with different speed distributions, also exists in this study. The criteria for site selection were the same as previous studies.

Passetti and Fambro (1999) tried to develop speed prediction models for passenger cars on horizontal curves (see Table 2.1). The speed database contained 51 curves (39 simple curves + 12 curves with spiral) from six states in the U.S. (minimum 100 observations per site). The equipment used was a counter/classifier equipped with piezoelectric sensors. The authors also compared the speed at the curves with and without spirals and found no statistically significant difference. The criteria for site selection were the same as previous studies.

Fitzpatrick et al. (2000a) developed speed prediction models for passenger cars taking into consideration the effect of 3D alignments (see Table 2.1). The speed database included 176 sites in six states in the U.S. (Minimum 100 observations per site). The equipment used for data collection consist of radar gun, Light Detecting And Ranging (LIDAR) gun (a new generation of the radar gun), and counter/classifier with on-pavement piezoelectric sensors. The developed models would be finally employed in the design consistency module of IHSDM. Due to the insufficiency of the observed speeds for trucks or recreational vehicles (almost ten observations per site), no specific models were developed for these types of vehicles; rather, the established passenger car models were verified for other vehicle types. However, Fitzpatrick et al. (2000a)

utilized the TWOPAS microscopic model to study the restraining effect of upgrades on the vehicles. This model helps researchers understand whether the speed given by the speed prediction models can be achieved. Similar to the study by Passetti and Fambro (1999), Fitzpatrick et al. (2000a) did not find any statistically significant difference between the curves with and without transitions.

In addition, the authors developed some equations for the acceleration/deceleration rates of the vehicles negotiating the curved sections where the commonly used value of 0.85 m/s^2 was found to be an overestimation of the actual driver's acceleration/deceleration performance. Among the 176 sites whose speed data were available, 103 sites were used for model development, 41 sites were used to cross-validate the models of this study, and 32 sites were used to validate the models developed by Krammes et al. (1995). The criteria used for site selection were as follows:

- Not in proximity of towns or developed areas.
- Lane width: 2.74 m – 3.66 m.
- Radius of curve: 110 m – 3500 m.
- AADT: 500 – 4000 Veh/day.
- Longitudinal grade: –10% to +10%.
- Design speed $\leq 120 \text{ km/h}$.
- Posted speed: 75 km/h – 115 km/h

Andueza (2000) presented regression models to predict the speed of the passenger cars on curves and tangents (see Table 2.1). The speed database included 39 sites (21 curves + 18 tangents) whose speed data were collected by a radar gun in Venezuela (30 to 64 observations for each site). The criteria used for site selection were as follows:

- Lane width = 3.65 m.

- Shoulder width ≥ 1.2 m.
- Longitudinal Grade $\leq 3\%$.

McFadden and Elefteriadou (2000) presented another regression equation for speed reduction from tangent to curve (see Table 2.1). Including 21 sites with a minimum of 75 observations per site, the speed database was a portion of the big database mentioned earlier in the study by Fitzpatrick et al. (2000a). Unlike other related models, McFadden and Elefteriadou (2000) presented a more realistic methodology to measure the speed reduction between the tangent and curved sections of a highway. They, in fact, introduced a new variable, 85MSR, defined as the 85th percentile maximum speed reduction from the middle of the tangent to the middle of the curve. To them, the 85MSR is, on average, two times greater than the difference between the 85th percentile speeds of the points at the middle of the tangent and the curved sections. For the precise investigation of speed reduction behaviour of the drivers, McFadden and Elefteriadou (2000) compared three different kinds of models as follows:

- The model that estimates the difference between the 85th percentile speed at the middle of the curve and the middle of the tangent (85S2).
- The model that estimates the difference between the maximum 85th percentile tangent speed and minimum 85th percentile curve speed using nine points on each curve; namely, middle of the curve, quarter one and quarter three of the curve, point of tangency, point of curvature, and four other points before the point of curvature at every 50 m (85S9) (see Figure 2.2).
- The model that estimates 85th percentile maximum speed reduction from the middle of the tangent to the middle of the curve using each individual driver speed profile (85MSR).

The comparison of the speed reduction values estimated by any of these three approaches showed that the 85S2 (the method which is currently used to evaluate the consistency of the design) gives the lowest speed reduction among all methods. The highest value, on the other hand, can be obtained using 85MSR. In addition, the hypothesis testing were carried out and proved that there was a significant difference between all the three aforementioned models at the 95% confidence level.

Figure 2.2: Data collection locations at each curve
(McFadden and Elefteriadou, 2000)

McFadden and Elefteriadou (2000) used a new generation of the radar gun, called lidar gun, for the purpose of speed data collection. This equipment has two distinguished advantages over the radar gun. First, for every reading it gives the exact location of that reading and makes the researcher able to track a vehicle at different points of the curve. It is noteworthy that this is an advantage over the ordinary types of

the radar guns and not the professional one like STALKER ATS that has this capability in its software. Second, the lidar gun takes advantage of LASER beams that are moving in a straight line compared to Radio frequency beams that are being spread in a conical shape. Thus, the readings of a lidar gun are more accurate than those of the radar gun due to not being distracted by other vehicles.

Hassan et al. (2000) verified the models developed by Lamm and Choueiri (1987) and Krammes et al. (1995) using the data from six curves in Highway 61 in Canada. For the purpose of speed data collection, a radar gun was used for 30 minutes at each curve. The methodology to verify the previous models was to calculate the allowable speed of vehicles based on 3D sight distance, vehicle stability, and driver comfort and then, compare these values with the predicted operating speeds. Whenever the allowable speeds were larger than the predicted operating speeds, the alignment was called safe. In this regard, the model by Krammes et al. (1995) was preferred over the other model. Hassan et al. (2000) finally concluded the necessity of considering 3D alignment in developing operating speed models. They also recommended the value of 102.0 km/h as an average 85th percentile speed value for use on tangents.

Polus and Fitzpatrick (2000) tried to develop the speed prediction models for tangent sections. Using the same database as the study by Fitzpatrick et al. (2000a), the authors divided the database to two main categories, short and long tangent. Then, they introduced a parameter, called Geometric Measure of Tangent Section (GM) for each category. GM was introduced as a function of the radii of the preceding (R_{Pre}) and succeeding curves (R_{Suc}) and the tangent length (TL). The inclusion of GM in a speed prediction model resulted in a relatively low relationship to the dependent variable (R^2 between 0.228 and 0.322).

As a result, Polus and Fitzpatrick (2000) decided to categorize the database to four categories with more similar conditions. These four categories were small radii and

small tangent length, small radii and intermediate tangent length, intermediate radii and tangent length, and large tangent length and any radii. Models were developed for each of the four mentioned categories and showed higher correlation to *GM* (R^2 between 0.553 and 0.838). However, two deficiencies can be stated about the mentioned models. First, they were developed based on very low number of cases (seven cases for one of the models). Second, the maximum value considered for the tangent speed varies from 98.4 km/h to 105 km/h that does not seem to be reasonable (many drivers may drive faster).

Gibreel et al. (2001) presented a set of speed prediction models for passenger cars on curves taking into consideration the effect of 3D alignment (see Table 2.1). The speed database contained nineteen curves (10 crest combinations + 9 sag combinations) whose speed data were collected by a radar gun on Highways 61 and 102 in Ontario, Canada. In the study by Gibreel et al. (2001), the sag combinations were the horizontal curves overlapping with a sag vertical curve. The same interpretation is true about the crest combinations. In view of the accuracy of the equations, five different equations were established for each curve. These equations predict the vehicles' speed at 60 m to 80 m before the point of tangent-to-spiral, point of spiral-to-curve, point of the middle of the curve, point of curve-to-spiral, and 60 m to 80 m after the point of spiral-to-tangent separately.

Cross-validation of the models was carried out using three curves out of nineteen curves (the other sixteen curves had been used for model development). The results of this validation showed a very good conformance between the predicted and observed speed values. However, verification of other earlier models did not reach satisfactory results. The result of the study by Gibreel et al. (2001) proved that 2D models introduced by Lamm et al. (1990), Islam and Seneviratne (1994), Krammes

(1997), etc. were far removed from the observed data; they either highly overestimated or underestimated the actual vehicles' speed. The site selection criteria were as follows:

- No influence of intersection.
- No influence of other adjacent sections.
- No physical features that create an abnormal hazard such as narrow bridges.
- Marked pavement and paved shoulder.
- Guardrail protected sections in case of having embankment height greater than 3 m.
- Pavement width ≥ 7.5 m.
- Shoulder width ≥ 1.8 m.
- Longitudinal grade $\leq 6\%$.
- AADT $\leq 10,000$ veh./d.
- Design Speed ≤ 110 km/h.

Jessen et al. (2001) presented two different speed prediction models for vertical crest curves; one for the case of limited sight distance and the other one for the case of non-limited sight distance (see Table 2.1). The speed database was formed using a counter/classifier for speed data collection of passenger cars on 70 sites in Nebraska. Jessen et al. (2001) showed that the posted speeds in highways have the most influence on the operating speed of the vehicles. Cross-validation of the models was carried out using eight sites out of 70 sites.

A summary of the previously developed speed prediction models are reflected in Table 2.1. This table includes author, year of development, types of vehicles, country, and regression models together with their coefficients of determinations. The meanings of all abbreviations are shown in the nomenclature.

Table 2.1: Previously developed speed prediction models

No.	Author	Country	Vehicle Type	Equations	R ²
1	Taragin (1954)	U.S.A.	PC	$V_{85} = 88.87 - \frac{2554.76}{R}$	0.860
2	McLean (1978)	Australia	PC	$V_{85} = 101.20 - 0.0675CCR = 101.2 - \frac{2730}{R}$	0.870
3	McLean (1979)	Australia	PC	$V_{85} = 53.80 + 0.464V_F - \frac{3260}{R} + \frac{85000}{R^2}$	0.920
4	Kerman et al. (1982)	U.K	PC	$V_{85} = V_a - \frac{V_a^3}{398R}$	0.910
5	Guideline (1984)	Germany	PC	$V_{85} = 60 + 39.70 \times e^{-0.00358CCR}$ LW=3.5 m	0.790
6	Glennon et al. (1985)	U.S.A.	PC	$V_{85} = 103.96 - \frac{4524.94}{R}$	0.840
7	Setra (1986)	France	PC	$V_{85} = 102/[1 + 346/(57300/CCR)^{1.5}]$	N/A
8	Lamm and Choueiri (1987)	U.S.A.	PC	$V_{85} = 88.72 - 0.084CCR$ LW=3.0 m	0.846
				$V_{85} = 89.55 - \frac{2862.69}{R}$	0.753
				$V_{85} = 92.69 - 0.080CCR$ LW=3.3 m	0.731
				$V_{85} = 93.83 - \frac{2955.40}{R}$	0.746
				$V_{85} = 95.77 - 0.076CCR$ LW=3.6 m	0.836
				$V_{85} = 96.15 - \frac{2803.70}{R}$	0.824
				$V_{85} = 94.39 - \frac{3188.57}{R} = 93.85 - 0.045CCR$	0.787
				$V_{85} = 55.84 - \frac{2809.32}{R} + 0.634LW + 0.053SW + 0.0004AADT$	0.842

Table 2.1: Previously developed speed prediction models (Cont'd)

No.	Author	Country	Vehicle Type	Equations	R ²
9	Kanellaidis et al. (1990)	Greece	PC	$V_{85} = 109.09 - \frac{3837.55}{R}$	0.647
				$V_{85} = 32.20 + 0.839 V_d + \frac{2226.9}{R} - \frac{533.6}{\sqrt{R}}$	0.925
				$V_{85} = 129.88 - \frac{623.1}{\sqrt{R}}$	0.777
10	Lamm (1993)	Germany	PC	$V_{85} = 10^6 / (8270 + 7.20CCR)$	0.730
11	Ottesen and Krammes (1994)	U.S.A.	PC	$V_{85} = 103.04 - 0.0477CCR = 103.70 - \frac{3403}{R}$	0.800
12	Talarico and Morrall (1994)	Canada	PC	$V_{85} = e^{(4.561 - 0.00566DC)}$	0.631
13	Islam and Seneviratne (1994)	U.S.A.	PC	$V_{85} = 95.41 - 1.48DC - 0.012(DC)^2$	0.990
				$V_{85} = 103.30 - 2.41DC - 0.029(DC)^2$	0.980
				$V_{85} = 96.11 - 1.07DC$	0.980
14	Krammes et al. (1995)	U.S.A.	PC	$V_{85} = 103.66 - 1.95DC$	0.800
				$V_{85} = 102.45 - 1.57DC + 0.0037L_c - 0.10 DF$	0.820
				$V_{85} = 41.62 - 1.29DC + 0.0049L - 0.12 DF + 0.95V_{tangent}$	0.900
15	Lamm et al. (1995a)	Greece	PC	$V_{85} = 10^6 / (10150.1 + 7.676CCR)$	0.810
16	Choueiri et al. (1995)	Lebanon	PC	$V_{85} = 91.03 - 0.050CCR$	0.810

Table 2.1: Previously developed speed prediction models (Cont'd)

No.	Author	Country	Vehicle Type	Equations	R ²
17	Al Masaeid et al. (1995)	Jordan	PC, LT, HT	$\Delta V_{85PC} = 3.64 + 1.78DC$	0.510
				$\Delta V_{85LT} = 2.00DC$	0.690
				$\Delta V_{85T} = 4.32 + 1.44DC$	0.420
				$\Delta V_{85ALL} = 3.30 + 1.58DC$	0.620
				$\Delta V_{85ALL} = 1.84 + 1.39DC + 4.39PC_{Con} + 0.07G^2$	0.770
				$\Delta V_{85ALL} = \frac{5081}{R_2} - \frac{5081}{R_1}$	0.810
18	Voigt (1996)	U.S.A.	PC	$\Delta V_{85ALL} = 108.30 - \frac{3498}{L_T} - 0.71(\frac{DF_1 \times DF_2}{DF_1 + DF_2})$ Common Tangents	0.720
				$V_{85} = 99.61 - \frac{2951.37}{R}$	0.840
19	Abdelwahab et al. (1998)	Jordan	PC	$\Delta V_{85} = 0.9433DC + 0.0847 DF$	0.920
20	Passetti and Fambro (1999)	U.S.A.	PC	$V_{85} = 103.90 - \frac{3020.50}{R}$	0.680
21	Fitzpatrick et al. (2000a)	U.S.A.	PC	$V_{85} = 106.30 - \frac{3595.29}{R}$ HC: $0 \leq G < 4$ or, HC + Sag VC	0.920
				$V_{85} = 96.46 - \frac{2744.49}{R}$ HC: $4 \leq G < 9$	0.560
				$V_{85} = 100.87 - \frac{2720.78}{R}$ HC: $-9 \leq G < 0$	0.590
				$V_{85} = 101.90 - \frac{3283.01}{R}$ HC + LSD Crest VC	0.780

Table 2.1: Previously developed speed prediction models (Cont'd)

No.	Author	Country	Vehicle Type	Equations	R ²
21	Fitzpatrick et al. (2000a)	U.S.A.	PC	$V_{85} = 111.07 - \frac{175.98}{K}$	0.540
				$V_{85} = 100.19 - \frac{126.07}{K}$	0.680
22	Ottesen and Krammes (2000)	U.S.A.	PC	$V_{85} = 102.44 - 1.57DC - 0.012L_c - 0.01DCxL_c$	0.810
23	Andueza (2000)	Venezuela	PC	$V_{85} = 98.25 - \frac{2795}{R} - \frac{894}{R_a} + 7.486DC + 9.308L_T$	0.840
				$V_{85} = 100.69 - \frac{3032}{R} + 27.819L_T$	0.850
24	McFadden and Elefteriadou (2000)	U.S.A.	PC	$85MSR = -14.90 + 0.144V_{85T} - \frac{954.55}{R} + 0.0153L_T$	0.712
				$85MSR = -0.812 + \frac{998.19}{R} + 0.017L_T$	0.603
				$V_{85(S1)} = 91.81 + 0.010R + 0.468\sqrt{L_v} - 0.006G_1^3 - 0.878\ln(A) - 0.826\ln(L_o)$	0.980
				$V_{85(S2)} = 47.96 + 7.217\ln(R) + 1.534\ln(L_v) - 0.258G_1 - 0.653A - 0.008L_o + 0.020\exp(e)$	0.980
25	Gibreel et al. (2001)	Canada	PC	$V_{85(S3)} = 76.42 + 0.023R + 0.00023K^2 - 0.008\exp(A) - 0.000123L_o^2 + 0.062\exp(e)$	0.940
				$V_{85(S4)} = 82.87 + 0.011R + 2.067\ln(K) - 0.361G_2 - 0.0001091L_o^2 + 0.036\exp(e)$	0.950
				$V_{85(S5)} = 109.45 - 1.257G_2 - 1.586\ln(L_o)$	0.790

Table 2.1: Previously developed speed prediction models (Cont'd)

No.	Author	Country	Vehicle Type	Equations	R ²
25	Gibreel et al. (2001)	Canada	PC	$V_{85(C1)} = 82.29 + 0.003R - 0.05 DF + 3.441 \ln(L_v) - 0.533G_1 + 0.017 \exp(e) - 0.000097(L_0)^2$	0.940
				$V_{85(C2)} = 33.69 + 0.002R + 10.418 \ln(L_v) - 0.544G_1 + \frac{8.699}{[\ln(A+1)]} + 0.032 \exp(e) - 0.011L_0$	0.970
				$V_{85(C3)} = 26.44 + 0.251\sqrt{R} + 10.381 \ln(L_v) - 0.423G_1 + \frac{6.462}{[\ln(A+1)]} + 0.051 \exp(e) - 0.028(L_0)$	0.980
				$V_{85(C4)} = 74.97 + 0.292\sqrt{R} + 3.105 \ln(K) - 0.850G_2 + 0.026 \exp(e) - 0.00017(L_0)^2$	0.900
				$V_{85(C5)} = 105.32 - 0.418G_2 - 0.123\sqrt{L_0}$	0.830
26	Jessen et al. (2001)	U.S.A.	PC	$V_{85} = 86.80 + 0.297 V_P - 0.614G_1 - 0.00239ADT$	LSD 0.540
				$V_{85} = 72.10 + 0.432 V_P - 0.00212ADT$	NLSD 0.420
27	Donnell et al. (2001)	U.S.A.	T	$V_{85(1)} = 56.1 + 0.117R - 1.15G_1 + 0.006L_{T1} - 0.000097(L_{T1})(R)$	0.613
				$V_{85(2)} = 78.4 + 0.0140R - 1.40G_2 - 0.00724L_{T2}$	0.562
				$V_{85(3)} = 75.1 + 0.0176R - 1.48G_2 - 0.00836L_{T2}$	0.600
				$V_{85(4)} = 74.5 + 0.0176R - 1.69G_2 - 0.00810L_{T2}$	0.611
				$V_{85(5)} = 83.1 - 2.08G_2 - 0.00934L_{T2}$	0.577

2.5 Design Consistency in North American Design Guides

After reviewing the various methods recommended for design consistency evaluation, it seems timely to look at the way the North American design guides consider this issue. In this connection, two references are studied here; namely the latest version of the AASHTO Green Book (2001) and the latest version of the Canadian guide (TAC, 1999).

2.5.1 AASHTO Green Book (2001)

AASHTO Green Book (2001) considers the consistency of the design implicitly. In its third chapter, "Elements of Design", in a section called, "General Controls for Horizontal Alignment", the following statements can be found:

- "In alignment developed for a given design speed, the minimum radius of curvature for that speed should be avoided wherever practical. The designer should attempt to use the generally flat curves, saving the minimum radius for the most critical conditions."
- "Consistent alignment should always be sought."
- "Sharp curves should not be introduced at the ends of long tangents."
- "Sudden changes from areas of flat curvature to areas of sharp curvature should be avoided."
- "Where sharp curvature is introduced, it should be approached, where practical, by a series of successively sharper curves."

As it can be realized from the above statements, there is no clear explanation or quantified formulation about the evaluation of design consistency in the AASHTO Green Book. In other words, the interpretation of some words remains vague, e.g., "sharp" and

“flat” curvature, or “long tangents”. As a result, it is up to the designer how to consider the design consistency issue according to his/ her own judgment.

2.5.2 TAC Guide (1999)

TAC guide (1999) discusses the issue of design consistency explicitly in a separate chapter. Three principles were suggested for design consistency evaluation in TAC guide (1999): cross section consistency, driver workload consistency, and operating speed consistency. In order to consider cross section consistency, designers are encouraged to apply consistent cross section dimensions; namely lane and shoulder width. Driver workload consistency is just introduced in the TAC guide (1999), however, it is not recommended due to the fact that its parameters are difficult to quantify. Operating speed consistency, on the other hand, is introduced in a more quantitative manner. In this regard, the following statements have been cited in the TAC guide (1999):

- “Individual drivers vary their operating speed to adjust to features encountered along the road, such as intersections, accesses and curves in the alignment. The greater and more frequent are the speed variations, the higher is the probability of collision.”
- “A designer can enhance the safety of a road by producing a design which encourages operating speed uniformity.”

In an effort to ensure the operating speed uniformity of the design, prediction of the speed values for different highway geometric features is essential. Thus, the speed prediction model for the middle of the curve developed by Krammes et al. (1995) has been recommended in the TAC (1999). Using this model and an assumed value of 0.85 m/s^2 for acceleration and deceleration rate, designers would be able to draw the speed profile for a section of road including some curves and tangents. Looking at the speed

profile, a designer can determine the speed reduction between the two successive features of the road alignment. It is expected that the higher the speed reduction, the worse the safety performance of the road as shown in Figure 2.3 by TAC (1999). However, it is stated that the graph should be used with caution, because of the database used; in this regard, using the local data is recommended wherever possible.

What is lacking in the TAC guide (1999) is a clear criterion for design consistency evaluation. Therefore, the designer does not have a tool to translate the speed reduction to collision rate. As a result, the following three questions would still remain unanswered in the TAC guide:

- 1) Is the speed prediction model developed using the speed data of American roads, applicable to the Canadian roads?
- 2) Can the designer calculate the speed reduction while the speed distribution for the tangent section and the middle of the curve are different and the related 85th percentile speeds cannot be simply subtracted from each other?
- 3) Does the fact that speed reduction is proportional to collision rate mean that greater speed reduction is equivalent to more perilous section of a road?

Regarding the abovementioned unanswered questions in the TAC guide (1999), one can understand that the operating design consistency practice in Canada is still in its preliminary stages. Therefore, the author tries, in this study, to find some logical ways to answer those questions related to the deficiencies of speed prediction models. However, dealing with the criteria for design consistency evaluation is out of the scope of this study and can be stated as a recommendation for future studies.

Figure 2.3: Mean collision rate versus mean speed difference between geometric elements (TAC, 1999)

2.6 Pitfalls of Previous Speed Prediction Models

Looking at the speed prediction models developed before, it was realized that almost all the models suffer from common pitfalls as follows:

1. The majority of the models (25 out of 27 models) were developed concentrating on speed prediction for passenger cars and models, which can predict the speed for light trucks or heavy trucks are lacking. The only studies carried out with the purpose of estimating truck speed were the studies by Leisch and Leisch (1977), Al-Masaeid et al. (1995), and Donnell et al. (2001).
2. Although most of the research was carried out using a large database (e.g., a large number of curves), the number of observed vehicles for each curve ranged

from 30 to 100. As a result, the accuracy of the observed 85th percentile speed values is questionable.

3. The data collection device used to record vehicle speed was, in most of the cases, a manually operated radar gun. The utilization of radar gun is often accompanied with three problems. First, to record the speed of a vehicle, an observer should read from the LCD screen of the radar gun and another person should write those readings; as a result, human error may be introduced into the process of data collection. Second, the speed measured by a radar gun at the curved sections has some degrees of inaccuracy due to the fact that the reading angle is different from zero and is changing along the curve (cosine error). Third, it is almost impossible not to be visible by the drivers, and therefore the speeds obtained by this equipment might not be the representative of the actual speeds. Thus, much care should be employed in using a radar gun on curves.
4. The effect of intersections or driveways has not been considered in the previous models. However, in Canada's rural highway network a curve, which is not in proximity of either an intersection or a driveway is rarely found.
5. The majority of the models are dealing with the horizontal alignments only, while a recent study by Hassan et al. (2000) showed that considering the effect of 2D alignments only, instead of the actual 3D alignment, may dramatically overestimate or underestimate the actual speed values. There is not much research that has been conducted taking into consideration the effect of 3D alignment.
6. Few studies (Al-Masaeid et al. 1995; Abdelwahab et al. 1998; McFadden and Elefteriadou, 2000) focused on developing equations for the speed reduction from a tangent to a curve. However, only one of them, McFadden and Elefteriadou (2000), focused on the realistic speed reduction model by tracking the vehicles

along a curve. The other two, just subtracted the 85th percentile speed value at the middle of the curve from that of the tangent. The latter methodology is not acceptable due to the difference between the speed distributions at the two mentioned points (Hirsh, 1987).

7. Previous studies have been done mainly in the United States and Europe. However, models reflecting the specific condition of Canadian rural roads (e.g., geographical, environmental, etc.) are limited.
8. The only research that considered the effect of 3D alignment as well as different vehicle classification and acceleration/deceleration rates is the work presented by Leisch and Leisch (1977). However, the suggested procedure is based on the information contained in AASHTO (1965) that is now updated.
9. The model introduced by Gibreel et al. (2001), can be useful only in the case of having a pronounced sag or crest curve. The reason is that it employs independent variables such as the length of vertical curve, the distance between the vertical point of intersection and horizontal point of intersection, rate of vertical curvature (the horizontal distance in meters required to effect one percent change in gradient), etc., that cannot be measured in case of having splines– a combination of sag and crest curves.
10. Few studies dealt with speed prediction model development for tangents; namely Islam and Seneviratne (1994), Andueza. (2000), Fitzpatrick et al. (2000a), Gibreel et al. (2001), and Donnell et al. (2001). Among the mentioned models, the model by Gibreel et al. (2001) is the only one developed using speed data of Canadian roads.

2.7 Conclusion

Design consistency is important due to its direct relationship to safety (TAC, 1999). In this chapter, different avenues to design consistency evaluation were studied. Among the methodologies introduced for design consistency evaluation, the operating speed methodology was paid special attention in the literature due to its simplicity, quantifiability, and applicability. The comparison between the operating speed methodology and other approaches resulted in introducing this method as the most efficient method in evaluating design consistency.

The consistency of a design, in this avenue, is evaluated by comparing the vehicular speeds at different points of a highway with each other and with the design speed values. High differences between the vehicle speeds at different points of a portion of highway or high differences between the operating speed and design speed values show an inconsistent design. It is clear that the first step toward operating speed consistency evaluation must be the development of some regression models capable of predicting the operating speeds at different features of a road (horizontal curves or tangents). Therefore, the various speed models developed by different researchers in the past 50 years were studied and summarized in this chapter.

Despite the fact that the operating speed methodology is a very promising approach, the related speed prediction models are still not advanced enough to be used in practice. The common pitfalls of the previously developed models were also presented in this chapter. Considering the mentioned deficiencies, this study is carried out to enhance the applicability of the operating speed methodology. In this regard, the models are to be developed for operating speed and speed differential from a tangent to a curved section.

Chapter 3: Research Methodology and Data Collection

Based on the literature review presented in the previous chapter, it is evident that the operating speed approach is the most efficient design consistency measure in terms of relative simplicity and quantifiability. With the aim of improving the current practice of design consistency evaluation, models capable of relating the speed values at different points of a horizontal curve to the geometric features of the curve should be developed. In order to investigate the realistic behaviour of the drivers while traversing a curve, it is also necessary to track each vehicle along a curve and model the speed differential from the straight section to different points on the curve.

3.1 Research Methodology

Regarding the mentioned pitfalls of the previous models, the current study focuses on the development of operating speed models and more importantly, speed differential models for passenger cars (PC), light trucks (LT) and heavy trucks (HT). In addition to the literature review presented in Chapter 2, the research methodology followed in this research consisted of the following steps:

1. Field data collection (first run).
2. Data preparation and preliminary analysis.
3. Field data collection (second run).
4. Final model development and verification.

3.1.1 Field Data Collection (First Run)

Having completed the literature review including the process of data collection utilized in previous studies, the scheme for the first run of field data collection was prepared. The scheme dictated no limitations for the site selection except that the curves

should belong to the non-built-up areas on two-lane two-way rural highways. For practical purposes, the locations should preferably be as close as possible to the City of Ottawa.

In order to obtain a wide range of geometric features, sixteen curves with various geometric characteristics (e.g., different radii, length, grade, etc.) were chosen on four different two-lane rural highways namely Highways 31, 43, 12, and 41 in Ontario, Canada. Five electronic counter/classifiers were used for speed data collection. These counters were installed at five different points of each horizontal curve. In addition, two other devices were used in the field data collection: a rain gauge to record the rain time of each day and a ball banking indicator to measure the superelevation rates of the curves. The first run of the data collection started on August 9, 2002 and finished on September 13, 2002.

3.1.2 Preliminary Analysis and Data Preparation

Having the speed database on the sixteen curves, the preliminary analysis of the data with the purpose of preparing the database for model development followed. The initial and final disturbances imposed to the database due to the presence of the crew in vicinity of the counters, led to the exclusion of the first and the last half an hour of the database. The 85th percentile speed value was then calculated for each point at each lane of traffic for the vehicles travelling under free-flow conditions on dry pavement.

For the purpose of speed differential model development, each vehicle was traced, while negotiating the curve, using the time stamps and the distance between the counters. Then, the vehicle speed change from the tangent section to different points on the curve was calculated. Excluding the initial and final disturbances, non-free-flow speed, and rain time data, the 85th percentile value of speed differential was calculated for each point at each lane of traffic.

Having the observed values of operating speeds and speed differentials, it was tried to develop a regression model that related each of these two sets of dependent variables to the geometric features of the curve using the Statistical Package for Social Science (SPSS, version 11.5). However, the model development did not lead to promising results. For the operating speed at the middle of the curve and the tangent section, no equation was found. It means that none of the independent variables showed statistically significant relationship to the dependent variable at the 95% confidence level. For the speed differential model, on the other hand, some models were found with fairly low R^2 values (almost 0.5). As a result, the second run of data collection seemed essential to achieve two main objectives; namely verifying the accuracy of the readings given by the counters, and adding some more curves to the speed database to enhance the predictability of the models.

3.1.3 Field Data Collection (Second Run)

Regarding the aforesaid statements, the second run of data collection started on April 30, 2003 and finished on May 9, 2003 with the same scheme as the first run. The accuracy of the counters' speed readings were checked using a radar gun for about an hour at the beginning of data collection. The counters' readings at the first hour, was then excluded from the rest of readings because they may have been influenced by the presence of the crew in proximity of the counters. In order to have a wide range of curves in the database, the four curves selected for the second run were chosen with different radii than the first sixteen curves.

3.1.4 Final Model Development and Verification

Adding the mentioned four curves to the database increased the total number of curves to twenty (or 40 sites considering the two lanes for each curve). The same

procedure was followed to process the data of the additional curves and make them ready for the model development. This time, a strong relationship was found between the speed differential and the independent variables ($R^2=0.889$) and all the independent variables appeared in the model were statistically significant at the 95% confidence level. However, still no relationship was found for the operating speed at the middle of the curve or the tangent section. Therefore, it was concluded that such a relationship does not exist for the collected data in spite of what have been stated in all the previous studies.

3.2 Data Collection Process

As stated earlier, after reviewing the process of data collection employed in previous studies the data collection scheme was prepared, and then the required equipment were ordered. Consequently, the curves were selected and located based on the as-built alignment data obtained from the Ministry of Transportation of Ontario (MTO) and the united county of Stormont, Dundas, and Glengarry (SDG). Traffic data, namely Average Annual Daily Traffic (AADT), was also obtained from the same authorities. The ranges of the geometric and traffic features of the selected curves are reflected in Table 3.1.

Table 3.1: Ranges of geometric and traffic features of the selected curves

Radius (m)	Length (m)	Superelevation Rate (%)	Longitudinal Grade (%)	AADT (Veh./day)	Speed Limit (km/h)
317 to 873	102 to 1002	1.95 to 7.81	-8.04 to 8.04	1562 to 8290	80

3.2.1 Site Selection

In order to have an acceptable database including a sufficient number of observations for the purpose of model development, speed data were collected for almost 24 hours for each curve. In this regard, twenty curves with various geometric characteristics namely, different radii, length, grade, etc. were chosen on four different two-lane rural highways namely, Highways 31, 43, 12, and 41 in Ontario, Canada. In selecting the sites for data collection, it was tried to include different highway classes such as King's Highways (41), Secondary Highways (31 and 43), and County Roads (12) in Ontario's Eastern Region.

The speed data collection was first carried out on sixteen curves (the first run of data collection from August 9, 2002 to September 13, 2002) and then on the next four curves (the second run of data collection from April 30, 2003 to May 9, 2003). The justification for splitting the data collection process to two runs was that the analysis of the data of the first sixteen curves did not reach good results (no strong relationship was found between the dependent and independent variables). Therefore, it was decided to collect the data for four more curves with different radii range using two different data collection devices (counter/classifier and radar gun) at the same time. In fact, the second run of data collection helped the author to achieve two goals; calibrating the speed data of the counters, and adding four curves to the database with the purpose of probable improvement of the models.

Three out of the four mentioned highways (Highways 31, 43, and 12) are in the jurisdiction of the united county of SDG and the last one (Highway 41) is in the jurisdiction of MTO Eastern Region. It is noteworthy that all the selected curves are located in non-built-up areas of the mentioned highways. The locations of the curves as well as their townships, stationing, radius and length, and spiral length are given in Table

3.2. Also, the maps showing the exact location of each highway including the nearby townships and highways are presented in Figures 3.1 to 3.5.

Table 3.2: Locations and geometric features of the selected curves

Curve No.	Highway No.	PI Station (ft or km)	Township	Data Collection Date	Curve Radius (m)	Curve Length (m)	Spiral Length (m)
1	31	27+731.038	Mountain	09-Sep-02	388.00	527.49	60.96
2	31	682+20.77// 121+04.34	Williamsburgh// Winchester	11-Sep-02	436.59	162.37	60.96
3	31	621+53.91	Williamsburgh	12-Sep-02	873.19	473.96	60.96
4	31	583+91.94	Williamsburgh	13-Sep-02	873.19	464.19	60.96
5	43	115+82.16	Mountain	19-Aug-02	873.19	604.65	60.96
6	43	220+46.43	Mountain	20-Aug-02	388.00	203.99	60.96
7	43	254+49.29	Mountain	21-Aug-02	582.12	101.60	60.96
8	43	262+51.60	Mountain	23-Aug-02	582.12	110.91	60.96
9	12	38+51.72	Finch	26-Aug-02	698.55	352.96	0.00
10	12	79+33.66	Finch	27-Aug-02	436.59	233.04	0.00
11	12	286+90.81	Finch	29-Aug-02	317.52	443.34	0.00
12	12	305+58.89	Finch	30-Aug-02	317.52	445.47	0.00
13	41	11+120.19	Grattan	03-Sep-02	582.13	111.20	60.96
14	41	10+816.41	Grattan	04-Sep-02	388.08	159.45	60.96
15	41	18+591.79	Grattan	05-Sep-02	436.59	100.14	45.72
16	41	19+051.01	Grattan	06-Sep-02	436.59	204.66	45.72
17	31	26+621.125	Mountain	30-Apr-03	349.00	472.38	60.96
18	12	110+87.82	Osnabruck	05-May-03	367.66	579.98	0.00
19	41	26+043.167	Wilberforce	07-May-03	537.35	836.89	60.96
20	41	27+621.685	Wilberforce	08-May-03	635.05	1001.89	60.96

Figure 3.1: Highway 31 location

Figure 3.2: Highway 43 location

Figure 3.3: Highway 12 location

Figure 3.4: Highway 41 location (part1)

Figure 3.5: Highway 41 location (part2)

In order to closely observe the vehicle speed changes along a curve, five counters were installed at the following points of each horizontal curve (see Figure 3.6):

- 1) 100 m before the point of tangent-to-spiral or 100 m before the point of curve in absence of transition curves (approach tangent, AT)
- 2) At the point of spiral-to-curve or at the beginning of the curve in absence of transition curves (BC).
- 3) At the middle of the curve (MC).
- 4) At the point of curve-to-spiral or at the end of the curve in absence of transition curves (EC)
- 5) 100 m after the point of spiral-to-tangent or 100 m after the point of tangency in absence of transition curves (departure tangent, DT)

Figure 3.6: Counters' location at each horizontal curve

It should be noted that the installed counters can be used to collect the speed data at the two lanes of traffic at the same time. In this regard, "lane 1" in the counters' recordings was introduced as the southbound (SB) or eastbound (EB) lane, while "lane 9" was introduced as the northbound (NB) or westbound (WB) lane.

3.2.2 Data Collection Equipment

The equipment used for data collection included the followings:

- 1) Five TCC/540 electronic counter/classifiers with two pneumatic tubes per counter
- 2) Stalker ATS radar gun
- 3) Rain gauge
- 4) Digital ball banking indicator

The counter/classifiers were used to collect vehicle spot speed plus the exact time stamp, wheelbase (WB), and the number of axles. Each counter was connected to two pneumatic tubes (1.50 m apart) that were fixed to the pavement for almost 24 hours. The tubes were fixed to the asphalt pavement and the gravel shoulder using Figure-8 restraints and nailpegs (see Figure 3.7a, b, &c).

As an accurate, easy to calibrate device, the radar gun was used to calibrate the readings of the counters. This equipment was used for a short time (almost an hour) to make sure that the speed values given by the counters were precise enough for the model development. The radar gun was mounted on a tripod while connected to a laptop computer in order to send the information of the captured vehicles namely, speed values, the acceleration/ deceleration rate, and the distance to the computer (see Figure 3.7d).

A rain gauge was used in this study to record the rain time during each day of data collection (see Figure 3.7e). The reason for utilizing such equipment was that the focus of this study is on vehicular speed on dry pavement condition. Having the rain time of each day, the related data were excluded from the speed database.

Taking advantage of a digital ball banking indicator, the superelevation rates were measured at the middle of each curve for the two directions of traffic (see Figure 3.7f). The justification for using this device was that the superelevation rates were not available for many of the curves. Furthermore, it was highly possible that the superelevation values given in the as-built drawings had changed by the passage of time.

3.2.3 Data Collection Scheme

The first step to prepare the field data collection scheme was suggesting criteria based on which the curves would be selected. Looking at the literature, many items can be seen as the site selection criteria, e.g., longitudinal grade less than 5%, no influence of intersections, AADT between 400 and 5000 vehicles/day, etc. However, as stated earlier, in this study no limitation imposed on the site selection criteria except that the curves should belong to the non-built-up areas in two-lane two-way rural highways in Ontario's Eastern Region, and as close as possible to Ottawa.

a) Traffic counter/classifier

b) Pneumatic tubes

c) Figure-8 restraints

d) ATS Stalker radar gun

e) Rain gauge

f) Ball banking indicator

Figure 3.7: Equipment used in data collection

In selecting the sites, it was tried to choose the curves with an extensive radii range belonging to different highway classes. However, not many different curve radii were found in the as-built drawings (only ten different radii values).

A proposed scheme for field data collection is presented in this section. In preparing the scheme it was tried to follow the rules stated in MTO safety book. The field data collection scheme comprised of the following steps:

- 1) Finding the exact location of the five points on each curve based on the as-built alignment data.
- 2) Dropping a counter and two tubes beside each point (using the crew vehicle).
- 3) Wearing a safety vest and a hard hat.
- 4) Placing five traffic cones at each side of the road before the location of counter installation (see Figure 3.8).
- 5) Holding two stop/slow signs by two traffic control personnel (TCP) to show the “stop” side of the sign when the other two members of the team are working at the middle of the road, and “slow” side of the sign when they are working at the shoulders (see Figure 3.8).
- 6) Spreading the two tubes across the highway and fixing them with the distance of 1.5 m using two Figure-8 restraints at each end, nails, and nailpegs.
- 7) Connecting the tubes to the counter/classifier, synchronizing the counter with a laptop computer, and setting up the counter to start collecting the data at the desired time.
- 8) Securing the counters with chains and locks.
- 9) Measuring the lane width and the shoulder width for each lane of traffic.
- 10) Measuring the superelevation rate at each lane near the middle of the curve.
- 11) Installing the rain gauge as close as possible to the middle of the curve.

12) Picking up the tools, signs, and traffic cones and moving to the next point of the curve on foot.

13) Repeating the aforementioned steps for the other four points of each curve.

The counters were left in the field for almost 24 hours. At the beginning of the next day, the counters were picked up, the data were retrieved to the laptop computer, and then the same procedure was repeated for the next curve.

Basically, at the end of each week, the data of five curves were collected, except for delays due to loosening of tubes or heavy rain during installation. On each Saturday all equipment were gathered from the field and after the retrieval of the data the counters were plugged on campus to be charged and ready for the next weekday (e.g., Monday).

Figure 3.8: Traffic control personnel (TCP), safety cones, and stop/slow sign

3.2.4 Data Retrieval

At the beginning of each day, the data collected by the counters were retrieved to a laptop computer in the field. After that, the retrieved file was exported as an ASCII format that is readable as a text file (see Figure 3.9). However, in order to process the data, it was necessary to convert the file to the Microsoft Excel format (see Figure 3.10). As it can be seen in these figures, the output files include time stamp for each reading, date, speed, number of axles, wheelbase, lane of traffic, and location of counter in the curve (e.g. beginning of the curve, middle of the curve, etc).

The data recorded by the rain gauge was also retrieved every day as an Excel file. These data were then used to exclude that part of the database correspondent to the rain time of the day. The data of the digital ball banking indicator did not need to be retrieved, as it was just a number displayed on the device's LCD screen. The readings of the radar gun were simultaneously transferred to the laptop computer and saved in a separate file for each reading (the name of each file shows the exact time stamp).

3.2.5 Radar Gun Usage for Counter Calibration

A Stalker ATS radar gun was employed in this study so as to calibrate the electronic counters in the second run of data collection. Due to the fact that there was no way to calibrate the counters by themselves, it was necessary to calibrate them with an accurate equipment. Hence, a radar gun was used to record the speed of the same vehicles as one of the counters was collecting for almost one hour. After retrieving the data of the mentioned counter on the next day, a comparison was made between the readings of the two devices. This comparison showed that the counters are very promising devices in terms of accuracy (the difference between the readings given by the counters and the radar gun was less than 5%). As a result, the calibration of the counters was aborted.

```

Data For Site: H41TNC1
  Info 1:
  Info 2:
Storage Mode : Raw
Start Time   : 09/03/02 at 11:30
Stop Time    : 09/04/02 at 09:50
Data collected by a TCC-540 (ver 2.45)

Counter Serial Number: 0205-7463

Time= 11:30:00.00 Date= 09/03/02 (Sep 3, 2002 Tue)
1: Info: S      Mode: Normal   Sensors: Axle-Axle   Sensor spacing: 150cm
9: Info: N      Mode: Normal   Sensors: Axle-Axle   Sensor spacing: 150cm
9: 11:30:37.00  77.9kph, 2 Axles, Length= 241cm, 241cm
1: 11:30:52.00  80.1kph, 2 Axles, Length= 277cm, 277cm
1: 11:31:04.00  61.4kph, 2 Axles, Length= 247cm, 247cm
9: 11:31:22.00  91.1kph, 2 Axles, Length= 250cm, 250cm
9: 11:31:52.00  74.6kph, 2 Axles, Length= 302cm, 302cm
1: 11:32:31.00  65.8kph, 2 Axles, Length= 332cm, 332cm
9: 11:33:46.00  81.2kph, 2 Axles, Length= 283cm, 283cm
9: 11:33:47.00  83.4kph, 2 Axles, Length= 274cm, 274cm
1: 11:34:08.00  68.0kph, 2 Axles, Length= 436cm, 436cm
9: 11:34:22.00  86.7kph, 2 Axles, Length= 244cm, 244cm
1: 11:35:54.00  54.9kph, 7 Axles, Length=1963cm, 390cm 155cm 558cm 186cm 485cm 189cm
1: 11:35:58.00  62.5kph, 2 Axles, Length= 287cm, 287cm
9: 11:36:05.00  84.5kph, 2 Axles, Length= 250cm, 250cm
9: 11:36:09.00  79.0kph, 2 Axles, Length= 238cm, 238cm
1: 11:36:56.00  82.3kph, 2 Axles, Length= 162cm, 162cm
9: 11:36:59.00  83.4kph, 2 Axles, Length= 338cm, 338cm
1: 11:37:26.00  85.6kph, 2 Axles, Length= 335cm, 335cm
9: 11:37:48.00  102.0kph, 2 Axles, Length= 332cm, 332cm
9: 11:37:50.00  99.9kph, 2 Axles, Length= 256cm, 256cm
9: 11:37:52.00  100.9kph, 2 Axles, Length= 244cm, 244cm
9: 11:39:06.00  93.3kph, 2 Axles, Length= 283cm, 283cm
9: 11:39:31.00  90.0kph, 2 Axles, Length= 253cm, 253cm
1: 11:40:12.00  85.6kph, 2 Axles, Length= 283cm, 283cm
9: 11:40:36.00  75.7kph, 2 Axles, Length= 296cm, 296cm
9: 11:40:43.00  45.0kph, 2 Axles, Length= 259cm, 259cm
9: 11:41:26.00  100.9kph, 2 Axles, Length= 311cm, 311cm
1: 11:41:28.00  77.9kph, 2 Axles, Length= 268cm, 268cm
9: 11:42:25.00  83.4kph, 2 Axles, Length= 265cm, 265cm
9: 11:42:27.00  81.2kph, 2 Axles, Length= 256cm, 256cm
1: 11:42:47.00  97.7kph, 2 Axles, Length= 268cm, 268cm
1: 11:42:48.00  96.6kph, 2 Axles, Length= 268cm, 268cm
9: 11:42:53.00  87.8kph, 2 Axles, Length= 235cm, 235cm
1: 11:43:59.00  54.9kph, 3 Axles, Length= 686cm, 549cm 137cm
1: 11:44:00.00  53.8kph, 2 Axles, Length= 122cm, 122cm
9: 11:44:45.00  94.4kph, 2 Axles, Length= 235cm, 235cm
1: 11:44:52.00  75.7kph, 2 Axles, Length= 314cm, 314cm
9: 11:45:40.00  83.4kph, 3 Axles, Length= 600cm, 253cm 347cm
1: 11:45:44.00  84.5kph, 2 Axles, Length= 271cm, 271cm
1: 11:46:28.00  97.7kph, 2 Axles, Length= 283cm, 283cm

```

Figure 3.9 : Counter output file in ASCII format

On the other hand, an electronic counter/classifier is much more convenient for use in such studies. In case of using counters, the data collection team do not have to conceal themselves, because the counters automatically collect and save the data, and there is no need for the presence of any operators. In addition, speed data collection becomes feasible for a longer time (24 hours) that leads to having a larger database for the model development. Moreover, using the counter makes the data free of the troubles associated with the radar gun utilization such as difficulty to target the vehicles in the

curved section due to the cosine error, bias of the observed data because of the presence of the crew in vicinity of the curve, etc.

Lane	Date	Time	Speed (km/h)	No of Axles	Length (cm)	Axle Spacing (cm)					
9	03/09/2002	11:30:37 AM	77.9	2	241	241					
1	03/09/2002	11:30:52 AM	80.1	2	277	277					
1	03/09/2002	11:31:04 AM	61.4	2	247	247					
9	03/09/2002	11:31:22 AM	91.1	2	250	250					
9	03/09/2002	11:31:52 AM	74.6	2	302	302					
1	03/09/2002	11:32:31 AM	85.8	2	332	332					
9	03/09/2002	11:33:46 AM	81.2	2	283	283					
9	03/09/2002	11:33:47 AM	83.4	2	274	274					
1	03/09/2002	11:34:08 AM	68.0	2	436	436					
9	03/09/2002	11:34:22 AM	86.7	2	244	244					
1	03/09/2002	11:35:54 AM	54.9	7	1963	390	155	558	186	485	189
1	03/09/2002	11:35:58 AM	62.5	2	287	287					
9	03/09/2002	11:36:05 AM	84.5	2	250	250					
9	03/09/2002	11:36:09 AM	79.0	2	238	238					
1	03/09/2002	11:36:56 AM	82.3	2	162	162					
9	03/09/2002	11:36:59 AM	83.4	2	338	338					
1	03/09/2002	11:37:26 AM	85.6	2	335	335					
9	03/09/2002	11:37:48 AM	102.0	2	332	332					
9	03/09/2002	11:37:50 AM	99.9	2	256	256					
9	03/09/2002	11:37:52 AM	100.9	2	244	244					
9	03/09/2002	11:39:06 AM	93.3	2	283	283					
9	03/09/2002	11:39:31 AM	90.0	2	253	253					
1	03/09/2002	11:40:12 AM	85.6	2	283	283					
9	03/09/2002	11:40:36 AM	75.7	2	296	296					
9	03/09/2002	11:40:43 AM	45.0	2	259	259					
9	03/09/2002	11:41:26 AM	100.9	2	311	311					
1	03/09/2002	11:41:28 AM	77.9	2	268	268					
9	03/09/2002	11:42:25 AM	83.4	2	265	265					
9	03/09/2002	11:42:27 AM	81.2	2	256	256					
1	03/09/2002	11:42:47 AM	97.7	2	268	268					
1	03/09/2002	11:42:48 AM	96.6	2	268	268					
9	03/09/2002	11:42:53 AM	87.8	2	235	235					
1	03/09/2002	11:43:59 AM	54.9	3	686	549	137				
1	03/09/2002	11:44:00 AM	53.8	2	122	122					
9	03/09/2002	11:44:45 AM	94.4	2	235	235					
1	03/09/2002	11:44:52 AM	75.7	2	314	314					
9	03/09/2002	11:45:40 AM	83.4	3	600	253	347				
1	03/09/2002	11:45:44 AM	84.5	2	271	271					
1	03/09/2002	11:46:28 AM	97.7	2	283	283					

Figure 3.10: Counter output file in Microsoft Excel format

Chapter 4: Database Preparation and Preliminary Analysis

In this chapter, the steps followed to prepare the database for model development are explained. These steps include the screening of the raw data, vehicle classification, AADT estimation, longitudinal grade calculation, and preliminary data analysis.

4.1 Screening of Raw Data

The main objective of this step was to screen the raw data collected by the counters and exclude the data that could have been disturbed.

4.1.1 Initial and Final Disturbances

Initial and final disturbances might have been introduced into the data by the presence of the crew dressed with traffic vests in vicinity of the counters while they were collecting the speed data of the vehicles. Based on observations during these periods, drivers clearly decreased their speeds when they saw the traffic control personnel standing or walking in the data collection sites. Drivers may tend to believe that there is a relationship between these personnel and traffic enforcement, and as a result of that they might have altered their driving behaviour.

Regarding the abovementioned issue, the readings of the first and the last 30 minutes of the speed database were considered to include initial and final disturbances, respectively, and therefore were excluded. Initial disturbance happened in time of installing the counters while the final disturbance happened in time of picking up the counters on the next day.

4.1.2 Sensor Miss

Although the counters are accurate and convenient devices for the purpose of speed data collection, they have some weak points. For example, they may not be able to give the speeds of two successive vehicles that are very close to each other. They may also not be able to give the speeds of the vehicles that are travelling very slowly. In such cases they just give a message, "Sensor Miss", that shows the counter was not able to measure the speed of the related vehicle. The number of sensor miss cases was very small and did not exceed 0.16 % at any specific site. Thus, before continuing the data processing, these cases were eliminated from the database.

4.1.3 Nighttime Data

As stated before, this study focuses on modelling operating speed and speed differentials as influenced only by the geometric features of the road. Thus, that portion of the database corresponding to speeds collected at night should be eliminated as it may contain some differences in the speeds of vehicles caused by the lack of enough ambient light. Considering an assumed margin of safety, it was decided to exclude the records of the database from half an hour before sunset to half an hour after sunrise of the next day. Exact sunrise and sunset times for each specific day were obtained from the Internet site: <http://www.cmpsolv.com/los/sunset.html>

4.1.4 Wet Pavement Data

Another issue that should be thought of in terms of the preparation of the database is taking out the speed readings collected during wet pavement condition. Considering an assumed margin of safety, the rain time data plus fifteen minutes afterward were taken out. The elimination of rain time data incorporated two curves

(curves 5 and 13) out of the total of twenty curves of the database. According to the output files of the rain logger, these curves had 140 and 50 minutes of rain, respectively.

4.1.5 Non-Free-Flow Traffic

The last item needed to be addressed in terms of data screening was the vehicles impeded by slower vehicles ahead. Such vehicles in the database generated by the traffic counters, can be identified using the headway, defined as the time gap between the two successive vehicles in the stream of traffic. Based on several studies carried out before, the value of five seconds was suggested as the cut-off value at or above which, the free-flow condition occurs.

Although the headway value of five seconds has been widely accepted by the researchers around the globe as a logical criterion for the determination of free-flow condition, further analysis was carried out in this study to verify this value. To reach this goal, the speed data of the twenty curves were entered to a SPSS file, and a comparison was made among the operating speeds (85th percentile speeds) of the vehicles having different headway values. This analysis showed that, in most cases, the operating speed increased by the increase of headway up to three or four seconds and after that, it became constant. Therefore, the previously suggested value of five seconds was confirmed in this study noting that the percentage of vehicles with a headway of three and four seconds did not exceed 8.0% of the whole database.

4.2 Vehicle Classification

Due to the fact that the counters' output files only give the number of axles, wheelbase (WB) and axle spacing of each vehicle, a procedure should be undertaken to classify the vehicles based on these fields. In this study, the vehicle classification was carried out based on the procedure introduced by Hassan (2003). After reviewing the

catalogues of more than 200 car makes and models in Canada, Hassan (2003) proposed the following criteria for vehicle classification:

- Passenger Car (PC): Number of Axles = 2 and $2\text{m} \leq \text{Wheelbase} < 3\text{m}$
- Light Truck (LT) : Number of Axles = 2 and $3\text{m} \leq \text{Wheelbase} < 4\text{m}$
- Heavy Truck (HT) : Number of Axles > 2 and $\text{Wheelbase} \geq 4\text{m}$
- Not Applicable (N/A): Other cases

Accounting for almost 65% of the whole database, passenger cars formed the most frequently observed vehicle in the database. Light trucks and heavy trucks, on the other hand, composed almost 20% and 11% of the database, respectively. The last category, “not applicable”, incorporated almost 4% of the database and corresponded to exceptional cases such as two-axle vehicles with extremely low speeds or very large or small wheelbase.

4.3 AADT Estimation

Since the Average Annual Daily Traffic (AADT) is one of the independent variables used in some previous speed prediction models especially on tangents, it was necessary to determine the AADT on the sections considered in the study. For the majority of the curves employed in this study (14 out of 20 curves), the AADT were not available for the recent years as the most recent data were for the years 1990 – 1994 .

Therefore, it was suggested to calculate the AADT based on the number of vehicles recorded by the counters and then, using the hourly, daily, and monthly expansion factors. This time, the problem was that the mentioned expansion factors were not available at the united county of SDG.

As a result, AADT calculation for the year 2002 or 2003, depending on the data collection date for each site, was carried out using the AADT of the years 1990 to 1994. In this regard, the growth factor of AADT was calculated for each two successive years,

and the average growth factor was calculated. Assuming that the mentioned average growth factor would remain constant for the next eight or nine years, the AADT was estimated for the year 2002 or 2003.

4.4 Longitudinal Grade Calculation

In this study, the effect of 3D alignment on the operating speeds of vehicles was considered. The longitudinal grade for the highways, which have pronounced vertical alignments (sag or crest vertical curves or uphill-downhill sections), can be easily obtained from the as-built drawings. However, as it is the case for most of the Ontarian two-lane two-way rural roads, the vertical alignment follows the natural trend of the topographic characteristics of the ground. This type of vertical alignment, called “spline grades”, in fact, consists of a combination of mild sag and crest curves that best fits the topography. Clearly, the utilization of splines reduces the construction expenses of the alignment.

Therefore, to calculate the longitudinal grade for a specific section of a highway, the average values of the longitudinal grades for smaller sections of the road were calculated. In each of these small sections, any of the three shapes (sag, crest, or straight section) could be easily distinguished. This was done in either of the two ways; manually by using the as-built drawings, or entering the elevation data of each curve to the VAFIT software developed by Easa et al. (1998) and obtaining the corresponding separated sections. In the latter method, a spline grade is simulated by the software, as a combination of straight, sag, and crest sections.

4.5 Pavement Serviceability Index (PSI)

In order to account for the effect of pavement condition on drivers' choice of speed, pavement serviceability index (PSI), defined as a physical measure of pavement

roughness (Garber and Hoel, 2002), should be considered in modelling as an independent variable. PSI normally ranges from 2.0 to 4.5. When a pavement is originally constructed, it is in a very good condition (PSI = 4.5). However, as the number of traffic loading increases, the PSI value diminishes to a value near 2.0 or 2.5 (minimum acceptable value). In this case, the pavement is considered for rehabilitation.

The necessity of such a variable becomes clearer considering the fact that the pavement condition for some of the selected sites was dramatically poor as it had lots of cracks and potholes. As an attempt to consider PSI as a representative of pavement condition in modelling the operating speed or speed differential, these values were requested from the MTO. However, despite the fact that the data of interest were available for some of the curves, the authorities refused to release such data.

4.6 Preliminary Data Analysis

After preparing the speed database and the required independent variables e.g., longitudinal grade, AADT, etc., a preliminary analysis was carried out prior to the model development. In fact, in this section, it was tried to achieve the following objectives:

1. Comparing the speeds of vehicles belonging to different classes.
2. Comparing the speeds of vehicles travelling on the two lanes of traffic.
3. Comparing the initial readings given by the counters with the rest of database.

Achieving the first objective, the author would be able to differentiate between the models that would be developed (perhaps two or three sets of model would be considered to cover different vehicle classes). The possible difference between the speeds of vehicles travelling on the two lanes of traffic would show the necessity of having a dummy variable that enters the effect of curve direction into the models. Finally, the comparison between the initial readings (e.g., first 100 readings) with the rest of readings would be carried out to verify the consistency of speed measurement.

4.6.1 Impact of Vehicle Classification

The Analysis of Variance (ANOVA) was carried out to examine the effect of vehicle classification on the observed operating speeds. This analysis was carried out to compare the differences between the mean speed values for the three following combinations: (PC, LT, and HT), (PC/LT and HT), and (PC and LT). First, the ANOVA was carried out among all vehicle classes in the database (PC, LT, and HT). The results showed that in 63% of cases there was a statistically significant difference, at the 95% confidence level, between the mean speeds of vehicles belonging to different classes. The results of this ANOVA are presented in Table 4.1

Second, the ANOVA was carried out between the speed values of PC/LT with HT. As shown in Table 4.2, the results showed that in 62% of the cases there was a statistically significant difference between the speeds of the mentioned vehicles. Therefore, the null hypothesis saying that there is no difference between the mean speeds of such vehicles can be rejected at the 95% confidence level. Third, a comparison was made between the speed values related to PC and LT while the third category, HT had been excluded from the database. It was, then, concluded that in majority of the cases (88%) there was no statistically significant difference between the speeds of PCs and LTs. The results of ANOVA for the comparison between PC and LT are presented in Table 4.3.

Therefore, the preliminary analysis reconfirmed the necessity of the classification of the vehicles. It was also concluded that there was no statistically significant difference between the mean speeds of the light trucks and passenger cars. As a result, the two latter categories could be combined together and thought of as one category (PC/LT). In short, for the purpose of model development, only two classes of vehicles were considered: PC/LT and HT.

Table 4.1: Summary of ANOVA – all vehicle classes

Curve No.	Lane	p-value at point				
		AT	BC	MC	EC	DT
1	SB	0.177	N/A	0.204	N/A	0.150
	NB	0.008	N/A	0.006	N/A	0.004
2	SB	0.028	N/A	0.222	N/A	0.016
	NB	0.010	N/A	0.087	N/A	0.858
3	SB	< 0.001	N/A	0.006	N/A	< 0.001
	NB	< 0.001	N/A	0.009	N/A	0.021
4	SB	0.018	N/A	< 0.001	N/A	0.022
	NB	0.128	N/A	0.508	N/A	0.074
5	EB	0.532	0.574	0.185	0.825	0.389
	WB	0.001	0.001	0.062	0.057	< 0.001
6	EB	0.009	0.189	< 0.001	0.025	0.232
	WB	0.209	0.152	0.324	0.871	0.449
7	EB	< 0.001	< 0.001	< 0.001	0.109	< 0.001
	WB	0.082	0.001	0.015	0.037	0.226
8	EB	< 0.001	< 0.001	0.009	0.013	< 0.001
	WB	0.011	0.172	0.861	0.485	0.398
9	SB	< 0.001	0.002	0.018	0.004	0.002
	NB	0.032	0.013	< 0.001	< 0.001	< 0.001
10	SB	< 0.001	< 0.001	0.001	0.001	0.002
	NB	0.006	0.262	0.008	0.083	0.016
11	SB	< 0.001	< 0.001	< 0.001	0.001	0.004
	NB	0.003	0.002	0.003	< 0.001	0.001
12	SB	0.003	0.002	0.004	< 0.001	< 0.001
	NB	0.011	0.001	< 0.001	< 0.001	0.286
13	SB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
	NB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
14	SB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
	NB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
15	SB	0.036	0.256	0.764	0.413	0.781
	NB	0.088	0.089	0.001	0.002	< 0.001
16	SB	0.552	0.260	0.249	0.289	0.560
	NB	0.014	< 0.001	0.071	0.001	0.009
17	SB	< 0.001	N/A	0.882	N/A	0.063
	NB	0.105	N/A	0.121	N/A	0.393
18	SB	< 0.001	N/A	0.249	N/A	0.001
	NB	0.254	N/A	0.334	N/A	0.001
19	SB	0.165	N/A	0.018	N/A	< 0.001
	NB	0.470	N/A	0.556	N/A	0.060
20	SB	0.486	N/A	0.302	N/A	0.470
	NB	0.489	N/A	0.360	N/A	0.386

For the p -values less than 0.05, the null hypothesis can be rejected.
 "N/A" refers to the curves in which, only three counters were installed

Table 4.2: Summary of ANOVA – HT versus the rest of classes

Curve No.	Lane	p-value at point				
		AT	BC	MC	EC	DT
1	SB	0.115	N/A	0.172	N/A	0.064
	NB	0.002	N/A	0.002	N/A	0.001
2	SB	0.114	N/A	0.412	N/A	0.296
	NB	0.003	N/A	0.157	N/A	0.751
3	SB	< 0.001	N/A	0.001	N/A	< 0.001
	NB	0.155	N/A	0.002	N/A	0.037
4	SB	0.035	N/A	0.094	N/A	0.019
	NB	0.495	N/A	0.276	N/A	0.051
5	EB	0.268	0.401	0.204	0.960	0.247
	WB	0.003	0.055	0.068	0.029	< 0.001
6	EB	0.101	0.479	< 0.001	0.007	0.088
	WB	0.077	0.235	0.937	0.602	0.519
7	EB	< 0.001	< 0.001	< 0.001	0.036	< 0.001
	WB	0.156	< 0.001	0.006	0.012	0.089
8	EB	< 0.001	0.001	0.009	0.012	< 0.001
	WB	0.008	0.193	0.849	0.473	0.463
9	SB	< 0.001	< 0.001	0.006	0.001	0.001
	NB	0.021	0.003	< 0.001	< 0.001	< 0.001
10	SB	< 0.001	< 0.001	< 0.001	< 0.001	0.001
	NB	0.002	0.469	0.007	0.095	0.025
11	SB	< 0.001	< 0.001	< 0.001	0.002	0.012
	NB	0.001	0.001	0.001	< 0.001	0.014
12	SB	0.001	< 0.001	0.001	< 0.001	< 0.001
	NB	0.003	< 0.001	< 0.001	< 0.001	0.776
13	SB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
	NB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
14	SB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
	NB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
15	SB	0.012	0.163	0.641	0.277	0.782
	NB	0.479	0.067	0.009	0.002	< 0.001
16	SB	0.335	0.106	0.095	0.133	0.292
	NB	0.009	0.012	0.135	0.028	0.017
17	SB	< 0.001	N/A	0.973	N/A	0.024
	NB	0.218	N/A	0.131	N/A	0.575
18	SB	< 0.001	N/A	0.126	N/A	< 0.001
	NB	0.154	N/A	0.382	N/A	< 0.001
19	SB	0.588	N/A	0.008	N/A	< 0.001
	NB	0.740	N/A	0.408	N/A	0.019
20	SB	0.493	N/A	0.333	N/A	0.293
	NB	0.563	N/A	0.513	N/A	0.306

For the p -values less than 0.05, the null hypothesis can be rejected.
 "N/A" refers to the curves in which, only three counters were installed

Table 4.3: Summary of ANOVA – PC versus LT

Curve No.	Lane	p-value at point				
		AT	BC	MC	EC	DT
1	SB	0.318	N/A	0.247	N/A	0.519
	NB	0.568	N/A	0.485	N/A	0.726
2	SB	0.031	N/A	0.130	N/A	0.008
	NB	0.638	N/A	0.093	N/A	0.633
3	SB	0.440	N/A	0.887	N/A	0.954
	NB	< 0.001	N/A	0.624	N/A	0.059
4	SB	0.059	N/A	< 0.001	N/A	0.154
	NB	0.053	N/A	0.674	N/A	0.204
5	EB	0.834	0.498	0.185	0.512	0.460
	WB	0.027	0.002	0.137	0.297	0.626
6	EB	0.011	0.100	0.104	0.635	0.944
	WB	0.957	0.119	0.129	0.948	0.252
7	EB	0.843	0.936	0.222	0.830	0.159
	WB	0.070	0.679	0.354	0.568	0.755
8	EB	0.022	0.034	0.096	0.155	0.026
	WB	0.135	0.128	0.607	0.323	0.224
9	SB	0.229	0.804	0.427	0.506	0.561
	NB	0.165	0.809	0.752	0.259	0.862
10	SB	0.106	0.311	0.201	0.211	0.386
	NB	0.447	0.114	0.129	0.139	0.072
11	SB	0.105	0.070	0.098	0.047	0.020
	NB	0.270	0.253	0.722	0.739	0.006
12	SB	0.640	0.972	0.834	0.401	0.438
	NB	0.548	0.456	0.517	0.098	0.112
13	SB	0.678	< 0.001	0.496	0.328	0.441
	NB	0.090	0.166	0.186	0.060	0.136
14	SB	0.764	0.007	0.948	0.078	0.928
	NB	0.333	0.104	0.696	0.532	0.996
15	SB	0.575	0.395	0.555	0.464	0.536
	NB	0.039	0.220	0.009	0.072	0.014
16	SB	0.604	0.784	0.969	0.644	0.830
	NB	0.193	0.002	0.080	0.003	0.054
17	SB	0.544	N/A	0.601	N/A	0.060
	NB	0.488	N/A	0.144	N/A	0.203
18	SB	0.013	N/A	0.508	N/A	0.270
	NB	0.405	N/A	0.235	N/A	0.351
19	SB	0.072	N/A	0.287	N/A	0.058
	NB	0.186	N/A	0.494	N/A	0.706
20	SB	0.326	N/A	0.230	N/A	0.524
	NB	0.307	N/A	0.200	N/A	0.348

For the p -values less than 0.05, the null hypothesis can be rejected.

"N/A" refers to the curves in which, only three counters were installed

4.6.2 Impact of Lane of Traffic

Another issue that should be addressed here is the influence of the lane of traffic. Looking at the 85th percentile speed values for the two lanes of traffic for the same curve, it can be realized that there is, in most of the cases, a substantial difference between these speeds. Therefore, the ANOVA was carried out to support this idea from the statistical point of view. The results of ANOVA showed that in 80% of the cases there was a statistically significant difference between the mean speeds of the two lanes of traffic. The results of ANOVA for all the curves are presented in Table 4.4.

Considering the fact that there is a significant difference between the speeds of the two directions, even for the cases in which no influence of longitudinal grade can be seen, this variation can be ascribed to the direction a curve is turning (left turn or right turn curve). As a result of what was concluded in this preliminary analysis, an independent variable, "curve direction", which had the value of one for the right turn curves and zero for the left turn curves, was introduced and considered in the modelling.

4.6.3 Consistency of Speed Measurement

The final issue considered among the measures undertaken before the model development, was comparing a speed sample including the first 100 readings of each counter with the rest of readings. The rationale behind such an effort was that there was a possibility that the counters' sensors (pneumatic tubes) had become loose and sagged after passage of many vehicles. This analysis was carried out only for the first sixteen curves, because the main reason for this comparison was to verify the consistency of speed readings by the counters prior to the second run of data collection. In addition, for the last four curves, a radar gun was utilized at the beginning of each day. As a result the speed readings by the counters might have been influenced by the presence of the crew and the radar gun.

Table 4.4: Summary of ANOVA – effect of lane of traffic

Curve No.	<i>p</i> -value at point				
	AT	BC	MC	EC	DT
1	< 0.001	N/A	< 0.001	N/A	< 0.001
2	< 0.001	N/A	< 0.001	N/A	< 0.001
3	< 0.001	N/A	< 0.001	N/A	< 0.001
4	< 0.001	N/A	< 0.001	N/A	< 0.001
5	< 0.001	0.156	< 0.001	0.006	< 0.001
6	< 0.001	< 0.001	< 0.001	0.898	0.844
7	< 0.001	< 0.001	0.001	< 0.001	< 0.001
8	< 0.001	0.856	< 0.001	0.003	0.590
9	< 0.001	< 0.001	0.006	0.005	< 0.001
10	0.304	< 0.001	0.845	0.138	0.003
11	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001
12	< 0.001	< 0.001	0.324	< 0.001	< 0.001
13	< 0.001	0.008	< 0.001	< 0.001	< 0.001
14	< 0.001	< 0.001	< 0.001	0.587	0.234
15	< 0.001	< 0.001	< 0.001	0.993	< 0.001
16	0.199	< 0.001	< 0.001	< 0.001	0.977
17	< 0.001	N/A	0.640	N/A	< 0.001
18	< 0.001	N/A	0.157	N/A	0.082
19	< 0.001	N/A	< 0.001	N/A	0.019
20	0.008	N/A	< 0.001	N/A	< 0.001

For the *p*-values less than 0.05, the null hypothesis can be rejected.
 "N/A" refers to the curves in which, only three counters were installed

The ANOVA showed that in 72% of the cases, there was no statistically significant difference, at the 95% confidence level, between the means of the sample including the first 100 readings and the rest of speed database. Therefore, the sensors remained in a proper shape during the data collection period (almost 24 hours). The results of ANOVA for the first sixteen curves are given in Table 4.5.

In summary, this chapter dealt with the possible measures to prepare the database for model development (operating speed model and speed differential model) as well as some preliminary analysis of the data. In this connection, the screening of the data was carried out to exclude every possible factor that can influence drivers' choice of speed rather than the geometric features of the alignment. In addition, the ANOVA was employed to investigate the differences between the speeds of the vehicles belonging to different classes, the effect of lane of traffic, and the consistency of speed measurement.

Table 4.5: Summary of ANOVA – consistency of speed measurement

Curve No.	Lane	p-value at point				
		AT	BC	MC	EC	DT
1	SB	0.004	N/A	< 0.001	N/A	0.262
	NB	0.238	N/A	0.699	N/A	0.292
2	SB	0.453	N/A	0.357	N/A	0.460
	NB	0.083	N/A	0.026	N/A	0.170
3	SB	< 0.001	N/A	< 0.001	N/A	< 0.001
	NB	< 0.001	N/A	0.080	N/A	0.026
4	SB	0.011	N/A	0.003	N/A	0.009
	NB	< 0.001	N/A	0.261	N/A	0.235
5	EB	0.131	0.293	< 0.001	0.059	0.079
	WB	0.916	0.512	0.102	0.946	0.941
6	EB	0.267	0.283	0.471	0.110	0.095
	WB	0.324	0.753	0.477	0.851	0.811
7	EB	0.526	0.743	0.466	0.591	0.531
	WB	0.063	0.062	0.852	0.068	0.348
8	EB	0.115	0.027	0.038	0.541	0.147
	WB	< 0.001	< 0.001	0.002	0.065	0.494
9	SB	< 0.001	0.002	< 0.001	< 0.001	< 0.001
	NB	0.333	0.004	0.052	0.124	0.012
10	SB	0.554	0.009	0.048	0.001	< 0.001
	NB	0.514	0.367	0.081	0.112	0.690
11	SB	0.062	0.092	0.001	0.509	0.031
	NB	0.366	0.711	0.104	0.788	0.864
12	SB	0.203	0.199	0.728	0.074	0.528
	NB	0.652	0.869	0.767	0.409	0.708
13	SB	0.481	0.637	0.516	0.546	0.425
	NB	0.016	0.017	0.002	0.005	0.013
14	SB	0.843	0.882	0.881	0.763	0.168
	NB	0.220	0.477	0.581	0.868	0.850
15	SB	0.984	0.488	0.408	0.603	0.757
	NB	0.633	0.571	0.615	0.451	0.474
16	SB	0.862	0.415	0.092	0.187	0.464
	NB	< 0.001	< 0.001	< 0.001	< 0.001	< 0.001

For the p -values less than 0.05, the null hypothesis can be rejected.

"N/A" refers to the curves in which, only three counters were installed

Chapter 5: Operating Speed Model Development

The steps taken to develop a model capable of predicting the vehicles' operating speed at different points on a horizontal curve are elucidated in this chapter. The first step in this regard was to calculate the 85th percentile speed values for the different classes of vehicles under free-flow conditions, dry pavement, and during daytime based on the observed individual readings stored in the Excel files.

After calculating the 85th percentile values (dependent variable) for each site, it was necessary to gather all the available geometric and traffic characteristics of each site (independent variables). Some independent variables were directly derived from the as-built drawings, some were measured in the field, and some were calculated as a function of other existing variables. Independent variables are listed as follows:

- Curve radius (r)
- Circular curve length (L_C)
- Spiral curve length (L_S)
- Spiral flag (*sp-flag*): A dummy variable that is equal to 1 if there is a spiral, otherwise 0.
- Lane width (LW)
- Shoulder width (SW)
- Longitudinal grade (G)
- Superelevation rate (e)
- Average annual daily traffic ($AADT$)
- Speed on approach tangent ($V_{tangent}$)
- Intersection flag (*int-flag*): A dummy variable that is equal to 1 if there is an intersection, otherwise 0.

- Driveway flag (*drv-flag*): A dummy variable that is equal to 1 if there is a driveway, otherwise 0.
- Curve direction (*curve-dir*): A dummy variable that is 1 in case of a right-turn curve, otherwise 0.
- Tangent Length (*TL*)
- Circular curve deflection angle (*DFC*): $\frac{L_c}{R}$
- Curve deflection angle (*DF*): $\frac{L_c + L_s}{R}$
- Curvature change rate (*CCR*)
- Preceding curve radius (*pre-radius*)
- Succeeding curve radius (*suc-radius*)

In addition to these independent variables, some transformations of those variables were also considered in the modelling namely, $e^{(a+b \times CCR)}$, $\ln(r)$, $(r)^2$, $(r)^3$, $1/(r)$, $1/(r^2)$, $\sqrt{V_{tangent}}$. The mentioned variables covered almost all the independent variables employed for speed prediction modelling in the past 50 years. Among all the independent variables used for modelling, r , L_c , and L_s were obtained directly from the as-built drawings. On the other hand, e , LW , and SW were measured in the field. Finally, G and $AADT$ were calculated as explained in Chapter Four.

5.1 Calculation of 85th Percentile Speed

Having the individual observed speed values of the vehicles, it was possible to calculate the 85th percentile speed as the representative of operating speed correspondent to each site (each lane of traffic for a specific curve). In order to calculate these values, a macro was written in Microsoft Excel environment.

The purpose of writing such a macro was to perform the following three tasks automatically. First, the headway values were calculated and the impeded vehicles, those with the headway of less than five seconds, were excluded. Second, the vehicles were categorized to two categories namely, PC/LT and HT regarding the classification criteria mentioned in Chapter Four. Third, the 85th percentile speed value was automatically calculated for the two vehicle groups at each point at each lane of traffic, and was presented in a table. In such a table, the number of vehicles by the use of which, the 85th percentile speed had been calculated was also shown.

This macro was applied for all curves, and consequently the operating speed values were calculated for all points on all curves (see Tables 5.1 and 5.2). The speed values in these tables, in fact, form the dependent variable for the model development. It should be noted that the speed data for the first sixteen curves were collected in the first run of data collection and the remaining four curves were included in the second run.

In Tables 5.1 and 5.2, some cells are defined as "Not Available (N/A)"; this happened after one of the counters was stolen in time of installation. It was decided, then, to install three (instead of five) counters at the AT, MC, and DT of each curve. As shown in Table 5.2, the number of observations for heavy trucks, in most of the cases, was too low for proper determination of operating speeds (58 observations on average). Therefore, the model development for heavy trucks was aborted due to the lack of acceptable number of observations.

Having determined the dependent and independent variables, they were entered to the Statistical Package for Social Science (SPSS v.11.5) to carry out a regression analysis. The analysis was carried out at two stages: for the first sixteen curves (using the data collected at the first run), and for all the twenty curves (using the data collected at the two runs of data collection).

Table 5.1: Observed 85th percentile speed values for PC/LT

Curve No.	Lane	V _{AT} (km/h)	V _{BC} (km/h)	V _{MC} (km/h)	V _{EC} (km/h)	V _{DT} (km/h)	No. of Observations
1	SB	100.9	N/A	96.6	N/A	106.4	756
	NB	90.0	N/A	94.4	N/A	95.5	739
2	SB	116.3	N/A	98.8	N/A	96.6	710
	NB	86.7	N/A	94.4	N/A	90.0	724
3	SB	95.5	N/A	100.9	N/A	93.3	703
	NB	118.5	N/A	98.8	N/A	103.1	595
4	SB	105.3	N/A	115.2	N/A	95.5	778
	NB	102.0	N/A	95.5	N/A	96.6	675
5	EB	92.2	98.8	104.2	97.7	97.7	494
	WB	100.9	97.7	94.4	98.8	102.0	540
6	EB	102.0	103.1	97.7	95.5	100.9	690
	WB	92.2	91.1	87.8	95.5	96.6	744
7	EB	100.9	93.3	94.4	94.4	95.5	896
	WB	96.6	103.1	95.5	98.8	98.8	849
8	EB	98.8	96.6	96.4	102.0	100.9	977
	WB	94.4	97.7	98.8	102.0	98.8	889
9	SB	102.0	94.4	100.9	103.1	96.6	489
	NB	95.5	102.0	97.7	103.8	102.0	456
10	SB	100.9	98.8	100.9	94.4	100.9	503
	NB	102.0	106.4	99.9	98.8	98.8	451
11	SB	87.8	96.6	97.7	102.0	102.0	539
	NB	103.5	102.0	94.4	92.2	102.0	488
12	SB	102.4	95.5	96.6	97.7	85.6	529
	NB	99.9	90.0	95.5	94.4	99.9	431
13	SB	98.8	104.2	90.0	88.9	91.1	294
	NB	99.9	104.2	100.9	103.1	105.3	301
14	SB	94.4	97.7	91.1	98.8	94.4	301
	NB	110.8	97.7	104.2	94.4	94.4	253
15	SB	100.9	99.9	99.9	103.2	125.1	166
	NB	124.0	110.8	114.0	108.6	113.0	130
16	SB	111.9	114.1	103.1	115.2	101.5	323
	NB	111.9	92.2	98.8	100.2	102.0	276
17	SB	106.4	N/A	95.5	N/A	102.0	696
	NB	96.3	N/A	94.6	N/A	98.9	479
18	SB	94.4	N/A	97.7	N/A	96.6	218
	NB	99.9	N/A	98.8	N/A	100.9	190
19	SB	110.8	N/A	102.0	N/A	102.0	464
	NB	95.5	N/A	98.8	N/A	101.2	330
20	SB	100.9	N/A	102.8	N/A	105.3	587
	NB	103.1	N/A	92.2	N/A	99.9	375

Table 5.2: Observed 85th percentile speed values for HT

Curve No.	Lane	V _{AT} (km/h)	V _{BC} (km/h)	V _{MC} (km/h)	V _{EC} (km/h)	V _{DT} (km/h)	No. of Observations
1	SB	99.1	N/A	93.7	N/A	105.3	72
	NB	85.6	N/A	90.0	N/A	93.3	101
2	SB	116.3	N/A	99.1	N/A	96.6	86
	NB	83.2	N/A	91.1	N/A	86.7	100
3	SB	92.2	N/A	99.4	N/A	92.5	83
	NB	114.1	N/A	97.7	N/A	100.2	84
4	SB	99.0	N/A	109.7	N/A	92.1	79
	NB	99.9	N/A	93.0	N/A	96.2	83
5	EB	91.1	97.5	102.7	96.6	95.5	88
	WB	96.6	93.6	91.1	98.0	98.8	67
6	EB	97.7	99.9	93.3	92.2	98.8	116
	WB	91.1	89.6	86.4	93.3	94.4	94
7	EB	95.5	87.8	91.7	88.9	91.9	92
	WB	91.1	97.7	91.1	94.8	96.2	74
8	EB	94.4	92.2	92.1	97.8	96.6	104
	WB	93.3	98.8	98.8	103.1	100.9	152
9	SB	96.4	90.0	96.5	98.6	91.1	42
	NB	95.3	95.5	91.1	99.8	96.8	45
10	SB	98.9	95.5	97.3	94.2	100.9	34
	NB	94.6	99.6	94.1	92.6	93.0	37
11	SB	80.1	88.0	90.0	94.4	96.6	53
	NB	95.5	93.7	87.8	83.4	95.5	45
12	SB	95.5	90.0	90.0	91.1	80.4	72
	NB	94.4	84.5	90.0	91.1	98.0	45
13	SB	86.2	86.0	73.5	72.4	73.3	48
	NB	88.9	92.2	91.5	94.4	94.8	45
14	SB	78.7	78.7	76.8	84.5	85.2	64
	NB	103.1	97.7	99.4	87.8	87.8	63
15	SB	93.6	94.4	93.6	98.7	122.9	27
	NB	119.7	105.5	107.2	101.7	106.0	34
16	SB	105.6	109.5	97.7	109.7	97.7	48
	NB	107.5	90.0	96.7	97.7	99.9	36
17	SB	99.9	N/A	94.4	N/A	96.6	71
	NB	96.6	N/A	92.2	N/A	97.2	69
18	SB	89.5	N/A	88.6	N/A	89.0	19
	NB	88.1	N/A	88.5	N/A	90.3	13
19	SB	109.7	N/A	103.1	N/A	100.5	55
	NB	92.4	N/A	97.9	N/A	102.9	58
20	SB	97.7	N/A	99.9	N/A	103.1	67
	NB	99.9	N/A	90.0	N/A	97.7	63

5.2 Speed Prediction Model (First Data Run)

In this stage, the stepwise technique was employed to establish a relationship between the dependent variable and the independent variables. Thus, those independent variables that are not statistically significant, at the 95% confidence level, would be automatically excluded from the model.

As an attempt to develop a model for each of the five points on the curved sections, the data of curves 5 – 16 were used. The reason was that for the rest of the curves, the speed data were not available for all the five points (see Table 5.1). Taking out four sites as potential outliers, the developed models for each of these points are presented in Table 5.3. As it can be realized from the table, no strong relationship was found for the points BC and EC. In addition, despite having a relatively high R^2 value, the model established for the MC lacks what is believed to be the most important variable (r) or any transformation of that. Moreover, some of the independent variables appeared in the models, do not seem logical; namely the radius of the succeeding curve may not affect the speed at the middle of the curve, or G_{tan} may not influence the speed at the departure tangent, or the radius of the preceding curve may not influence the speed at the end of the curve.

Table 5.3: Speed prediction models using data of the curves 5-16

Points	Model	R^2	SEE
AT	$V_{85} = -62.48 + 47.75 (LW) - 0.0015 (AADT) + 6.80 (sp-flag)$	0.697	3.72
BC	No relationship was found	N/A	N/A
MC	$V_{85} = 96.93 - 0.790 (G_{tan}) + 0.00069 (suc_radius)$	0.601	2.60
EC	$V_{85} = 97.05 - 0.00126 (pre_radius)$	0.288	4.80
DT	$V_{85} = 98.71 - 0.628 (G_{tan})$	0.215	4.16

R^2 : Coefficient of determination
 SEE : Standard Error of the Estimate
 G_{tan} : Longitudinal grade at the approach tangent

Due to the fact that the speed data were not available for BC and EC at four out of sixteen curves, it was decided to focus on the other three points on which speed data were available for all sites; namely AT, MC, and DT. However, even for these three points, the results did not seem to be promising in spite of having relatively good R^2 for some points (AT). This may have resulted from the overfitting phenomenon because of the relatively low number of points (twenty points). Therefore, further investigations were deemed necessary before recommending any of these models. Consequently, the second run of data collection was suggested in order to add four more curves (eight sites), with radii different from those of the first sixteen curves, to the database. It should be noted that the selected curves in the first run of data collection did not have sixteen different radii because of the limited range of radii available in the as-built drawings.

5.3 Speed Prediction Model (Second Data Run)

Since the data collected in the second run increased the sample size on three points only (AT, MC, and DT), the model development was just considered for these points. Having four more curves in the database, the author expected to see an improvement in predictability of the operating speed models. However, the new analysis showed no improvement in the models. In case of considering all the twenty curves, no relationship was found for any of the three aforementioned points. Even after the exclusion of six sites with relatively high or low speed values as potential outliers, the R^2 values were limited to 0.291 (see Table 5.4). This confirmed the previous assumption that the models presented in Table 5.3, may have overfitted that data. In addition, no relationship was established between the operating speed and r or any transformation of it.

Table 5.4: Speed prediction models using data of all twenty curves excluding outliers

Points	Model	R ²	SEE
AT	No relationship was found	N/A	N/A
MC	$V_{85} = 97.73 - 0.779 (G_{13})$	0.291	3.12
DT	$V_{85} = 98.86 - 0.640 (G_{13})$	0.127	4.35

R² : Coefficient of determination
 SEE : Standard Error of the Estimate
 G₁₃ : Average longitudinal grade from AT to MC

Regarding the fact that this finding – no relationship established between the operating speed and geometric features of the curve – was in explicit contrast with all the previous studies in the past decades, the calculated 85th percentile values were recalculated to reassure that there was no flaw. This time, the classification of vehicles and determination of the vehicles travelling at free-flow speed, was carried out using SPSS v. 11.5 (instead of using macros). Not surprisingly, the results were exactly the same as the first time calculation.

As another remedy, the curves were categorized based on the class of highways they belonged to. In this regard, three categories were formed; Highway 12 including five curves as a county road, Highway 41 including six curves as a King's Highway, and the combination of Highways 31 and 43 with a total of nine curves as Secondary Highways. After running the regression analysis, models were developed for each of these categories. However, except for the case of Highway 12, no strong relationship was found for the operating speed at the three desired points (see Table 5.5).

Looking at Table 5.5, one can realize that even limiting the conditions to a certain class of highway did not improve the models. The only strong relationship was found for the five curves of Highway 12, and cannot be generalized to all highways. In other words, despite having a high R² value for some curves, the models presented in Table 5.5 cannot be recommended as reliable speed prediction models.

Table 5.5: Speed prediction models using data of all curves (3 different categories)

Hwy	Points	Equation	R ²	SEE
12	AT	$V_{85} = -97.19 + 52.32 (LW) + 18.26 (drv_flag) + 0.0018 (Pre_radius) - 1.47 (SW)$	0.974	1.05
	MC	$V_{85} = 101.4 - 490053 / (r^2)$	0.505	1.62
	DT	No relationship was found	N/A	N/A
41	AT	No relationship was found	N/A	N/A
	MC	No relationship was found	N/A	N/A
	DT	No relationship was found	N/A	N/A
31&43	AT	$V_{85} = 95.69 + 8.17 (curve_dir)$	0.258	7.34
	MC	$V_{85} = 89.33 + 0.0136 (r)$	0.284	4.87
	DT	$V_{85} = 96.58 + 3.83 (Int_flag)$	0.254	3.46

R² : Coefficient of determination
 SEE : Standard Error of the Estimate

Since no reliable model was found to predict the speed at the middle of the curve, it seemed essential to draw the scatter plot of the operating speed at the middle of the curve versus the curve radius for all the curves in the database to see if there is any trend (see Figure 5.1).

As shown in Figure 5.1, no clear trend can be observed in the scatter plot of the operating speed at the middle of the curve against the radius. As a final remedy, the scatter plots of speed at MC versus radius were drawn for different radii ranges, namely ($r < 400\text{m}$), ($r < 500\text{m}$), ($r < 600\text{m}$), ($500 < r < 600$), etc. However, no trend was found in any of the mentioned scatter plots. Therefore, there was no statistically significant relationship, at the 95% confidence level, between the speed at the middle of the curve

and the geometric features of the curve such as radius, length, etc. for the sites considered in this study. Such a finding can be justified regarding the following facts:

- Drivers' choice of speed depends on the length of trip, urgency of trip, and familiarity with the road and level of speed enforcement that cannot be easily addressed by the speed prediction models.
- Such a potentially dominating factor as the perception of speed enforcement in the studies utilizing a radar gun might conceal other factors that normally influence drivers' choice of speed. As a result, that part of the speed database including the drivers who are travelling faster than the speed limit could be excluded. It can be the reason why models showing a strong relationship between the operating speed and the geometric features of the curve were developed for the previous studies in which radar gun was used as the speed data collection device.

Figure 5.1: Observed 85th percentile speed at MC versus radius

5.4 Operating Speed on Approach Tangent

As stated earlier, every effort to account for the variation of approach tangent speed had failed and no reliable equation could be obtained from the database. As the final endeavour, the tangent sections were divided into two categories: non-independent tangents and independent tangents. On the whole, ten out of forty sites have non-independent tangents. Regression analysis was carried out to find a possible relationship between the speed at the approach tangent and other independent variables. However, the analysis showed no statistically significant correlation, at the 95% confidence level, between the approach tangent speed and any of the independent variables. Thus, the value of 103.0 km/h, the average of the speeds on the 30 independent tangents, was recommended as an average 85th percentile speed on independent tangents. On the other hand, the value of 95.75 km/h, the average of the speeds on the ten non-independent tangents, was recommended as an average 85th percentile speed on non-independent tangents.

It is noteworthy that the ANOVA showed that there is a statistically significant difference, at the 95% confidence level, between the recommended value in this study (103.0 km/h) and the values suggested by Lamm and Choueiri (1987), 94.7 km/h, or Krammes et al. (1995), 97.9 km/h for independent tangents. However, it is in a good agreement with the value recommended by Hassan et al. (2000), 102.0 km/h. The observed speed values on independent tangents as well as the recommended average values are shown in Figure 5.2. As shown in the figure, most of the speed observations were above the average values recommended by Lamm and Choueiri (1987) and Krammes et al. (1995). For non-independent tangents, on the other hand, no recommended average value was found in the literature. Thus, the scatter plot of

observed speeds on non-independent tangents in this study including the recommended average speed value is presented in Figure 5.3.

Figure 5.2: Observed 85th percentile speed on independent tangents

Figure 5.3: Observed 85th percentile speed on non-independent tangents

In this chapter it was tried to establish some models to predict the speed at different points of horizontal curves based on the geometric features of the curve. Since the number of observations for heavy trucks was not enough for model development, this task was limited to passenger cars (PC) and light trucks (LT). The regression analysis resulted in no reliable equation for the speed at any of the five points of horizontal curves. In this regard, many efforts were put to find any possible relationships between the dependent and independent variables. For example, the curves belonging to different highway classes were analyzed separately, the second run of data collection was carried out with the purpose of adding some points for model development, and a scatter plot showing the relationship between the operating speed and curve radii was drawn. However, none of these attempts led to a promising result.

The findings of this chapter should not be viewed pessimistically when a more realistic methodology that scrutinizes drivers' behaviour while negotiating the curves, is to be introduced in the next chapter. Typical operating speed values were recommended for independent and non-independent tangents. It should be noted that these values are needed for the speed differential model presented in Chapter Six.

Chapter 6: Speed Differential Model Development

As stated in the literature, a more realistic parameter for evaluating drivers' performance on horizontal curves should be the "speed differential" between tangent and curve (Hirsh, 1987; McFadden and Elefteriadou, 2000). Speed differential is, in fact, calculated for each individual vehicle in the stream of traffic. Consequently, the 85th percentile value is calculated for these observations and introduced as 85th percentile speed differential ($\Delta_{85}V$).

Hirsh (1987) argued that calculating the speed differential by the simple subtraction of the two related 85th percentile values on tangent and curve would not give reasonable results. The rationale behind this statement was suggested to be related to the dissimilarity of speed distributions at the two sections (more variations were expected for the speed distribution at the middle of the curve). The author, however, believes that more variations are expected to be experienced on the tangent sections due to the fact that, theoretically, there is no confining feature as it is the case on the curves (the same was concluded in the study by Fitzpatrick et al., 2000b regarding the speed distribution measures).

Moreover, according to Hirsh (1987), each driver responds differently to the horizontal curve based on his/her desirable tangent speed and the actual side friction factor. Thus, the 85th percentile driver at one section is not necessarily the same 85th percentile driver at the second section. Regarding the mentioned problems, Hirsh (1987) suggested that the full distribution of speed changes as incurred by each driver should be examined to calculate the speed differential value. However, he considered this concept only from the theoretical point of view. In the conclusion of his paper, Hirsh (1987) stated: "While the proposed approach (speed differential) is appealing on a

theoretical basis, its practical implementation is pending on the availability of appropriate databases “

Following Hirsh's proposed methodology, McFadden and Elefteriadou (2000) introduced a new parameter, 85th percentile of maximum speed reduction (*85MSR*). This parameter was proposed in view of bringing the Hirsh's proposed methodology from the state of research to the state of practice. As mentioned earlier, McFadden and Elefteriadou (2000) were able to track the vehicles negotiating curved sections using three lidar guns, and subsequently, the individual speed changes were determined. However, as stated earlier, the following deficiencies can be found in that research.

- The readings of lidar gun may not be reliable due to the fact that it is highly probable that the drivers were being influenced by the presence of the operators. As a result, the observed speed differential values might not represent the actual performance of the drivers on the curved sections.
- The mentioned study was carried out in the United States and the developed model should be examined for its applicability on Canadian roads.
- The speed database used in that study included only 21 sites (100 observations per site) and may not be suitable for speed reduction model development.
- By the definition, *85MSR* cannot show speed increase, because it is calculated by subtracting the maximum speed on tangent and minimum speed on curve. However, some examples can be seen in the database of the present study, where there is an increase of speed from a tangent to a curve.

Considering the mentioned pitfalls, a new methodology to investigate the realistic drivers' performance on curves is presented in this chapter. In order to achieve this goal, first, the methodology for calculating the 85th percentile speed differential is explained. Other sections of this chapter include speed differential model development, model

diagnostics, and finally, model verification. It should be noted that the independent variables used for speed differential model development, were exactly the same as those employed for operating speed model development as explained in Chapter Five.

6.1 Calculation of 85th Percentile Speed Differential

As mentioned before, the data collection scheme dictated using five counters on each curve. However, after collecting the speed data of the first twelve curves, one of the counters was stolen. Therefore, only the three points believed to be the most critical, were chosen for counter installation on the latest eight curves. These locations were the approach and departure tangents plus the point of the middle of the curve (AT, DT, and MC). The rationale behind choosing the mentioned points was having a symmetric distribution of counters for the two lanes as well as considering the locations where the extreme speed values might be seen (points 1, 3, and 5 in Figure 3.6). In summary, for the first twelve curves, the speed data were available at five points on each curve, while for the other eight curves the data were available at three points only.

Among the mentioned points, half of them were used to investigate the performance of the drivers while they were approaching the curve at each lane. For example, in case of having five points, the first three points would cover the vehicle path from the approach tangent to the middle of the curve for one lane of traffic (points 1, 2, and 3 respectively). Similarly, for the other lane of traffic, the other three points (the point of MC is common between the two directions) would cover the vehicle path from the approach tangent to the middle of the curve (points 5, 4, and 3 respectively). In case of dealing with three points, points 1 and 3 were considered for one direction and points 5 and 3 were considered for the other direction of travel.

Since all the five (or three) counters were synchronized with the laptop computer, as mentioned in Section 3.2.3, the feasibility of tracing every individual vehicle

approaching the curve was logically expected. However, there were some problems that prevented the tracking of all individual vehicles negotiating the curve. First, there was at least one intersection or driveway on the majority of the curves whose speed data were collected. Such a configuration could have caused some of the vehicles recorded by one counter, not to be recorded by the second or third counter in case of turning to an intersection or a driveway. Second, the timer circuits implemented in the electronic counters were not very precise. This was observed through synchronizing the counters with the laptop computer in the morning and finding the time error of up to 36 seconds in the next day for some counters.

Due to the aforementioned problems, the necessity of establishing an algorithm to track the vehicles as they negotiated the curve from one point to the other can be understood. Such an algorithm would involve a series of logical statements to trace the vehicles using the recorded features of the vehicles namely, time stamp, number of axles, and wheelbase. Therefore, a vehicle-tracking macro including the following three conditions was created in Microsoft Excel environment.

- 1) The time gap between the two observations recorded by two successive counters should be limited to an estimated value.
- 2) The number of axles recorded by the two successive counters should be the same.
- 3) The difference between the two recorded wheelbases should be in the range of ± 50 cm.

The first condition, limited time gap, is the most tangible evidence for tracing a vehicle along a curve. In fact, the expected time interval between two observations recorded by two successive counters can be calculated using the distance between the counters and the average speed of the two observations. In this connection, the speeds of the vehicles recorded by the first counter were first printed in specific cells in an Excel

file. Then, the tracking started from the first vehicle recorded by the second counter; having the speed of this vehicle as well as that of the first vehicle recorded by the first counter, the average speed and the expected time interval were calculated. If this time interval was in agreement with the difference between the related observed time stamps, with an acceptable tolerance, the vehicle would pass the first condition. Otherwise, other vehicles in the queue should be tried one by one.

In an ideal world, the time difference calculated from the speed formula (time = distance/average speed) should be exactly the same as the difference between the two related time stamps given by the counters. However, due to the inaccuracy of the electronic timers, this did not happen in reality. As a result, it was necessary to introduce a tolerance value (Δt) to account for the uncertainties in speed and time measurements. The proper range for this value was found by trial and error and ranged from six to fifteen seconds depending on the length of the curve.

Once a vehicle successfully passed the first condition, the other two conditions were examined. In this regard, the second condition necessitated the number of axles to be identical. This condition was implemented as a complementary statement to the first condition. It was, in truth, used to compensate for the possible confusion imposed by the vehicles traversing the curve very close to each other, and as a result their time stamps could be very similar to each other. The third condition, limited difference between the wheelbases, can be also thought of as a complementary condition to the previous ones. The recommended tolerance range, ± 50 cm, accounted for the inaccuracy of the wheelbase calculated by the counters. This inaccuracy may have originated from the limited precision in installing the sensors (pneumatic tubes), and also the assumptions made in the calculation of wheelbase.

However, if a vehicle recorded by the first counter did not meet the first condition, it would be marked as a non-traceable vehicle and would not go through the rest of

conditions. Failing in the first test (condition1) means that there was no vehicle, among the vehicles recorded by the second counter, to match the mentioned vehicle in terms of the time interval. Therefore, it seemed logical that no more tests should be carried out for such a non-traceable vehicle. It should be noted that such a vehicle was not tracked over the third counter either.

A vehicle was flagged as a traceable, provided that the three mentioned conditions were satisfied while it passed through the three points (two points in time of having three counters). When a tracking flag was activated (flag=1) for a vehicle, it meant that the vehicle had already been traced and should be excluded from the queue of the vehicles to be traced. After marking all the vehicles as either traceable or non-traceable, the traceable percentage was calculated and the post-processing tasks continued only for the traceable vehicles. Before post-processing stage, the information about the vehicles that were traced including, time stamp, speed, number of axles, and wheelbase, were printed in a separate Excel sheet. This step made it much easier for the model developer to refer to the processed data and check the suitability and accuracy of the methodology.

The post-processing tasks were automatically performed using another macro. These tasks consisted of vehicle classification, headway calculation, speed differential calculation, and tabulation of the final results. Vehicle classification was carried out using the same procedure as stated in Section 4.2. The three classes of vehicles (PC, LT, and HT) were defined in the macro while PC/LT was considered as one class. The headway calculation was carried out for the traceable vehicles and consequently the vehicles with the headway value of less than five seconds were disregarded.

Having in hand the traceable vehicles travelling at the free-flow speed, three speed differential values were calculated for each vehicle: the speed differential from point 1 to point 2 (ΔV_{12}), the speed differential from point 2 to point 3 (ΔV_{23}), and the

speed differential from point 1 to point 3 (ΔV_{13}). Obviously, for the curves on which only three counters were installed, the only available speed differential value was ΔV_{13} . Speed differential values for different vehicle classes were calculated using two macros, one for PC/LT and the other one for HT. As an example of speed differential calculation, a sample of the original Excel file as well as the output file resulted from running the macro for curve 10 are shown in Tables 6.1 and 6.2.

There are some issues in Table 6.2 that need more explanations. First, the word “N/A” can be seen in the place of the first headway value. The reason is that for the first reading of the database, the headway value cannot be calculated. Second, some of the calculated speed differential values are negative. This means that there was an increase of speed from the tangent section to the middle of the curve. Such a trend was observed for some parts of the database and is in clear contrast with all previous studies defining the general concept of “speed differential” as “speed reduction”. Third, in the sample file presented in Table 6.2, the vehicles were traced over all five points; however, if a vehicle was traced over the first three points only, it was still marked as “Traceable”. Finally, the “tracking flag” and “headway flag” had the value of 1 if the vehicle was traceable and travelling at the free-flow speed respectively. Otherwise, these parameters had the value of zero.

Table 6.1: A sample of original Excel file

Location	Lane	Time Stamp	Speed (km/h)	No. of Axles	WB (cm)
AT (Point 1)	1	11:30:27 AM	94.4	2	219
	1	11:30:30 AM	90.0	2	241
	1	11:30:53 AM	104.2	2	351
	1	11:31:12 AM	40.6	5	1554
	1	11:32:30 AM	90.0	2	369
BC (Point 2)	1	11:30:31 AM	92.2	2	216
	1	11:30:33 AM	87.8	2	238
	1	11:30:56 AM	95.5	2	344
	1	11:31:21 AM	37.3	5	1454
	1	11:32:34 AM	88.9	2	360
MC (Point 3)	1	11:30:35 AM	96.6	2	223
	1	11:30:37 AM	88.9	2	247
	1	11:31:00 AM	97.7	2	357
	1	11:31:32 AM	37.3	5	1454
	1	11:32:38 AM	95.5	2	372
EC (Point 4)	1	11:30:40 AM	90.0	2	207
	1	11:30:43 AM	83.4	2	229
	1	11:31:05 AM	90.0	2	332
	1	11:31:43 AM	37.3	5	1433
	1	11:32:43 AM	93.3	2	344
DT (Point 5)	1	11:30:44 AM	94.4	2	216
	1	11:30:48 AM	87.8	2	238
	1	11:31:10 AM	94.4	2	344
	1	11:31:55 AM	38.4	5	1460
	1	11:32:47 AM	100.9	2	363

Table 6.2: A sample of vehicle tracking macro output

Vehicle #	Points	Speed (km/h)	No. of Axles	Length (cm)	Time Stamp	Tracking Flag	Headway (Second)	Headway Flag	Vehicle Class	$\Delta V_{1,2}$	$\Delta V_{1,3}$	$\Delta V_{2,3}$
1	1	94.4	2	219	11:30:27	1	N/A	N/A	PC			
	2	92.2	2	216	11:30:31							
	3	96.6	2	223	11:30:35							
	4	90.0	2	207	11:30:40							
	5	94.4	2	216	11:30:44							
2	1	90.0	2	241	11:30:30	1	3	0	PC			
	2	87.8	2	238	11:30:33							
	3	88.9	2	247	11:30:37							
	4	83.4	2	229	11:30:43							
	5	87.8	2	238	11:30:48							
3	1	104.2	2	351	11:30:53	1	23	1	LT	8.70	6.50	-2.20
	2	95.5	2	344	11:30:56							
	3	97.7	2	357	11:31:00							
	4	90.0	2	332	11:31:05							
	5	94.4	2	344	11:31:10							
4	1	40.6	5	1554	11:31:12	0	19	1				
	2											
	3											
	4											
	5											
5	1	90.0	2	369	11:32:30	1	78	1	LT	1.10	-5.50	-6.60
	2	88.9	2	360	11:32:34							
	3	95.5	2	372	11:32:38							
	4	93.3	2	344	11:32:43							
	5	100.9	2	363	11:32:47							

The last post-processing task carried out by the related macro was the tabulation of the final results. This included the calculated 85th percentile value of the speed differentials ($\Delta_{85}V$) for the two lanes of traffic, the number of observations, etc. A summary of the final results is presented in Tables 6.3 and 6.4 for PC/LT and HT respectively. The reason for the existence of some “not available” cells is that on some curves only three counters were available.

The statistic “ $\Delta_{85}V$ ”, the 85th percentile speed differential, is introduced, for the first time, in this study. It is defined as *the differential speed not exceeded by 85% of the drivers travelling under free-flow condition*. This statistic, as mentioned earlier, is calculated by taking the 85th percentile value of all the individual speed differential values. It is noteworthy that $\Delta_{85}V$ is fundamentally different from ΔV_{85} . The latter statistic is calculated by simply subtracting the two 85th percentile speeds of the two related successive elements.

Looking at Table 6.3, one can realize that the number of PC/LT observations for some of the curves namely, 14, 15, 16, and 19 may not be enough for accurate determination of $\Delta_{85}V$. In such cases, the calculated 85th percentile speed differential values were disregarded for model development. In the case of HT (Table 6.4), however, the number of observations was too low for proper determination of $\Delta_{85}V$, and therefore speed differential model development for HT was aborted.

Table 6.3: Observed 85th percentile speed differential values for PC/LT

Curve No.	Lane	$\Delta_{85}V_{1-2}$ (km/h)	$\Delta_{85}V_{2-3}$ (km/h)	$\Delta_{85}V_{1-3}$ (km/h)	No. of Observations
1	SB	N/A	N/A	10.95	731
	NB	N/A	N/A	0.00	694
2	SB	N/A	N/A	20.80	672
	NB	N/A	N/A	-2.20	699
3	SB	N/A	N/A	6.60	314
	NB	N/A	N/A	28.94	145
4	SB	N/A	N/A	6.50	321
	NB	N/A	N/A	16.50	241
5	EB	-3.30	-1.10	-6.60	461
	WB	5.50	5.50	9.90	511
6	EB	2.20	11.00	13.10	631
	WB	5.72	3.30	8.80	633
7	EB	20.83	2.20	20.80	419
	WB	-3.30	8.80	4.40	786
8	EB	3.30	2.20	4.40	922
	WB	-1.10	1.10	-1.65	698
9	SB	8.80	-0.61	7.70	478
	NB	-2.20	6.60	2.20	416
10	SB	4.40	0.00	4.40	472
	NB	-0.10	8.80	6.60	435
11	SB	-4.40	1.10	-5.50	504
	NB	15.45	12.15	23.10	114
12	SB	9.90	12.16	21.96	134
	NB	11.10	7.70	19.93	127
13	SB	-1.10	16.40	13.20	248
	NB	1.10	4.40	4.25	288
14	SB	13.20	8.80	18.67	50
	NB	14.30	-3.30	9.90	239
15	SB	14.68	1.10	14.04	25
	NB	35.65	0.00	32.63	16
16	SB	10.68	20.18	11.00	45
	NB	20.80	-3.30	15.30	201
17	SB	N/A	N/A	14.30	589
	NB	N/A	N/A	12.10	455
18	SB	N/A	N/A	3.30	163
	NB	N/A	N/A	15.93	166
19	SB	N/A	N/A	21.97	50
	NB	N/A	N/A	6.60	142
20	SB	N/A	N/A	9.90	271
	NB	N/A	N/A	18.64	112

Table 6.4: Observed 85th percentile speed differential values for HT

Curve No.	Lane	$\Delta_{85}V_{1-2}$ (km/h)	$\Delta_{85}V_{2-3}$ (km/h)	$\Delta_{85}V_{1-3}$ (km/h)	No. of Observations
1	SB	N/A	N/A	9.65	42
	NB	N/A	N/A	-2.20	21
2	SB	N/A	N/A	N/A	0
	NB	N/A	N/A	-3.08	33
3	SB	N/A	N/A	N/A	0
	NB	N/A	N/A	N/A	0
4	SB	N/A	N/A	N/A	0
	NB	N/A	N/A	N/A	0
5	EB	-4.18	0.44	-5.20	13
	WB	3.30	7.10	9.30	18
6	EB	1.58	10.73	10.38	16
	WB	3.30	3.30	6.60	41
7	EB	4.4	-1.10	3.30	1
	WB	-4.95	7.15	1.10	11
8	EB	2.20	1.10	2.20	61
	WB	-2.20	0.00	-2.97	39
9	SB	6.49	-1.32	3.19	15
	NB	-1.10	3.08	0.00	9
10	SB	3.30	-1.10	2.80	17
	NB	-3.69	4.40	-1.10	10
11	SB	-3.30	-3.30	-6.60	1
	NB	N/A	N/A	N/A	0
12	SB	N/A	N/A	N/A	0
	NB	N/A	N/A	N/A	0
13	SB	N/A	N/A	N/A	0
	NB	0.00	3.30	2.20	33
14	SB	N/A	N/A	N/A	0
	NB	16.85	-2.47	13.59	12
15	SB	N/A	N/A	N/A	0
	NB	N/A	N/A	N/A	0
16	SB	N/A	N/A	N/A	0
	NB	N/A	N/A	N/A	0
17	SB	N/A	N/A	12.70	37
	NB	N/A	N/A	16.50	25
18	SB	N/A	N/A	-1.98	7
	NB	N/A	N/A	8.96	10
19	SB	N/A	N/A	N/A	0
	NB	N/A	N/A	N/A	0
20	SB	N/A	N/A	N/A	0
	NB	N/A	N/A	N/A	0

At the end of this section, it seems plausible to compare the observed 85th percentile speed differential values ($\Delta_{85}V$) with their comparable values obtained by subtracting the two related 85th percentile speeds ($V_{85(i+1)} - V_{85(i)}$). This comparison is shown in Table 6.5 wherein the observed speed differential values between the different points on each curve are shown next to the value obtained from the simple subtraction of the operating speeds. As it can be seen in the table, the traditional way of calculating speed differential, mainly, underestimates the realistic values. This agrees with the results of the study by McFadden and Elefteriadou (2000).

There are, however, some cases for which the speed differential value is negative (the increase of speed). In these cases, the value given by the traditional way is, sometimes higher than the other value. This can be easily justified as the weakness of the traditional way in capturing the speed increase. In such cases, it can be said that the speed differential value given by the tracking system is the realistic value.

In order to have a better visualization of the difference between the speed differential values obtained from the two methods, the $\Delta_{85}V_{1-3}$ is drawn versus $\Delta V_{85(1-3)}$ (see Figure 6.1). As it can be seen in the figure, a linear trend can be observed in the scatter plot that can be quantified as:

$$\Delta_{85}V_{1-3} = 7.55 + 0.97 \times \Delta V_{85(1-3)} \quad (R^2 = 0.77) \quad (6.1)$$

The Equation 6.1 shows that for the speed data observed in this study, there is a linear relationship between the speed differentials obtained using the two mentioned methods. As expected, the realistic value of speed differential ($\Delta_{85}V_{1-3}$) is greater than the speed change calculated by subtracting the two related operating speeds ($\Delta V_{85(1-3)}$).

Table 6.5: Comparison between the speed differential values using two methods

Curve No.	Lane	Points 1 and 2		Points 1 and 3		Points 2 and 3	
		$\Delta_{85}V_{1-2}$	$V_{85(1)} - V_{85(2)}$	$\Delta_{85}V_{1-3}$	$V_{85(1)} - V_{85(3)}$	$\Delta_{85}V_{2-3}$	$V_{85(2)} - V_{85(3)}$
1	SB	N/A	N/A	10.95	4.3	N/A	N/A
	NB	N/A	N/A	0.00	-4.4	N/A	N/A
2	SB	N/A	N/A	20.80	17.5	N/A	N/A
	NB	N/A	N/A	-2.20	-7.7	N/A	N/A
3	SB	N/A	N/A	6.60	-5.4	N/A	N/A
	NB	N/A	N/A	28.94	19.7	N/A	N/A
4	SB	N/A	N/A	6.50	-9.9	N/A	N/A
	NB	N/A	N/A	16.50	6.5	N/A	N/A
5	EB	-3.30	-6.6	-6.60	-12.0	-1.10	-5.4
	WB	5.50	3.2	9.90	6.5	5.50	3.3
6	EB	2.20	-1.1	13.10	4.3	11.00	5.4
	WB	5.72	1.1	8.80	4.4	3.30	3.3
7	EB	20.83	7.6	20.80	6.5	2.20	-1.1
	WB	-3.30	-6.5	4.40	1.1	8.80	7.6
8	EB	3.30	2.2	4.40	2.4	2.20	0.2
	WB	-1.10	-3.3	-1.65	-4.4	1.10	-1.1
9	SB	8.80	7.6	7.70	1.1	-0.61	-6.5
	NB	-2.20	-6.5	2.20	-2.2	6.60	4.3
10	SB	4.40	2.1	4.40	0.0	0.00	-2.1
	NB	-0.10	-4.4	6.60	2.1	8.80	6.5
11	SB	-4.40	-8.8	-5.50	-9.9	1.10	-1.1
	NB	15.45	1.5	23.10	9.1	12.15	7.6
12	SB	9.90	6.9	21.96	5.8	12.16	-1.1
	NB	11.10	9.9	19.93	4.4	7.70	-5.5
13	SB	-1.10	-5.4	13.20	8.8	16.40	14.2
	NB	1.10	-4.3	4.25	-1.0	4.40	3.3
14	NB	14.30	13.1	9.90	6.6	-3.30	-6.5
16	NB	20.80	19.7	15.30	13.1	-3.30	-6.6
17	SB	N/A	N/A	14.30	10.9	N/A	N/A
	NB	N/A	N/A	12.10	1.7	N/A	N/A
18	SB	N/A	N/A	3.30	-3.3	N/A	N/A
	NB	N/A	N/A	15.93	1.1	N/A	N/A
19	NB	N/A	N/A	6.60	-3.3	N/A	N/A
20	SB	N/A	N/A	9.90	-1.9	N/A	N/A
	NB	N/A	N/A	18.64	10.9	N/A	N/A

Figure 6.1: The scatter plot of $\Delta_{85}V_{1-3}$ versus $\Delta V_{85(1-3)}$.

Similar to the procedure followed for operating speed model development (Chapter Five), the regression analysis was carried out at two stages using SPSS v. 11.5: for the first sixteen curves (using the data collected at the first run), and for all twenty curves (using the data collected at the two runs of data collection).

6.2 Speed Differential Model (First Data Run)

In this stage, the ordinary least squares general linear regression was used for the model development. In this connection, a stepwise technique was employed to establish a possible relationship between the dependent variable and the independent variables. Using this technique, those independent variables that are not statistically significant, at the 95% confidence level, would be automatically excluded from the model. There are three dependent variables for which the model development is

considered; speed differential from AT to BC ($\Delta_{85}V_{1-2}$), speed differential from BC to MC ($\Delta_{85}V_{2-3}$), and speed differential from AT to MC ($\Delta_{85}V_{1-3}$).

Having the data of the first sixteen curves, the model development for the mentioned dependent variables can be considered only for curves 5–16. As stated earlier, the three speed differential values ($\Delta_{85}V_{1-2}$, $\Delta_{85}V_{2-3}$, and $\Delta_{85}V_{1-3}$) were available only for the mentioned curves (see Table 6.3). For the four other curves (1–4), the only available speed differential value was $\Delta_{85}V_{1-3}$. Excluding four sites due to not having enough number of observations (see Table 6.3), models correlating the speed differential to some of the independent variables were found as shown in Table 6.6. In order to have a better understanding of the non-linear model established for $\Delta_{85}V_{1-3}$, the graphs relating this model to a logical range of radii (e.g., 300 m – 1000 m) for three different values of tangent speeds namely, 90, 100, and 110 km/h are shown in Figure 6.2. As it can be seen in the figure, the radius of the curve has a great effect on the speed differential value for the radii less than 400 m, while for the radii greater than 400 m no relationship exists between the mentioned variables. The impact of the tangent speed can be also easily realized in the figure regarding the fact that there is a clear shift between the graphs belonging to each category of tangent speed.

Table 6.6: Speed differential models using data of the curves 5–16

Model	R ²	SEE
$\Delta_{85}V_{1-2} = -93.95 + 1.00 (V_{tangent})$	0.529	5.54
$\Delta_{85}V_{2-3}$: No relationship was found	N/A	N/A
$\Delta_{85}V_{1-3} = -91.86 + 0.99 (V_{tangent}) + e^{-12.5 + 4679.23 / r}$	0.572	N/A

R² : Coefficient of determination

SEE : Standard Error of the Estimate

Figure 6.2: The relationship between the radius of the curve and $\Delta_{85}V_{1-3}$ for curves 5–16

As it can be realized from Table 6.6, no relationship was found for the speed change from BC to MC ($\Delta_{85}V_{2-3}$). Speed change from AT to BC ($\Delta_{85}V_{1-2}$), on the other hand, correlated relatively well to the speed at the approach tangent. However, this relationship could not be improved after the second run of data collection because of the lack of the related data. Finally, for the speed differential from AT to MC ($\Delta_{85}V_{1-3}$), a fairly good relationship was found with the approach tangent speed and r . This relationship can be enhanced using four more curves in the second run of data collection. As a result, the next section would focus on the improvement of the model for $\Delta_{85}V_{1-3}$.

6.3 Speed Differential Model (Second Data Run)

As explained in Section 6.2, the only speed differential parameter that can be considered for the whole database was $\Delta_{85}V_{1-3}$. Among the 40 sites whose speed data were available, five sites should be excluded due to the lack of enough number of

observations (see Table 6.3). Therefore, the total number of applicable sites became 35. With the purpose of finding the best model, four different data sets were chosen from the database and for each of them a regression analysis was carried out.

The first data set included all 35 curve sites, while in the second data set three sites (13, 14, and 26) were excluded due to their proximity to another curve. The third data set is the same as the second one except that six additional sites (1, 2, 9, 10, 17, and 18) were excluded for model verification. Finally, the fourth data set was formed by taking out two sites (6 and 13) as outliers (based on the graph of standardized residuals versus standardized predicted values) in addition to the same six sites excluded in the third data set.

The resultant established speed differential model for each data set is reflected in Table 6.7 (Models 1– 4 respectively). It is noteworthy that the percentage of sites eliminated as outliers was 6% - 9% of the whole database, which is much less than the acceptable value of 20%. Statistical summary of the significant variables is depicted in Table 6.8.

Regarding Table 6.7, some explanations about the dependent and independent variables are essential. First, a positive value of speed differential means a speed reduction from the tangent section to the middle of the curve, and the negative value means an increase of speed. Second, the independent variables appear in the models in order of their contribution to the model. The speed at the tangent section, for instance, made the largest contribution to the R^2 value of all the models. Third, the elimination of outliers substantially improved the predictability of the models.

The non-linear Model introduced in Table 6.7 (Model 1a) is, in fact, the improved version of the model introduced in Table 6.6 for $\Delta_{85}V_{1-3}$, using all the curves. As it can be seen, there is a small difference in the resultant coefficients and R^2 values. To have a better visualization of the non-linear model established for $\Delta_{85}V_{1-3}$, the graphs relating

this model to a logical range of radii (e.g., 300 m – 1000 m) for three different values of tangent speeds namely, 90, 100, and 110 km/h are shown in Figure 6.3. Also, the observed speed differential values are shown in the same figure to make it easier for the reader to understand the goodness of fit for the presented model. As it can be seen in the figure, the radius of the curve has a great effect on the speed differential value for the radii less than approximately 450 m, but this effect tends to diminish after the 450-m-radius. The impact of the tangent speed can be also easily realized in the figure regarding the fact that there is a clear shift between the graphs belonging to each category of tangent speed.

Figure 6.3: The relationship between the radius of the curve and $\Delta_{85}V_{1-3}$ for all curves

Comparing the R^2 values as well as the standard error of estimates (SEE) for the models presented in Table 6.7, Model 3 is recommended due to having the highest R^2 and lowest SEE value. It is clear that further investigation is necessary in terms of model diagnostics and verification (see Sections 6.4 and 6.5). It should be emphasized that all

the independent variables are statistically significant at the 95% confidence level (see Table 6.8). In order to make it easy for designers to decide about the independent variables that are to be included in the recommended model, the base models that can be derived from that model (Model 3) are shown in Table 6.9.

Table 6.7: Speed differential models using data of all curves

No.	Model	R ²	SEE
1	$\Delta_{85}V_{1-3} = -174.47 + 18.89(\sqrt{V_{\text{tangent}}}) + 0.09(DFC) - 3.66(SW)$	0.671	5.06
1a	$\Delta_{85}V_{1-3} = -83.63 + 0.93(V_{\text{tangent}}) + e^{-8.93 + 3507.10/r}$	0.640	N/A
2	$\Delta_{85}V_{1-3} = -204.00 + 22.01(\sqrt{V_{\text{tangent}}}) + 0.10(DFC) - 5.25(SW) + 4.14(drv_flag) + 1.41(G_{13}) - 4.80(curve-dir) + 0.0006(pre-rad)$	0.879	3.36
3	$\Delta_{85}V_{1-3} = -198.74 + 21.42(\sqrt{V_{\text{tangent}}}) + 0.11(DFC) - 4.55(SW) - 5.36(curve-dir) + 1.30(G_{13}) + 4.22(drv_flag)$	0.889	3.21
4	$\Delta_{85}V_{1-3} = -184.71 + 20.12(\sqrt{V_{\text{tangent}}}) + 0.10(DFC) - 4.34(SW) - 5.67(curve-dir) + 0.89(G_{13}) + 3.19(drv_flag)$	0.854	3.31

R² : Coefficient of determination

SEE : Standard Error of the Estimate

G₁₃ : Longitudinal grade from point 1 to point3

Table 6.8: Statistical summary of the significant variables

Model No.	Independent Variables	Coefficients	t-statistic	p-value	MSE
1	Constant	-174.468	-6.817	< 0.001	25.599
	$\sqrt{V_{\text{tangent}}}$	18.887	7.512	< 0.001	
	DFC	0.090	2.995	0.005	
	SW	-3.664	-2.254	0.031	
2	Constant	-204.003	-11.261	< 0.001	11.296
	$\sqrt{V_{\text{tangent}}}$	22.015	12.120	< 0.001	
	DFC	0.098	3.989	0.001	
	SW	-5.252	-4.504	< 0.001	
	drv_flag	4.139	2.997	0.006	
	G ₁₃	1.408	3.619	0.001	
	curve_dir	4.796	-3.461	0.002	
	pre_radius	< 0.00158	2.068	0.050	
3	Constant	-198.739	-10.601	< 0.001	10.291
	$\sqrt{V_{\text{tangent}}}$	21.419	11.436	< 0.001	
	DFC	0.110	4.595	< 0.001	
	SW	-4.546	-4.185	0.001	
	curve_dir	-5.359	-3.846	0.001	
	G13	1.300	3.485	0.002	
	drv_flag	4.222	2.930	0.009	
4	Constant	-184.707	-8.722	< 0.001	10.938
	$\sqrt{V_{\text{tangent}}}$	20.123	9.450	< 0.001	
	DFC	0.100	4.217	< 0.001	
	SW	-4.336	-3.911	0.001	
	curve_dir	-5.665	-3.995	0.001	
	G13	0.889	3.151	0.005	
	drv_flag	3.192	2.140	0.045	

MSE: Mean of Squared Errors

G₁₃ : Longitudinal grade from point 1 to point3

Table 6.9: Base models derived from Model 3

Sub-Model No.	Independent Variables	Coefficients	t-statistic	p-value	R ²	SEE
1	Constant	-151.492	-4.877	< 0.001	0.534	5.838
	$\sqrt{V_{\text{tangent}}}$	16.200	5.241	< 0.001		
2	Constant	-168.100	-5.821	< 0.001	0.634	5.283
	$\sqrt{V_{\text{tangent}}}$	17.419	6.136	< 0.001		
	DFC	0.087	2.512	0.019		
3	Constant	-163.986	-6.129	< 0.001	0.701	4.883
	$\sqrt{V_{\text{tangent}}}$	17.758	6.756	< 0.001		
	DFC	0.106	3.183	0.004		
	SW	-3.623	-2.218	0.037		
4	Constant	-183.659	-7.384	< 0.001	0.775	4.330
	$\sqrt{V_{\text{tangent}}}$	20.098	8.059	< 0.001		
	DFC	0.108	3.661	0.001		
	SW	-4.102	-2.810	0.011		
	curve_dir	-4.888	-2.640	0.015		
5	Constant	-193.007	-8.815	< 0.001	0.838	3.767
	$\sqrt{V_{\text{tangent}}}$	20.987	9.571	< 0.001		
	DFC	0.131	4.862	< 0.001		
	SW	-4.326	-3.399	0.003		
	curve_dir	-5.634	-3.451	0.003		
	G ₁₃	1.217	2.784	0.011		
6	Constant	-198.737	-10.601	< 0.001	0.889	3.207
	$\sqrt{V_{\text{tangent}}}$	21.419	11.436	< 0.001		
	DFC	0.110	4.595	< 0.001		
	SW	-4.546	-4.185	0.001		
	curve_dir	-5.359	-3.846	0.001		
	G ₁₃	1.300	3.485	0.002		
	drv_flag	4.222	2.930	0.009		

R² : Coefficient of determination

SEE : Standard Error of the Estimate

V_{tangent} : Speed at the approach tangent (km/h)G₁₃ : Longitudinal grade from point 1 to point 3

The logical explanation about the obtained coefficients in this model is presented here. With the increase of tangent speed, the value of speed differential will go up. This is logical because the drivers driving at higher speeds at the tangent section need more speed adjustment when they encounter the curve. The deflection angle of the curve also directly affects the speed differential. This can be justified by the fact that more speed reduction is imposed on the drivers negotiating a curve with higher deflection angle. When the shoulder width increases, the speed differential will decrease. This can be explained by the fact that drivers face less speed reduction in the alignment with wider shoulder width as they think that they can easily move to the shoulder if necessary. In the case of having a right turn curve, the speed differential will be less than that in the case of a left turn curve, the fact that was observed in the videos taken from the curves. Together with the increase of average longitudinal grade, between the tangent section and the middle of the curve, the speed differential will increase. This can be justified by the fact that drivers will encounter more speed reduction on uphill sections. Finally, in case of having some driveways on a curve, the speed differential will increase as the drivers may be more cautious about the possible cars coming from those driveways.

6.4 Model Diagnostics

After establishing a model for predicting speed differential, verifying the underlying assumptions of the “general linear model” is inevitable. In this regard, three classical linear regression model (CLRM) assumptions, namely linearity, homoscedasticity, and independence should be checked (Tabachnick and Fidell, 2001). Looking at the scatter plot of the standardized residuals versus standardized predicted values (Figure 6.4), one can realize that the assumption of linearity is met. The reason is that the scatter points have spread almost symmetrically about a line passing through the zero value of standardized residuals and is parallel to the x-axis.

“The assumption of homoscedasticity is that the variability in scores for one continuous variable is roughly the same at all values of another continuous variable.” (Tabachnick and Fidell, 2001). From Figure 6.4, it can also be understood that there is no problem of heteroscedasticity, defined as the failure of homoscedasticity. In this case, the reason is observing no specific trend (e.g., a fan shaped trend) in the scatter plot. Therefore, the assumption of homoscedasticity was also satisfied. The last assumption, independence, was also met because the speed database consists of the curves randomly selected from the population.

Since the CLRM assumptions are all met for the recommended model, the ordinary least square equation is BLUE (Best Linear Unbiased Estimator). In addition to the CLRM assumptions, another issue, multicollinearity, should be checked for the model. For this purpose, the correlation factors among the different independent variables belonging to the developed model were checked (see Table 6.10). It was, then, concluded that there was no strong relationship among the employed independent variables. In addition, as shown in Table 6.10, the related tolerance values ranged from 0.753 to 0.916 (any value greater than 0.5 is acceptable). As a result, the multicollinearity was not an issue of concern. Therefore, the recommended model passed all the required diagnostics.

Table 6.10: Pearson correlation factors and tolerance values

Independent Variable	Pearson correlation factors						Tolerance
	$\sqrt{V_{\text{tangent}}}$	<i>DFC</i>	<i>SW</i>	<i>curve_dir</i>	<i>G₁₃</i>	<i>drv_flag</i>	
$\sqrt{V_{\text{tangent}}}$	1.000	-0.171	0.014	0.352	-0.036	-0.153	0.822
<i>DFC</i>	-0.171	1.000	0.242	-0.063	-0.288	0.367	0.753
<i>SW</i>	0.014	0.242	1.000	-0.108	-0.036	0.150	0.916
<i>curve_dir</i>	0.352	-0.063	-0.108	1.000	0.129	-0.132	0.835
<i>G₁₃</i>	-0.036	-0.288	-0.036	0.129	1.000	-0.173	0.878
<i>drv_flag</i>	-0.153	0.367	0.150	-0.132	-0.173	1.000	0.842

Figure 6.4: Scatter plot of standardized residuals versus standardized predicted values

6.5 Model Verification

The cross validation of the developed model is presented in this section. The purpose of doing this task was verifying the predictability of the model for the part of the database that has not been used in model development. As explained in Section 6.3, six sites out of 32 sites (18.8%) were put aside for model verification. As the first step toward the model verification, the bar chart showing the observed and predicted speed differentials for all the sites is shown in Figure 6.5. As stated earlier, sites 1, 2, 9, 10, 17, and 18 were used for model verification and the rest of the sites were employed in model

development. As it can be seen in the figure, there is a good conformance between the predicted and observed speed differential values for the majority of cases.

In order to have a better understanding of the relationship between the observed and predicted values, these two parameters were drawn versus each other in Figure 6.6. In this figure, the horizontal axis shows the observed value while the vertical axis shows the predicted value by Model 3. Again, most of the points are spread very closely to the line of $y = x$. As a result, Model 3 of Table 6.7 passes the verification test and its applicability is confirmed.

Figure 6.5: Comparison between the observed and predicted values

Figure 6.6: Predicted versus observed speed differential values

Chapter 7: Conclusions and Recommendations

7.1 Summary of the Research

In this study, the essentiality of highway geometric design consistency evaluation was first investigated. Then, the operating speed methodology for evaluating the consistency of design was scrutinized. This methodology was chosen, among other existing approaches, due to its simplicity and quantifiability. Based on the literature review, other methodologies either have not evolved enough to be used as the only design consistency measure or are too complicated to be quantified. For instance, the alignment indices method is believed to be useful only as a complementary approach to the operating speed approach. Driver workload, on the other hand, is very difficult to quantify. In addition, the procedure includes some subjective ratings for which no clear assessment criterion has been found to date.

As mentioned earlier, the current design practice in North America does not ensure the consistency of the design. AASHTO (2001) just implies the concept, and it is the responsibility of the designer to consider the design consistency. TAC (1999), on the other hand, briefly addresses some of the design consistency evaluation measures, but no criterion is presented. In addition, the recommended model for speed prediction in the TAC guide is, in fact, a model established based on the U.S. speed database that may not be applicable to the Canadian roads.

Based on the literature, the consistency of a design, according to the operating speed approach, is evaluated using one of two ways. In the first method, the operating speed and the design speed for an element of a highway are compared. The second method, on the other hand, considers the difference between the operating speeds at two successive elements of a highway. In both cases, the related differences are limited

to certain values that are given by the existing criteria. Therefore, the first step toward the design consistency evaluation is the development of the models to predict the operating speed values as well as the speed differential values.

Various speed prediction models developed in different countries in the past years have been introduced in this study. Regarding a vast range of variation in the models' obtained coefficients and the fact that they are mostly established in the U.S. and Europe, it seemed essential to develop speed prediction models using the speed data of Canadian roads. In this regard, the speed data collection was carried out in two stages for twenty curves on four different rural highways in Ontario. In the first stage, the speed data of sixteen curves were collected and the speed data of the other four curves were gathered through the second stage. The second run of data collection was carried out to check the accuracy of the counters using a radar gun, and enhancing the predictability of the models by adding a number of sites to the database.

Speed data collection was done using five counters (or three counters in some cases) on each horizontal curve and its tangents. Such an arrangement of counters made it possible to trace the vehicles traversing the curves and observe the variation of speed between the specified points. In addition to speed data, the lane width, shoulder width and superelevation rates were measured in the field for the twenty curves. Moreover, a rain gauge was implemented in the field to record the exact time of the day in which it was raining. Having this information in hand, the speed values recorded at the rain time of each day were excluded. Since the scope of the study was limited to the two-lane two-way rural highways in daytime and free-flow condition, the nighttime and the non-free-flow speed data were eliminated as well.

The analysis of variance showed that there was a statistically significant difference between the observed speeds of different vehicle classes. Therefore, the model development was done for two various classes of vehicles; PC/LT and HT. The

distinction between the different vehicle classes were made using the observed number of axles and wheelbase of vehicles and taking advantage of the catalogues of more than 200 car makes in Canada. It is noteworthy that the model development for heavy trucks was aborted due to the lack of enough number of observations. Two different kinds of models were established during this study; operating speed models, and speed differential models.

The model development started using the screened database. First, the operating speed models were developed for the five points (or three points) on each curve. For each point, different data sets were tried to get the model that best fits the speed data. In this connection, some curves with extremely high or low observed speeds were excluded as outliers. Then, the observed speed differential values were calculated using a macro in Microsoft Excel environment. Having the observed speed differential values, the 85th percentile speed differential values were calculated and used for model development. Finally, the models were developed to predict the speed change between different points of a horizontal curve. Due to the fact that one of the independent variables used in the modelling was speed at the approach tangents, average values of the observed 85th percentile speeds for independent tangents, or non-independent tangents employed in this study, were recommended.

The mentioned models can be used for design consistency evaluation based on the operating speed methodology. The development of these models was in fact a contribution to a broader research initiative at Carleton University for design consistency evaluation. Obviously, in order to relate the consistency evaluation to the different measures, the development of reliable models that can predict operating speed and speed differential while accounting to local conditions was inevitable.

7.2 Findings of the Research

The findings of this research can be summarized as below:

- Based on the analysis of variance, it was concluded that there is a statistically significant difference, at the 95% confidence level, between the observed operating speeds of vehicles in the two directions of traffic for the same curve. The same was true for the observed speed differential values belonging to the two directions of traffic in a curve. This happened in the curves with a very mild longitudinal grade (Less than 1 %), therefore the difference between the speeds cannot be ascribed to the effect of grade. This study concluded that such a difference may have originated from the curve direction (e.g., right turn or left turn curve).
- Based on the analysis of variance, it was also concluded that there is a statistically significant difference, at the 95% confidence level, between the operating speeds and also, speed differential values of the vehicles belonging to different classes. In this regard, the passenger cars were found to have similar speeds to the light trucks. As a result, the two mentioned groups can be treated as one classification. However, both classes showed statistically significant differences with the heavy trucks.
- Despite having almost 24 hours of speed data for each curve, the number of observations for heavy trucks was not enough for model development. Thus, the model establishment for heavy trucks was aborted.
- The more general concept of *speed differential*, compared to *speed reduction*, was introduced in this study. This emphasizes the fact that in some curves there was an increase of speed from the approach tangent to the middle of the curve.

- Unlike previous studies, this research showed that there was no relationship between the radius of the curve, or any transformation of that, and the speed at the middle of the curve.
- No relationship was found, at the 95% confidence level, between the speed at the middle of the curve and the geometric features of a horizontal curve.
- No relationship was found, at the 95% confidence level, between the speed at the approach tangent and the geometric and traffic features of the section. Therefore, the values of 103.0 km/h and 95.7 km/h were recommended as the average values of the observed 85th percentile speeds at the independent and non-independent tangents respectively.
- The equipment used in this study for the purpose of speed data collection was electronic counter/classifier. This equipment is much preferable for use to the radar gun used in the majority of previous studies. The reason is that in case of using the radar gun, the possibility of being visible by the drivers is extremely high. Therefore, as observed in part of this study, drivers clearly reduce their speeds when they realize that a traffic control personnel is monitoring the traffic. This imposed speed reduction, make the speed database unreal (e.g. with very few variation in observed speeds).
- Calculating the speed differential by simply subtracting the two pertinent 85th percentile speed values, underestimates the real value of speed changes. This is in agreement with what has been stated by Hirsh (1987), and McFadden and Elefteriadou (2000).
- A more realistic methodology for speed differential calculation is tracking each individual vehicle while traversing a horizontal curve.

- Speed differential from an approach tangent to the middle of the curve, was found as a function of approach tangent speed, deflection angle, shoulder width, curve direction, longitudinal grade, and driveway flag. The independent variables named here in order of their contribution to the model. Therefore, the approach tangent speed made the biggest contribution to the speed differential model.
- The parameters of 3D alignment such as longitudinal grade turned out to be significant in speed differential model. This agrees with the results of the study by Hassan et al. (2000) in which the significance of considering 3D alignments was emphasized.

7.3 Recommendations for Future Research

The following is recommended as future works in this area:

- The potential driver's choice of speed is affected by the forces imposed on his/her vehicle as he/she negotiates the curve. This requires the examination of friction forces and lateral acceleration.
- Models should be developed for predicting the speed at the tangent sections. Such models can then be used to estimate the approach tangent speed to be included in speed differential models developed in this study.
- A data collection scheme for a period of at least 48 hours should be considered in order to have enough number of observations for heavy trucks.
- Video cameras can be mounted in the field to contribute to the process of vehicle classifications and tracking.
- The trend of vehicles' speed at night should be studied in order to find a possible difference with daytime speed database, and consequently develop a set of models capable of predicting operating speed or speed differential at day or night.

- The developed speed differential model should be verified using a larger speed database including other two-lane rural highways in Canada.
- Criteria to evaluate the consistency of design in terms of the difference between the operating speeds of two successive elements or between the operating and design speed of an element should be introduced in North American guidelines.

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