Effect of Horizontal Alignment on Driver Speed Behaviour on Different Road Classifications

Submitted by

Bin Nie, B. Eng.

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

Department of Civil and Environmental Engineering

Carleton University

Ottawa, Ontario

The Master of Applied Science in Civil Engineering Program is a joint program with the University of Ottawa, administrated by the Ottawa-Carleton Institute for Civil Engineering

Your file Votre référence
ISBN: 978-0-494-18328-1
Our file Notre référence
ISBN: 978-0-494-18328-1

This thesis is dedicated to:

My lovely son, *ZhanPeng Nie*

ABSTRACT

The operating speed method has proved highly effective in achieving consistent alignments on the highway facility. Accordingly tremendous research efforts have been drawn to operating speed prediction worldwide over the past decades. However, most of the previous research concentrated on two-lane rural highways only, and there has been little study conducted on other road types. Some prediction models are questionable due to the bias or human errors induced by manual speed measurements. Driver speed behaviour may be misinterpreted because of the limited and discrete observation points on the study site. This research proposed a field experiment to analyze driver speed behaviour on the most common road types in Eastern Ontario, including freeway interchanges. Speed prediction models were firstly developed, using real driving data for two-lane rural highways and urban/suburban roads, respectively. The models consider driver speed behaviour when negotiating horizontal curves. Driver behaviour on freeway interchanges was also investigated.

ACKNOWLEDGEMENTS

The author would like to thank his colleagues Dalia Said and Wael Moustafa. Without their cooperation and diligent efforts, the field study would have not be smoothly conducted and accomplished on time. Special thanks go to volunteer drivers, who participated in experiments, and those standing at the GPS control point, for their selfless sacrifice and supports.

The author would like to thank McMartin, Ken, Conley, Stanley, Whitehome, Jim, and Trudel, Pierre, at Civil and Environmental Laboratory of Carleton University. Without their seamless collaboration in setting up the test vehicle, the fulfillment of this research was certainly not possible.

The author would like to express his thanks to Stephen M, Perkins, at Survey and Mapping of the City of Ottawa, for providing roadway data used in this research. The author would like to thank Dr. Dan E, Patterson, at Carleton University, Ottawa. His contributions to the database construction in this study are highly appreciated.

The author would like to express his sincere gratitude to the thesis supervisor, Professor Yasser Hassan, whom for his timely supervision, patience, encouragement, and his invaluable time, I was able to deliver this work.

Financial support by Natural Sciences and Engineering Research Council of Canada (*NSERC*) is gratefully acknowledged. The experiment equipment used in this research was made available by funds from the Canada Foundation Innovation (*CFI*) and Ontario Innovation Trust (*OIT*).

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CHAPTER 1: INTRODUCTION

1.1 Background

Growing economies, global urbanization, and technology revolutions over the last century have stimulated the development of transportation engineering systems. In today's world, undoubtedly, transportation is one of the signs of the modernization of a nation. Highways supercede other transport modes in ground transportation networks, particularly in the industrialized countries like Canada and the United States, as one might say: they are countries on wheels.

Despite the benefits resulting from the development of highway engineering, a growing concern has brought more attention to road safety due to the fact that traffic collisions have become a major source of social and economic losses in recent years. In the United States, each year nearly 42,000 people are killed on highways and 3 millions are injured. This annual highway death toll is equivalent to a jet airliner crashing and killing everyone on board every single day (Special Report 260 TRB, 2001). According to Ontario Road Safety Annual Report (2003), 831 fatalities were reported on Ontario's roads in the year of 2003, while the number of serious injuries requiring hospitalization was 3,848 and the number of minor injuries seen in an emergency room reached 30,401. Notwithstanding the non-existence of collision-free roads, the highway geometric design deserves more efforts to reduce social pains and sufferings due to injuries, property damage, or fatalities attributed to traffic collisions.

The current design process applied in North American design guides concentrates on the individual elements for which the geometric parameters should be selected or

determined to satisfy the minimum requirement. This approach has shown its weakness due to potential geometric inconsistencies and cannot guarantee that the design product would conform to the expectations of road users. Starting in European countries, such as Switzerland, Germany and France, the concept of design consistency was introduced and emphasized in the last decades. By treating a certain section of the highway as a whole, design consistency requires the design of successive elements to produce a harmonious driving performance without surprising events in that section (Gibreel et al, 1999).

Although these practices are significant and promising, state of the art design consistency still needs to be refined and closely examined for the purpose of its implementation in the geometric design process. Through an overall literature review, which is presented in detail in the next chapter, some common limitations as well as shortcomings remaining in the previous research and the North American design guides are depicted as follows:

- Data collection procedures are mostly questionable and may compromise the reliability and accuracy of the data observations. For instance, using a radar gun to measure the operating speed suffers two major problems. The first is that the reliability of the data collection could be weakened as the exposure of the personnel with the experiment device (i.e. radar gun) could possibly be perceived as speed enforcement and thus forcing drivers to change their speed behaviour. The second problem arises from the reduction in accuracy of the data observations, specifically on the horizontal curve. The deviation between the reading beam of the radar gun and the driving direction of the vehicle on the curved section results in a "cosine error" that reduces accuracy of the data collection.

- Most of the previous studies focused on the highways located in rural areas. Furthermore, the study sites were selected by restricting some physical constraints, such as the presence of intersections or the change in number of lanes, as much as possible. Admittedly, this is advantageous in modelling speed trends. However, simplifying and minimizing the conditions could spontaneously reduce the practical applicability of their studies. There has been little research carried out in a densely populated area where traffic collisions occur more often.

- The current design guides in North America adopt the design speed approach. According to a mail-out survey conducted in 1999, some states in the United States have taken into account the anticipated operating speed as one of the factors used to select the design speed value. However, the question of to what degree the design speed values need to be revised, if necessary, should be explicitly addressed. In Canada, the Transportation Association of Canada design guide (TAC, 1999) explicitly addresses the concept of geometric design consistency in a separate chapter. Nevertheless, how to implement the concept remains relatively vague. The answer should depend on a close investigation of operating speeds on Canadian roads, an issue that has not received enough attention in previous research. In this sense, the importance of comprehensive and continuous studies on operating speeds based on different road classes is evident.

1.2 Objectives

As explained earlier, the challenges that professionals and engineers are now facing would attract continuous studies with an attempt to assist in the improvement of road safety

for our society. The main purpose of this research is to investigate driver speed behaviour in terms of speed selection based on different road classifications through an experimental approach. The findings of this study will hopefully lead to:

- Recognition of the factors influencing the driver choice of speed on the most common road classes in Eastern Ontario.
- Development of prediction models based on the specific road class, using more reliable and accurate speed data.

To fulfill these objectives, thirty volunteer drivers were recruited to drive an instrumented test vehicle along the pre-designed test route. The test vehicle was equipped with a variety of devices capable of recording driver behaviour such as the use of fuel pedal, brake, and steering, operating speeds and so on during the period of experiment. The test route was selected in a manner that four common types of the road in Eastern Ontario were included plus freeway interchanges. Using GPS techniques, driver actual trajectories were tracked based on the instantaneous coordinates of the test vehicle provided by GPS receivers.

1.3 Scope of Research

The recruitment of test drivers was limited to those with G licenses ensuring data samples capable of representing the majority of driver populations in Eastern Ontario. The experiments were conducted during daytime and under normal weather conditions. Nighttime data and speed behaviour on rainy days were not considered in this study. For practical reasons, the test route was selected close to the City of Ottawa. The terrain in the area is relatively level. Therefore, a logical assumption being made in this research is to

ignore the impacts of vertical alignments (i.e. grade and vertical curves) of the road on driver speed behaviour. The investigation of driver speed behaviour was limited to passenger cars only.

1.4 Thesis Organization

This thesis comprises seven chapters. The first chapter presents research background, the study interest, the scope of the study, the objectives of the study, and the thesis structure. The second chapter contains a literature review covering a comprehensive review of previous research relevant to this study. A review of current North American geometric design guides is also included. Chapter 3 consists of two parts. The first part involves the introduction of the test scheme, experiment instruments, and data collection process, and the second part discusses the database construction. Chapter 4 discusses the procedure of the study curve identification and data processing for the preparation of the data analysis in the following chapter. Chapter 5 presents sets of statistical models capable of predicting operating speeds and speed differentials in accordance with three road classes, namely two-lane rural highways, urban/suburban roads, and freeways. Chapter 6 provides the data analysis on the driver speed selection for freeway interchanges. The last chapter summarizes the research. Findings of the study, as well as recommendations for future works are also presented in the last chapter.

CHAPTER 2: LITERATURE REVIEW

As this study aims to investigate driver speed behaviour, the literature review is carried out in accordance with the areas relevant to this research. The design speed selection as well as its application in the current North American design guides, and international practices for alignment designs, are firstly discussed. The deficiencies of the traditional design speed approach are mentioned at the end of the first part of the literature review. Operating speed prediction is another area of interest in this chapter. The concept and the development of design consistency worldwide are briefly introduced. The discussions then concentrate on operating speed prediction. The promising aspects, as well as drawbacks of the previous research are spelled out. Finally, the potential contributions of this research are presented to conclude this chapter.

2.1 Highway Alignment and Speed

The alignment of a roadway facility contains three fundamental elements, in terms of horizontal alignment, vertical alignment, and cross section. The alignment design for each element along with the alignment coordination constitutes the physical features of the highway to accommodate roadway traffic. In the geometric design stage, selection of a speed parameter is the initial step upon which both the horizontal and vertical alignments are to be determined. In a highway system, the vehicular speed depends on a number of factors, such as the vehicle performance, driver capabilities, geometric features, weather conditions, prevailing traffic conditions, speed limit, and traffic control devices. It can also be affected by the urgency of the trip, familiarity of the driver to the road, etc. Designers have to take into consideration the foregoing factors, which can be highly correlated, to

achieve the requirements of mobility, accessibility, and safety for road users. The key point in design policy and practice, according to Krammes (2000), is the speed upon which the alignment of a roadway should be based.

In the United States, the American Association of State Highway and Transportation Officials (AASHTO) policy uses the design speed as the basis for alignment designs, which at one time, had been widely adopted by most national design guides. However, through the practices of highway alignment designs over 50 years, there has been a growing concern related to disparities between the selected design speed and the speeds at which drivers actually operate their vehicles. With the acceptance of the fact that this problem mainly arises from the direct use of the traditional design speed approach, some countries, particularly in Europe and Australia, revised and refined their design procedures by incorporating operating speeds into their design guides and practices.

2.1.1 Design Speed Approach

The history of the design speed approach can be traced back to early 1930s. Both Germany and the United States started to apply the design speed in the highway geometric design procedure by using a particular speed compatible with vehicle behaviours. By 1934, the common design speeds were 80, 100, and 130 km/h depending on the class of the road (Lay, 1992). Until today, the AASHTO design guide adheres to the classical design speed approach for alignment designs.

The design speed was defined in the AASHTO policy (1994) as "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favourable that the design features of the highway govern". More recently, this definition

was replaced in the AASHTO policy (2001) with "a selected speed used to determine the various geometric alignment features on a roadway." This new definition is believed to resolve the liability concerns in case the actual operating speed exceeds the posted speed on a roadway. An overall design speed range is recommended in AASHTO (2004), ranging from 20 to 130 km/h with an incremental value of 10 km/h. The AASHTO policy strongly recommends the use of 10 km/h as the increments in design speed selection, as it notes "smaller increments would result in little distinction in the dimensions of design elements between one design speed and the next higher design speed" (AASHTO, 2004). Table 2-1 duplicates the recommended design speed values in AASHTO (2004).

Table 2-1: Design Speed in AASHTO (2004) (Exhibit 2-29).

Metric	US Customary
Design speed (km/h)	Corresponding design speed (mph)
20	15
30	20
40	25
50	30
60	40
70	45
80	50
90	55
100	60
110	70
120	75
130	80

To select an appropriate value for a design speed, the ASSHTO policy emphasizes two factors, in terms of road classification and terrain type. The AASHTO policy further explains some principles related to the selection of a design speed in correspondence with

the driver's expectancy, road class, land development (urban versus rural), as well as topographic constraints. For example:

- The selected design speed should be consistent with the speed that drivers are likely to expect on a given highway facility.
- The selected design speed should fit the travel desire and habits of nearly all drivers expected to use a particular facility.
- A pertinent consideration in selecting design speeds is the average trip. The longer the trip length, the higher the desire for traffic movements.
- On rural highways and on high-type urban facilities, the selection of an appropriate design speed is particularly important. In many arterial streets, vehicle speeds are limited or regulated more by the presence of large volumes of vehicles and by traffic control devices, rather than by the physical characteristics of the street. In such cases, the selection of a design speed is less critical to safety and efficient operation.

To apply the design speed in alignment designs, the AASHTO policy provides some descriptive guidance, such as: all of the pertinent features of the highway should be related to the selected design speed to achieve a well-balanced design; although the selected design speed establishes the limiting values of curve radius and minimum sight distance that should be used, flatter horizontal curves or greater sight distances are preferable wherever the topographic constrains and economic conditions permit; and introduction of a next lower design speed in response to changes in terrain and other physical controls

should be made gradually, allowing drivers to change their speed without abrupt manipulations.

Clearly, the design speed selection as well as its application in the AASHTO policy is such a subjective and qualitative process that designers have to determine the design parameters based on their own judgement and experiences. Fitzpatrick and Carlson (2002) noted that "AASHTO guidelines also discuss other factors such as operating speed, adjacent land use, and safety, but provide little, if any, quantitative guidance related to how such factors should be considered or how they affect the selection process of a design speed value." As a result, one can state that the AASHTO policy fails to provide an explicit and quantifiable procedure that a designer should follow.

Although the latest AASHTO policy (2004) makes few changes in design speed selection and its application for alignment designs, professionals and researchers in the United States have made tremendous efforts to remedy the deficiencies arising from the adoption of the conventional design speed approach in the past decades. These research efforts are mainly reflected in the works by Leisch and Leisch (1977), Lamm and Smith (1994), Krammes (2000), and Fitzpatrick and Carlson (2002).

Leisch and Leisch (1977) claimed that the direct use of the current design philosophy could lower the drivers' expectations, resulting in considerable operating speed variations, particularly for a low speed alignment. In addition, the design speed should be selected to meet the driver's natural tendency and appear to be reasonable to the driver. Consequently, the authors suggested the use of a 15 km/h rule to examine speed differentials along the alignment, taking into considerations the potential average operating speed variations between automobiles, the change in design speeds, and the variance of the dynamic

capabilities between passenger cars and trucks. The application of Leisch and Leisch's methodology requires plotting a speed profile, which involves determining the average running speed on tangents, the average running speed on horizontal curves, and selecting the rates of acceleration and deceleration for vehicles. The speed data and the rates of acceleration and deceleration can be determined using a number of tables and nomographs adapted and extrapolated from the AASHTO (1965) guide. The speed profiles accounted for both horizontal and vertical alignments, and were established for different vehicle classes. By checking the speed profile against the 15 km/h rule, the road section with excessive running speed differentials could be identified and the corresponding alignment should be adjusted to eliminate them.

Lamm et al (1988) recognized the effects of the interaction of adjacent elements on speed reductions. The authors noted that the change in the 85th percentile speed from a tangent into a horizontal curve has a strong relationship with the length of the approach tangent. Based on research conducted both in the United States and Germany, Lamm and Smith (1994) presented a curvilinear alignment design process to promote the vehicle's operational consistency on the roadway. According to the authors, as opposed to long tangent-short curve type of alignment, short tangent-long curve type of alignment (curvilinear alignments) should be encouraged to overcome the problem of inconsistent alignments induced by the adoption of the design speed approach.

The speed data collected from the United States highways in 1978, 1991 and 1992, proved the considerable disparity between the inferred design speed and the anticipated operating speed. After comparing the AASHTO (1994) policy with international practices of selecting a design speed, Krammes (2000) proposed feasible countermeasures to reduce

the disparity between the design and the actual speed in four areas: (1) selection of an appropriate design speed, (2) revisions to the design-speed concept to improve operating speed consistency, (3) uniformity of horizontal curve design, and (4) curve information systems for drivers.

According to Fitzpatrick and Carlson (2002), a mail-out survey carried out in 1999 across states, reported that five major factors, such as functional classifications, legal speed limit, legal speed limit plus 5 or 10 mph (8.1 to 16.2 km/h), traffic volume, and anticipated operating speed, are usually considered when selecting a design speed by the respondents. The factor recommended in the AASHTO (1994) policy, terrain type, was only occasionally considered as one important factor. The study revealed that over one-third of states in the United States had incorporated the anticipated operating speed into their design practices. In addition, Fitzpatrick and Carlson (2002) studied the factors suggested by the AASHTO policy for design speed selection, and proposed four potential solutions to improve the design speed selection. These solutions were limiting the range of design speed values within each functional class, functional area, or terrain type; using anticipated posted or operating speed (or using anticipated posted or operating speed plus a preset incremental increase); incorporating a feedback loop that would check the predicted speed along an alignment; and managing speeds on the tangent section by controlling the tangent length.

In Canada, TAC (1999) defines the design speed as "a speed selected as a basis to establish appropriate geometric design elements for a particular section of road", which agrees with that in the AASHTO (2004) guide. In selecting a design speed, the TAC (1999) guide emphasizes the functional classification of the road, adjacent land use (urban versus

rural), and the consideration of interruptions from opposite traffic (divided or undivided). Design speeds recommended in Table 1.3.2.1 in the TAC guides (1999) are duplicated in Table 2-2 in this chapter. Differing from the AASHTO design guides, the TAC policy explicitly addresses the limitations of the use of design speed approach. To overcome the deficiencies, a supplementary solution is presented in Chapter 1.4 of the policy, which will be further discussed in this chapter.

In addition, some provinces in Canada have started to consider the effect of speed limits on selection of a design speed. For example, according to the field survey conducted on Alberta rural highways, 85 percent of the drivers observed exceeded the speed limits on highway facilities by 6 to 10 km/h. It was stated in the Alberta guidelines (Alberta Infrastructure and Transportation, 1999) that the normal speed limit on the finished roadway is an important consideration in selecting design speed. It is desirable that the design speed exceeds the normal speed limit by a margin of at least 10 km/h. The Ontario Design Guide (1993) specified that the design speed should be higher than the proposed legal speed by 20 km/h, unless circumstances warrant a reduction.

Table 2-2: Design Speed in TAC (1999) (Table 1.3.2.1).

	Design Speed (km/h)	Local	Collector	Classification Arterial	Expressway	Freeway
rural	50	RLU50				
	60	RLU60	RCU60			
	70	RLU70	RCU70			
			RCD70			
	80	RLU80	RCU80	RAU80		
			RCD80	RAD80		
	90	RLU90	RCU90	RAU90		
			RCD90	RAD90		
	100	RLU100	RCU100	RAU100		
			RCD100	RAD100		RFD100
	110	RLU110	RCU110	RAU110		
			RCD110	RAD110		RFD110
	120			RAU120		
				RAD120		RFD120
	130			RAU130		
				RAD130		RFD130
urban	30	ULU30				
	40	ULU40				
	50	ULU50	UCU50	UAU50		
			UCD50			
	60		UCU60	UAU60		
			UCD60	UAD60		
	70		UCU70	UAU70		
			UCD70	UAD70		
	80		UCU80	UAU80		
			UCD80	UAD80	UED80	UFD80
	90			UAD90	UED90	UFD90
	100			UAD100	UED100	UFD100
	110				UED110	UFD110
	120					UFD120

RLU: Rural Local Undivided.

RCU: Rural Collector Undivided; RCD: Rural Collector Divided.

RAU: Rural Arterial Undivided; RAD: Rural Arterial Divided.

RFD: Rural Freeway Divided.

ULU: Urban Local Undivided.

UCU: Urban Collector Undivided; UCD: Urban Collector Divided.

UAU: Urban Arterial Undivided; UAD: Urban Arterial Divided.

UED: Urban Expressway Divided.

UFD: Urban Freeway Divided.

2.1.2 Operating Speed Approach

As explained previously, the traditional design speed approach is based on the design speed concept, with an assumption that the road users will travel at a speed compatible with the design speed. Therefore, no formal check for the actual speed characteristics in terms of the operating speed is needed. The operating speed design approach, on the other hand, bases the selection of the design parameters on actual operating speeds on the roadway facility. The operating speed is defined in the AASHTO guide (2004), as "the speed at which drivers are observed operating their vehicles during free-flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or geometric features". With the recognition of discrepancies between the operating and design speed, some countries have incorporated the actual driver speed behaviour in terms of anticipated operating speed into alignment design practices. The following summarizes international design practices based on the operating speed approach among Australia and some European countries.

Australian guidelines depart from the traditional design speed approach for a low-speed alignment design (i.e. <100 km/h). The 85th percentile speed replaces the design speed as the basis of alignment design. For high speed alignment (i.e.≥100 km/h), the traditional design speed concept is retained because field studies indicated that operating speeds were lower than the design speed in a high speed environment. According to Krammes and Garnham (1995), an iterative procedure with seven major steps is applied to check for and resolve the design inconsistency on successive elements. The difference of design speeds greater than 10 km/h between successive elements will lead to alignment modifications. Sight distance should be adequate for the design speed (estimated 85th

percentile speed). In selecting superelevation rates for curves with radii larger than the minimum, the frictional demand with the proposed rate at the design speed (85th percentile speed) should not exceed specified maximum values.

The German guidelines also use an iterative method to balance the design and operating speeds for rural highway alignment design. Design speed is firstly used, as in the United States and Canada, to determine the minimum curve radius of horizontal curves, the maximum grades, as well as the minimum *K*-values (length corresponding to 1 percent change in the grade) for crest vertical curves. The 85th percentile operating speed is then estimated, and should be balanced with the design speed. In case the difference exceeds 20 km/h, the design speed needs to be increased, or the initial alignment must be revised to reduce the expected operating speed (Brenac, 1996). The operating speed on rural two-lane highways is estimated based on the curvature change rate (*CCR*: the absolute sum of the angular change in direction divided by the total length of the segment) and pavement widths. Furthermore, the German guidelines specify a limit of 10 km/h, above which the difference of the estimated operating speeds on successive elements is unacceptable. If the speed differential between successive road sections exceeds 10 km/h, a transitional section should be introduced to minimize the operating speed variations.

In Switzerland, the design policy incorporates design speed and the project speed (a surrogate of the expected operating speed) in the design process. The design speed is selected based on the road type and land use density (urban versus rural), and used to determine the minimum radii for horizontal curves and maximum grades. Project speed is used to examine the speed difference on successive elements, and to choose superelevation rates and sight distance parameters. An excessive speed difference should be reduced, to

ensure the speed consistency on a certain section of the roadway. The project speed profile is developed based upon three independent variables, which are the maximum speed on tangents, the speed on curves (assumed to be constant on the curve), and the deceleration and acceleration rates when entering and departing horizontal curves (Krammes and Garnham, 1995).

The British practices emphasize the effects of the alignment and the layout constraints on operating speed in selecting a design speed. For example, the alignment constraint is a function of "bendiness" defined as the total degree of curvature per km and the harmonic mean of available sight distance, whereas layout constraint is a function of the access density and the road class (Polus et al, 1995). For rural highways, the design speed is based on the 85th percentile speed for passenger cars on the wet pavement. For urban/suburban roads, the design speed is established based upon the appropriate speed limit. The United Kingdom develops a structured system, in which design speeds are coordinated in such a quantitative manner that a certain design speed value that accommodates 85th percentile drivers should be appropriate for the 50th percentile traffic of the next higher design speed and for the 99th percentile traffic of the next lower design speed.

In Italy, the design speed is a function of the type of road and its cross section features. The design guide allows the design speed to vary between different elements. However, such variances must be controlled to avoid a potential risk. For instance, by giving a design range rather than designating a certain speed value, designers are able to take advantage of the given freedom fitting a suitable alignment into the surroundings. A maximum length of

straights based on design speed is suggested in the Italian design guides (Krammes and Garnham, 1995).

The previous French standards in terms of "Instruction" had five road categories, with the design speed ranging from 40 to 120 km/h for each road category. Having realized that the "Instruction" may ignore the effect of alignment on actual driver speed behaviour, and permit higher speeds, the French design guide was modified in 1994. The new French standards specify three new road categories and limit the range of design speed to 20 km/h for each of them. Further, the guidelines specify the minimum radius following long tangents so as to avoid an extremely high operating speed ahead of a sharp curve (Krammes and Garnham, 1995).

Based on these summaries, the procedures for alignment designs may be somewhat different among the aforementioned countries. Nevertheless, they share at least two common characteristics, which are: (1) operating speed is viewed as an important factor which should be taken into account for selecting an appropriate design speed; (2) disregarding the details, a feedback loop is included in the design guides to achieve design consistency. The design feedback loop generally consists of the following steps:

- A preliminary alignment design is conducted based on the traditional design speed concept.
- Expected operating speeds are estimated for the initial alignment.
- An Iterative procedure is carried out to achieve operating speed consistency.
- The alignment design is finalized when design speed consistency is achieved.

From the above discussions, it is clear that modern highway geometric design has noticeably deviated from the traditional design guides. When selecting a design speed, the existing North American design guides concentrate on the single element without considering the alignment coordination between adjacent elements. Fitzpatrick and Carlson (2002) stated that the use of the classical design speed approach could create several problems, which are: inconsistent design speed along alignment; operating speeds higher than the design speed; operating speed variations on successive elements; and posted speed limit greater than the design speed. With the recognition of these problems, a systematic theory, in terms of geometric design consistency, has been developed in recent years. The design consistency is believed to be an effective tool to improve the quality of the highway geometric design. According to Gibreel et al (1999), achieving highway geometric design consistency is an important issue in the design and evaluation of rural highways to attain smooth and safe traffic operation. Therefore, the literature review in the next section moves to the discussion of knowledge and practice of design consistency, with an emphasis on operating speed prediction.

2.2 Highway Alignment and Geometric Design Consistency

2.2.1 Definition of Design Consistency

Design consistency has been defined by many researchers, linking the highway geometric features to safety performances on highways. For example, Nicholson (1998) considered design consistency as the ability of the highway geometry to conform to driver expectations. Gibreel et al (1999) defined the design consistency as one that ensures successive geometric elements to be coordinated in a manner to produce

harmonious driver performance without surprising events. In addition, Gibreel et al (1999) proposed a framework for highway design consistency, in which both design and traffic elements are considered and related to design consistency bases. The authors categorized the previous research work into three main areas, namely speed considerations, safety considerations, and performance considerations. These areas involve a diversity of aspects, ranging from operating speed prediction, vehicle stability, driver workload and anticipation, and highway aesthetics. As mentioned earlier, many countries have updated their design guides based on their domestic research on design consistency. In North America, the role of design consistency has been reinforced in the Canadian design guide (TAC, 1999). A concise comparison with regard to design consistency between two North American design guides - AASHTO (2004) and TAC (1999) - is provided hereinafter.

2.2.2 Design Consistency in North American Design Guides

In spite of the abundant research concerning geometric design consistency in the United Sates, the latest version of AASHTO (2004) still has not adopted virtually any of this work. In view of promoting a consistent alignment design, only little qualitative description can be found, such as "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", and "where sharp curvature is introduced, it should be approached, where practical, by a series of successively sharper curves". With these unclear regulations, designers may still have difficulties to determine the appropriate design parameters that could lead to consistent geometric features.

On the contrary, the Canadian design policy (TAC, 1999) provides explicit considerations on alignment design consistency in Chapter 1.4. An update of this chapter has been suggested by Easa (2002). This update suggests two categories of consistency measures, in terms of system and local measures. System consistency measures contain four components: cross section consistency, operating speed consistency, alignment geometry consistency, and driver workload consistency, while local measures have two: design-speed margin consistency and vehicle stability consistency.

The cross section consistency requires the designer to apply consistent cross section dimensions such as lane and shoulder width along a certain section of the roadway. In case changes are unavoidable, the change should be introduced gradually. Alignment geometry consistency can be evaluated using such alignment indices as: average radius (*AR*), ratio of an individual curve radius to the average radius (*CRR*), and average rate of vertical curvature (*AVC*). For driver workload consistency, a quantitative methodology in terms of visual demand is included. Among the system consistency measures, TAC (1999) addresses operating speed consistency in a quantitative manner. In the suggested revision of the guide, operating speed prediction models developed by Fitzpatrick and Collins (2000) in the United States are adopted. These models account for the effect of grade, vertical curves and sight distance on the estimated speeds. The details of speed profile construction are depicted and illustrated.

At the local consistency level, the revised TAC policy introduced two measures: design speed margin consistency and vehicle stability consistency. Harmonizing the operating speed and the selected design speed is an important issue covered in the suggested revision. A regression equation is recommended to estimate the design speed for redesigning an

existing alignment based on curvature change rate (*CCR*, gon/km). This model originates from the work by Morrall and Talarico (1994). The vehicle stability is examined by the difference between the side friction assumed and demanded. The concern of vehicle stability will arise if the assumed side friction is less than the value demanded. The evaluation criteria are suggested based on the work by Lamm et al (1999).

2.2.3 Design Consistency Evaluation Approaches

In view of design consistency evaluation, there are four commonly used methodologies among most of the previous research, namely operating speed consistency, alignment indices, vehicle's dynamic stability, and driver workload (Gibreel et al, 1999, Fitzpatrick et al 2000b). According to Fitzpatrick et al (2000b), these four methods can further be divided into two simple classes: speed-based and non-speed-based approaches. The speed-based class corresponds to the operating speed and design speed consistency, whereas the non-speed-based class contains the other three components. Because of the limits of this research, discussions are dedicated to speed-based approaches, with a focus on the operating speed prediction.

The speed-based evaluation approaches basically consist of two groups: speed distribution measures and operating speed approaches. The speed distribution measures base the design consistency evaluation on descriptive statistics of the speed distribution, such as: the mean speed, variance, standard deviation, coefficient of variation, skewness, and kurtosis. The operating speed methods, on the other hand, deal with design speed margin consistency and operating speed consistency for the individual and multiple elements of the roadway, respectively.

2.2.3.a Speed Distribution Methods

The free flow speed, representing the desired speed at which a driver wishes to drive, is hypothesized to be normally distributed. Six descriptive statistics are used to describe the speed sample. They are the mean speed (km/h), variance (km/h), standard deviation (km/h), coefficient of variation, skewness, and kurtosis. Among the six statistics, variance, skewness, and kurtosis are three important measures to study the sample of speed observations. Any deviation from perfect normal distribution (Kurtosis = 3, skewness = 0) or significant change in speed distribution may be the result of alignment inconsistency. Fitzpatrick et al (2000b) conducted a field study regarding speed distribution with four proposed assumptions:

- Roadway geometry can be used to predict speed distribution measures.
- Design and/or posted speed can be used to predict speed distribution measures.
- The relationship between successive features can be used to predict speed variance.
- Increased speed variance can be used as an indicator of design inconsistency.

Speed data were collected on horizontal curves and the preceding tangents distributed in four regions of the United Sates. However, the results of the preliminary analysis did not support the above mentioned hypothesis. For example, no strong relationships were found between highway geometry (e.g., horizontal curves or preceding tangents) and the speed distribution measures, such as the mean speed, variance, skewness and kurtosis. The standard deviation of the speed samples decreased as the curve radius decreased. However, this finding was only applicable to the conditions where the posted speed was in excess of

the design speed. No strong relationship was recognized with speed variance, as well. The authors finally concluded that speed distribution was not a desirable alternative for design consistency evaluation. Although there might be some relationships between speed measures and alignment features, the data variations indicated that such relationships were not statistically significant.

2.2.3.b Operating Speed Methods

Numerous research works related to speed considerations have adopted the operating speed as the basis to evaluate geometric design consistency. The reason of using the operating speed mainly arises from its simplicity, quantifiability, and applicability. Researches agree that highway geometric features have significant impacts on driver speed behaviour, which are reflected in the operating speed variations. Road safety studies revealed that there has been a close correlation between highway alignment and accident rates. Wilson (1968) and Babkov (1975) reported that the accident rate on sharp curves (curve radius <200 m) are about 4 to 5 times that on flatter curves (curve radius >900 m).

Lamm and Choueiri (1987) found that accident rate variations were most sensitive to the radius of curve (degree of curve). Accordingly, Lamm et al (1988) suggested rating criteria to evaluate design consistency, linking the degree of curve (*DC*) to accident rates. Later, Lamm et al (1991) recommended Safety Criteria I and II, relating the difference between the operating speed and the design speed and operating speed variations to accident records. The authors designated the thresholds of the difference between the operating speed and the design speed corresponding to three levels of the design (Good, Fair, and Poor) as Safety Criterion I. This measure is used to assess the design consistency for the single element. Similarly, Safety Criterion II was established by assigning the

thresholds of operating speed variations on successive elements corresponding to three levels of the alignment design. Hence, road designers could recognize roadway elements susceptible to design inconsistency, with Safety Criteria I and II.

Hassan et al (2005) conducted a study in order to establish safety criteria applicable for Canadian roads. Based on the data collected from seven two-lane rural highways in Eastern Ontario, negative binomial regression analysis was performed, relating the design speed margin consistency and operating speed consistency to the collision frequency. The authors recommended operating speed consistency as a consistency evaluation criterion, according to the result of the regression analysis. More details on this work were also provided in Awatta et al (2006).

For the application of operating speed methods in design consistency evaluation, the role of operating speed prediction has been evident. Subsequently, tremendous research efforts have been brought to operating speed prediction worldwide over the past decades. The following section presents the existing research works with respect to operating speed modelling, including the study procedures and their main findings.

2.2.4 Operating Speed Modelling

As discussed earlier, Leisch and Liesch (1977) proposed a new design process in the United States to assist in the improvement of alignment design, based on operating speed consideration. For establishing the speed profile, vehicle acceleration or deceleration was assumed to occur on the upstream or downstream tangents; a constant speed was assumed to be maintained on the curved section; and acceleration and deceleration rates were extracted or extrapolated from AASHTO guide of 1965.

Mclean (1978) conducted a speed study in Australia. The study was limited to two-lane rural highways and only passenger car speeds were considered. The database contained 120 horizontal curves. However, the sample size as well as the equipment used for speed measurement was not provided. A speed prediction model was then established, using regression analysis. The operating speed was found to be correlated to curve radius (*R*) or curvature change rate (*CCR*). The author also noticed that the operating speed tended to be lower than the design speed in the high speed environment (>90 km/h). On the other hand, the operating speed exceeded the design speed in the low speed environment (<90 km/h). Using regression analysis, Mclean (1979) developed another speed prediction model based on the curve radius and the desired speed on the approach tangent. These findings resulted in the revision of the Australian design guide by replacing the traditional design speed with 85th percentile speed for alignment designs in low speed conditions (<100 km/h).

Lamm and Choueiri (1987) presented a set of regression models, corresponding to different lane widths, to estimate operating speed on horizontal curves. The study utilized speed data of passenger cars on 261 sites on two-lane rural highways in the state of New York. The conditions applied in the site selection included no influence of intersections and hazardous physical features (e.g. narrow bridges), consistent lane and shoulder width, marked pavement with paved shoulder, grades limited to less than 5 percent, and relatively low *AADT* value (400 – 5000 vehicles/day). The speed measurements were made with stop watches. The 85th percentile operating speed was found to be sensitive to curvature change rate or the curve radius for all curves with different lane widths. The lane width, shoulder width, and *AADT* value were included as secondary variables in the speed estimation

models. The authors also recommended 94.4 km/h as a suitable speed value for tangent sections.

Krammes et al (1995b) established speed prediction models for horizontal curves based on the observed speed data for passenger cars. The database contained 284 study sites across five states in the United States. Site selection criteria included low to moderate volume rural collectors or minor arterials, grades less than 5 percent, design speed lower than 100 km/h, lane width between 3.05 to 3.66 meters, and shoulder width between 0 to 2.44 meters. Study sites consisted of 138 horizontal curves, 78 paired tangent-curves, and 68 tangents. The sample size fluctuated between 50-100 observations, with speed data collected using radar guns. The models were improved later by Krammes (1997), and the models have been included in the Interactive Highway Safety Design Model program (*IHSDM*) in the United States. The authors reported that the estimated 85th percentile speed is dependent upon the degree of curvature, length of curve, deflection angle, and approach tangent speed. However, the attempts to predict operating speeds on tangent sections were unsuccessful. Therefore, a speed value of 97.9 km/h was recommended as a desired speed on long tangents (Krammes et al, 1995b).

Al-Masaeid et al (1995) presented a set of regression models capable of predicting operating speed differences on two adjacent geometric features, corresponding to different vehicles (passenger cars, light trucks, and heavy trucks). The study was limited to two-lane rural highways. The presence of intersections and physical features, which could result in operating speed reductions, was precluded. The pavement condition, in terms of Present Serviceability Rating (*PSR*), and shoulder width were identical on the successive elements under investigations. The study sites consisted of 57 simple curves (i.e. an individual

horizontal curve preceded by an independent tangent) and 36 continuous curves (i.e. a pair of curves connected with a non-independent tangent). Speed data were collected using a 40-m trap in free-flow conditions, with a minimum of 93 observations per site. The operating speed differences were calculated by subtracting the 85th percentile values of the speed on the adjacent elements. Separate models were presented corresponding to single curves and continuous curves, respectively. The study led to four major findings: (1) the speed reduction on simple curves was significantly affected by the degree of curve, gradient, length of vertical curve within the horizontal curve, and pavement condition, (2) the ratio of curve radii had the greatest contribution to speed reductions on continuous curves, (3) the operating speed on the common tangent was highly correlated to the length of the common tangent, and (4) the speed reduction influenced by degree of curve on flatter curves for all vehicles and for each type of vehicles (i.e. truck and passenger car) was not significant.

Abdelwahab et al (1998) presented another set of models in Northern Jordan, relating the passenger car speed reduction to horizontal curve parameters. Study sites were selected to satisfy several conditions: free from the influence of intersections; without the presence of lateral hazards (e.g. narrow bridge); consistent cross section dimensions on successive elements, exclusion of the continuous curves; and free-flow traffic. The database contained 46 curves from five two-lane rural highways with a minimum of 35 speed measurements for each site. The speed data were collected using a stop watch. The speed reduction took the difference of the 85th percentile speed between the approach tangent and the middle of the curve.

Passetti and Fambro (1999) attempted to determine the effects of spirals on operating speeds by comparing the difference between the observed speed on spiraled curves and the speed on simple circular curves. The database contained 12 spiraled curves and 39 simple circular curves on two-lane rural highways from six states in the United States. The speed data were collected using piezoelectric sensors connected to traffic counters/classifiers or using radar guns. The minimum sample size was 100 measurements per site and the speed observations were taken during daytime on dry pavements. Only passenger car speeds under free-flow conditions were included in the data analysis. The authors learned that spiral transitions did not significantly affect the operating speed at which drivers traversed horizontal curves on two-lane rural highways.

Fitzpatrick et al (2000a) presented speed prediction models for passenger cars based on the speed data from six states in the United States. The study was conducted at 176 sites, among which 103 sites were used for model development whereas 41 sites were used for model validation. Speed prediction models developed by Krammes et al (1995b) were also examined in this study, using the speed observations at 32 sites from the database. The minimum sample size for each site was 100 speed measurements. Radar gun, Lidar gun, and pavement piezoelectric sensors connected to counters/classifiers were used to collect speed data. Conditions applied for the site selection were: outside developed areas, lane width between 2.74-3.66 meters, radius range between 110-3500 meters, *AADT* volume between 500-4000 vehicles/day, grades between -10 percent and +10 percent, design speed lower than 120 km/h, and posted speed between 75-115 km/h. The study indicated that the 85th percentile speed was correlated to the radius of the horizontal curve, or to the rate of vertical curvature, and could be affected by longitudinal grades, combinations of

horizontal alignment with different types of vertical alignments, as well as the limited sight distance. Furthermore, the influence of the horizontal curve with and without spiral sections on speed estimations was not statistically significant. This supported the conclusion of the study by Passetti and Fambro (1999).

Mathematical models for passenger cars in Venezuela were introduced by Andueza (2000). The study was carried out for a two-lane rural highway. The road was two 3.65 meter lanes with paved shoulders of 1.20 meters or greater. The grades of the sections under investigation were less than 3 percent. The study sites involved 21 curves and 18 tangents. The speeds were measured at the middle of the curve and on the tangent in two directions with radar gun. Sample size fluctuated between 30 and 64 speed observations per site. Only passenger car speeds under free-flow condition were included. The author reported that the 85th percentile speed on the curve was a function of the degree of curvature, sight distance, the length of the preceding tangent, and the radius of the preceding curve. The 85th percentile tangent speed was dependent on the radius of the previous curve and the length of the approach tangent. On the other hand, the relationship between superelevation and observed speed data was not statistically significant.

McFadden and Elefteriadou (2000) studied speed reductions from the approach tangent into horizontal curves. The speed data were collected at 21 sites on two-lane rural highways in the United States. At each site, at least 75 vehicles were measured. Only isolated horizontal curves with a leading tangent having a minimum length of 200 meters were included in the study, and the vertical alignment was limited to ±5%. Data collections were restricted to passenger cars under free-flow condition. Vehicle speeds were measured using Lidar guns (Light Detecting and Ranging Device) and cosine errors were corrected.

The authors proposed a new methodology that was able to trace the individual driver speed behaviour from the approach tangent to middle of the horizontal curve. A new parameter, called *85MSR* (85th percentile maximum speed reduction) was introduced based on the accumulative frequency distribution of the maximum speed reduction. The authors also compared *85MSR* with *85S2*, which is the difference between operating speeds on tangent and curve as currently used in design consistency evaluation. The results showed that *85MSR* was two times greater than *85S2*. This finding supported Hirsh's hypothesis (1987): "the use of 85th percentile speed for evaluating design consistency tended to underestimate speed reduction experienced by individual drivers".

Hassan et al (2000) proposed a practical procedure to evaluate the alignment consistency by comparing the difference between the estimated operating speed and the maximum allowable speed. The maximum allowable speed was defined as the maximum speed at which the driver can operate safely, and was determined based on *3D* available sight distance (Hassan et al, 1997), vehicle dynamic requirements, and driver comfort. The operating speed was estimated using previous speed models developed by Lamm and Choueiri (1987) and Krammes et al (1995b). A field study was carried out at six horizontal curves on Highway 61 in Canada to validate the previous speed prediction models. The study showed that Krammes et al models were preferable to another candidate as Krammes et al models yielded estimated speeds closer to observed speed values. Hassan et al (2000) also suggested 102 km/h as a desired tangent speed, based on the observed speeds in the field.

Following these findings, Gibreel et al (2001) presented a set of models accounting for the effects of three-dimensional alignments. Two types of *3D* alignment combinations

were selected to develop the models. The combinations were a sag vertical curve combined with a horizontal curve and a crest vertical curve combined with a horizontal curve. The study sites consisted of 10 crest vertical curve combinations and 9 sag vertical curve combinations, located on Highways 61 and 102 (two-lane rural highways) in Ontario, Canada. Vehicle speeds were observed in two driving directions, thus resulting in a total of 18 sites for sag combinations and 20 sites for crest combinations. Speed data were collected using radar guns for one hour at five locations along each travel direction per site. Only free-flow passenger car speeds were considered. Five regression models were established, corresponding to five locations for each alignment combination through linear regression analysis. It was learned that the 85th percentile speed was significantly affected by horizontal curve radius, deflection angle of the horizontal curve, horizontal distance between the point of horizontal intersection and the point of vertical intersection, length of vertical curve, gradients, algebraic difference in grades, and superelevation rates. The predicted values showed a good agreement with the observed speeds. In addition, the *2D* models by Lamm et al (1990), Islam and Seneviratne (1994), and McFadden and Elefteriadou (1997) were subjected to verifications using the field speed data in this study. The results indicated noticeable differences between estimated data and actual speeds. The *2D* models were found to either underestimate or overestimate the vehicle's actual speed.

Jessen et al (2001) investigated the operating speeds in the vicinity of crest vertical curves. Field studies were conducted on horizontal tangent sections of two-lane rural highways in Nebraska. Seventy sites at crest vertical curve were selected, and 8 of the 70 sites were used for model verifications. The sample size at each site had 275 vehicle observations. The speed data were collected using traffic counters and classifiers, and were

screened to meet certain conditions. The final data were passenger car speeds under free flow conditions during daytime hours on dry pavements. Two regression models were developed, corresponding to two conditions: at a limited sight distance and at a desired sight distance, respectively. The results showed that, at the limited sight distance locations, approach tangent speed affected the operating speed, and that posted speed limits had the most impacts on drivers' speed choice no matter if the desired sight distance was available or not. *ADT* (average daily traffic) also showed some influence on operating speeds when approaching the crest vertical curve, where the operating speed decreased with the increase of the *ADT*.

Misaghi and Hassan (2005) investigated the operating speed at 20 horizontal curves located on Highways 31, 43, 12, and 41 in Eastern Ontario. Electronic counters/classifiers were used to measure vehicle speeds, and to distinguish the vehicle classes. The speed data on rainy days, during nighttime, and with traffic disturbance (non-free-flow condition), were removed from the database. The authors introduced a new variable called $\Delta_{85}V$, which was defined as the speed differential that the 85 percent of drivers do not exceed, based on the individual driver's speed profile. The authors concluded that ΔV_{85} seriously underestimated the actual speed change as hypothesized by other researchers. A linear relationship between $\Delta_{85}V$ and ΔV_{85} was established in this study, showing a difference between these two parameters as high as 7.55 km/h. The approach tangent speed was reported as the most important factor contributing to the speed reduction from the approach tangent into the curve. The speed change could also be influenced by other factors, such as deflection angle, shoulder width, curve direction, longitudinal grade and the presence of a driveway.

Crisman et al (2005) presented a series of operating speed prediction models for two-lane rural roads in Italy. The models were able to estimate the operating speed for tangent sections and horizontal curves, respectively. Operating speeds were collected on 30 circular curves and at the midpoint of 27 tangents, using laser equipment. The database contained extensive vehicle speed data for a period of 5 years along six two-lane rural roads in the north-east of Italy. For developing speed prediction models, only passenger car speed data under free-flow conditions were considered, and they were free from the influence of grades (< 4%) and the presence of intersections or private accesses. According to the authors, the actual driver speed behaviour was controlled not only by highway geometric features alone, but also by the overall road environment, that could be identified by the curvature change rate (*CCR*, gon/km) over a certain section of the roadway. Based on this assumption, numerical models were firstly established to predict the environmental speed (V_{env}), defined as the maximum speed of the 85th percentile speeds surveyed on long tangents or generous curves belonging to a homogenous road section. V_{env} was then used as an independent variable to develop operating speed models. The developed models suggested that the operating speed on the curve was a function of curve radius and V_{env}, whereas the maximum operating speed reached on dependent tangents was dependent upon the operating speed on the previous curve and the length of the tangent.

Figueroa and Tarko (2005) studied driver behaviour before and after horizontal curves in the state of Indiana. The study resulted in the development of four prediction models, estimating operating speeds on tangents, on curves and on acceleration and deceleration tangent- curve transition sections of two-lane rural highways. The research methodology was proposed based on an assumption that drivers would start to reduce speed from a

desired speed (V_T) with a constant deceleration rate (d) on the approach tangent, and maintain a constant speed of (V_C) on the curve. Drivers would then start to accelerate steadily with a rate of (a) from the curved section through the departure tangent. The operating speed models for transition sections were formed, depending on operating speeds on the tangent and the curve, the mean acceleration and deceleration rates, the length of the transition sections, and the portion of the transition sections on the tangent. As the mean acceleration and deceleration rates and the portion of the transition sections on the tangent were uncertain variables, free-flow speed data were observed and used to calibrate them, following an iterative process.

Speed observations were made with either laser guns or traffic classifiers on a total of 158 sites, located on horizontal curves and on tangents at different distances from horizontal curves. On average, 360 speed observations were made per site. During the calibration process, the sites were classified into four groups that corresponded to model development on tangents, curves, and acceleration and deceleration transition sections. The site classification criteria were based on their locations with respect to tangents, horizontal curves, and deceleration or acceleration transition sections.

The regression analysis yielded that the speed model on curves was sensitive to curve sight distance, residential driveway density, degree of curvature, and superelevation rate. The speed model on tangents was found to be affected by a variety of factors, including the posted speed, percentage of trucks, sight distance, highway grade, adjacent land use, presence of intersections, and roadway cross sectional features. The operating speed on transition sections was correlated to the estimated or observed speed on tangents and curves and the distance of the site to the beginning of the curve or to the end of the curve (l_d

and l_a). The models also explained that 65.5 percent of the deceleration transition and 71.6 percent of the acceleration transition occurred on the tangent before and after the curve, respectively. The mean deceleration rate of 0.7 m/s^2 (2.4 ft/s^2) and acceleration rate of 0.5 m/s^2 (1.6 ft/s^2) were estimated for a 16 km (10 mph) speed reduction.

Canadian researchers Park and Saccomanno (2006) studied the correlations between variables included in the conventional single-level speed prediction models, and found that the correlation within group may violate the assumption of residual independence assumed in linear regression models. The result would be biased parameter estimates and mis-estimated standard errors. They suggested use of multi-level models to enhance the reliability for operating speed prediction. To validate this assertion, passenger car speeds under free-flow conditions were collected from 18 tangent-curve two-lane rural highway sections, and the site selection criteria were similar to those in the study by McFadden and Elefteriadou (2000). The operating speed on tangents, which was believed to have impacts on the responder, was included in the proposed two-level model as the 1st level predictors, whereas highway sections with various geometric features were treated as the 2nd level predictors. The speed reduction (*85MSR*) on the other hand, was employed as the responder. Regression analysis yielded a positive result as the variance component at each level of the model was significantly reduced.

This study did not result in the development of a speed prediction model. However, it provided some insights into theoretical advantages of using multi-level models for the operating speed or the speed differential prediction. With multi-level speed prediction models, the potential contributions of factors at each level to operating speed variations can be identified. Thus, engineers or decision makers would be able to look at those factors,

and choose, for example, either decreasing the degree of curvature, or modifying driver speed behaviour by introducing photo radar, which was suggested as the most cost-effective measure for treatment.

2.3 Summary

The materials provided in this chapter cover the existing literature related to highway geometric design consistency. The discrepancy between the classical design speed approach and international practices for alignment design implies the weaknesses that current North American design guides experience. It has been evident that the design consistency theory is a promising alternative supplementary to traditional design approach. However, translating the theory to practical applications faces some great challenges that need to be solved. Otherwise, the benefits of the application of design consistency will be compromised. With regard to operating speed prediction, several issues need to be closely examined before it can be effectively implemented.

- **Study site selection criteria:** Almost all of the study sites were selected at curves with no intersections or change in number of lanes, except for studies by Misaghi (2003) and Figueroa and Tarko (2005). According to Hassan (2004), out of 91 curves surveyed in Eastern Ontario, 63 curves were coupled with intersections or turning lanes. Thus, the existing models were less effective for over half of the road segments as mentioned above.
- **Model applicability:** The previous research related to operating speed prediction focused on two-lane rural highways. There has been little research conducted on other types of roads, particularly in highly developed areas, where traffic

collisions often occur. Besides, most of the models were developed based on speed data collected in other countries. Therefore, their applicability for Canadian roads needs to be verified using local data.

- **Speed prediction on tangents:** Due to the difficulty of speed prediction in tangent sections (Polus et al, 2000), there has been little statistical model successful to predict tangent speeds. Instead, desired speeds on tangents were assumed by some researchers, such as 94.9 km/h, 97.9 km/h, and 102.0 km/h recommended by Lamm and Choueiri (1987), Krammes et al (1995b), and Hassan et al (2000). It should be noted that the suggested tangent speeds were based on studies conducted in different countries (the United States and Canada), where the geometric characters as well as physical constraints might not be the same. Hassan's suggestion seems to be more realistic as it has reflected the appearance of safer and faster cars. However, lacking analytical models for speed prediction on tangents requires the assumption to be justified based on a larger database. Another extremity of tangent speed estimation is less effective in practical use due to the complexity of the model itself. For instance, the model presented in the work by Figueroa and Tarko (2005) includes over 10 independent variables. Requisition of such diverse information in reality might not be feasible.

- **Measured speed versus actual speed:** the speed measurement generally relies on the use of Radar/Lidar guns, stop watch, speed trap, and traffic counters/classifiers. The effect of manual measurements was once quantified by Hassan (2004), using Misaghi's database (2003). The difference of the observed speed with and without using the radar gun could be as high as 7 km/h. Therefore, the manual speed

measurement could possibly reduce the accuracy as well as the reliability of the database used for operating speed estimations.

- **Data reduction:** empirical studies such as those by Mcfadden and Elefterriadou (2000) and Misaghi (2003) validated Hirsh's hypothesis that use of ΔV_{85} would result in underestimation of the speed reduction from the tangent into the curve. Thereby, they introduced new parameters *85MSR* and $\Delta_{85}V$, based on the individual driver's speed profile. Park and Saccomanno (2006) compared parameters ΔV_{85} *and* *85MSR*, and explained the reason why *85MSR* was always greater than ΔV_{85}, from a theoretical perspective. According to the authors, ΔV_{85} assumed that speed samples on the tangent and the curve are independent, while *85MSR* accounted for the dependency between two samples through the covariance term. Thus, neglecting the correlation between samples at two locations caused loss of information during sample aggregation by ΔV_{85} measure. The authors finally suggested that *85MSR* should replace ΔV_{85} to establish design consistency along a roadway. Admittedly, *85MSR* and $\Delta_{85}V$ are two logical measures. However, it should be noted that $\Delta_{85}V$ only explains the speed differential from the approach tangent to the middle of the curve, but does not account for the speed behaviour after the midpoint of the curve. In addition, *85MSR* uses nine points, while $\Delta_{85}V$ uses two points for speed measurements. One may argue that there might be too little observation to precisely capture driver speed behaviour on the curve. For instance, in case the minimum speed does not

take place at the middle of the curve, $\Delta_{85}V$ could possibly underestimate the speed reduction as well.

Considering the above arguments, the necessity of having a comprehensive investigation on driver speed behaviour based on actual driving data, becomes more evident. In this study, speed data are collected from four common types of roads in Eastern Ontario. Driver speed behaviour is studied using real driving data. With the fulfillment of the objectives, the results would contribute to enlarging the existing database in Canada, with which the speed model development and verification, and the design speed selection can be performed in a more effective and reliable way. Secondly, the study of design consistency on other road classes in addition to rural two-lane highways will facilitate future research related to more complex and advanced highway networks in today's world.

CHAPTER 3: DATA COLLECTION AND DATABASE PREPARATION

To investigate driver speed behaviour using actual driving data, an experiment was designed and executed on the selected highway facilities in Eastern Ontario from August to October, 2005. Driver behaviour measures such as: the use of fuel pedal, brake and steering, longitudinal and lateral acceleration, and instantaneous speed of the test vehicle were recorded during the period of the experiment, using different instruments installed in a regular vehicle. The trajectory of the test vehicle was tracked with GPS technique. Upon the completion of each test run, the collected data were transmitted to a Geographic Information System (GIS) software Arcview 3.2a, with which the database was constructed and prepared for the analysis.

This chapter comprises two sections. The first section describes the data collection process, covering the experiment scheme, selection of the test route, driver recruitment, as well as an introduction to the equipment used in the experiment. The second section discusses the database construction. The software used for data integration, the database preparation process, and extraction of horizontal curve parameters are depicted in detail.

3.1 Data Collection

3.1.1 Experiment Scheme

The field experiment was divided into two stages, namely experiment preparation and operation. The preparation process included the selection of the test route, equipment installation and calibration, and driver recruitment. During the experiment operation, the

recruited drivers drove the instrumented vehicle along the pre-designed test route, while the installed test instruments recorded the driver's performance in response to the highway geometry, the physical features, adjacent traffic, and other factors they perceived. The total trip length for each test run was approximately 112 kilometers, and the duration of the test, on average, lasted for one hour and half. When the field work was completed, the collected data were downloaded to a laptop immediately. The raw data stored in the experiment devices were then discarded, and the test instruments were reconnected for the next run.

Three persons were required for each test run: two operators responsible for operating the test equipment and one volunteer driver. Field tests were carried out on four road classes plus freeway interchanges. The selected test route included the most common road types in Eastern Ontario. The experiments were arranged during daylight time (after sunrise, and before sunset), and under normal weather conditions (i.e. dry pavement). The field experiments were scheduled to include the data for rush and non-rush hours, and both on weekdays and weekends.

Test drivers were asked to drive the test vehicle in a way compatible with their inherent driving patterns. The commencement and completion of the test were disguised from the awareness of the drivers. The data collection seamlessly started after travelling for some distance to allow drivers to become familiar with the specific test vehicle.

3.1.2 Selection of the Test Route

The test route was intentionally selected to cover four common types of roads in Eastern Ontario, plus freeway interchanges in connection with adjacent roadways. For practical purpose, the test route was chosen as close to the City of Ottawa as possible. There are a total of seven different roads involved in the test route. Four out of these roads cross rural areas where light traffic is expected. The remaining three roads are located nearby the CBD area of the City of Ottawa, where moderate to heavy traffic is predominant. The selected road types are as follows:

- Urban freeway (Highway 417).
- Two-lane rural highways (Provincial Route 7, Regional Routes 3 and 6).
- Rural freeway (Highway 416).
- Urban/Suburban (Regional Routes 12 and 73).

The road types associated with the road names and the trip lengths are presented in Table 3-1. The test route, represented by a solid black line, is illustrated in Figure 3-1. The red arrows show the driving direction of the test vehicle. Data collection started at the beginning of the entry ramp at Bronson interchange to Highway 417, and ended at the intersection between Prince of Wale's Drive and Hog's Back Road.

Table 3-1: Selected Test Route.

Road Type	Road Name	Length (m)
Urban Freeway	Highway 417	24,885
Two-Lane Rural Highway	Highway 7 (Provincial Route 7)	9,095
Two-Lane Rural Highway	Dwyer Hill RD. (Regional Route 3)	22,862
Two-Lane Rural Highway	Roger Steven DR. (Regional Route 6)	19,122
Rural Freeway	Highway 416	16,473
Urban/Suburban Road	Fallowfield RD. (Regional Route 12)	8,523
Urban/Suburban Road	Prince of Wales DR.(Regional Route 73)	6,766
Interchange Entry Ramp	Bronson to Highway 417	280
Interchange Exit Ramp	Highway 417 to Highway 7	2,254
Interchange Entry Ramp	Regional Route 6 to Highway 416	365
Interchange Exit Ramp	Highway 416 to Regional Route 12	620
		Total Length: 112,245

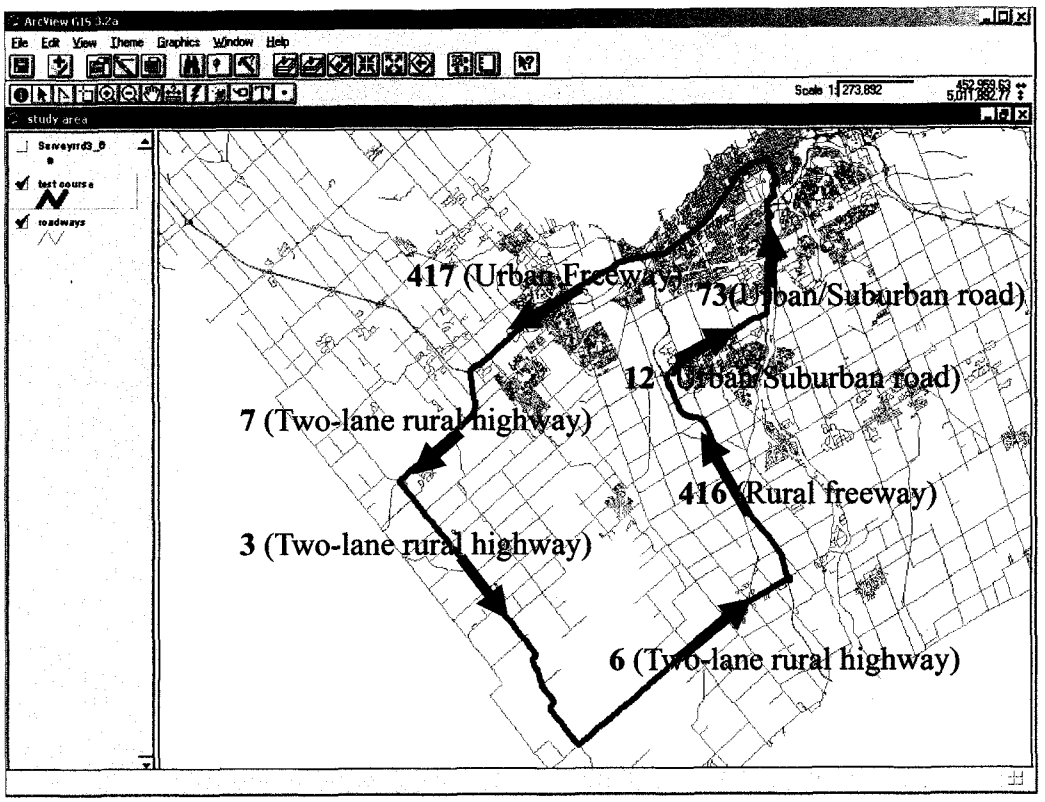


Figure 3-1: Map of the Test Route.

As shown in the above figure, the driving direction is described as follows:

- Highway 417: from Bronson interchange to Exit No. 145 of Highway 417.
- Provincial Route 7: from Exit No. 145 of Highway 417 to intersection with Regional Route 3.
- Regional Route 3: from intersection with Provincial Route 7 to intersection with Regional Route 6.
- Regional Route 6: from intersection with Regional Route 3 to Roger Steven interchange of Highway 416.
- Highway 416: from Roger Steven interchange to Exit No. 66 of Highway 416.
- Regional Route12: from Exit No.66 of Highway 416 to intersection with Regional Route 73.
- Regional Route73: from intersection with Regional Route12 to intersection with Hog's Back Road.

3.1.3 Driver Recruitment

To obtain actual driving data, volunteer drivers were recruited to participate in the test driving. In selecting the test drivers, the selected driver sample should be able to reflect the majority of the driver populations on the road. Second, some minimum sample size needs to be satisfied. Based on the above considerations, experienced drivers with G licenses were considered as the qualified drivers mainly because G license drivers were believed to represent the high percentage of the driver populations on Ontario's roadway facilities.

Thirty qualified drivers were recruited in this study to meet the requirement of the minimum sample size.

Before the test run, one of the research team members advised each driver of the objectives of the experiment and the driving task. Experiment instruments installed inside the test vehicle were briefly introduced. The map of the test route was handed out, showing driving directions to the driver. Test drivers were told to drive following their natural driving habits with no special tasks that need to be performed during the experiment. After completing the test run, the test drivers were asked about the experience during the test run, such as the degree of comfort with the test vehicle and the road. The results of this survey showed that the test drivers were all comfortable with the test vehicle during the experiment.

3.1.4 Experiment Equipment

The test vehicle was a Ford Windstar minivan, in which a number of instruments were installed to capture driver behaviour. As this research aims to investigate driver speed behaviour, only those instruments relevant to the research interest of this study will be discussed. The primary equipment consisted of a Corsa data acquisition box and GPS receivers. The Corsa data acquisition box was capable of detecting and recording lateral and longitudinal accelerations of the vehicle, and the use of fuel pedal, brake, and steering. It was also able to measure the instantaneous speed of the test vehicle. The Global Positioning System (GPS) receivers were used to track driver actual path with the measured coordinates. Several supplementary devices were also deployed in the experiment, including a Lidar gun and a video camera. The Lidar gun was used to measure

the distance, as well as the relative speed between the test vehicle and the vehicle in front. A video camera was installed in the test vehicle, recording the driver's front view during the test trip. The summary of the experiment equipment is presented in Table 3-2.

Table 3-2: Summary of the Test Equipment.

System	Equipment	Function
Corsa Data Acquisition Box	Throttle Sensor	Detecting the use of fuel pedal
	Brake Sensor	Detecting the use of brake
	Steering Sensor	Detecting the use of steering
	Accelerometer	Detecting lateral and longitudinal accelerations
	Speed Sensor	Recording instantaneous speed
GPS Receiver	GPS (Static)	Detecting errors
	GPS (Rover)	Tracing driver path
Lidar Gun		Recording speed and distance
Video Camera		Recording driver's front view

3.1.4.a Corsa Data Acquisition System and Speed Sensor

The Corsa data acquisition system is designed to collect the vehicle's kinematic and dynamic characters, and the driving manipulation data. It contains both the hardware and software components. The hardware component - Corsa box has three outlets that are able to receive the electronic signals (raw data) from different sensors. One of the three outlets receives electronic signals from the sensors connected to the brake pedal, fuel pedal, and steering device of the test vehicle, respectively. Another outlet is connected to a control box, which was held by the experimenter during the test run. The control box acts as a stop watch, recording the time of the occurrence of events, such as the start and end of the experiment, the presence of construction zones, or occasional events that might influence

the normal driving performance. The last outlet is linked to a magnetic speed sensor attached to the left rear wheel of the test vehicle.

The software component - a software provided with Corsa box allows the users to configure the proposed measures (e.g. throttle, brake, steering, and speed), process the raw data, and produce the data output as specified by the user in the configuration file. In this research, the speed measurement was configured to be recorded every 0.2 seconds. In other words, there were five speed measurements available within one second. In addition, a time stamp corresponding to each measurement was provided in the output file.

A magnetic speed sensor (SS1) was used in this study to measure the instantaneous speed of the test vehicle. For sensor installation, four magnets were firstly mounted on the moving part of the rear wheel. Then, the magnetic sensor was assembled on a bracket attached to the vehicle body in a manner that the flat end faces the moving part. The gap between the sensor and the moving magnets should be less than 6.35 mm (0.25 inches) ensuring the best operation. Another end of the speed sensor was connected to the Corsa box. When the wheel turns, the attached magnets will cut magnetic fields, resulting in changes in electronic signals that can be detected by the speed sensor. The Corsa system then converts the signal changes (pulses) to a speed value. The magnitude of the speed depends on the signal changing frequency (pulses per mile). For example, the higher the value of the "pulses per mile", the greater the instantaneous speed.

The speed sensor calibration involved calculating the value of "pulses per mile" and inputting "pulses per mile" into the configuration file of Corsa software. The calibration procedure follows four steps:

- The tire rolling circumference (inches) was firstly determined.

- Number of tire revolution per mile was computed through dividing 63,360 (inch/mile) by the measured tire rolling circumference.
- "Pulses per mile" was obtained by multiplying the "number of tire revolution per mile" by the number of magnets used. "Pulses per mile" was then divided by 3,600 to get the value "Scale" (Hertz/mph) which was used as an input in the configuration file.
- The result in the last step was input into the "Scale" field in the configuration file to complete the calibration.

The test vehicle (Ford Windstar), in which the test instruments were installed is shown in Figure 3-2. Figures 3-3 through 3-5 show the Corsa data acquisition box associated with the sensors used in the experiment. An example of the Corsa output file is illustrated in Figure 3-6.

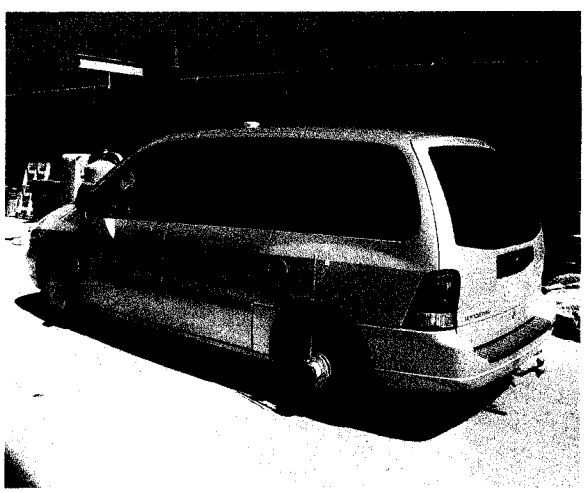


Figure 3-2: Test Vehicle (Ford WindStar).

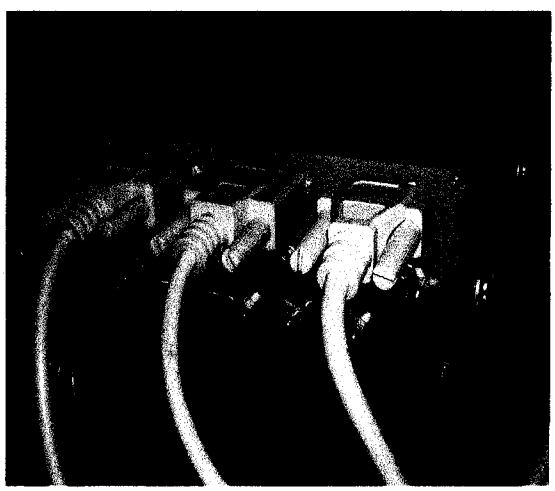


Figure 3-3: Corsa Data Acquisition Box.

Figure 3-4: Magnetic Speed Sensor.

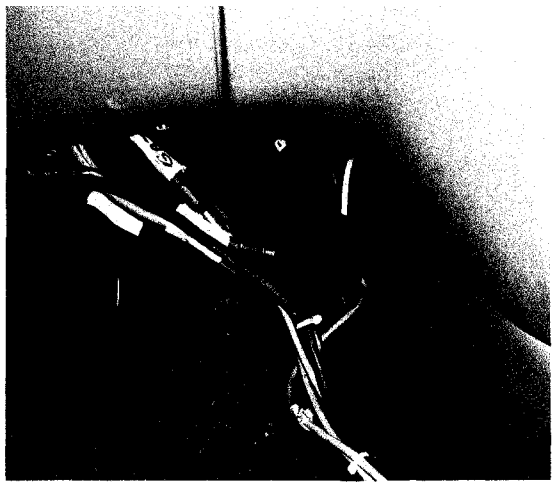


Figure 3-5: Sensors for Throttle (Fuel Pedal), Brake, and Steering.

Driver2

	A	B	C	D	E	F	G	H	I
1	Driver2.dat start=0.000 end=4744.200								
2	Time	Lateral	LonG	Box Temp	RPM	Speed	Brake	Throttle	Steering
3	0	0.032	1.063	-22.7	0	19.347	0.572	35.429	0.538
4	0.2	0.032	0.167			40.411	0.572	33.429	0.43
5	0.4	0.032	0.178			41.359	0.572	30.571	0
6	0.6	0.072	0.169			41.752	0.429	30	-2.903
7	0.8	0.067	0.158			41.605	0.572	29.714	-3.333
8	1	0.017	0.151	-22.7		44.084	0.572	25.714	-4.194
9	1.2	-0.012	0.137			43.879	0.572	20.857	-6.559
10	1.4	-0.039	0.098			44.658	0.572	21.143	-10.108
11	1.6	-0.081	0.094			44.333	0.572	25.429	-13.441
12	1.8	-0.148	0.075			44.198	0.572	29.714	-12.688
13	2	-0.15	0.091	-22.7		44.553	0.572	30.571	-12.258
14	2.2	-0.129	0.114			45.15	0.572	31.143	-12.258
15	2.4	-0.124	0.116			45.553	0.429	34.571	-12.258
16	2.6	-0.117	0.121			46.031	0.572	37.429	-11.398
17	2.8	-0.107	0.123			46.474	0.572	38.286	-10.323
18	3	-0.098	0.121	-22.7		46.96	0.572	39.429	-9.032
19	3.2	-0.091	0.119			48.048	0.572	42	-7.097
20	3.4	-0.062	0.128			48.593	0.572	44.286	-6.237
21	3.6	-0.067	0.144			49.227	0.572	45.429	-6.022
22	3.8	-0.076	0.128			49.707	0.572	45.714	-3.871
23	4	-0.034	0.116	-22.7		51.052	0.572	46.571	-2.366
24	4.2	-0.001	0.139			51.428	0.572	46.571	-1.935
25	4.4	0.001	0.132			51.98	0.572	46.571	-0.323
26	4.6	0.017	0.13			52.692	0.572	46.857	0.323
27	4.8	0.029	0.132			54.035	0.572	46.571	1.075
28	5	0.041	0.128	-22.7		54.472	0.572	46.571	1.72
29	5.2	0.048	0.123			54.98	0.572	46.571	2.366
30	5.4	0.069	0.126			55.81	0.572	46.571	3.118
31	5.6	0.088	0.121			56.597	0.572	46.571	3.441

Figure 3-6: Corsa Data Output File (Excel format).

3.1.4.b GPS System

A GPS system is comprised of three main components: the GPS receiver, the GPS terminal, and the SKI-Pro software. The GPS receiver contacts the satellites and receives the satellite signals during the period of the experiment. The GPS terminal allows the user to configure and specify the survey tasks. SKI-Pro software is a post-processing computer software used to solve the ambiguity problem, and to transform the initial coordinates (WGS84) into the local coordination system (e.g. UTM).

It should be noted that as the satellite signals propagate through the space to earth, they are affected by the atmosphere, resulting in the degradation in accuracy of observations, which is defined as the ambiguity problem. To overcome this ambiguity problem, two sets of Leica GPS Systems were used in this study. The first one was placed at a fixed point with known coordinates (GPS control point), detecting the errors due to the atmosphere effect on satellite signals. The second GPS receiver was then installed in the test vehicle, performing kinematic measurements to track the trajectory of the test vehicle.

To ensure an accurate survey, the baseline length between two GPS receivers should be minimized. This is mainly because the shorter the baseline length, the more likely it is that the atmosphere through which the satellite signals pass to two receivers will be identical, thus alleviating most of the atmospheric effects. In this sense, the GPS control point was selected in a manner that the distance between the GPS control point and the furthest end of the test route was controlled not to exceed 20 km, a threshold below which the ambiguity problem can be fully solved with SKI-Pro software.

The GPS receivers for both the static and kinematic measurements were configured to record five readings within one second. Thus, the position of the vehicle (driver trajectory)

could be determined every 0.2 seconds. At the end of each test run, the observed GPS data were exported to the SKI-Pro program, which is capable of processing the raw data, including solving the ambiguity problem and converting the initial readings on the basis of WGS84 datum to local coordinate system. In this study, the output of the GPS data was converted to UTM datum, in conformity with the data format for the roadway centerline provided by the City of Ottawa.

The GPS receivers (Static and Kinematic Units) and the GPS control point are shown in Figures 3-7, 3-8, 3-9 and 3-10. An example of the processed GPS data is given in Figure 3-11, in Excel format.

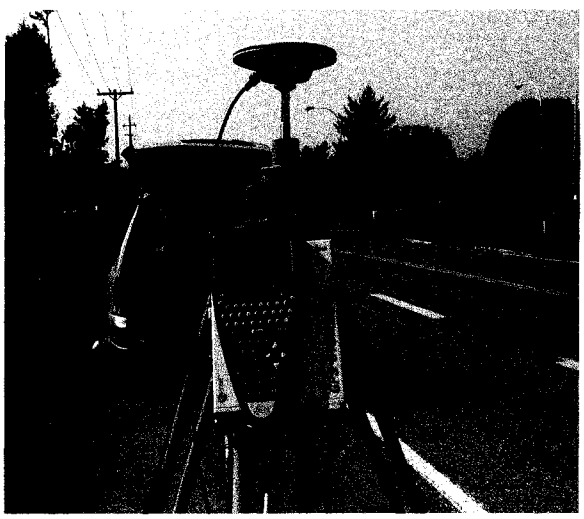


Figure 3-7: GPS Receiver (Static).

Figure 3-8: GPS Receiver Configuration.

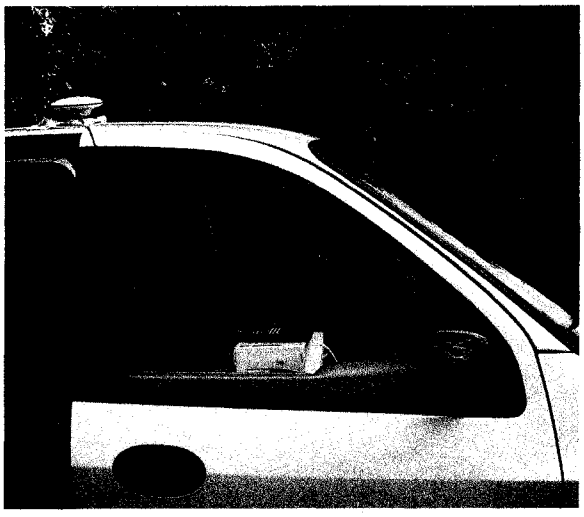


Figure 3-9: GPS Receiver (Rover).

Figure 3-10: GPS Control Point.

Driver2GPS

	A	B	C	D	E	F	G	H	I	J	K
1	Point Id	Date	Time	Ambiguity	Status	Type	Solution	type	Frequency	Easting	Northing
2	06990825_1420536	08/25/2005	10:20:54	yes	Moving	Phase	Iono	free	(L3)	445460.5163	502592
3	06990825_1420538	08/25/2005	10:20:54	yes	Moving	Phase	Iono	free	(L3)	445460.5183	502592
4	06990825_1420540	08/25/2005	10:20:54	yes	Moving	Phase	Iono	free	(L3)	445460.5125	502592
5	06990825_1420542	08/25/2005	10:20:55	yes	Moving	Phase	Iono	free	(L3)	445460.5086	502592
6	06990825_1420544	08/25/2005	10:20:55	yes	Moving	Phase	Iono	free	(L3)	445460.5072	502592
7	06990825_1420546	08/25/2005	10:20:55	yes	Moving	Phase	Iono	free	(L3)	445460.5066	502592
8	06990825_1420548	08/25/2005	10:20:55	yes	Moving	Phase	Iono	free	(L3)	445460.5112	502592
9	06990825_1420550	08/25/2005	10:20:55	yes	Moving	Phase	Iono	free	(L3)	445460.5117	502592
10	06990825_1420552	08/25/2005	10:20:56	yes	Moving	Phase	Iono	free	(L3)	445460.5125	502592
11	06990825_1420554	08/25/2005	10:20:56	yes	Moving	Phase	Iono	free	(L3)	445460.5117	50259
12	06990825_1420556	08/25/2005	10:20:56	yes	Moving	Phase	Iono	free	(L3)	445460.5082	502592
13	06990825_1420558	08/25/2005	10:20:56	yes	Moving	Phase	Iono	free	(L3)	445460.5092	502592
14	06990825_1420560	08/25/2005	10:20:56	yes	Moving	Phase	Iono	free	(L3)	445460.5152	502592
15	06990825_1420562	08/25/2005	10:20:57	yes	Moving	Phase	Iono	free	(L3)	445460.5133	502592
16	06990825_1420564	08/25/2005	10:20:57	yes	Moving	Phase	Iono	free	(L3)	445460.5084	502592
17	06990825_1420566	08/25/2005	10:20:57	yes	Moving	Phase	Iono	free	(L3)	445460.5105	50259
18	06990825_1420568	08/25/2005	10:20:57	yes	Moving	Phase	Iono	free	(L3)	445460.5079	50259
19	06990825_1420570	08/25/2005	10:20:57	yes	Moving	Phase	Iono	free	(L3)	445460.5084	502592
20	06990825_1420572	08/25/2005	10:20:58	yes	Moving	Phase	Iono	free	(L3)	445460.5096	502592
21	06990825_1420574	08/25/2005	10:20:58	yes	Moving	Phase	Iono	free	(L3)	445460.5135	502592
22	06990825_1420576	08/25/2005	10:20:58	yes	Moving	Phase	Iono	free	(L3)	445460.5098	502592
23	06990825_1420578	08/25/2005	10:20:58	yes	Moving	Phase	Iono	free	(L3)	445460.5131	502592
24	06990825_1420580	08/25/2005	10:20:58	yes	Moving	Phase	Iono	free	(L3)	445460.5089	502592
25	06990825_1420582	08/25/2005	10:20:59	yes	Moving	Phase	Iono	free	(L3)	445460.5113	502592
26	06990825_1420584	08/25/2005	10:20:59	yes	Moving	Phase	Iono	free	(L3)	445460.5142	502592
27	06990825_1420586	08/25/2005	10:20:59	yes	Moving	Phase	Iono	free	(L3)	445460.5151	502592
28	06990825_1420588	08/25/2005	10:20:59	yes	Moving	Phase	Iono	free	(L3)	445460.5143	502592
29	06990825_1420590	08/25/2005	10:20:59	yes	Moving	Phase	Iono	free	(L3)	445460.5068	502592

Driver2

Figure 3-11: GPS Data Output File (Excel Format).

3.1.4.c Lidar Gun and Video Camera

A Lidar gun (SpeedLaser©) was used during the experiment to measure the distance and the relative speed between the test vehicle and the impeding vehicle in front, which may exist on the same lane as the test vehicle. The distance measured was given in meters. The Lidar gun uses a technology called Light Detection and Ranging, and is capable of measuring the range and the speed of a target up to 1,220 meters. The measured distance between the test vehicle and the leading vehicle in the same driving lane was used for identifying the free-flow condition.

A video camera was installed to record the driver's front view during the test run. It is helpful in identifying occasional events encountered on the road, which may affect driver normal behaviour, such as: the presence of construction zones, intersections, and so on. The Lidar gun and video camera are shown in Figure 3-12.

Figure 3-12: Video Camera and Lidar Gun.

3.1.5 Equipment Coordination

Since more than one instrument were used in this study, it was imperative to coordinate all the equipment together. Before the commencement of each test, time synchronization was performed by ensuring that the time on the laptop and Lidar gun were compatible with that of the GPS receiver. Upon the completion of the test, the collected data with different experiment devices (i.e. Corsa system, GPS system, and Lidar gun) were exported to a laptop, where the data were saved in Excel format. The data file for each test was identified with the driver ID.

3.2 Database Preparation

The database prepared for this study contains two major data sets. The first type is the operating speed collected by Corsa magnetic speed sensor associated with both temporal and spatial references. The second data type refers to highway horizontal geometric features, in terms of curve parameters. The ArcView computer software for GIS applications was deployed to work as a platform, on which the observed speeds, GPS data (drivers' trajectory), and the coordinates of the roadway centerline were combined. Unlike other research where curve parameters were usually obtained from as-built drawings, in this study, the highway geometry was extracted based on the coordinates of the roadway centerline, using RoadFit extension (Imran et al, 2006) in Arcview environment.

3.2.1 Introduction of Arcview 3.2a

Arcview 3.2a is a specialized computer software that allows the user to visualize, explore, query, and analyze the data spatially. It is a powerful tool for users to deal with the spatially referenced data in Arcview environment. It supports a variety of data formats

such as vector data (e.g. shape files), raster data (e.g. scanned maps or satellite images), and tabular data (e.g. dBase files). The shape file data in this study could be the view of the highway networks in the City of Ottawa area, whereas the tabular data are attribute tables containing the numeric characteristics corresponding to the spatial features, such as the coordinates of a point on the road centerline or the length of a highway segment. The advantages of managing data in Arcview 3.2a environment are:

- Arcview 3.2a is capable of reading the tabular data in various formats, such as *dBase* files, *ARC/INFO* tables, and comma or tab delimited *ASCII* test files. It is advantageous to import the data into Arcview software from different sources.
- Arcview 3.2a enables two tabular data files to be linked or joined to the existing shape file based on a common identifier field.
- Arcview 3.2a supports add-on software products in terms of Extensions, which would greatly expand the capability of Arcview.
- The attribute table can be readily exported to other application computer software, such as a spreadsheet, in either *dBase* or *ASCII* format for post-processing analysis.

In addition, the shape file and the corresponding attribute table are interactively correlated. The selection of any record in the attribute table will automatically highlight its physical feature in the view, and vice versa. This valuable function makes the database visible to the user, and is particularly useful in extracting data from the database.

3.2.2 Extraction of Horizontal Curve Features

Highway geometric data in terms of curve parameters are usually acquired from the as-built drawings. However, the as-built drawings were unfortunately not available for most of the selected roadways in this research. Subsequently, an alternative method was proposed by extracting curve information based on the coordinates of the road centerline provided by the City of Ottawa, using RoadFit extension developed by Imran et al (2006).

The RoadFit extension is capable of determining the radius, the length of horizontal curves, as well as the length of spirals if they are applied, based on the given coordinates of points on the road centerline. The extraction of horizontal features consisted of several major steps as follows:

- The shape file of a road centerline was loaded into Arcview 3.2a.
- The target horizontal curve consisting of the approach tangent, the curved section, and the departure tangent was selected.
- RoadFit Extension was run by pressing the button on the tool bar in Arcview 3.2a, and curve parameters were determined.
- The curve report was generated, and saved to a specified location.

The RoadFit extension dialog box, an example of extracted curves on Regional Route 3, and the curve report are illustrated in Figures 3-13 through 3-15.

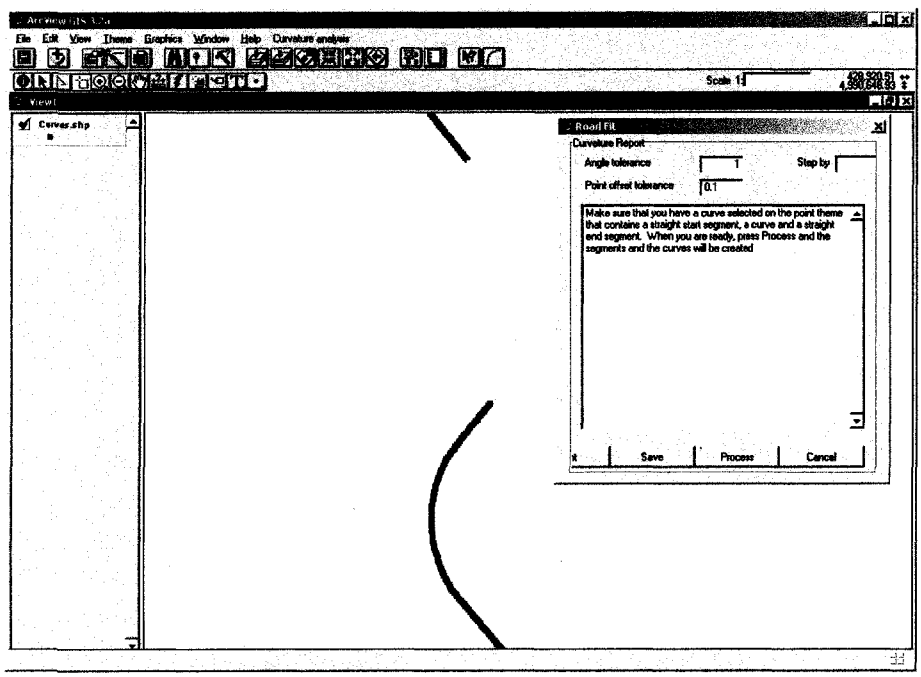


Figure 3-13: Target Horizontal Curve and RoadFit Extension Dialog Box.

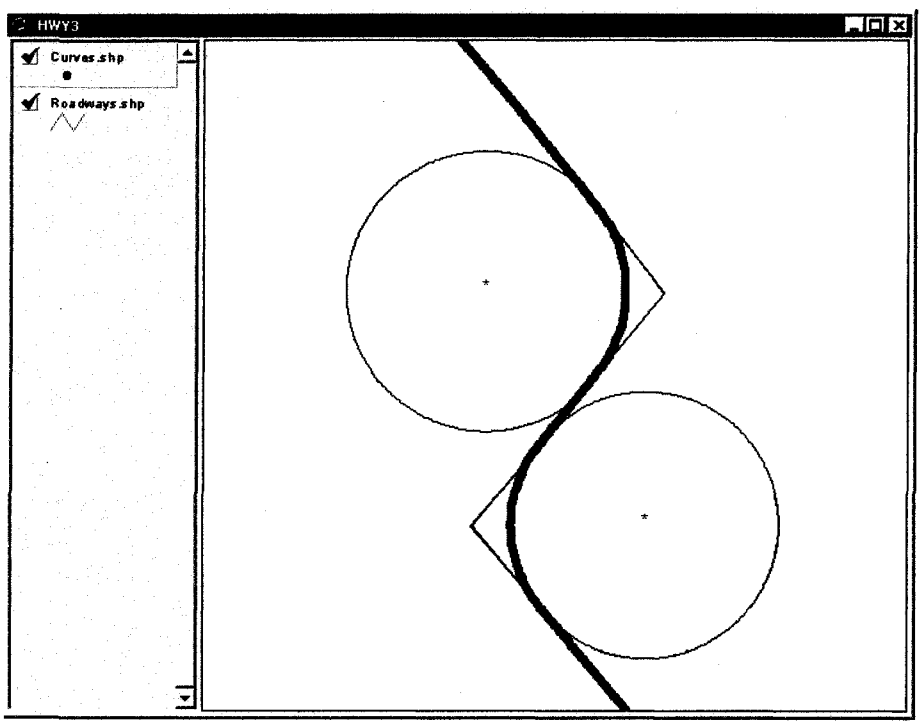


Figure 3-14: Example of Extracted Horizontal Curves on Regional Route 3.

```
Curvature Report:
horizontal alignment.apr Fri Nov 11 22:19:04 2005


Circular Curve Report

Number of points: 39
Average point spacing: 4.87166
Deviation of vector to indicate curve change 1 degrees
Step size (points to skip) 1
Entry points : 6
Exit points  : 17
Curve points : 15

Entry regression
        Y = 5.43773e+006 + (-1.03781 ) X
        R-squared = 1
Exit regression
        Y = 6.16974e+006 + (-2.74651 ) X
        R-squared = 1

Entry Azimuth :136.063
Exit Azimuth  :159.994

Initial circle estimates
Center X, Center Y, Radius
428276.96, 4993062.50, 137.172

Non-linear circular regression results
Center X, Center Y, Radius
428240.98, 4993038.41, 180.17
```

Figure 3-15: Example of Curve Report.

To verify the accuracy and reliability of the extracted curve data, curve parameters from the as-built drawings available for Highways 417, 7, and 416 were compared with the extracted curve values. Curve data from 18 curves were compared and the comparison results are summarized in Table 3-3. The results showed a relatively small difference between them. In general, the average difference of curve radius for all compared curves was 11 percent. The total curve length (including both the circular curve and the spiral) between two measures had an average difference of 13.2 percent. Therefore, this method was accepted and applied to the remaining roadways selected in this study.

Table 3-3: Extracted Curve Data versus Actual Curve Data.

Road / Curve ID	Plan			RoadFit Extension			
	R (m)	L_c (m)	L_s(m)	R (m)	L_c (m)	L_{s1}(m)	L_{s2}(m)
417C1	698.55	N/A	/	801.66	359.60	/	14.99
417C2	2,555.67	N/A	/	2,211.64	250.00	/	204.39
417C3	698.55	202.88	/	488.74	104.99	25.00	30.00
417C4	873.19	469.73	/	869.31	499.98	/	/
417C5	1,600.00	325.42	58.25	863.21	148.18	/	70.00
417C6	2,000.00	491.78	86.61	1,756.90	240.00	220.00	90.00
417C7	2,000.00	358.52	66.61	2,045.95	220.00	130.00	15.00
417C8	873.19	143.41	60.96	916.02	170.00	70.00	25.00
417C9	2,413.11	N/A	/	2,402.16	440.00	95.00	/
416C1	1,800.00	699.62	68.06	1,810.15	844.99	/	/
416C2	1,000.00	214.94	67.60	1,299.08	589.97	/	/
416C3	1,000.00	918.34	67.60	981.37	639.95	199.99	269.99
416C4	1,000.00	940.34	67.60	978.12	739.98	/	234.99
7C1	1,200.00	1,106.25	75.00	1,189.09	1,073.56	160.00	/
Ramp 7C1	475.00	94.05	84.21	447.83	104.99	35.00	49.99
Ramp 7C2	700.00	1,020.65	57.14	548.22	384.09	184.99	244.87
Ramp 416C1	150.00	81.83	48.00	151.42	94.97	20.00	19.99
Ramp 416C2	90.00	50.00	54.44	98.00	49.95	25.00	30.00

N/A: Information is not available in the plans.

R: Curve radius

L_c: Circular curve length

L_s: Total spiral length

L_{s1}: Entry spiral length

L_{s2}: Exit spiral length

3.2.3 Data Integration

As explained in the previous section, operating speeds (instantaneous speeds) were measured by the Corsa system, whereas curve data were extracted based on the coordinates of the actual road centerline. The next question was how to relate the speed data to highway horizontal alignments. The data integration was accomplished through a stepwise procedure that is presented below:

- The operating speed, the GPS data, and the roadway centerline were loaded into Arcview 3.2a.
- The GPS data (driver's trajectory) were joined to the Corsa data (operating speed) by performing "Table Join" in Arcview 3.2a, based on the time stamp, which is a common identifier in these two data sets. A new dataset was then created after the tables were joined.
- The created dataset was designated as a source table whereas the roadway centerline was assigned as a destination table. These two tables were combined again by performing "Table Join" the second time. Afterwards, the created new dataset was converted to the shape file to make it visible. It needs to be pointed out that the execution of "Table Join" in this step is a special type of "Table Join" in Arcview defined as "Spatial Join", which combines two tables on the basis of the shape identifier (i.e. point shape) in the source and destination tables.

Following this procedure, the speed database was completely developed, containing the operating speed corresponding to each point on the roadway centerline, associated with the time reference. The database construction has made use of some advantages of

Arcview 3.2a computer software, and the finished database was ready to be exported to other application software, such as a spreadsheet, for the post-processing analysis. In addition to the speed data, the database contains another component, in terms of horizontal curve parameters extracted by the RoadFit Extension in Arcview 3.2a, as previously explained. The database construction process is exhibited in Figure 3-16. An example of the speed database is given in Figure 3-17.

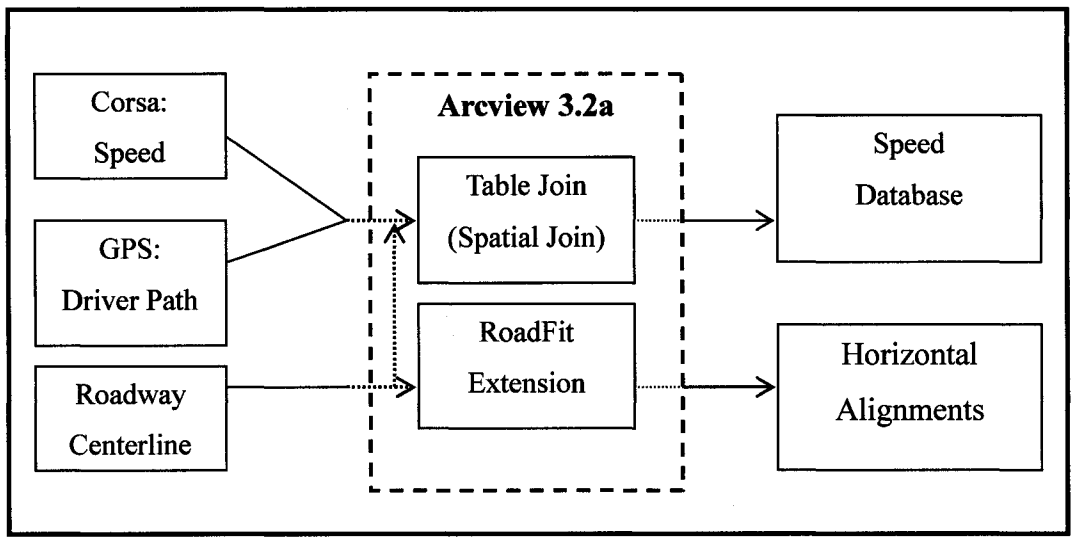
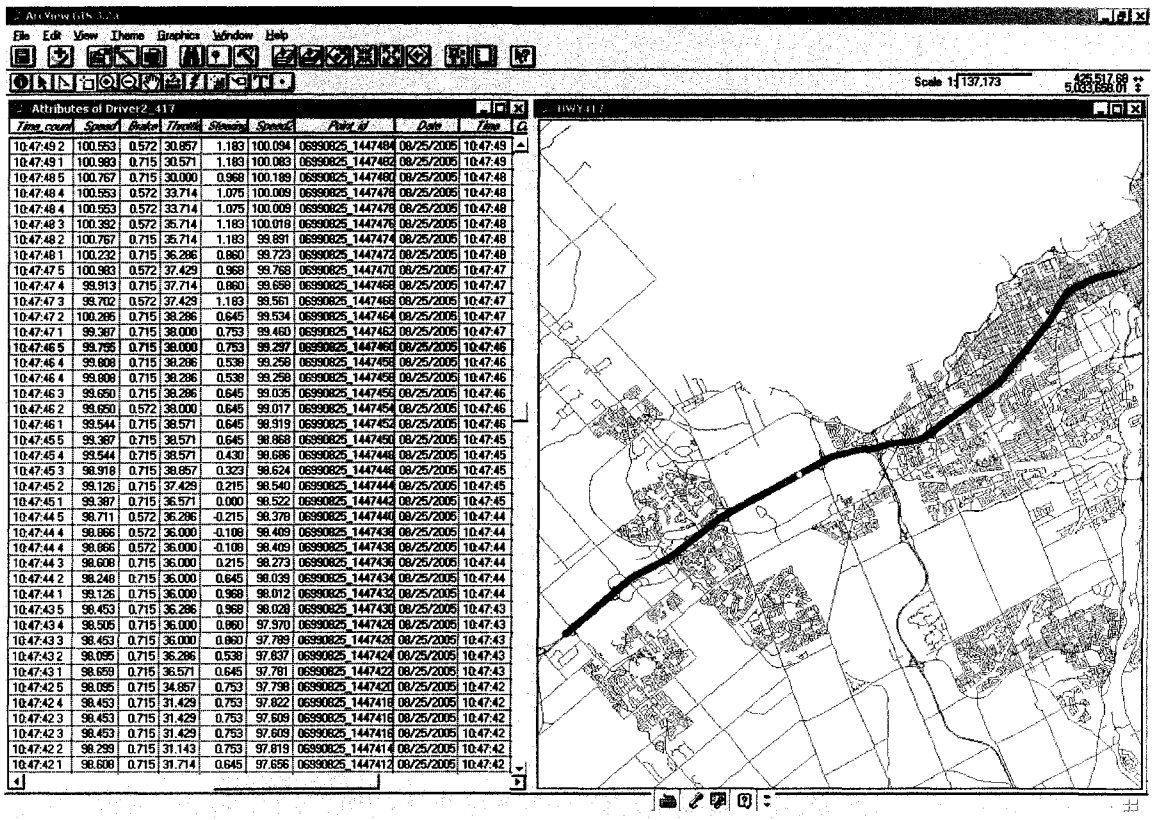


Figure 3-16: Database Construction Process.



Time_count	Speed	Brake	Throttle	Steering	Speed2	Point_id	Date	Time
10:47:49 2	100.553	0.572	30.857	1.183	100.094	06990825_1447484	08/25/2005	10:47:49
10:47:49 1	100.983	0.715	30.571	1.183	100.083	06990825_1447482	08/25/2005	10:47:49
10:47:48 5	100.767	0.715	30.000	0.968	100.189	06990825_1447480	08/25/2005	10:47:48
10:47:48 4	100.553	0.572	33.714	1.075	100.009	06990825_1447478	08/25/2005	10:47:48
10:47:48 4	100.553	0.572	33.714	1.075	100.009	06990825_1447478	08/25/2005	10:47:48
10:47:48 3	100.392	0.572	35.714	1.183	100.018	06990825_1447476	08/25/2005	10:47:48
10:47:48 2	100.767	0.715	35.714	1.183	99.891	06990825_1447474	08/25/2005	10:47:48
10:47:48 1	100.232	0.715	36.286	0.860	99.723	06990825_1447472	08/25/2005	10:47:48
10:47:47 5	100.983	0.572	37.429	0.968	99.768	06990825_1447470	08/25/2005	10:47:47
10:47:47 4	99.913	0.715	37.714	0.860	99.658	06990825_1447468	08/25/2005	10:47:47
10:47:47 3	99.702	0.572	37.429	1.183	99.561	06990825_1447466	08/25/2005	10:47:47
10:47:47 2	100.286	0.715	38.286	0.645	99.534	06990825_1447464	08/25/2005	10:47:47
10:47:47 1	99.387	0.715	38.000	0.753	99.460	06990825_1447462	08/25/2005	10:47:47
10:47:46 5	99.755	0.715	38.000	0.753	99.297	06990825_1447460	08/25/2005	10:47:46
10:47:46 4	99.608	0.715	38.286	0.538	99.258	06990825_1447458	08/25/2005	10:47:46
10:47:46 4	99.608	0.715	38.286	0.538	99.258	06990825_1447458	08/25/2005	10:47:46
10:47:46 3	99.650	0.715	38.286	0.645	99.035	06990825_1447456	08/25/2005	10:47:46
10:47:46 2	99.650	0.572	38.000	0.645	99.017	06990825_1447454	08/25/2005	10:47:46
10:47:46 1	99.544	0.715	38.571	0.645	98.919	06990825_1447452	08/25/2005	10:47:46
10:47:45 5	99.387	0.715	38.571	0.645	98.868	06990825_1447450	08/25/2005	10:47:45
10:47:45 4	99.544	0.715	38.571	0.430	98.686	06990825_1447448	08/25/2005	10:47:45
10:47:45 3	98.918	0.715	38.857	0.323	98.624	06990825_1447446	08/25/2005	10:47:45
10:47:45 2	99.126	0.715	37.429	0.215	98.540	06990825_1447444	08/25/2005	10:47:45
10:47:45 1	99.387	0.715	36.571	0.000	98.522	06990825_1447442	08/25/2005	10:47:45
10:47:44 5	98.711	0.572	36.286	-0.215	98.378	06990825_1447440	08/25/2005	10:47:44
10:47:44 4	98.866	0.572	36.000	-0.108	98.409	06990825_1447438	08/25/2005	10:47:44
10:47:44 4	98.866	0.572	36.000	-0.108	98.409	06990825_1447438	08/25/2005	10:47:44
10:47:44 3	98.608	0.715	36.000	0.215	98.273	06990825_1447436	08/25/2005	10:47:44
10:47:44 2	98.248	0.715	36.000	0.645	98.039	06990825_1447434	08/25/2005	10:47:44
10:47:44 1	99.126	0.715	36.000	0.968	98.012	06990825_1447432	08/25/2005	10:47:44
10:47:43 5	98.453	0.715	36.286	0.968	98.028	06990825_1447430	08/25/2005	10:47:43
10:47:43 4	98.505	0.715	36.000	0.860	97.970	06990825_1447428	08/25/2005	10:47:43
10:47:43 3	98.453	0.715	36.000	0.860	97.789	06990825_1447426	08/25/2005	10:47:43
10:47:43 2	98.095	0.715	36.286	0.538	97.837	06990825_1447424	08/25/2005	10:47:43
10:47:43 1	98.659	0.715	36.571	0.645	97.781	06990825_1447422	08/25/2005	10:47:43
10:47:42 5	98.095	0.715	34.857	0.753	97.798	06990825_1447420	08/25/2005	10:47:42
10:47:42 4	98.453	0.715	31.429	0.753	97.822	06990825_1447418	08/25/2005	10:47:42
10:47:42 3	98.453	0.715	31.429	0.753	97.609	06990825_1447416	08/25/2005	10:47:42
10:47:42 3	98.453	0.715	31.429	0.753	97.609	06990825_1447416	08/25/2005	10:47:42
10:47:42 2	98.299	0.715	31.143	0.753	97.819	06990825_1447414	08/25/2005	10:47:42
10:47:42 1	98.608	0.715	31.714	0.645	97.656	06990825_1447412	08/25/2005	10:47:42

Figure 3-17: Example of Speed Database (Driver 2 on Freeway 417).

3.2.4 Operating Speed Verification

The GPS timing is highly accurate due to the use of an atomic clock. For Leica GPS System 500 that was used in this research, the accuracy of timing reaches 0.1 seconds. This merit was used to validate the measured speed from the Corsa device. To fulfill the speed verification, a second speed (V_2) was introduced. Because the GPS receiver was configured to record the coordinates of a point on the driver's path at the time interval of 0.2 seconds, for two successive points, the distance in between can be computed by the following formula:

$$S = \sqrt{(x_i - x_{i+1})^2 + (y_i - y_{i+1})^2} \qquad (3.1)$$

Where

S = Horizontal distance between two successive points (m)

x_i, x_{i+1} = Coordinate X (Easting) for point i and next point i+1,

y_i, y_{i+1} = Coordinate Y (Northing) for point i and next point i+1,

Then, the average speed for the vehicle travelling between these two successive points was calculated using Equation 3.2 given below:

$$V_2 = \frac{1000 \times S}{0.2 \times 3600} \qquad (3.2)$$

Where

V_2 = Calculated speed (km/h)

0.2 = Time elapsed between two successive points (seconds)

1000 / 3600 = A constant used to convert the speed unit from m/s to km/h.

Having the second speed (V_2) in hand, the difference between the measured speed (Corsa speed) and the calculated speed (V_2) was checked. Speed verification was applied to all the test drivers for thirty trips. The result showed a near perfect agreement between these two speeds. The mean value of the difference for all drivers was 0.758 km/h, and the average standard deviation was 0.946 km/h (Table 3-4). Therefore, the accuracy, as well as the reliability, of the operating speed collected by Corsa System has been justified.

Table 3-4: Statistics of the Difference between Corsa Speed and GPS Speed.

Driver ID	Mean (km/h)	Median (km/h)	Stdev (km/h)
2	0.537	0.436	0.639
3	0.742	0.474	0.932
4	1.208	0.623	1.610
5	1.110	0.682	1.690
6	0.931	0.545	1.255
8	0.940	0.536	1.246
9	0.789	0.537	0.953
10	0.941	0.568	1.231
11	0.513	0.421	0.574
12	1.087	0.784	1.152
13	1.140	0.637	1.432
14	0.839	0.506	1.102
15	0.537	0.456	0.662
16	0.523	0.436	0.630
17	0.578	0.402	0.888
18	0.456	0.350	0.622
19	0.982	0.591	1.283
20	0.557	0.441	0.669
21	0.606	0.460	0.763
22	0.507	0.438	0.575
23	0.602	0.495	0.709
24	0.723	0.483	1.028
25	0.791	0.540	0.977
26	0.796	0.551	0.964
27	0.573	0.465	0.683
28	1.174	0.728	1.426
29	0.521	0.432	0.714
30	0.651	0.494	0.769
31	1.040	0.705	1.224
32	0.700	0.591	0.724
Average Value	0.758	0.522	0.946

CHAPTER 4: IDENTIFICATION OF STUDY CURVES AND DATA PROCESSING

As noted in the literature review, many researchers have reported a high likelihood of traffic collisions on the curved sections of highways. For example, Collins and Krammes (1996) noted that the accident rates on horizontal curves were higher than those on tangent sections of two-lane rural highways. It was estimated that more than half of the fatalities on two-lane rural highways occurred on curved roadway sections (Lamm et al, 1991). Therefore, the curved sections on a roadway represent the most critical locations deserving a close examination. Subsequently, examining the driver speed behaviour in this study required determining the exact locations of horizontal curves and the associated speeds.

4.1 Identification of Study Curves

The establishment of the study curve range should consider a sufficient length of the section so as to monitor driver speed behaviour while traversing the horizontal curve. Therefore, each study curve was determined to have three components, in terms of the approach tangent, the entire curved section, and the departure tangent.

Previous research has noted the effect of the tangent length between adjacent horizontal curves on the tangent speed as well as the speed change from the tangent into the curve. The critical tangent length refers to the distance required to accelerate from a curve to the desired speed and then decelerate to the next curve (Collins and Krammes, 1996). Accordingly, two types of tangents are defined, namely independent and non-independent tangents, on the basis of whether the tangent length is long enough to allow drivers to accelerate to their desired speeds. In accordance with this notion, it was decided in this

research that the range of the study curve starts from 200 meters before the point of curve (*PC*) on the approach tangent, and ends 200 meters after the point of the tangent (*PT*) on the departure tangent. Based on the concept of independent and non-independent tangent, the study curves involved in this study are categorized into two groups: independent curves and non-independent curves. For the non-independent curves defined as curves connected with a tangent less than 200 meters to the preceding curve, the study curve begins from the end point of the preceding curve (*SAT*). If the curve is followed by a succeeding curve connected with a short tangent less than 200 meters, the study site ends at the start point of the succeeding curve (*EDT*).

Within the limits of each study curve, the speed profile for each individual test driver was studied. To accurately describe the speed characteristic and the speed change corresponding to different locations within the study site, some particular points were designated as observation points to track the operating speed of the test vehicle from the approach tangent through the departure tangent. The observation points are the middle of the approach tangent (*AT+100* or *SAT*, for independent and non-independent curves with a short preceding tangent less than 100 m, respectively), the beginning of the circular curve (*PC* or *SC*, for simple curves and curves with spirals, respectively), the middle of the circular curve (*MC*), the end of the circular curve (*PT* or *CS*, for simple curves and curves with spirals, respectively), and the middle of the departure tangent (*DT+100* or *EDT*, for curves with a following tangent greater than 200 m or shorter than 100 m, respectively). A typical study site is sketched in Figure 4-1.

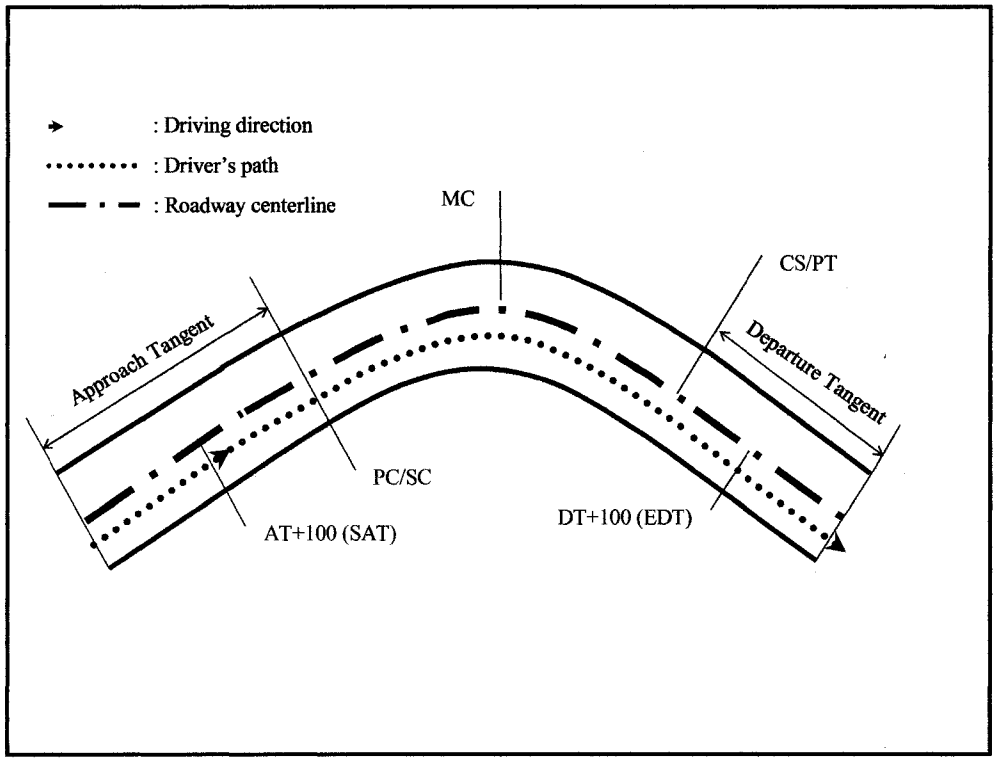


Figure 4-1: Sketch of A Typical Study Curve.

A total of 42 horizontal curves were identified on the test route using RoadFit Extension in Arcview 3.2a. More specifically, the test route included 13, 4, 14, and 11 curves located on two-lane rural highways, the rural freeway, urban/suburban roads, and the urban freeway, respectively, as illustrated in Figures 4-2 through 4-8. The geometric parameters of each curve are presented in Tables 4-1 through 4-4.

Figure 4-2: Study Curves on Urban Freeway 417.

Table 4-1: Study Curve Parameters on Urban Freeway 417.

Study Curve	Road Type	Radius (m)	Circular Curve Length (m)	Entry Spiral Length (m)	Exit Spiral Length (m)
417C1	Urban Freeway	802	360	/	15
417C2	Urban Freeway	2,212	250	/	204
417C3	Urban Freeway	489	105	25	30
417C4	Urban Freeway	869	500	/	/
417C5	Urban Freeway	863	148	/	70
417C6	Urban Freeway	1,757	240	220	90
417C7	Urban Freeway	2,046	220	130	15
417C8	Urban Freeway	916	170	70	25
417C9	Urban Freeway	2,402	440	95	/
417C10	Urban Freeway	3,147	1,042	/	/
417C11	Urban Freeway	2,152	502	35	/

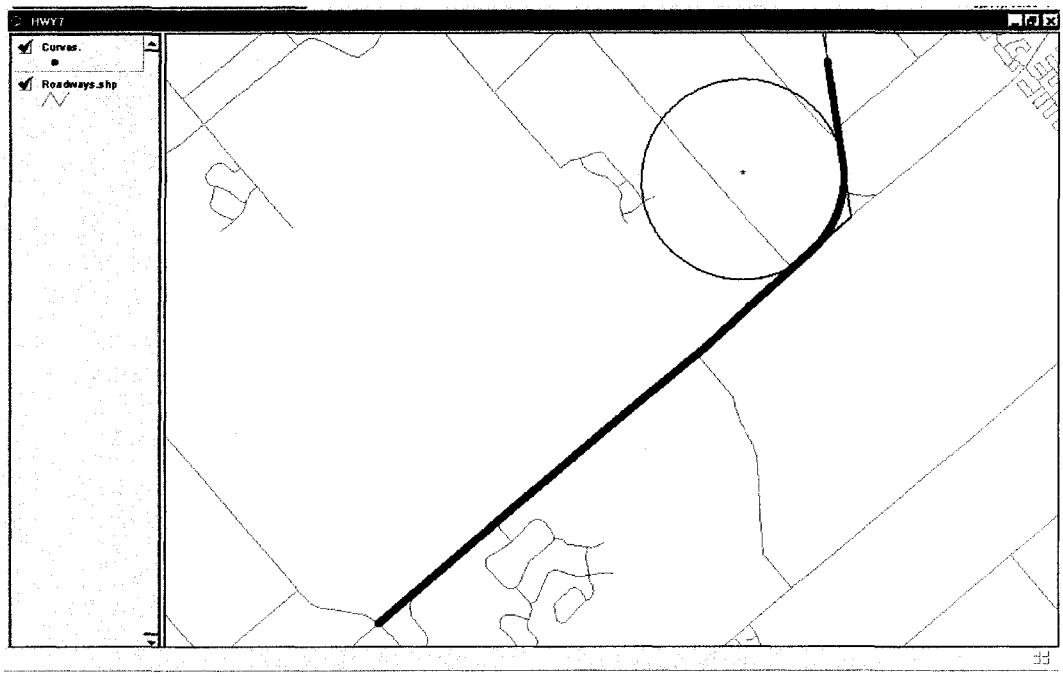


Figure 4-3: Study Curves on Provincial Route 7 (Two-Lane Rural Highway).

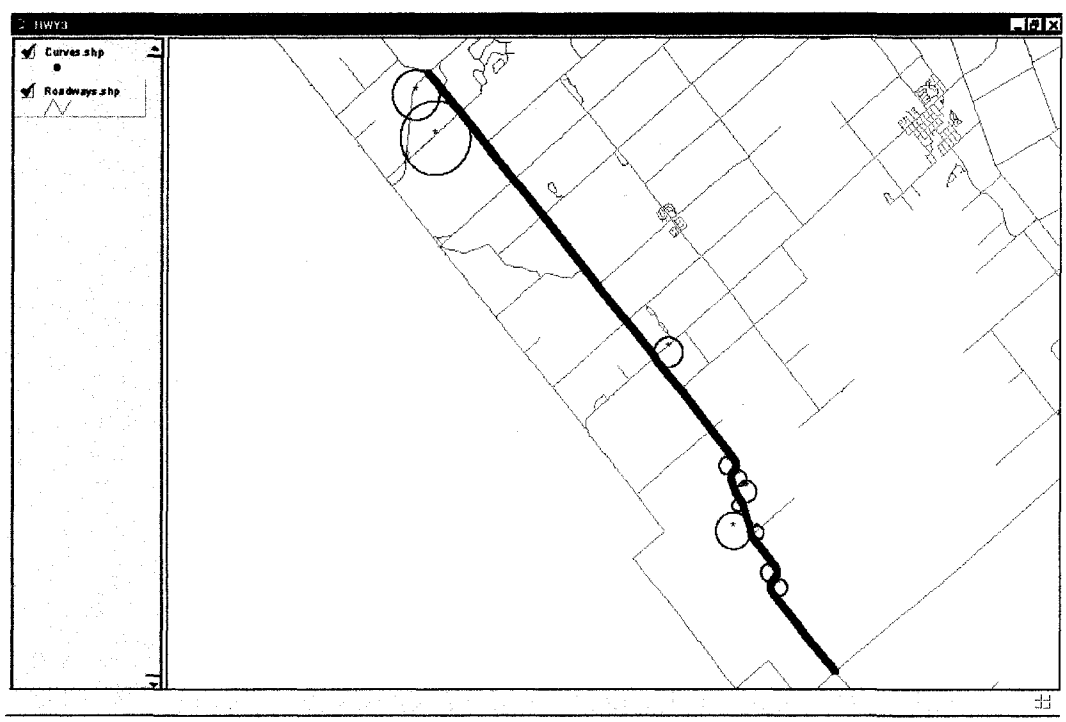


Figure 4-4: Study Curves on Regional Route 3 (Two-Lane Rural Highway).

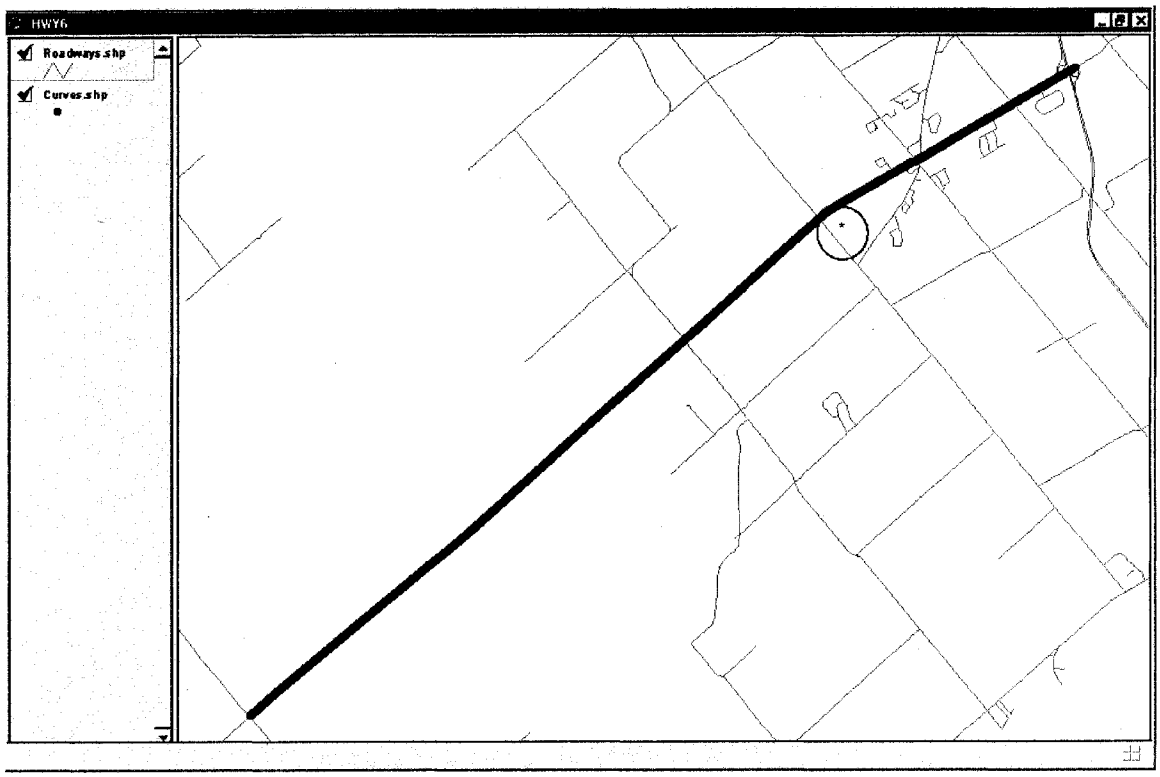


Figure 4-5: Study Curves on Regional Route 6 (Two-Lane Rural Highway).

Table 4-2: Study Curve Parameters on Two-Lane Rural Highways.

Study Curve	Road Type	Radius (m)	Circular Curve Length (m)	Entry Spiral Length Ls (m)	Exit Spiral Length Ls (m)
3C1	Two-lane Rural Highway	743	150	35	/
3C2	Two-lane Rural Highway	1,105	195	60	/
3C3	Two-lane Rural Highway	450	85	40	/
3C4	Two-lane Rural Highway	273	382	/	/
3C5	Two-lane Rural Highway	239	225	55	/
3C6	Two-lane Rural Highway	325	170	35	/
3C7	Two-lane Rural Highway	180	40	25	10
3C8	Two-lane Rural Highway	556	160	30	/
3C9	Two-lane Rural Highway	203	71	30	40
3C10	Two-lane Rural Highway	267	430	35	/
3C11	Two-lane Rural Highway	256	370	/	/
6C1	Two-lane Rural Highway	463	160	/	/
7C1	Two-lane Rural Highway	1,189	1,074	160	/

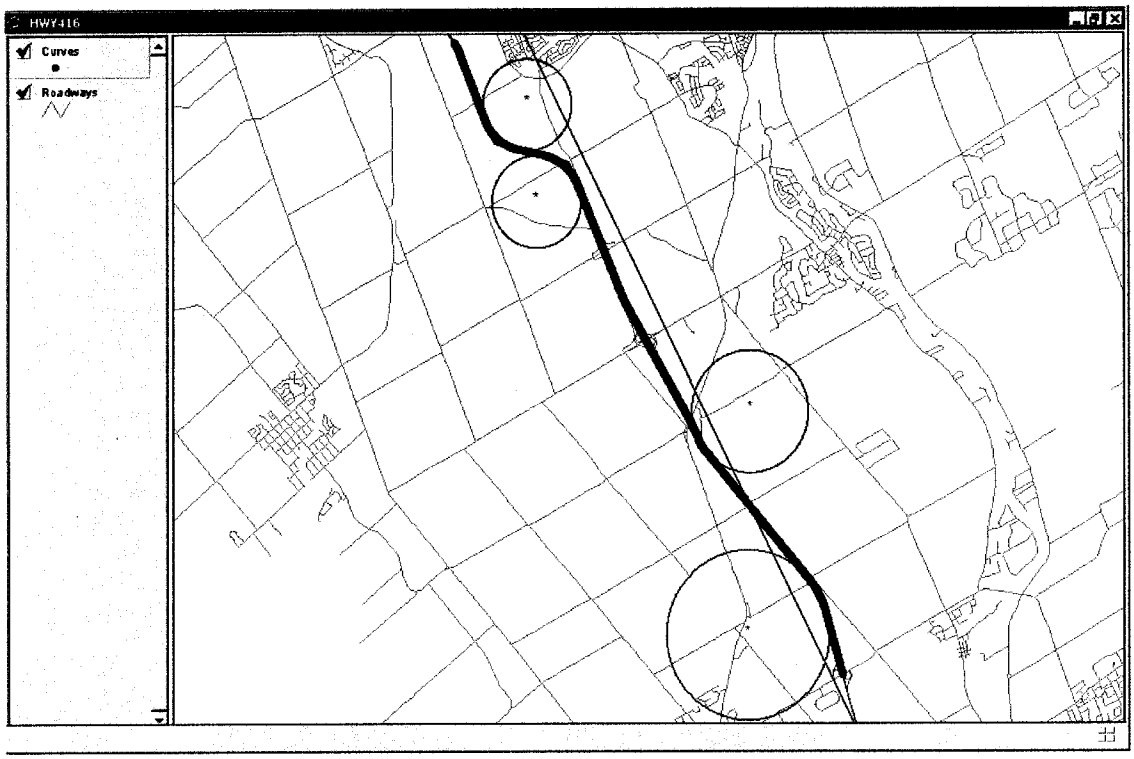


Figure 4-6: Study Curves on Rural Freeway 416.

Table 4-3: Study Curve Parameters on Rural Freeway 416.

Study Curve	Road Type	Radius (m)	Circular Curve Length (m)	Entry Spiral Length (m)	Exit Spiral Length (m)
416C1	Rural Freeway	1,810	845	/	/
416C2	Rural Freeway	1,299	590	/	/
416C3	Rural Freeway	981	640	200	270
416C4	Rural Freeway	978	740	/	235

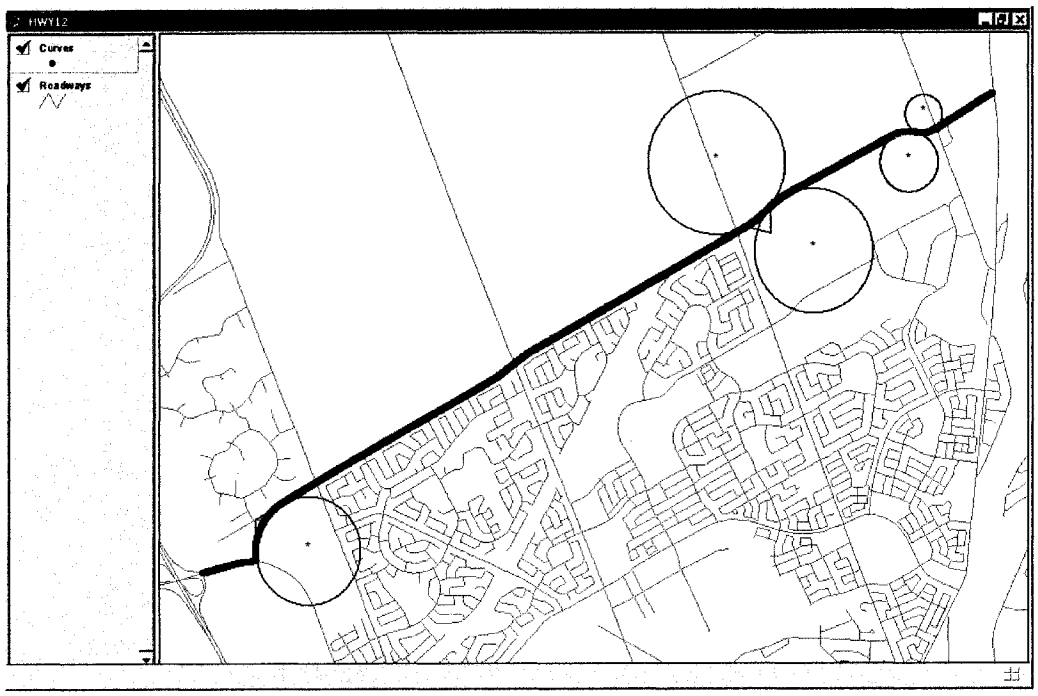


Figure 4-7: Study Curves on Regional Route 12 (Urban/Suburban Road).

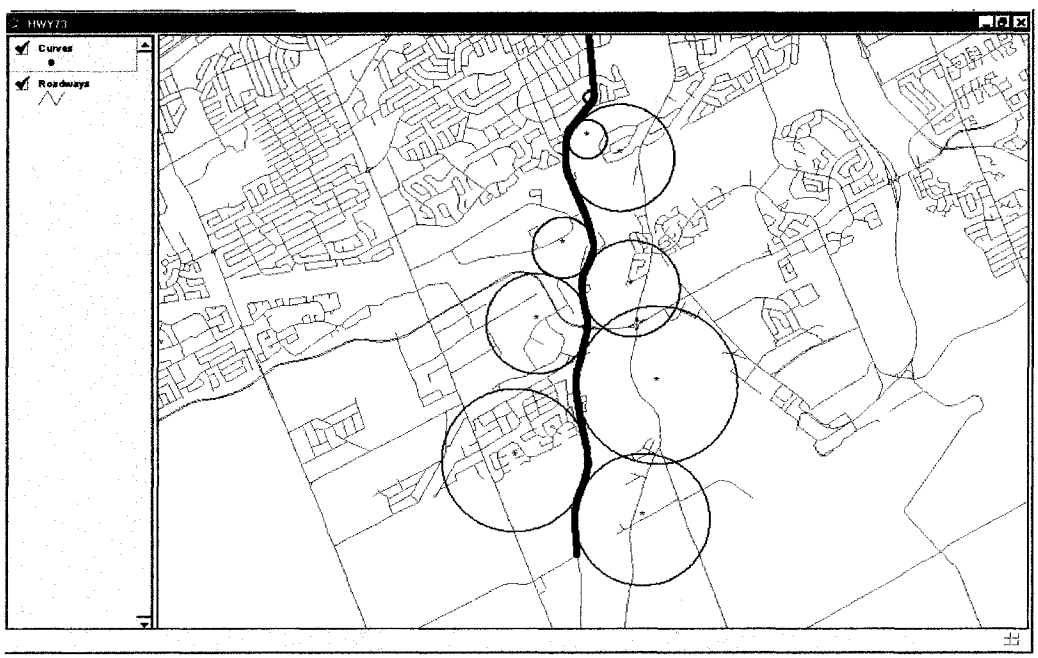


Figure 4-8: Study Curves on Regional Route 73 (Urban/Suburban Road).

Table 4-4: Study Curve Parameters on Urban/Suburban roads.

Study Curve	Road Type	Radius (m)	Circular Curve Length (m)	Entry Spiral Length (m)	Exit Spiral Length (m)
12C1	Urban/Suburban road	476	226	135	120
12C2	Urban/Suburban road	627	150	20	65
12C3	Urban/Suburban road	539	70	25	40
12C4	Urban/Suburban road	263	100	65	65
12C5	Urban/Suburban road	169	30	20	15
73C1	Urban/Suburban road	789	180	35	85
73C2	Urban/Suburban road	857	355	/	140
73C3	Urban/Suburban road	944	419	90	62
73C4	Urban/Suburban road	596	199	105	40
73C5	Urban/Suburban road	571	390	39	/
73C6	Urban/Suburban road	361	187	/	60
73C7	Urban/Suburban road	637	289	/	126
73C8	Urban/Suburban road	231	150	40	/
73C9	Urban/Suburban road	67	30	15	20

Based on the concept of the independent and non-independent curves explained earlier, 13 out of the 42 study curves were found to be non-independent curves with relatively short preceding tangents (i.e. < 200 m). Table 4-5 summarizes the information of these curves with their preceding tangent lengths.

Table 4-5: Non-independent Curve Parameters.

Curve ID	Road Type	Radius (m)	Circular Curve Length (m)	Spiral Length (m)	Preceding Tangent Length (m)
3C5	Two-lane Rural Highway	239	225	55	30
3C7	Two-lane Rural Highway	180	40	35	40
3C9	Two-lane Rural Highway	203	71	70	100
3C11	Two-lane Rural Highway	256	370	0	115
12C1	Urban/Suburban road	476	226	255	100
12C3	Urban/Suburban road	539	70	65	55
12C5	Urban/Suburban road	169	30	35	30
73C2	Urban/Suburban road	857	355	140	135
73C5	Urban/Suburban road	571	390	39	65
73C6	Urban/Suburban road	361	187	60	35
73C8	Urban/Suburban road	231	150	40	64
417C6	Urban Freeway	1,757	240	310	185
417C7	Urban Freeway	2,046	220	145	195

4.2 Quantification of Driver Speed Behaviour

4.2.1 Operating Speed and Speed Differential Parameters

As discussed in the literature review, driver behaviour can to a large extent be affected by the road geometric characteristics, particularly in the vicinity of horizontal curves. The driver speed behaviour can be described using the operating speed, normally represented by the 85th percentile value of speed distribution, and using the operating speed differential measured by different parameters. To quantify the driver speed behaviour, several operating speed and speed differential parameters were used in this research. These parameters can be explained as follows:

- V_{85_AT}: 85th percentile speed on the approach tangent. For study curves with a preceding tangent greater than 100 m, V_{85_AT} is the speed value at the point

AT+100 (i.e. 100 m before the beginning of the horizontal curve), except for one on Curve 3C11. The tangent speed was extracted at the point *AT+115* instead of *AT+100* (Table 4-5). For curves with an approach tangent less than 100 m, V_{85_SAT} is utilized, representing the operating speed at the start point of the approach tangent.

- V_{85_DT}: 85th percentile speed on the departure tangent. For curves with a departure tangent greater than 100 m, V_{85_DT} is the speed value at the point *DT+100* on the departure tangent (i.e.100 m after the end of the horizontal curve). For curves with a departure tangent less than 100 m, V_{85_EDT} is utilized, representing the operating speed at the end point of the departure tangent.
- $V_{85\_PC/SC}$: 85th percentile speed at the beginning of the circular curve.
- V_{85_MC}: 85th percentile speed at the midpoint of the circular curve.
- $V_{85\_CS/PT}$: 85th percentile speed at the end of the circular curve.
- ΔV_{85}: Operating speed differential calculated as the difference between V_{85_AT} and V_{85_MC}.
- $\Delta_{85}V_{(1\text{-}3)}$: Operating speed differential calculated as the 85th percentile value of individual speed differentials. Each individual speed differential in the distribution is taken as the difference between the speeds at the middle of the approach tangent and the middle of the circular curve for an individual driver.
- *85MSR*: Operating speed differential calculated as the 85th percentile value of individual maximum speed reductions. Each individual maximum speed reduction in the distribution is taken as the difference between the maximum

operating speed on the approach tangent and the minimum speed value within the curved section for an individual driver.

- *85MSI*: Operating speed differential calculated as the 85th percentile value of individual maximum speed increases. Each individual maximum speed increase in the distribution is taken as the difference between the maximum operating speed on the departure tangent and the minimum speed value within the curved section for an individual driver.

It should be noted that ΔV_{85} was obtained by simply subtracting the 85th percentile speeds at two locations. This procedure has been questioned by many researchers as it may underestimate the speed change from the tangent section into the curve. On the other hand, $\Delta_{85}V_{(1\text{-}3)}$ developed by Misaghi and Hassan (2005), and *85MSR* introduced by McFadden and Elefteriadou (2000), were calculated based on the individual speed profiles. In addition, *85MSI* is a new parameter introduced by the author, capable of explaining the speed change from the horizontal curve to the departure tangent. It was attained on the basis of the individual driver speed profiles, similar to $\Delta_{85}V_{(1\text{-}3)}$ and *85MSR*.

4.2.2 Quantifying Driver Speed Behaviour

The procedure to quantify driver speed behaviour was divided into two major steps, in terms of the extraction of speed data and the calculation of speed and speed differential measures. The first step of the procedure involved extracting speed data for all the sample drivers on each study curve from the speed database. The extracted speed data were then

exported to an Excel spreadsheet. The second step was to quantify driver speed behaviour through performing the following tasks.

- The speed profile for each driver was plotted individually. For each horizontal curve under investigation, plotting the individual speed profile was repeated for all test drivers (i.e. 30 drivers). The individual speed profiles on each study curve are presented in Appendix A.
- The 85th percentile speed value at each point along the study curve was calculated using the "Percentile" function in Excel.
- The operating speeds for points of interest, such as V_{85_AT}, V_{85_DT}, $V_{85\_PC/SC}$, $V_{85\_CS/PT}$ and V_{85_MC}, were extracted and compiled in an output table for the data analysis.
- The maximum speed value both on the approach and the departure tangents and the minimum speed value within the limits of the curved section were calculated for each individual driver on each individual study curve, using the "Maximum" and "Minimum" functions in Excel.
- Speed differential parameters were determined accordingly, using the results from the last step. For instance, the maximum speed reduction value for each driver was taken as the difference between the maximum speed on the approach tangent and the minimum speed on the curve. The *85MSR* for the whole driver population was then calculated using the "Percentile" function in Excel. The *85MSI* value was computed following the same procedure as for *85MSR*. The speed differential for each driver ($\Delta V_{(1\text{-}3)}$) was obtained by subtracting the speed values between two

points (i.e. *AT+100* or *SAT* and *MC*). The 85th percentile value ($\Delta_{85}V_{(1\text{-}3)}$) was calculated using the "Percentile" function in Excel.

A demo of quantifying driver speed behaviour is presented in the next section to demonstrate the procedure described above.

4.2.2.a Extracting Speed Data in Arcview 3.2a

To extract the speed data, the target curve should be highlighted in the View window, based on the stations corresponding to the range of the study curve. The attributes of the selected curve (e.g. operating speed, time stamp, and station) will be automatically highlighted in the attribute table of the curve under consideration. Then, the selected table (data) was exported to an Excel file in a comma-delimited file format. For the given example shown in Figure 4-9, the data for Driver 2 on Curve 3C10 from Highway 3 are highlighted. The range of the study curve is between station 18+822.11 and station 19+286.992, which corresponds to the section between *AT+200* m on the approach tangent and *DT+115* m on the departure tangent.

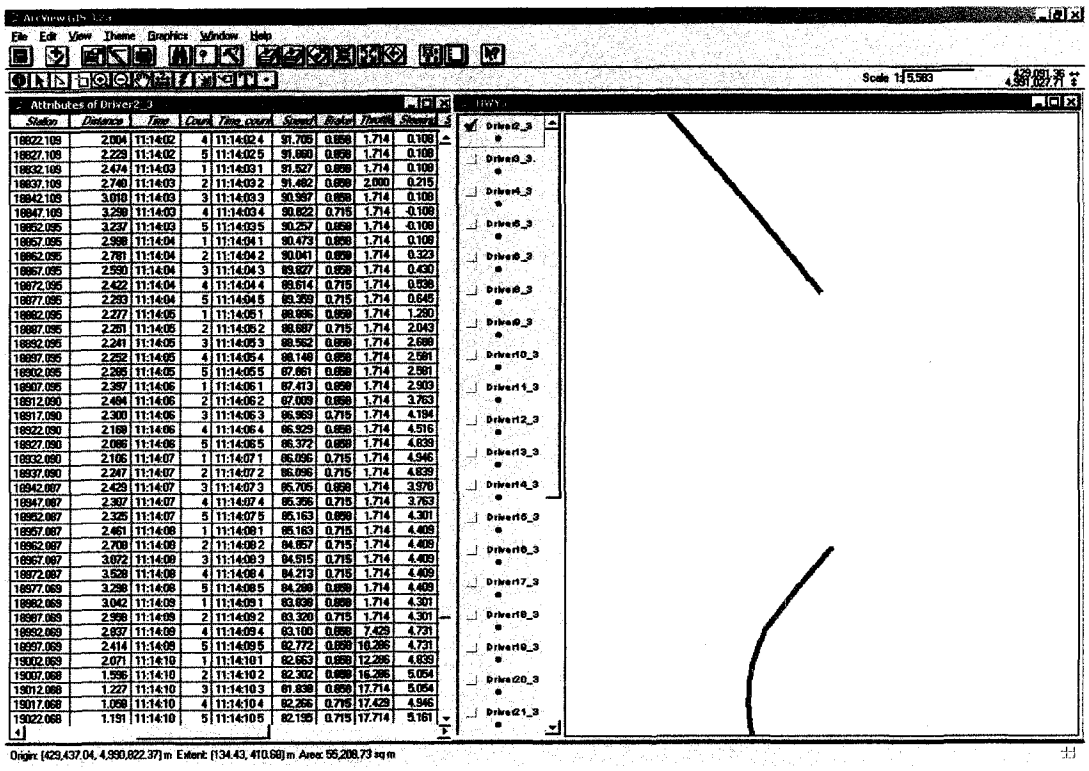
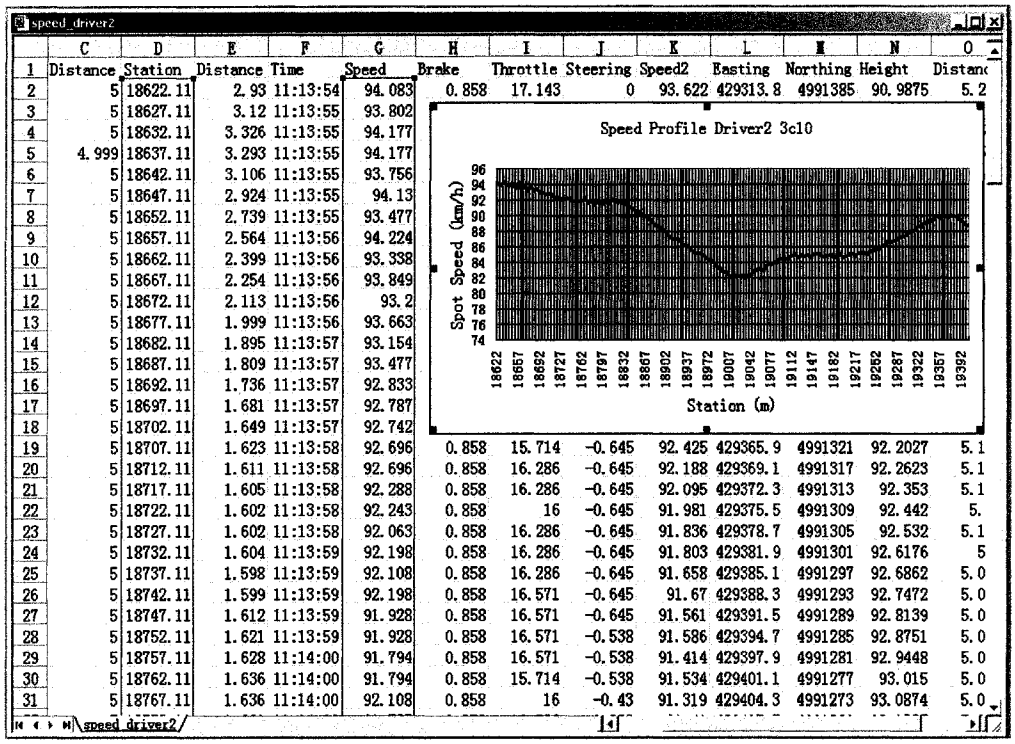


Figure 4-9: Example of Extracting Speed Data (Driver 2 on Curve 3C10).

4.2.2.b Individual Speed Profile

The individual driver's speed profile within the selected section of the study curve can be drawn in Excel, based on the extracted data, as shown in Figure 4-10.



	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Distance	Station	Distance	Time	Speed	Brake	Throttle	Steering	Speed2	Easting	Northing	Height	Distanc
2	5	18622.11	2.93	11:13:54	94.083	0.858	17.143	0	93.622	429313.8	4991385	90.9875	5.2
3	5	18627.11	3.12	11:13:55	93.802								
4	5	18632.11	3.326	11:13:55	94.177								
5	4.999	18637.11	3.293	11:13:55	94.177								
6	5	18642.11	3.106	11:13:55	93.756								
7	5	18647.11	2.924	11:13:55	94.13								
8	5	18652.11	2.739	11:13:55	93.477								
9	5	18657.11	2.564	11:13:56	94.224								
10	5	18662.11	2.399	11:13:56	93.338								
11	5	18667.11	2.254	11:13:56	93.849								
12	5	18672.11	2.113	11:13:56	93.2								
13	5	18677.11	1.999	11:13:56	93.663								
14	5	18682.11	1.895	11:13:57	93.154								
15	5	18687.11	1.809	11:13:57	93.477								
16	5	18692.11	1.736	11:13:57	92.833								
17	5	18697.11	1.681	11:13:57	92.787								
18	5	18702.11	1.649	11:13:57	92.742								
19	5	18707.11	1.623	11:13:58	92.696	0.858	15.714	-0.645	92.425	429365.9	4991321	92.2027	5.1
20	5	18712.11	1.611	11:13:58	92.696	0.858	16.286	-0.645	92.188	429369.1	4991317	92.2623	5.1
21	5	18717.11	1.605	11:13:58	92.288	0.858	16.286	-0.645	92.095	429372.3	4991313	92.353	5.1
22	5	18722.11	1.602	11:13:58	92.243	0.858	16	-0.645	91.981	429375.5	4991309	92.442	5.
23	5	18727.11	1.602	11:13:58	92.063	0.858	16.286	-0.645	91.836	429378.7	4991305	92.532	5.1
24	5	18732.11	1.604	11:13:59	92.198	0.858	16.286	-0.645	91.803	429381.9	4991301	92.6176	5
25	5	18737.11	1.598	11:13:59	92.108	0.858	16.286	-0.645	91.658	429385.1	4991297	92.6862	5.0
26	5	18742.11	1.599	11:13:59	92.198	0.858	16.571	-0.645	91.67	429388.3	4991293	92.7472	5.0
27	5	18747.11	1.612	11:13:59	91.928	0.858	16.571	-0.645	91.561	429391.5	4991289	92.8139	5.0
28	5	18752.11	1.621	11:13:59	91.928	0.858	16.571	-0.538	91.586	429394.7	4991285	92.8751	5.0
29	5	18757.11	1.628	11:14:00	91.794	0.858	16.571	-0.538	91.414	429397.9	4991281	92.9448	5.0
30	5	18762.11	1.636	11:14:00	91.794	0.858	15.714	-0.538	91.534	429401.1	4991277	93.015	5.0
31	5	18767.11	1.636	11:14:00	92.108	0.858	16	-0.43	91.319	429404.3	4991273	93.0874	5.0

Figure 4-10: Example of Individual Driver's Speed Profile (Driver 2 on Curve 3C10).

4.2.2.c Calculation of Speed and Speed Differential Parameters

The previous steps *4.2.2.a* and *4.2.2.b* need to be performed repeatedly until the speed profiles for all sample drivers are established. The extracted data for all the drivers are saved in the same file identified by the curve ID (i.e. Curve 3C10). Following the procedure explained earlier, operating speeds at the points of interest as well as speed differential parameters are attained. The calculation worksheets are illustrated in Figures 4-11 and 4-12. The final report for Curve 3C10 is presented in Table 4-6.

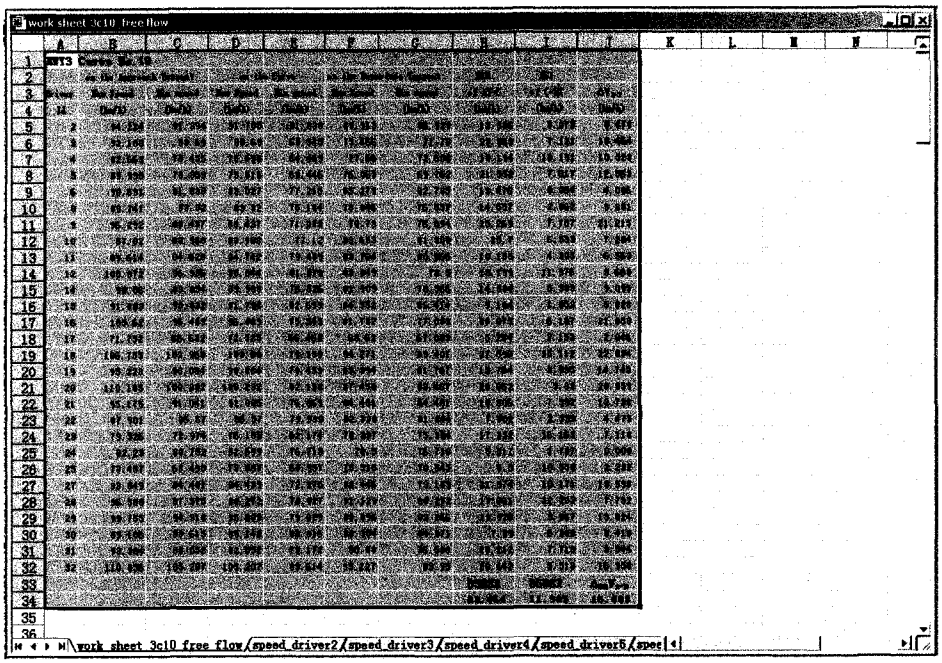


Figure 4-11: Calculation Worksheet for *85MSR*, *85MSI*, and $\Delta_{85}V_{(1\text{-}3)}$.

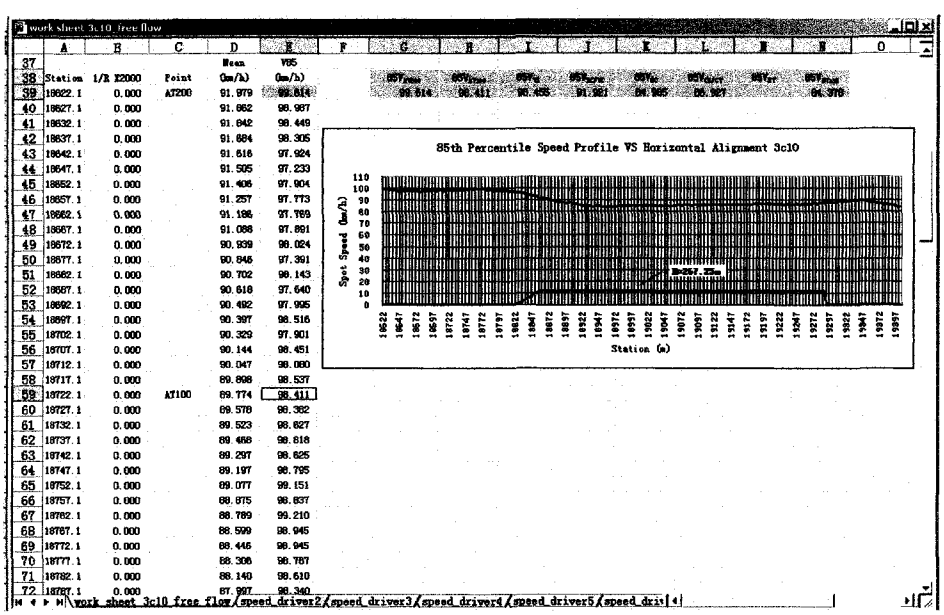


Figure 4-12: Calculation Worksheet for V_{85} and V_{85} Speed Profile.

Table 4-6: Final Report for Curve 3C10.

Curve ID	V_{85_AT} (km/h)	$V_{85\_PC/SC}$ (km/h)	V_{85_MC} (km/h)	$V_{85\_CS/PT}$ (km/h)	V_{85_EDT} (km/h)
3C10	98.411	91.921	84.985	86.927	84.378
	ΔV_{85} (km/h)	$\Delta_{85}V_{(1\text{-}3)}$ (km/h)	*85MSR* (km/h)	*85MSI* (km/h)	
	13.426	16.883	23.464	11.905	

4.3 Recognition of Free-Flow Conditions

In addition to highway geometric features, traffic flow is believed to play an important role affecting the driver speed choice on a roadway. Therefore, it was necessary to identify the cases where a specific speed could have been impeded by the prevailing traffic. As explained earlier, data generated by the front Lidar gun installed in the test vehicle were used to identify free-flow and non-free-flow speeds.

In accordance with previous research, identifying the free-flow condition was based on a 5-second time gap. The time gap is defined as the time elapsed for two successive vehicles passing a certain point under observation. Traffic having a time gap greater than 5 seconds is deemed as free flow. Time gap can be computed through dividing the distance between two successive vehicles as detected by the front Lidar gun by the operating speed of the trailing vehicle. During the experiment, the observer pressed the trigger of the Lidar gun when trailing a vehicle on the same lane, and the distance between the test vehicle and the impeding vehicle was recorded in meters. As the Lidar gun has a time stamp for each observation, the operating speed of the test vehicle with the same time reference can be identified in the speed database. The time gap was then calculated, and the calculation was performed in Excel environment. An example for the calculation of the time gap is given in Table 4-7. As shown in the table, the calculated time gaps suggest that the test vehicle

could have been affected by a front vehicle during the time period from 10:46:06 a.m. to 10:46: 39 a.m.

Table 4-7: Sample Calculation of the Time Gap (Driver 8 on Highway 7).

Distance (m)	Time Stamp	Speed (km/h)	Time gap (seconds)
84.98	10:46:06 AM	91.085	3.359
76.98	10:46:09 AM	91.394	3.032
65.27	10:46:13 AM	91.571	2.566
38.78	10:46:22 AM	91.616	1.524
N/A	10:46:26 AM	93.477	N/A
37.44	10:46:28 AM	93.477	1.442
N/A	10:46:34 AM	90.257	N/A
47.41	10:46:38 AM	86.214	1.980
49.02	10:46:39 AM	86.214	2.047

N/A: Distance readings from the Lidar gun are not available.

It should be noted that the accuracy of the range measurement could be reduced with the increase of the distance between two successive vehicles, when moving on a horizontal curve. The error mainly arises from the fact that the Lidar gun takes the chord distance between two vehicles instead of the arc distance on the vehicle's actual path. Nevertheless, this deficiency would not affect the accuracy of the Lidar gun readings in this study because the non free-flow condition only takes place when two successive vehicles are close enough for the chord distance to be very close to the actual spacing. The effort of identifying free-flow condition was applied to all the study curves on the test route, and the result is displayed in Table 4-8, including the total number of drivers experiencing traffic interruptions, and the driver ID whose speed was disturbed by the front traffic. As one expects, the number of drivers with speed interruptions increases when the test vehicle moved from the rural roads to the urban streets or freeways.

Table 4-8: Driver ID with Traffic Interruptions.

Road ID	Site ID	Numbers	Driver ID
3	3C1	2	5, 30
3	3C2	2	26, 30
3	3C4	2	13, 26
3	3C5	3	13, 15, 26
3	3C6	3	13, 15, 26
3	3C7	5	13, 15, 17, 26, 31
3	3C8	3	13, 15, 26
3	3C9	3	13, 15, 26
3	3C10	2	13, 26
3	3C11	3	13, 15, 26
6	6C1	9	5, 8, 18, 19, 26, 28, 30, 31, 32
7	7C1	14	3, 8, 9, 12, 13, 14, 18, 19, 20, 22, 25, 27, 28, 30
416	416C1	17	3, 5, 10, 11, 12, 13, 15, 18, 20, 21, 22, 24, 25, 26, 29, 31, 32
416	416C2	10	3, 5, 8, 11, 12, 14, 15, 20, 22, 31
416	416C3	16	6, 9, 12, 14, 15, 16, 17, 18, 19, 20, 22, 24, 26, 27, 29, 31
416	416C4	19	5, 6, 8, 9, 10, 13, 14, 15, 16, 17, 18, 19, 20, 22, 24, 27, 28, 29, 32
12	12C1	15	3, 6, 8, 9, 11, 14, 15, 16, 18, 19, 21, 23, 26, 31, 32
12	12C2	18	3, 9, 10, 11, 12, 13, 14, 15, 16, 17, 20, 21, 23, 25, 26, 28, 29, 31
12	12C3	19	3, 5, 6, 10, 11, 12, 13, 14, 15, 17, 20, 21, 23, 24, 25, 26, 28, 29, 32
12	12C4	16	4, 5, 6, 10, 11, 12, 14, 17, 18, 20, 21, 24, 26, 27, 28, 29
12	12C5	17	4, 6, 9, 11, 12, 14, 15, 17, 18, 21, 23, 24, 25, 26, 27, 28, 29
73	73C1	24	3, 4, 5, 6, 8, 9, 10, 11, 12, 13, 15, 17, 18, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 32
73	73C2	18	3, 4, 6, 8, 9, 11, 13, 15, 17, 18, 20, 21, 23, 26, 27, 28, 29, 32
73	73C3	20	4, 5, 8, 9, 11, 12, 13, 15, 17, 18, 20, 22, 23, 24, 26, 27, 28, 29, 31, 32
73	73C4	24	4, 5, 6, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 32
73	73C5	22	3, 4, 5, 6, 8, 9, 12, 13, 14, 15, 16, 17, 20, 21, 22, 23, 25, 26, 27, 28, 29, 32
73	73C6	20	3, 4, 5, 8, 9, 12, 13, 14, 15, 16, 17, 20, 21, 22, 23, 25, 26, 27, 28, 32
73	73C7	15	3, 4, 6, 8, 9, 11, 12, 13, 14, 15, 19, 20, 26, 27, 32
73	73C8	14	4, 6, 8, 9, 12, 13, 14, 15, 20, 23, 25, 26, 27, 32
73	73C9	16	6, 8, 9, 12, 13, 14, 15, 18, 20, 23, 25, 26, 27, 28, 31, 32
417	417C1	20	2, 4, 5, 6, 9, 11, 13, 14, 15, 16, 19, 21, 22, 24, 25, 26, 28, 29, 30, 32
417	417C2	22	2, 4, 6, 8, 9, 10, 11, 12, 13, 14, 15, 16, 19, 21, 22, 24, 25, 26, 28, 29, 30, 32
417	417C3	20	3, 4, 6, 8, 11, 12, 14, 15, 16, 17, 19, 21, 22, 23, 24, 25, 26, 29, 30, 32

Table 4-8: Driver ID with Traffic Interruptions (Cont'd).

417	417C4	25	3, 4, 5, 6, 8, 9, 10, 11, 12, 13, 14, 15, 16, 18, 19, 21, 22, 23, 24, 25, 26, 28, 29, 30, 32
417	417C5	17	3, 4, 5, 6, 8, 11, 13, 14, 16, 18, 21, 23, 24, 25, 26, 30, 32
417	417C6	20	3, 4, 5, 8, 10, 11, 12, 13, 14, 16, 18, 19, 21, 23, 24, 26, 28, 29, 30, 32
417	417C7	20	3, 4, 5, 6, 8, 11, 12, 13, 14, 16, 17, 18, 19, 21, 24, 26, 27, 28, 29, 32
417	417C8	21	3, 4, 5, 6, 9, 11, 12, 13, 14, 15, 17, 18, 19, 21, 23, 24, 25, 26, 28, 29, 32
417	417C9	21	3, 4, 6, 8, 10, 11, 12, 14, 17, 18, 19, 20, 21, 22, 23, 25, 27, 28, 29, 30, 32
417	417C10	25	3, 4, 5, 8, 9, 10, 11, 12, 13, 14, 15, 16, 18, 19, 20, 21, 22, 24, 25, 26, 27, 28, 29, 30, 32
417	417C11	21	3, 4, 5, 8, 9, 10, 11, 13, 15, 16, 19, 20, 22, 24, 25, 26, 27, 28, 29, 30, 32
417	417C11	21	3, 4, 5, 8, 9, 10, 11, 13, 15, 16, 19, 20, 22, 24, 25, 26, 27, 28, 29, 30, 32

4.4 Impact of the Presence of Intersections

In the highway network, various traffic control devices; such as traffic signals, stop signs, and yield signs; are used to guide and regulate the traffic to ensure safe and efficient operation at the intersections. The roadway traffic will have to respond to the presence of a traffic control device, and subsequently traffic control devices have vast impacts on the driver speed selection. For instance, an at-grade intersection with an all-way stop sign located right after the departure tangent would force the driver to bring the vehicle to a full stop. Thus, the normal speed trend (i.e. speed increase) on the departure tangent as it appears on the two-lane rural highway will be violated.

As drivers would behave differently when subject to the presence of intersections with traffic control devices, there is a possibility to recognize their impacts on driver speed behaviour by examining the driver speed profiles. Intersections that are located in the vicinity of the study site can be recognized from the digitized maps of the Ottawa Roadway

Networks provided by the City of Ottawa, and they were confirmed by the field survey and the videos recorded during the test runs. Study sites with the presence of intersections, associated with the traffic control type are summarized in Table 4-9.

Table 4-9: Study Curves with the Presence of Intersections.

Road ID	Site ID	Location of the Intersection	Control Type
Regional Route 3	3C1	Intersection with Provincial Route 7, before the approach tangent	Traffic Signal
Regional Route 3	3C2	Intersection with Crawford Side Road, in the curved section	All-way Stop Sign
Regional Route 3	3C3	Intersection with Frank Town road, on the approach tangent	All-way Stop Sign
Regional Route 12	12C1	Intersection with Strandherd Drive, on the approach tangent	Traffic Signal
Regional Route 12	12C2	Intersection with Woodroffe Avenue, on the approach tangent	Traffic Signal
Regional Route 12	12C5	Intersection with Merivale Road, on the departure tangent	Traffic Signal
Regional Route 73	73C3	Intersection with Deakin Street, on the exit spiral	Traffic Signal
Regional Route 73	73C4	Intersection with Huntclub Road, on the entry spiral	Traffic Signal

To demonstrate the impact of the presence of intersections associated with different types of traffic control devices on driver speed behaviour, the individual speed profiles for the test drivers were drawn at each of the above sites. At a signalized intersection, the driver sample was divided into two groups related to different speed trends on the basis of whether they were influenced by the signals. Figures 4-13 and 4-14 demonstrate the effects of a stop sign on driver speed behaviour at an intersection located before and on the

approach tangent, respectively. Two completely different speed trends can be found in Figures 4-15 and 4-16. The red solid line represents the 85th percentile speed at which drivers passed the intersection without the disturbance of traffic signals, whereas the blue dash line refers to the 85th percentile speed for those who experienced the interruption of traffic signals.

As shown in the figures, the speed trends vary greatly, depending upon the location of the intersection (i.e. before, within, or after the study site). It is also learned from the illustrations that the influence of traffic control devices (i.e. signals or stop signs) on driver speed behaviour could be extremely high. Therefore, speed observations with the interruptions by traffic signals and stop signs were removed before the data analysis was carried out, as they are not necessarily related to the highway geometric features.

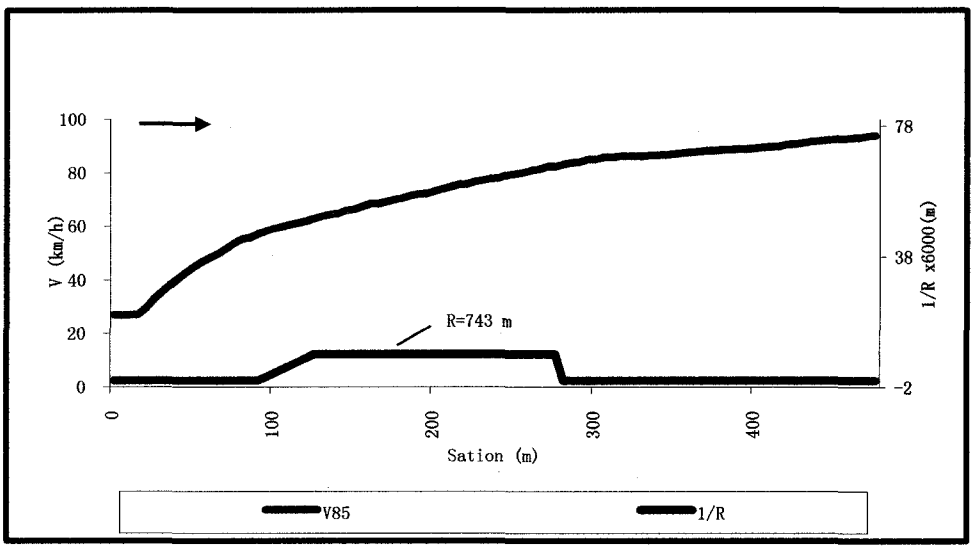


Figure 4-13: Speed Profile on Curve 3C1 (Stop Sign before the Approach Tangent).

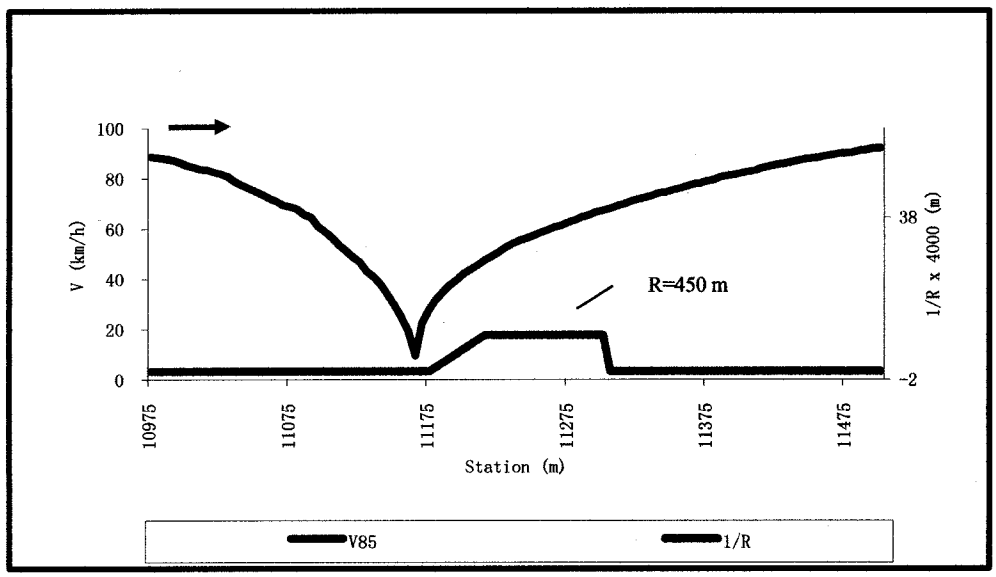


Figure 4-14: Speed Profile on Curve3C3 (Stop Sign on the Approach Tangent).

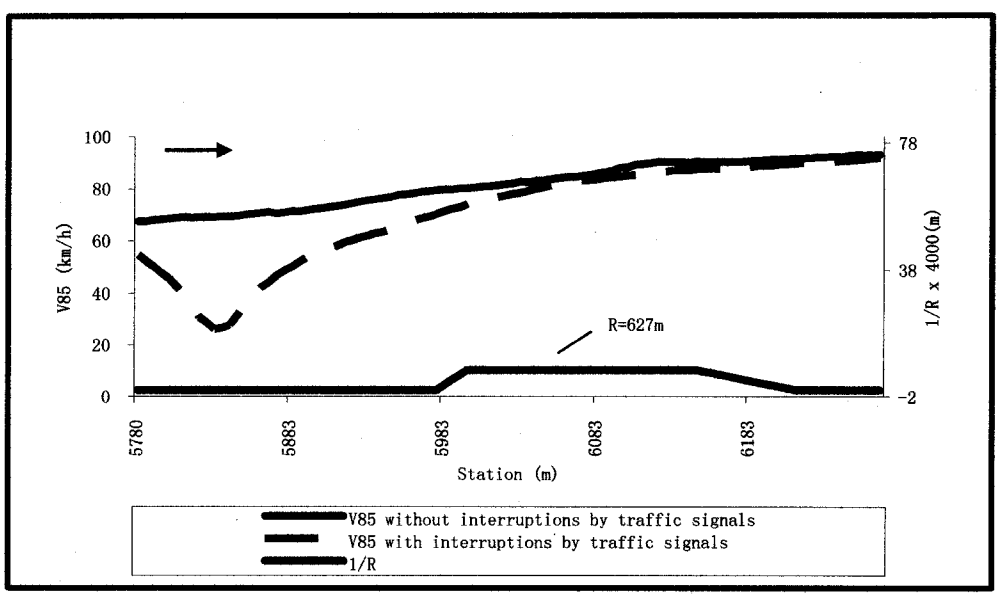


Figure 4-15: Speed Profile on Curve 12C2 (Traffic Signal on the Approach Tangent).

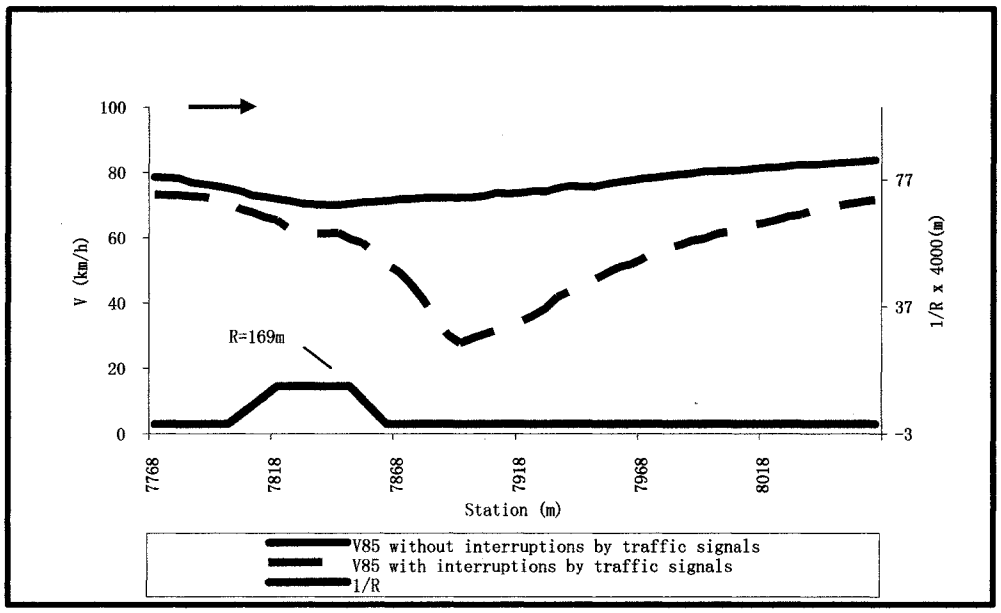


Figure 4-16: Speed Profile on Curve 12C5 (Traffic Signal on the Departure Tangent).

CHAPTER 5: MODELLING OPERATING SPEEDS AND SPEED DIFFERENTIALS

5.1 Introduction

On the basis of the work described so far, the major task of this chapter is to develop operating speed and speed differential models corresponding to different road classes, using actual driving data. This chapter contains three sections corresponding to each road class under investigation, which are two-lane rural highways, urban/suburban roads, and freeways (both rural and urban), respectively.

The development of operating speed and speed differential prediction models for each road class involved establishing a statistical relationship between the proposed dependent and independent variables through a regression analysis, using SPSS v11.5. The regression analysis was based on a common assumption that there is a generally linear relationship between the responder and predictors. Then, the developed models were subjected to model diagnoses, to justify the aforesaid hypothesis. SPSS v11.5 is a statistical package for social science, capable of handling the multiple regression analysis. In this study, the stepwise regression technique was used in the SPSS software so that those independent variables, which are not statistically significant at 95 percent confidence level, can be automatically removed from the models.

The dependent variables were defined, in this research, as operating speeds and speed differentials. The operating speed variables refer to the 85th percentile speeds at particular points along the study curve. The speed differential variables were quantified by measures:

ΔV_{85}, $\Delta_{85}V_{(1\text{-}3)}$, *85MSR*, and *85MSI*. For easy reference, all dependent variables used in this study are summarized as follows:

- V_{85_AT}: 85th percentile speed on the approach tangent (V_{85_AT+100} for the curve with a preceding tangent greater than 100 m, while V_{85_SAT} for the curve with a preceding tangent less than 100 m).
- V_{85_DT}: 85th percentile speed on the departure tangent (V_{85_DT+100} for the curve with a succeeding tangent length greater than 100 m, while V_{85_EDT} for the curve with a succeeding tangent less than 100 m).
- $V_{85\_PC/SC}$: 85th percentile speed at the beginning of the circular curve.
- V_{85_MC}: 85th percentile speed at the middle of the circular curve.
- $V_{85\_CS/PT}$: 85th percentile speed at the end of the circular curve.
- ΔV_{85}: Difference between 85th percentile speeds on the middle of the approach tangent and the middle of the curve.
- $\Delta_{85}V_{(1\text{-}3)}$: 85th percentile speed difference between the approach tangent and the middle of the curve.
- *85MSR*: 85th percentile value of the maximum speed reduction from the approach tangent to the curve.
- *85MSI*: 85th percentile value of the maximum speed increase from the curve to the departure tangent.

For the independent variables, horizontal alignment parameters were employed for the model development. The posted speed was also considered as an independent variable for predicting operating speeds and speed differentials on urban/suburban roads. In addition, a dummy variable was used to indicate the presence of an intersection in the vicinity of the study curve. The proposed predictors are listed below.

- r, $(1/r)$, $Ln(r)$, and r^2: Curve radius and its transformation.
- L_{AT}: Approach tangent length.
- L_{DT}: Departure tangent length.
- L_c: Circular curve length.
- L_{s1}: Entry spiral length.
- L_{s2}: Exit spiral length.
- $L = L_c + L_{s1} + L_{s2}$: Total curve length.
- L_c/r: Circular curve deflection angle.
- $(L_c + L_{s1}/2 + L_{s2}/2)/r$: Curve deflection angle.
- r_1/r_2: Ratio of radius between adjacent curves, where r_1 is the radius of the preceding curve and r_2 is the radius of curve under consideration..
- $CCRs = \dfrac{63700\ (Ls_1/2r + Lc/r + Ls_2/2r)}{Ls_1 + Lc + Ls_2}$: Curvature change rate.
- INT: A dummy variable that is equal to 1 if there is an intersection, otherwise 0.
- V_P: Posted speed

As mentioned earlier, the curve radius, the circular curve length, and the spiral lengths were extracted using the RoadFit Extension in Arcview 3.2a. Other independent variables related to curve parameters were computed accordingly in Excel for all the study curves.

5.2 Two-Lane Rural Highways

Provincial Route 7 and Regional Routes 3 and 6 on the selected test route are typical two-lane rural highways with a posted speed of 80 km/h. Site surveys were carried out to collect additional information such as adjacent land use, shoulder types, as well as pavement conditions for each study site. The rationale behind this effort is to confirm that the selected sites have homogenous physical conditions, so that the driver speed choice is affected by horizontal alignments alone.

Pavement conditions in the vicinity of study sites were considered as fair level since no major distresses such as potholes, rutting, or ravelling were evident in the pavement surface. The information with respect to physical features of each study site is presented in Table 5-1. The information of the presence of intersections is also included in the table. As mentioned earlier, the presence of intersections in the vicinity of the study sites was recognized with digitized maps provided by the City of Ottawa.

Table 5-1: Physical Features of Study Curves (Two-Lane Rural Highways).

Curve ID	Presence of Intersections	Posted Speed (km/h)	Pavement Condition	Shoulder Type	Adjacent Land use	Lane and Shoulder Width
3C1	√ (Signal)	80	Fair	Un-paved	Rural	Uniform
3C2	√ (All-way Stop Sign)	80	Fair	Un-paved	Rural	Uniform
3C3	√ (All-way Stop Sign)	80	Fair	Un-paved	Rural	Uniform
3C4	None	80	Fair	Un-paved	Rural	Uniform
3C5	None	80	Fair	Un-paved	Rural	Uniform
3C6	None	80	Fair	Un-paved	Rural	Uniform
3C7	None	80	Fair	Un-paved	Rural	Uniform
3C8	None	80	Fair	Un-paved	Rural	Uniform
3C9	None	80	Fair	Un-paved	Rural	Uniform
3C10	None	80	Fair	Un-paved	Rural	Uniform
3C11	None	80	Fair	Un-paved	Rural	Uniform
6C1	None	80	Fair	Un-paved	Rural	Uniform
7C1	None	80	Fair	Paved	Rural	Uniform

As shown in the above table, among the thirteen horizontal curves available on two-lane rural highways, Curves 3C1, 3C2, and 3C3 are located near intersections with a traffic light or stop signs. The other study curves are free from the presence of intersections. Due to the limited number of the study curves with an intersection in the vicinity, study curves 3C1, 3C2, and 3C3 were removed from the database prepared for the speed model development. On the other hand, the posted speed was not considered for model development on two-lane rural highways, as all the curves involved have a constant posted speed. Speed and speed differential parameters for the remaining 10 curves under free-flow conditions are summarized in Table 5-2.

Table 5-2: Operating Speed and Speed Differential Parameters (Two-Lane Rural Highways).

Curve ID	V_{85_AT} (km/h)	$V_{85\_PC/SC}$ (km/h)	V_{85_MC} (km/h)	$V_{85\_CS/PT}$ (km/h)	V_{85_DT} (km/h)	ΔV_{85} (km/h)	$\Delta_{85}V_{(1\text{-}3)}$ (km/h)	*85MSR* (km/h)	*85MSI* (km/h)	No of Observations
3C4	98.9	90.8	85.5	81.2	79.5	13.5	18.7	29.5	1.4	28
3C5	81.0	79.7	82.3	86.5	90.5	-5.4	-0.2	4.2	18.8	27
3C6	92.7	86.7	88.6	88.1	86.3	4.0	9.1	14.7	4.3	27
3C7	88.2	86.5	86.2	86.5	91.8	1.5	3.6	3.9	13.8	25
3C8	101.1	96.5	94.3	95.2	88.6	6.8	5.5	11.3	2.9	27
3C9	95.2	87.1	85.9	85.2	91.1	9.3	9.5	10.4	12.8	27
3C10	98.4	91.9	85.0	86.9	84.4	13.4	16.9	23.5	11.9	28
3C11	87.1	85.3	83.6	88.5	93.3	3.4	7.4	11.8	17.2	27
6C1	97.8	98.2	98.5	99.0	99.7	-0.7	2.3	9.7	18.5	21
7C1	104.8	105.6	105.7	105.5	107.2	-0.9	2.8	7.7	10.3	16
Average	94.5	90.8	89.6	90.3	91.2	4.5	7.6	12.7	11.2	25

In Table 5-2, a speed differential parameter with a negative sign represents a speed increase, at or above which 85 percent of driver populations experienced from the approach tangent into the middle of the curve. Some general speed trends can be obtained when taking a close look at the speed data in Table 5.2 and the individual speed profiles in Appendix A. They are described as follows:

- Drivers generally reduce their speeds from the approach tangent into the horizontal curve, and then start to accelerate at a certain point within the limits of the horizontal curve. Drivers are most likely to increase their speeds when departing the horizontal curve. The observed operating speed variations on the curve suggest that drivers do not maintain a constant speed within the limits of a horizontal curve.
- The points of the end of deceleration and the beginning of the acceleration on the curve vary among the study curves. The minimum operating speed does not necessarily take place at the midpoint of the circular curve.
- ΔV_{85} yields the lowest speed reduction, compared to *85MSR* or $\Delta_{85}V_{(1\text{-}3)}$. It is therefore once again proven to underestimate the speed changes while entering a horizontal curve from the approach tangent.
- A considerable disparity exists between *85MSR* and $\Delta_{85}V_{(1\text{-}3)}$, suggesting that the parameter $\Delta_{85}V_{(1\text{-}3)}$ may sometimes underestimate the speed differential from the tangent to the curve. This could happen when the minimum speed occurs after the midpoint of the circular curve, beyond which $\Delta_{85}V_{(1\text{-}3)}$ accounts for.

- A speed increase instead of a decrease was observed from the approach tangent into the horizontal curve on three sites. At the non-independent curve 3C5, despite a very small curve radius, along with a relatively high curve deflection angle, ΔV_{85} yields a speed increase of 5.4 km/h. One of the possible reasons is that drivers had overestimated the severity of the previous curve (Curve 3C4), resulting in an excessive speed reduction, and started to accelerate when realizing the overestimation. This is confirmed by the speed reduction values on Curve 3C4, where very high speed reductions, such as 13.5 km/h for ΔV_{85}, and 29.5 km/h for *85MSR*, were observed. Two other sites are independent curves with moderate to large curve radii, where speed increases from the approach tangent into the curve were observed. This suggests that flatter horizontal curves would encourage drivers to accelerate from the approach tangent to the curve.

After entering the speed and speed differential data, together with the proposed independent variables into SPSS v11.5, several base models that are significant at 95% confidence level were created. They are presented in Appendix B, along with the details of the regression results.

The next step then moved to model diagnosis for the purpose of justifying the underlying assumption of "general linear model". According to William and Terry (1996), the validity of the inferences associated with a regression analysis depends on the error term ε, satisfying certain assumptions: ε is normally distributed with a mean of 0, the variance σ^2 is constant, and all pairs of error terms are uncorrelated. A graphical display of regression residual was plotted in SPSS v.11.5, to validate the developed models. The Standardized regression residual was plotted on the vertical axis against the standardized predicted value

(dependent variable as a linear function of independent variables) on the horizontal axis. The test results showed that the assumptions concerning the error term ε were satisfied for all the models. An example of regression residual versus predicted value is illustrated in Figure 5-1. As one expects, the scatter points have no trends, no dramatic increases or decreases in variability and spread symmetrically about the line passing through the zero value of standardized residual, which is parallel to the horizontal axis.

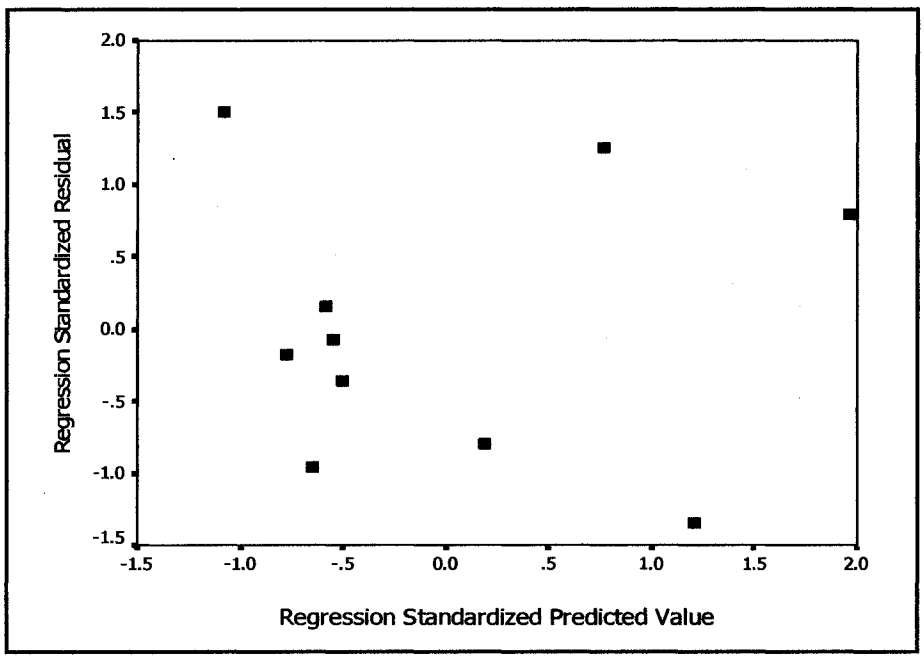


Figure 5-1: Example of Model Diagnoses (Model 3 in Table 5-3).

Referring to Table B-1 in the Appendix, Model 12 for $\Delta_{85}V_{(1\text{-}3)}$ looks illogical, since the curve radius or its transformations, which is believed to have great influences on driver speed selection when approaching the horizontal curve, is missing. Besides, no statistical relationship between ΔV_{85} and any of the proposed independent variables was found. Looking back at the database for the model development, it was found to consist of two

types of curves. Four out of the ten curves are non-independent curves (See Table 4-5), which have short preceding tangent lengths, whereas others are independent curves with relatively long preceding tangents. To improve the model development, it was decided to remove the four non-independent curves from the database. This effort produced a positive result. Five models for operating speeds and four models for speed differentials were developed exclusively for independent curves, which are shown in Table B-2. Afterwards, the developed models were subjected to diagnosis following the same procedure as for the models for all curves. Again, all the developed models for independent curves on two-lane rural highways passed the test.

Several issues regarding the developed base models need to be addressed for the purpose of recommending those that best represent the relationship between the dependent and independent variables for practical uses.

First, among the models for all curves on two-lane rural highways, Model 3 for the point of curve (*PC/SC*) is preferable as curvature change rate (CCR_s), which considers the transition of the curve, has been included in the model. Model 8 should be recommended for the point of *CS/PT* because it has the highest value of adjusted coefficient of determination (R^2), and the lowest standard error of the estimate (*SEE*). On the departure tangent, Model 9 shows a low correlation between the estimated speed and the predictor, and thus should be discarded.

Secondly, as shown in Table B-2, the development of the operating speed model on the approach tangent was not successful for the independent curves. This should have been expected since the only significant variable was the length of the approach tangent which should have no major effect on the tangent speed on independent tangents alone.

Based on the above discussion, those models which are believed to be advantageous to the practical use are recommended, as shown in Tables 5-3 through 5-6. The details of the regression results of these models are available in Appendix B.

Table 5-3: Operating Speed Models for All Curves (Two-Lane Rural Highways).

No	Model	df	Adjusted R^2	SEE
1	$V_{85\_AT} = 81.782 + 0.086L_{AT}$	8	0.661	4.24
2	$V_{85\_PC/SC} = 108.132 - 0.090CCR_s$	8	0.714	4.03
3	$V_{85\_MC} = 108.357 - 0.097CCR_s$	8	0.860	2.83
4	$V_{85\_CS/PT} = 102.238 - 0.092CCR_s + 0.039L_{DT}$	7	0.938	1.83
5	$V_{85\_DT} = 78.690 + 0.00001127r^2 + 0.066L_{DT}$	7	0.857	2.95

df: Degree of freedom

Adjusted R^2: Adjusted coefficient of determination

SEE: Standard error of estimate

Table 5-4: Speed Differential Models for All Curves (Two-Lane Rural Highways).

No	Model	df	Adjusted R^2	SEE
6	$85MSR = 17.857 - 0.080L_{DT} + 7.324DFC$	7	0.729	4.23
7	$85MSI = -0.410 + 0.078L_{DT}$	8	0.715	3.42

Table 5-5: Operating Speed Models for Independent Curves (Two-Lane Rural Highways).

No	Model	df	Adjusted R^2	SEE
8	$V_{85\_PC/SC} = 30.563 + 10.582Ln(r)$	4	0.776	3.14
9	$V_{85\_MC} = 110.386 - 6856.213(1/r)$	4	0.908	2.48
10	$V_{85\_CS/PT} = 111.404 - 7360.698(1/r)$	4	0.868	3.24
11	$V_{85\_DT} = 76.486 + 0.127L_{DT}$	4	0.767	5.03

Table 5-6: Speed Differential Models for Independent Curves (Two-Lane Rural Highways).

No	Model	df	Adjusted R^2	SEE
12	$\Delta V_{85} = -5.326 + 0.073CCR_s$	4	0.574	4.22
13	$\Delta_{85}V_{(1-3)} = -4.540 + 0.088CCR_s$	4	0.749	3.56
14	$85MSR = -0.658 + 0.107CCR_s$	4	0.760	4.22

The last step for the model development is the model interpretation, explaining the mathematical models with engineering terms. The following is provided for the recommended models listed in Tables 5-3 through 5-6.

- The operating speed value on the approach tangent is highly correlated with the approach tangent length. For independent curves, no relationship was found between the approach tangent speed and the geometric features.
- The operating speed on the curved section was found to have a statistically significant relationship with the curvature change rate (CCR_s). With the increase of

the curvature change rate, the speed on the curve decreases. In addition, the operating speed at the end of circular curves is also affected by the departure tangent length (L_{DT}). The longer the departure tangent length, the higher the operating speed. For independent curves, operating speeds on the curved section are correlated to curve parameters *Ln(r)* or *1/r*.

- The magnitude of the operating speed on the departure tangent depends on the square of the radius (r^2) and L_{DT}. Model 5 indicates that while traveling on a generous curve, followed by a long departure tangent, drivers will likely choose a higher operating speed. The departure tangent speed for the independent curve is positively proportional to the length of the departure length (L_{DT}).
- The speed reduction represented by *85MSR* was found to be dependent on L_{DT} and curve deflection angle (*DFC*). Obviously, a sharp curve will result in a greater speed reduction from the approach tangent into the curved section. The appearance of the variable departure tangent length in the model may explain the situation where drivers encounter continuous curves separated by a short tangent. When the length of this common tangent decreases, the speed reduction will increase accordingly from the approach tangent into the curve. This model implies that drivers would be more cautious to negotiate continuous curves than they do on isolated curves by reducing a large amount of the vehicle speed. For curves preceded by a long tangent, the speed differentials represented by ΔV_{85}, $\Delta_{85}V_{(1\text{-}3)}$, and *85MSR* are sensitive to the curvature change rate (CCR_s) without exceptions.
- The maximum speed increase (*85MSI*) from the curve to the departure tangent depends on the departure tangent length, as reflected in Model 7.

5.3 Urban/Suburban Roads

Regional Routes 12 (Fallowfield Road) and 73 (Prince of Wales Drive) are typical two-lane roadways located near densely populated areas. The posted speed (V_p) and presence of intersections (*INT*) were believed to play significant roles in influencing driving performances, in addition to roadway alignments. Therefore, it was reasonable to consider these factors when studying driver speed behaviour on urban/suburban roads. There are fourteen horizontal curves available on urban/suburban roads under investigation. Five of these curves are located on Regional Route 12, while the remaining nine curves belong to Regional Route 73. The physical features for the study sites are summarized in Table 5-7.

Table 5-7: Physical Features of Study Curves (Urban/Suburban Roads).

Curve ID	Presence of Intersections	Posted Speed (km/h)	Pavement Condition	Shoulder Type	Adjacent Land Use	Lane Width
12C1	√ (Signal)	80	Fair	Un-paved	Urban/ Suburban	Uniform
12C2	√ (Signal)	80	Fair	Un-paved	Urban/ Suburban	Uniform
12C3	None	80	Fair	Un-paved	Urban/ Suburban	Uniform
12C4	None	80	Fair	Un-paved	Urban/ Suburban	Uniform
12C5	√ (Signal)	80	Fair	Un-paved	Urban/ Suburban	Uniform
73C1	None	80	Fair	Un-paved	Urban/ Suburban	Uniform
73C2	None	80	Fair	Un-paved	Urban/ Suburban	Uniform
73C3	√ (Signal)	80	Fair	Un-paved	Urban/ Suburban	Uniform
73C4	√ (Signal)	60	Fair	paved	Urban/ Suburban	Uniform
73C5	None	60	Fair	paved	Urban/ Suburban	Uniform
73C6	None	60	Fair	paved	Urban/ Suburban	Uniform
73C7	None	60	Fair	paved	Urban/ Suburban	Uniform
73C8	None	60	Fair	paved	Urban/ Suburban	Uniform
73C9	None	60	Fair	Paved	Urban/ Suburban	Uniform

As shown in the above table, Curves 12C1, 12C2, 12C5, 73C3, and 73C4 are located in the vicinity of intersections with traffic lights. It should be noted that seven out of the fourteen curves are non-independent curves (See Table 4-5). Nevertheless, it was decided to combine independent and non-independent curves in one database for the development of analytical models. This is mainly because that successive curves separated by a short tangent are commonly used on urban/suburban roads. In addition, as urban/suburban roads serve traffic volumes with low to moderate speeds, the effect of different types of horizontal curves (i.e. independent and non-independent curves) on driver behaviour may not be as significant as that for two-lane rural highways. Furthermore, as explained in Chapter 4, the effect of traffic signals on driver speed behaviour has been demonstrated to be enormous. Hence, speed observations with interruptions by traffic signals were excluded as traffic control devices are not the factors relevant to the interest of this research.

Differing from the data analysis for two-lane rural highways, the posted speed (V_p) and presence of intersections (*INT*), as previously explained, were added to the database as independent variables. In addition, the prevailing traffic condition was believed to have impacts on driver speed selection on urban/suburban roads. Thus, driver behaviour could be influenced by more factors than roadway geometric features alone. Therefore, it is wise to accommodate the aforesaid factors when examining driver speed behaviour on urban and suburban roads. Specifically, analyzing driver speed behaviour and developing speed models for urban/suburban roads were conducted in correspondence with three specific cases described below:

- Case 1: Only free-flow speeds were used with the consideration of intersections.
- Case 2: All speed observations were used except for those disturbed by traffic signals, with the consideration of intersections.
- Case 3: All speed observations were used except for those disturbed by traffic signals, without considering the presence of intersections.

Table 5-8 presents the speed and speed differential measures under free-flow conditions, while Table 5-9 contains the measures combining free-flow speeds with non-free-flow speeds observed on each study curve.

Table 5-8: Free-Flow Speed Measures on Urban/Suburban Roads.

Curve ID	V_{85_AT} (km/h)	$V_{85\_PC/SC}$ (km/h)	V_{85_MC} (km/h)	$V_{85\_CS/PT}$ (km/h)	V_{85_DT} (km/h)	ΔV_{85} (km/h)	$\Delta_{85}V_{(1-3)}$ (km/h)	*85MSR* (km/h)	*85MSI* (km/h)	No of Observations
12C1	30.4	79.8	86.5	85.7	92.7	-56.1	-40.3	0.0	29.7	15
12C2	70.8	80.2	84.8	90.3	92.8	-14	-8.4	0.0	14.5	6
12C3	93.0	94.8	95.1	95.7	95.5	-6.7	0.6	1.5	8.8	11
12C4	87.1	82.3	81.3	79.9	73.5	5.7	9.9	23.5	0.11	14
12C5	78.5	71.7	70.0	70.2	77.3	8.5	7.5	13.3	17.6	6
73C1	80.0	80.0	82.8	85.1	87.0	-2.8	3.2	5.4	10.3	6
73C2	84.7	85.5	86.8	85.9	79.5	-2.1	1.0	10.6	6.3	12
73C3	77.3	73.0	73.7	69.4	68.7	3.6	7.9	18.0	13.2	6
73C4	66.2	71.7	70.0	74.5	74.9	-3.8	-3.8	6.0	18.8	2
73C5	77.5	76.7	77.3	76.5	77.8	0.2	4.0	24.2	10.1	8
73C6	76.1	77.2	74.8	75.8	79.0	1.2	3.4	8.0	11.9	10
73C7	78.1	76.7	74.9	70.8	68.8	3.1	5.7	17.4	8.2	15
73C8	69.9	67.8	67.1	68.2	71.7	2.7	8.2	10.9	14.2	16
73C9	64.5	60.2	59.6	59.0	65.4	4.9	8.8	11.1	13.2	14
Average	73.9	77.0	77.5	77.6	78.9	-4.0	0.6	10.7	12.6	10

Table 5-9: Free-Flow and Non Free-flow Speed Measures on Urban/Suburban Roads.

Curve ID	V_{85_AT} (km/h)	$V_{85\_PC/SC}$ (km/h)	V_{85_MC} (km/h)	$V_{85\_CS/PT}$ (km/h)	V_{85_DT} (km/h)	ΔV_{85} (km/h)	$\Delta_{85}V_{(1-3)}$ (km/h)	*85MSR* (km/h)	*85MSI* (km/h)	No of Observations
12C1	30.2	77.2	82.9	85.3	85.7	-52.7	-39.4	0.0	29.9	30
12C2	76.2	79.5	83.3	88.3	90.1	-7.1	-5.5	0.9	14.2	12
12C3	88.4	90.0	90.2	89.5	90.5	-1.8	0.3	1.5	9.0	30
12C4	86.5	82.0	80.1	79.3	71.9	6.5	9.9	25.6	0.0	30
12C5	78.6	72.6	71.0	70.5	79.6	7.6	6.2	11.3	18.7	11
73C1	76.6	79.0	82.8	85.9	87.1	-6.2	-1.4	2.7	14.4	30
73C2	88.2	88.4	88.6	86.9	83.7	-0.4	2.8	17.2	6.6	30
73C3	79.2	76.4	76.0	74.0	71.8	3.2	9.6	17.3	13.3	19
73C4	69.9	67.4	69.9	69.7	66.5	0.0	14.3	30.4	19.8	9
73C5	76.0	75.1	74.4	75.1	76.1	1.6	5.5	12.6	13.0	28
73C6	74.9	75.8	73.4	77.2	79.4	1.5	3.9	4.9	12.6	30
73C7	83.3	82.9	77.5	73.4	69.3	5.8	8.5	20.9	6.0	30
73C8	70.8	68.8	67.3	68.4	68.8	3.5	7.7	8.7	9.8	30
73C9	68.6	60.9	60.6	60.5	65.3	7.9	12.9	16.9	12.8	30
Average	74.8	76.9	77.0	77.4	77.5	-2.2	2.5	12.2	12.9	25

From the above tables and the individual speed profiles in Appendix A, driver speed behaviour on urban/suburban roads can be qualitatively described as follows.

- Operating speeds gradually decreased when the test vehicle moved from suburban area to the urban area, implying the population density and the posted speed have great impacts on driver speed selection.
- Operating speed values were noticeably lower than those observed on rural roads when moving on the study curves. Speed reductions from the tangent to the curve were not as significant as those observed on rural roads. Speed increases from the tangent into the curve were also observed on some curves, particularly at those located at intersections with traffic signals.
- The speed differential parameters substantially differ from each other, and *85MSR* yields the highest value, whereas ΔV_{85} produces the lowest value for a speed reduction.
- Interestingly, no significant disparity between the free-flow speeds alone and the combination of free-flow and non free-flow speeds was observed along study curves under investigation, as shown in Tables 5-8 and 5-9.

Following the same steps introduced in the previous section, the regression analysis was performed using the stepwise technique in SPSS v.11.5, in accordance with the three specific cases explained earlier. It should be noted that the study sites with free-flow speeds less than 10 observations were removed from the database in Case 1. The created base models that are statistically significant at 95% confidence level are available in Tables B-3 through B-5 in Appendix B, corresponding to the three circumstances (Cases 1, 2, and 3).

The summaries of the statistics for the independent variables are also included in the tables. The developed base models were then diagnosed using a graphical display of regression residual in SPSS v11.5 and all the models subject to verifications were proven to satisfy the assumption of "general linear model". An example of the model diagnosis for urban/suburban roads is illustrated in Figure 5-2.

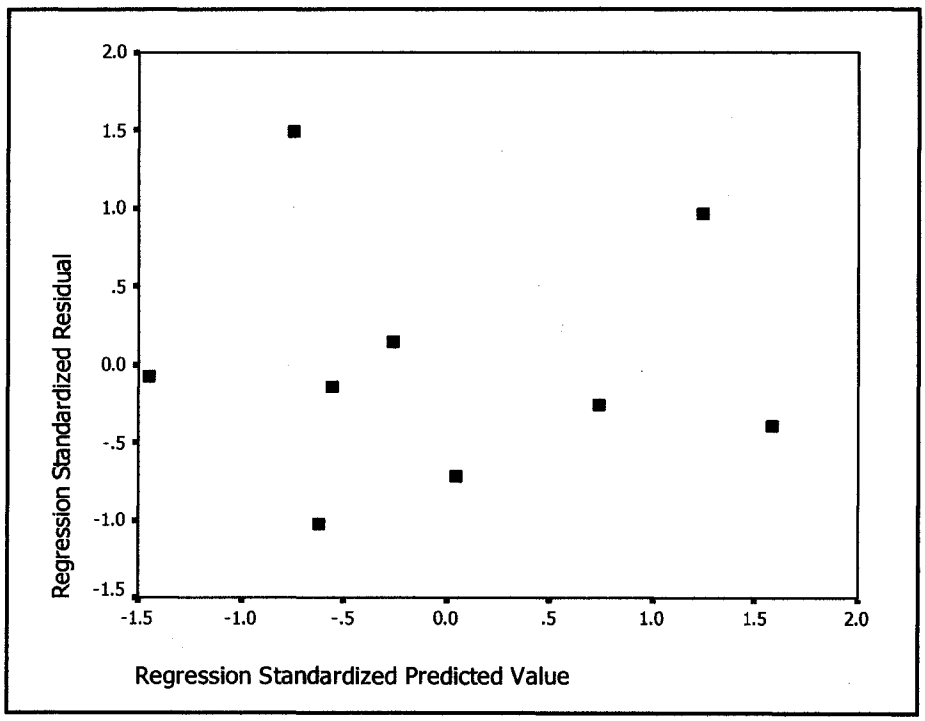


Figure 5-2: Example of Model Diagnoses (Model 6 in Table 5-14).

Regarding the developed models for urban/suburban roads, several issues need to be spelled out, and they are described as follows:

- Models for case 1 do not include the independent variable the presence of intersections (*INT*) mainly because most of the study curves with intersections

were removed due to the limited sample size (<10 observations). As a result, the variable *INT* does not appear in the models as expected.

- The development of the models for speed reductions in case 2 was relatively unsuccessful because 4 out of the 14 curves under consideration were observed to have speed increases from the approach tangent into the curve. This may have disturbed the model development.
- The attempt to develop models for the speed increase when departing a horizontal curve for cases 1 and 3 failed. No relationship was found between *85MSI* and any of the independent variables in both cases.

Having examined the models available for urban/suburban roads, those that best represent the relationship between the responder and predictors are recommended, as reflected in Tables 5-10 through 5-15.

Table 5-10: Operating Speed Models for Case 1 (Urban/Suburban Roads).

No	Model	df	Adjusted R^2	SEE
1	$V_{85\_AT} = 23.686 + 0.807V_p$	5	0.698	5.48
2	$V_{85\_PC/SC} = 60.643 - 28.819DFC + 0.540V_p$	4	0.872	4.07
3	$V_{85\_MC} = 54.738 - 29.031DFC + 0.618V_p$	4	0.931	3.12
4	$V_{85\_CS/PT} = 55.220 - 30.113DFC + 0.613V_p$	4	0.926	3.30
5	$V_{85\_DT} = 102.171 - 37.630DFC$	5	0.774	4.71

Table 5-11: Speed Differential Models for Case 1 (Urban/Suburban Roads).

No	Model	df	Adjusted R^2	SEE
6	$\Delta V_{85} = -10.671 + 17.309DFC$	5	0.868	1.58
7	$\Delta V_{85} = -6.991 + 15.963DFC - 0.018L_{DT}$	4	0.966	0.80
8	$\Delta_{85}V_{(1-3)} = -4.386 + 14.130DFC$	5	0.733	1.96
9	$85MSR = 24.401 - 0.800L_{DT}$	5	0.703	3.80
10	$85MSR = 13.605 - 0.070L_{DT} + 13.352DFC$	4	0.919	1.98

Table 5-12: Operating Speed Models for Case 2 (Urban/Suburban Roads).

No	Model	df	Adjusted R^2	SEE
1	$V_{85\_AT} = 53.432 + 0.483V_p - 9.085INT - 3.806r_1/r_2$	8	0.873	2.48
2	$V_{85\_PC/SC} = 58.397 - 6.691r_1/r_2 - 10.741INT + 0.458V_p$	8	0.927	2.32
3	$V_{85\_MC} = 53.358 - 6.215r_1/r_2 + 0.499V_p - 8.262INT$	8	0.841	3.43
4	$V_{85\_CS/PT} = 52.675 - 0.026CCR_s + 0.400V_p$	9	0.587	5.63
5	$V_{85\_DT} = 95.586 - 31.126DFC$	10	0.552	5.80

Table 5-13: Speed Differential Models for Case 2 (Urban/Suburban Roads).

No	Model	df	Adjusted R^2	SEE
6	$85MSI = 30.441 + 10.886INT - 0.271V_p - 0.029\,L_{AT}$	8	0.803	2.43

Table 5-14: Operating Speed Models for Case 3 (Urban/Suburban Roads).

No	Model	df	Adjusted R^2	SEE
1	$V_{85\_AT} = 49.709 + 0.546V_p - 4.121r_1/r_2$	5	0.936	2.02
2	$V_{85\_PC/SC} = -2.239 + 8.080Ln(r) + 0.486V_p$	5	0.908	2.99
3	$V_{85\_MC} = 58.589 - 24.797DFC + 0.522V_p$	5	0.839	4.00
4	$V_{85\_MC} = 53.710 - 10.464DFC + 0.559V_p - 4.495r_1/r_2$	4	0.992	0.870
5	$V_{85\_CS/PT} = 60.494 - 14.150DFC + 0.435V_p - 0.018CCR_s$	4	0.938	2.35
6	$V_{85\_DT} = 99.525 - 34.274DFC$	6	0.753	4.23

Table 5-15: Speed Differential Models for Case 3 (Urban/Suburban Roads).

No	Model	df	Adjusted R^2	SEE
7	$\Delta V_{85} = -6.280 + 9.418DFC + 0.023L_{AT}$	5	0.815	1.48
8	$\Delta V_{85} = -2.885 + 5.541DFC + 0.029L_{AT} - 5.240E\text{-}06r^2$	4	0.921	0.97
9	$\Delta_{85}V_{(1\text{-}3)} = -5.046 + 16.434DFC$	6	0.743	2.08
10	$\Delta_{85}V_{(1\text{-}3)} = -5.405 + 13.088DFC + 0.023\ L_{AT}$	5	0.871	1.48
11	$85MSR = 1.559 + 0.100L_{AT}$	6	0.764	3.98

The recommended models for urban/suburban roads are interpreted as follows:

- In case 1, the operating speed on the approach tangent was found to be dependent on the posted speed (V_p), whereas the speed on the departure tangent is based on curve deflection angle (*DFC*). When moving on the curve, operating speed was found to be affected by curve deflection angle (*DFC*), in addition to V_p.

- The variables that were found to have impacts on speed differential estimations are *DFC* and L_{DT}. *DFC* positively contributes to all the three speed differential measures, in terms of ΔV_{85}, $\Delta_{85}V_{(1\text{-}3)}$, and *85MSR*. In addition, the departure tangent length was found to affect *85MSR* estimation. This agrees with the finding on two-lane rural highways in this study.

- In case 2, the presence of intersections along with a low speed limit (V_p) would result in a low operating speed when approaching a horizontal curve. The approaching speeds from the tangent to the midpoint of the curve were found to be negatively correlated to the ratio of curve radii between the curve under consideration and its preceding curve. The operating speed prediction after the middle of the curve is based on the values of CCR_s, *DFC* and V_p.

- The presence of intersections, a low advisory speed (V_p), and a relatively short approach tangent would help in increasing the value of *85MSI*.

- In case 3, the posted speed (V_p) was found to be a significant variable affecting operating speed estimations when negotiating a horizontal curve. The operating speed on the approach tangent is based on the ratio of curve radii (r_1/r_2) between the approach curve and its preceding curve, in addition to V_p. Independent variables that affect the operating speeds within the curved section include *Ln(r)*, V_p, *DFC*, r_1/r_2, and CCR_s. The operating speed on the departure tangent depends on the value of *DFC*.

- For speed reduction estimations, independent variables *DFC*, L_{AT}, and r^2 were found to influence the value of ΔV_{85}. Model 10 in Table 5-15 suggests that the

value of $\Delta_{85}V_{(1\text{-}3)}$ would increase when approaching a severe curvature preceded by a long tangent. Model 11 in Table 5-15 indicates that *85MSR* is correlated to the length of the approach tangent (L_{AT}). This coincides with the major findings in many previous studies conducted on rural roads.

5.4 Freeways

In this section, a similar analysis procedure was performed with an attempt to develop statistical models capable of predicting the operating speed in a high speed environment, based on the freeway horizontal geometry. There are a total of 11 horizontal curves on Highway 417, whereas 4 horizontal curves are located on Highway 416. Operating speeds were collected on the selected study curves.

The first effort was made by treating these two freeways separately, as they are located in different areas with distinct traffic volumes. For example, Highway 417 serves a significant commuting traffic volume across the City of Ottawa in the east-west direction. Highway 416 is a typical rural freeway, on which low to moderate traffic volumes predominate. However, no positive results were obtained for both freeways, implying that there is no statistically significant relationship between the dependent and independent variables. With the recognition of the fact that the failure may have resulted from the insufficient sample size, particularly for Highway 416, as only four study curves were available for the model development, study sites were combined from both freeways to form a larger sample. The operating speeds, as well as the speed differential parameters, were compiled in the database for regression analysis, which are reflected in Tables 5-16, with the number of free-flow speed observations.

Table 5-16: Operating Speed and Speed Differential Parameters (Freeways).

Curve ID	V_{85_AT} (km/h)	$V_{85\_PC/SC}$ (km/h)	V_{85_MC} (km/h)	$V_{85\_CS/PT}$ (km/h)	V_{85_DT} (km/h)	ΔV_{85} (km/h)	$\Delta_{85}V_{(1-3)}$ (km/h)	*85MSR* (km/h)	*85MSI* (km/h)	No of Observations
417C1	114.6	111.7	108.5	110.2	110.8	6.0	7.9	12.6	9.2	9
417C2	110.8	111.0	112.9	114.9	113.0	-2.1	3.5	8.3	14.2	7
417C3	116.3	113.2	113.7	115.1	118.2	2.6	3.6	6.3	8.9	10
417C4	113.6	109.5	110.9	114.3	115.0	2.8	2.5	10.5	7.5	4
417C5	110.4	108.6	109.5	110.2	111.0	0.9	4.8	8.7	N/A	13
417C6	111.0	114.8	113.2	113.6	117.4	-2.2	-0.1	7.1	N/A	10
417C7	114.6	114.8	115.5	116.7	114.5	-0.9	5.0	6.8	5.3	10
417C8	112.6	111.8	112.4	112.2	116.0	0.2	2.9	4.4	5.2	9
417C9	119.2	119.4	118.6	117.0	116.3	0.6	3.3	5.5	3.4	9
417C10	108.5	108.3	110.1	110.1	118.4	-1.6	3.0	8.7	9.2	5
417C11	113.0	113.3	117.1	121.6	120.6	-4.1	-0.8	4.0	13.7	9
416C1	117.5	117.9	122.0	127.8	129.0	-4.5	-1.2	3.7	12.6	13
416C2	132.0	130.8	127.3	127.0	127.3	4.8	5.8	9.0	6.7	20
416C3	124.7	125.9	124.3	125.6	125.8	0.4	6.7	12.6	8.7	13
416C4	120.2	116.0	118.2	121.0	124.6	2.0	4.6	11.6	12.5	11
Average	115.9	115.1	115.6	117.2	118.5	0.3	3.4	8.0	9.0	10

As one expects, the operating speeds on freeway curves are considerably higher than those observed on two-lane rural and urban/suburban roadways. In addition, both the speed reduction and speed increase, when approaching and departing a curve, are much lower than those on other types of the roadway. This implies that there was a less operating speed fluctuation on freeway curves, as shown in Table 5-16 and the individual speed profiles available in Appendix A. Furthermore, the speed observations at some curves, particularly for Curves 417C4 and 417C10, are extremely small. Therefore, the aforesaid two curves with limited sample size were removed from the database for developing models.

After entering the independent and dependent variables into SPSS v.11.5 the second time, four models were established. However, the relationship between the explanatory variables and proposed responder is obviously not significant because of the very low coefficient of determination (Tables 5-17 and 5-18). The models listed in the tables indicate that there might be certain relationships between the freeway geometry and speed measures, but not strong enough to be recommended for practical uses. The details of the statistics for the independent variables are provided in Table B-6 in Appendix B.

Table 5-17: Operating Speed Models (Freeways).

Position	Model	df	Adjusted R^2	SEE
AT	$V_{85\_AT} = 111.023 + 11.417DFC$	11	0.217	5.45
DT	$V_{85\_DT} = 111.101 + 0.013L$	11	0.410	4.44

Table 5-18: Speed Differential Models (Freeways).

Parameter	Model	df	Adjusted R^2	SEE
ΔV_{85}	$\Delta V_{85} = -3.583 + 0.082CCR_s$	11	0.433	2.27
$85MSR$	$85MSR = 5.003 + 6.929DFC$	11	0.319	2.31

Although the development of analytical models for freeways is relatively unsuccessful, some valuable findings can still be gained from the practice:

- The weak correlation between freeway alignments and the driver speed behaviour reveals that the geometric design is not the dominant factor contributing to driver speed choice on freeways.
- The operating speed inconsistency on existing freeways is not as significant as what it exhibits on the other road types.
- Referring to Table 5-16, the average values of the observed operating speeds on both the tangent section and the curve exceed the posted speed (100 km/h) by substantial amounts (>15 km/h). Although the Ontario design guides use a 20 km/h above the posted speed rule to determine the design speed, with the appearance of faster cars, the safety margin of design speed can be reduced, resulting in the concern of safe operations on existing freeways in near future.

CHAPTER 6: DRIVER BEHAVIOUR ON FREEWAY INTERCHANGES

The freeway interchange is a transitional section connecting two adjacent roadway facilities. It consists of two basic components: the acceleration/deceleration lane located on the main segment of the freeway and the entry/exit ramp. In this chapter, driver behaviour on freeway interchanges, in terms of operating speeds, speed variations, as well as acceleration/deceleration rates, is studied. The study sites on the selected test route include two acceleration/deceleration lanes and two entry/exit ramps. They belong to Highways 417 and 416, respectively.

The procedure of the data extraction and reduction was similar to that for the main types of the road as explained in the previous chapter. Again, the individual speed profiles were drawn and examined. It needs to be pointed out that the drivers' speed profiles were not available on the acceleration lane of Highway 416 and the deceleration lane for Highway 417. This is because the GPS receivers lost contact with satellites when passing an overpass located at the abovementioned two locations. As a result, the trajectory of the test vehicle was missing, and accordingly the instantaneous speed of the test vehicle could not be related to the roadway geometric elements (i.e. stations).

Some interruptions of speed measurements may also result from the Corsa power failure. For example, the speed measurements for drivers 19, 21, and 27 were distorted on the acceleration lane of Highway 417. Therefore, the speed observations for these drivers were removed from the database before the data analysis was conducted.

Another concern that needs to be addressed is the non-free-flow speed observations. Because of a very high traffic volume at some particular locations on the test route, such

as the deceleration lane as well as the exit ramp of Highway 417, most of the sample drivers were observed to be impeded by front traffic during the period of experiment. For instance, only three drivers (Drivers 4, 18, and 28) were found to have free-flow speeds. Therefore, it was decided to combine free-flow and non free-flow speeds for the data analysis.

Due to the limited study sites on the test route, developing speed models for freeway interchanges was not possible. However, it is still valuable to study driver behaviour on the interchanges by starting from a descriptive analysis. This chapter is divided into two sections to discuss driver behaviour on the acceleration/deceleration lanes and ramps of the freeways under investigation, respectively.

6.1 Entry/Exit Ramps

Driver behaviour was investigated on two types of ramps located on Highways 417 and 416. The study areas are illustrated in Figures 6-1 through 6-4. The driving direction of the test vehicle is from the beginning of the ramp (*BOR*) to the gore area of the entry ramp, or to the end of the exit ramp. The limits of each study site were identified with the digitized maps provided by the City of Ottawa. The alignment parameters of the ramps were extracted using RoadFit Extension in Arcview 3.2a. Table 6-1 summarizes the length of the study area as well as the information of the extracted curves.

Table 6-1: Study Area and Curve Parameters of Entry/Exit Ramps.

ID	Length of Study Area (m)	Curve ID	Curve Radius (m)	Circular Curve Length (m)	Entry Spiral Length (m)	Exit Spiral Length (m)
Ramp 1	280	/	/	/	/	/
Ramp 2	365	6-416C1	71.62	119.87	54.90	84.79
Ramp 3	2253.92	417-7C1	447.83	104.99	35.00	50.00
		417-7C2	548.22	384.09	184.99	244.87
Ramp 4	619.902	416-12C1	151.42	94.97	20.00	19.99
		416-12C2	98.00	49.95	25.00	30.00

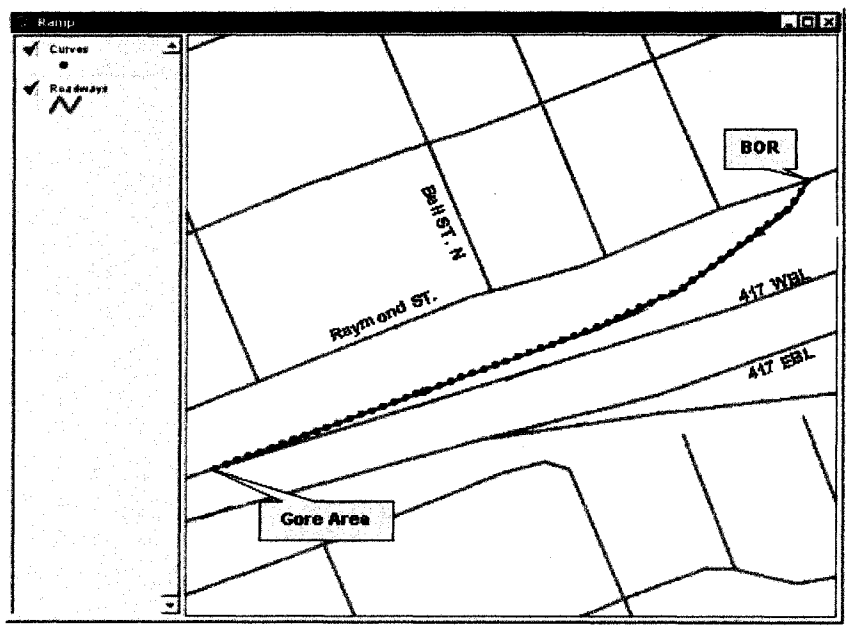


Figure 6-1: Entry Ramp of Highway 417 (Ramp 1).

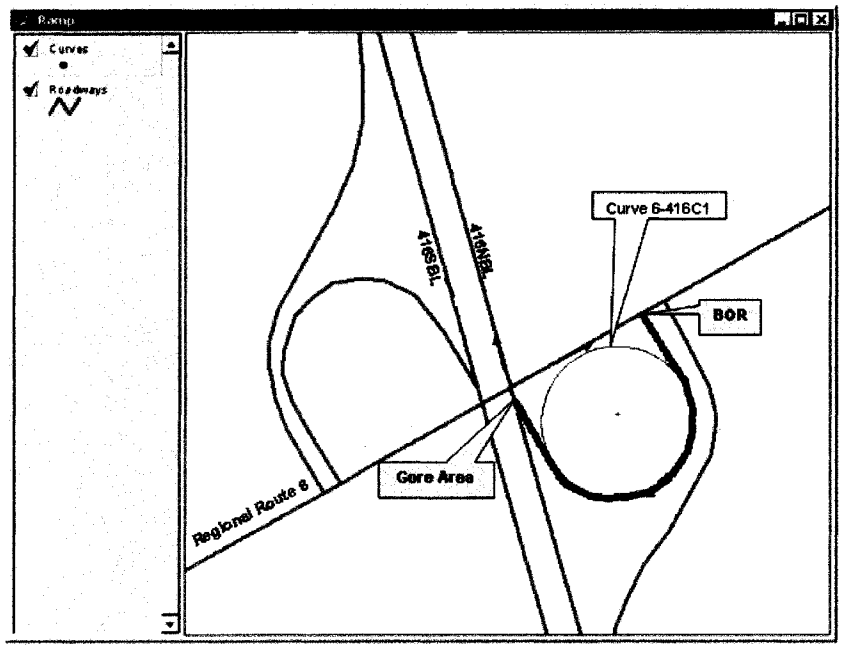


Figure 6-2: Entry Ramp of Highway 416 (Ramp 2).

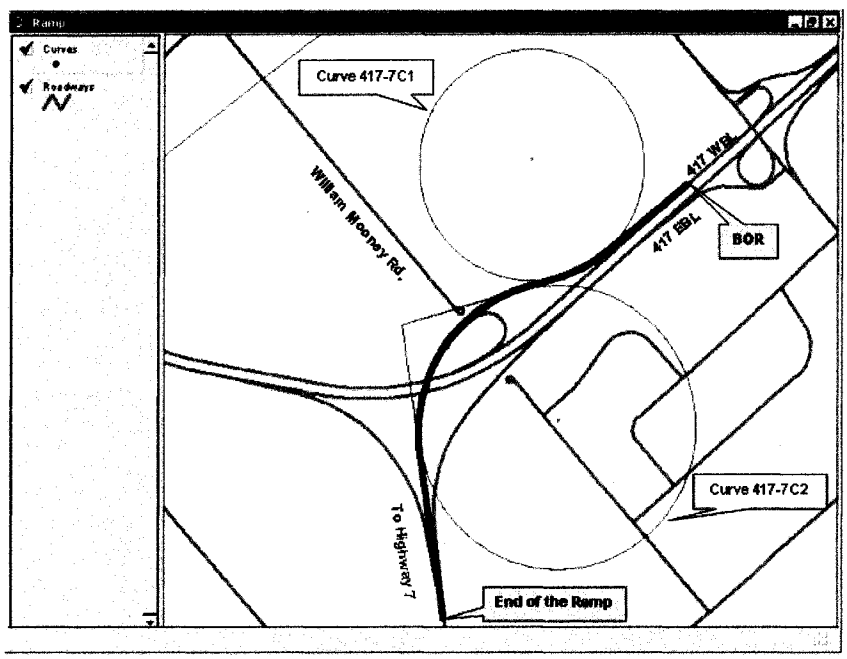


Figure 6-3: Exit Ramp of Highway 417 (Ramp 3).

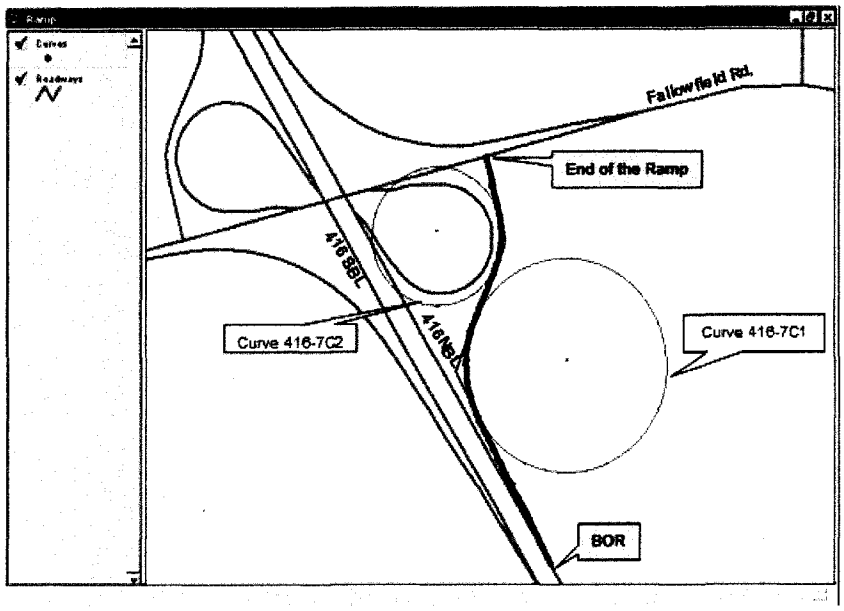


Figure 6-4: Exit Ramp of Highway 416 (Ramp 4).

6.1.1 Data Reduction

Several speed and speed differential measures were employed to quantify driver speed behaviour on freeway ramps, such as the 85th percentile speed, *85MSR*, and *85MSI*. Driver acceleration/deceleration manipulations were described using overall acceleration/deceleration rates. An overall acceleration/deceleration rate refers to the speed change during the time interval of two speed observations.

For the entry ramp of Highway 417 without a ramp curve, these two speed observation locations are the beginning of the ramp (*BOR*) and the ramp gore area (the physical nose of a ramp). Operating speeds at these two locations were extracted from the driver's speed profiles. Not surprisingly, the maximum speed values always took place at the end of the ramp (gore area) whereas the minimum operating speeds were usually observed at the beginning of the ramp (*BOR*), for an entry ramp, and vice versa for an exit ramp.

For ramps with horizontal curves such as Curves 6-416C1, 417-7C1, 417-7C2, 416-12C1, and 416-12C2, the overall deceleration rates was calculated through dividing the maximum speed reduction (*85MSR*) from the approach tangent into the curve by the time elapsed between these two speed observations, while the overall acceleration rate was obtained through dividing the maximum speed increase (*85MSI*) by the time interval from the curve to the departure tangent. The data reduction was performed in Excel following the same procedure introduced in the previous chapter of this thesis. Driver speed profiles are illustrated in Figures 6-5 through 6-8 for each study ramp. The driving direction flows from the beginning of the ramp to the gore area or to the end of study range, as shown in the figures.

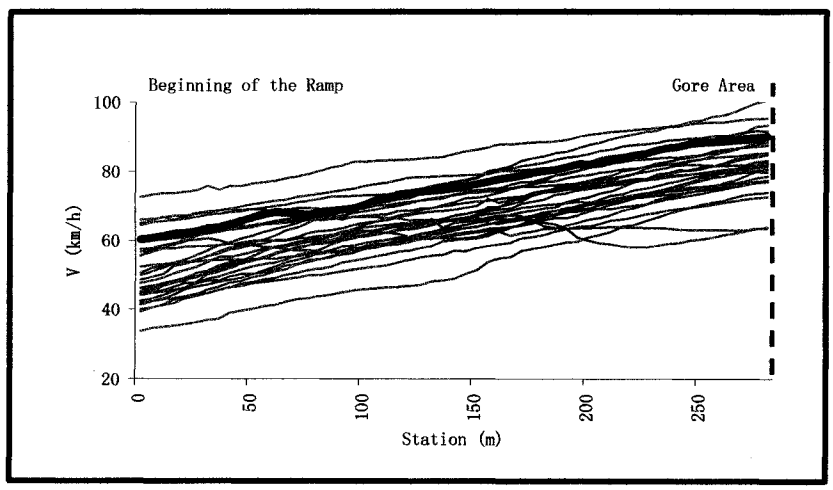


Figure 6-5: Speed Profiles for Entry Ramp of Highway 417.

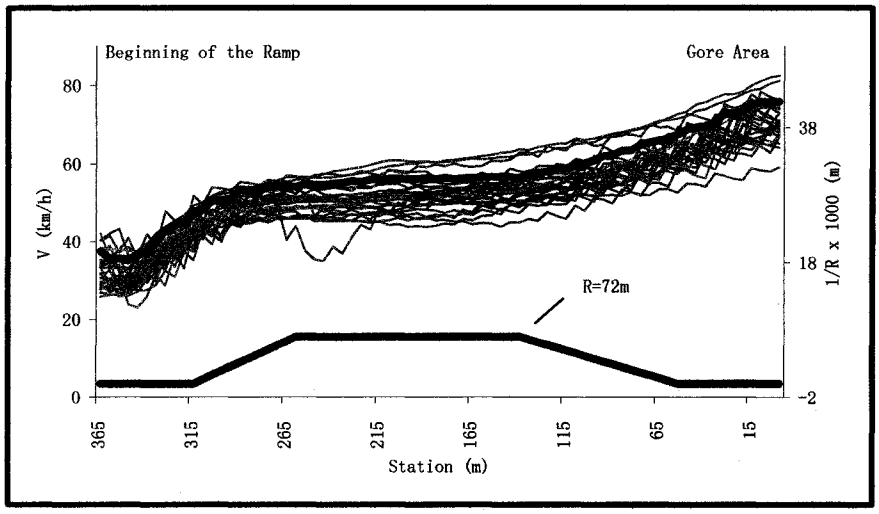


Figure 6-6: Speed Profiles for Entry Ramp of Highway 416.

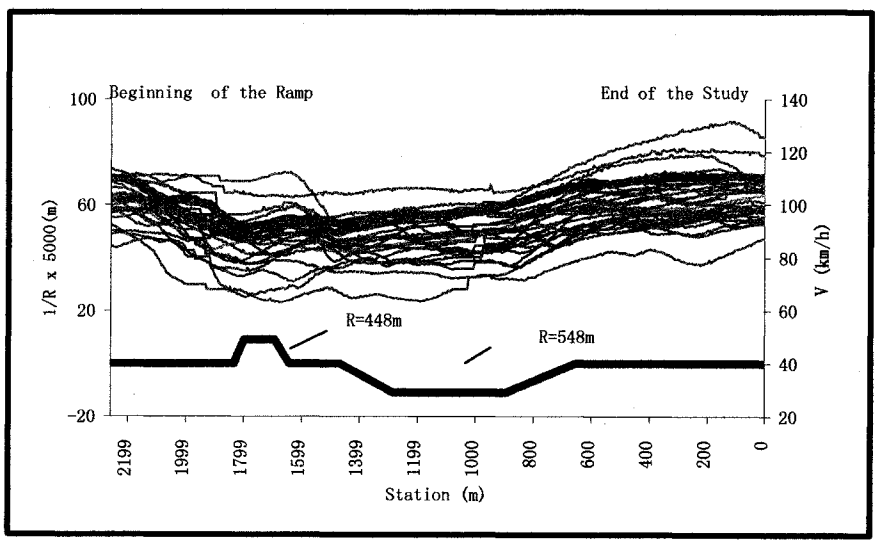


Figure 6-7: Speed Profiles for Exit Ramp of Highway 417.

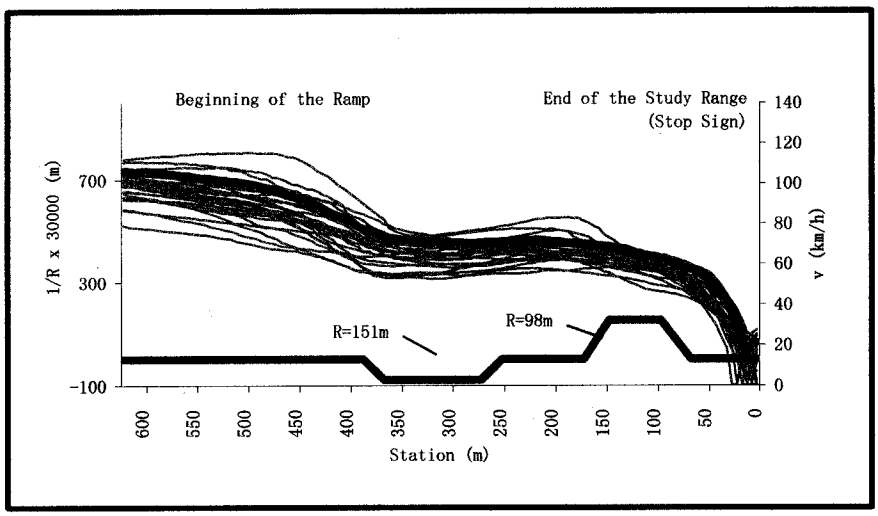


Figure 6-8: Speed Profiles for Exit Ramp of Highway 416.

Table 6-2 includes the speed measures as well as the associated acceleration/deceleration rates, explaining driver behaviour when moving on the entry ramps to Highways 417 and 416. Driver behaviour on the exit ramps of these two freeways is described as shown in Table 6-3.

Table 6-2: Operating Speeds and Acceleration/Deceleration Rates on Entry Ramps.

Ramp ID	V_{BOR} (km/h)	V_g (km/h)	*85MSR* (km/h)	*85MSI* (km/h)	*d'* (m/s^2)	*a'* (m/s^2)	Sample Size
1	60.26	89.96	/	43.82	/	0.81	27
2	37.43	75.76	2.69	33.50	2.03	0.44	30

V_{BOR}: 85th percentile speed at the beginning of the ramp

V_g: 85th percentile speed at the gore area of the ramp (End of the ramp)

85MSR: Maximum speed reduction on the approach tangent of Curve 6-416C1 (Figure 6-6)

85MSI: Maximum speed increase when departing Ramps 1 and 2 (Figures 6-5 and 6-6)

a': Overall acceleration rate corresponding to *85MSI*

d': Overall deceleration rate corresponding to *85MSR*

Table 6-3: Operating Speeds and Acceleration/Deceleration Rates on Exit Ramps.

Ramp ID	$85MSR_1$ (km/h)	$85MSR_2$ (km/h)	$85MSI$ (km/h)	a' (m/s^2)	d'_1 (m/s^2)	d'_2 (m/s^2)	V_g (km/h)	Sample Size
3	24.26	/	26.62	0.17	0.41	/	110.05	30
4	41.88	53.71	/	/	1.03	2.65	106.37	30

$85MSR_1$: Maximum speed reduction when entering the ramp (Figure 6-7 and 6-8)

$85MSR_2$: Maximum speed reduction when departing Curve 416-12C2 (Figure 6-8)

d'_1: Overall deceleration rate corresponding to $85MSR_1$

d'_2: Overall deceleration rate corresponding to $85MSR_2$

$85MSI$: Maximum speed increase when departing Curve 417-7C2

V_g: 85th percentile speed at the gore area of the ramp (Beginning of the exit ramp)

6.1.2 Driver Behaviour on Entry/Exit Ramps

The findings with respect to driver behaviour on the entry/exit ramps of the freeways under investigation are summarized as follows:

- The gore speeds at the end of the entry ramps were found to be lower than the posted speed of the freeways. This implies that the application of the acceleration lane after the entry ramp is necessary to allow drivers to continuously accelerate ensuring a safe merging operation.
- The overall acceleration rate (0.81 m/s^2) on the ramp of Highway 417 is near two times greater than that (0.44 m/s^2) on the ramp to Highway 416. This may arise from the fact that the ramp length (280 m) of Highway 417 is shorter than that (365 m) on Highway 416.
- An extremely high deceleration rate (2.03 m/s^2) was observed when approaching the entry ramp to Highway 416. This is mainly because there is no transitional

section connecting Regional Route 6 with the ramp. Therefore, drivers were forced to decelerate drastically right before entering the ramp.

- According to the individual speed profiles, two typical types of acceleration patterns were found to be adopted by drivers on the ramp to Highway 417. The first can be described as a straight-line acceleration, and the second can be viewed as a step-by-step acceleration. The straight-line acceleration indicates that a uniform acceleration rate was used and maintained during the acceleration period. In the case that the step-by-step acceleration was applied, drivers used multiple acceleration rates before the completion of acceleration. Examples of the acceleration patterns are illustrated in Figures 6-9 and 6-10.
- The gore speeds at the end of the deceleration lanes (beginning of the exit ramps) were observed to be higher than the posted speed (100 km/h). This suggests that the actual operating speeds at which drivers travel on freeways obviously exceed the speed limit.
- Significant speed reductions were observed when departing the main segment of freeways. The speed reductions could be mostly attributed to the change in road class (from a freeway to a highway) with different advisory speeds.
- The speed reduction on Curve 416-12C1 (41.88 km/h) was noticeably greater than that (24.26 km/h) observed on Curve 417-7C1. This demonstrates that highway geometry to a large extent affects driver choice of speed, as the ratio of curve radii of these two curves reaches 3.6. The second speed reduction with a

severe deceleration rate (2.65 m/s^2) on Curve 416-12C2 was induced by the stop sign at the end of the ramp.

- Drivers were found to gradually increase their speeds when departing the exit ramp of Highway 417. It should be noted that the speed adjustment on the exit ramp of Highway 417 was subjected to non-free-flow conditions as moderate to heavy traffic predominates on it.

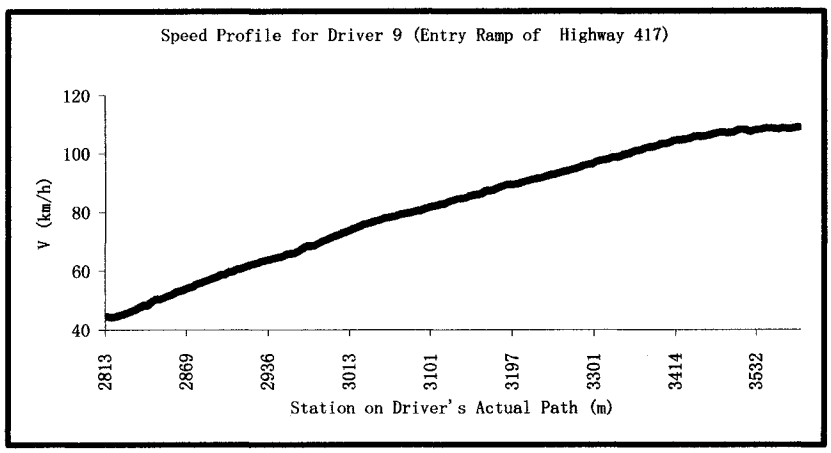


Figure 6-9: Example of Straight-Line Acceleration.

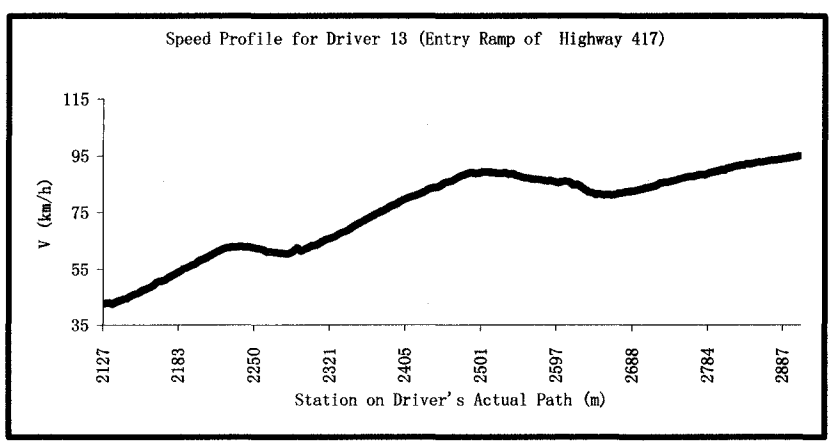


Figure 6-10: Example of Step-By-Step Acceleration.

6.2 Acceleration/Deceleration Lanes

Acceleration/Deceleration lanes are important elements of the freeway interchange, on which a variety of vehicle movements, such as: merging, diverging or changing lanes should be safely accommodated. Clearly, vehicular movements on the acceleration/deceleration lanes of freeway interchanges are more complicated than those on other roadway types. In this connection, there is a need to carry out an exclusive speed study on the freeway acceleration/deceleration lanes.

In this section, driver acceleration/deceleration behaviour, which reflects the degree of comfort of driving manoeuvres, was also investigated in addition to the speed study. The beginning and end of the acceleration/deceleration lane were designated as the points of interest to extract speed data. The acceleration/deceleration rate was computed based on the speed variation and time interval between these two observations. Another research interest - the location where drivers change lanes (i.e. the point at which vehicles enter or depart the acceleration/deceleration lane), was missing because the time of changing lanes was not marked during the experiment.

Two acceleration and deceleration lanes were involved in this study, and they are located on Highways 417 and 416, respectively. The stations as well as the length of the acceleration/deceleration lanes were originally identified from the plans provided by the Ontario Ministry of Transportation (*MTO*). Afterwards, the initial plan stations were transferred to the stations on the digitized map used in this research, based on a common point such as the gore area where an acceleration lane starts or a deceleration lane ends. The study areas are presented in Figures 6-11 through 6-14, with a red arrow showing the driving direction. The study range associated with the stations is summarized in Table 6-4.

Table 6-4: Study Area for Acceleration/Deceleration Lanes.

ID	Lane Length (m)	Plan Stations
Acceleration Lane on Highway 417	122	27+500 - 27+378
Deceleration Lane on Highway 417	444*	28+300 - 27+856
Acceleration Lane on Highway 416	438	17+730 – 18+168
Deceleration Lane on Highway 416	184	17+926 – 18+110

*: The length of 444 m includes the section from the end of the entry ramp of the upstream interchange to the end of the deceleration lane.

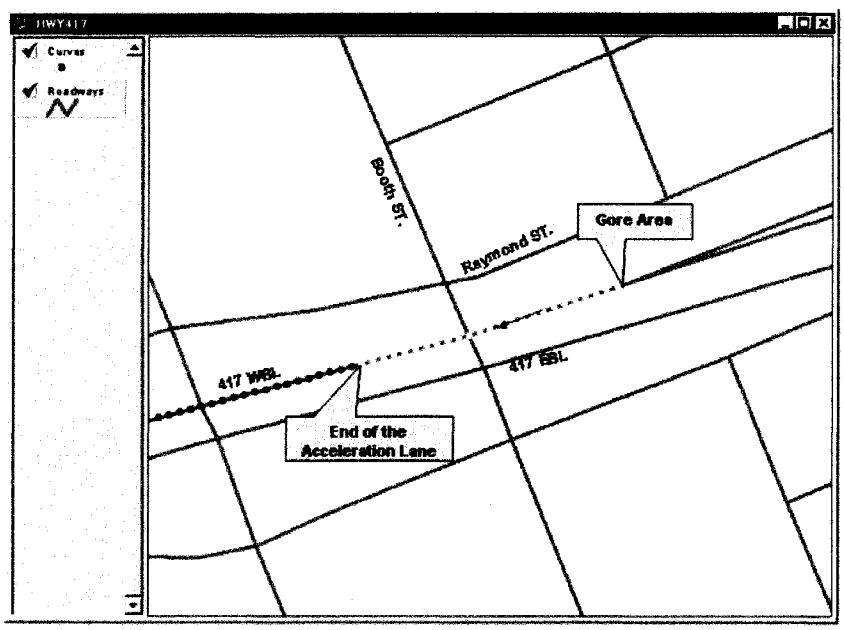


Figure 6-11: Acceleration Lane on Highway 417.

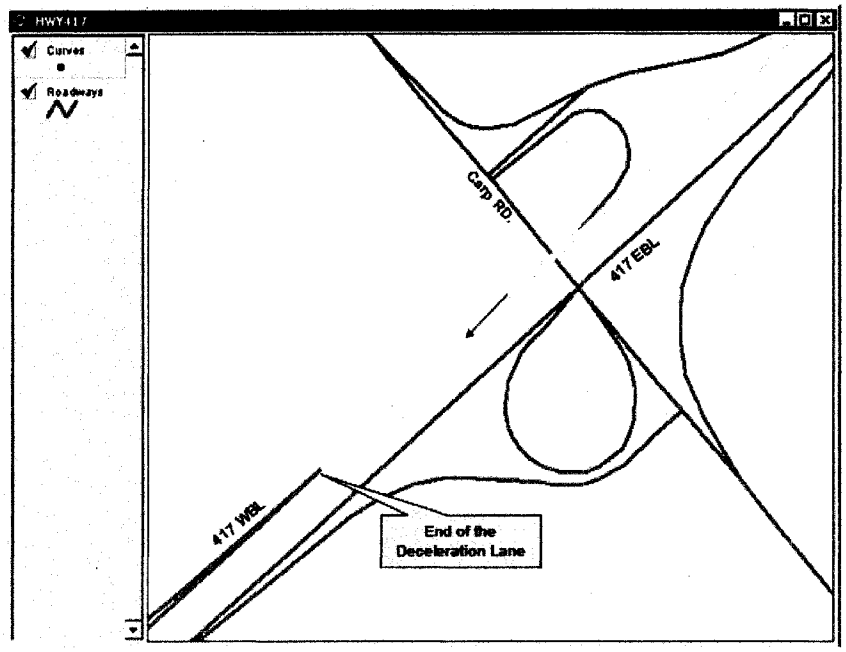


Figure 6-12: Deceleration lane on Highway 417.

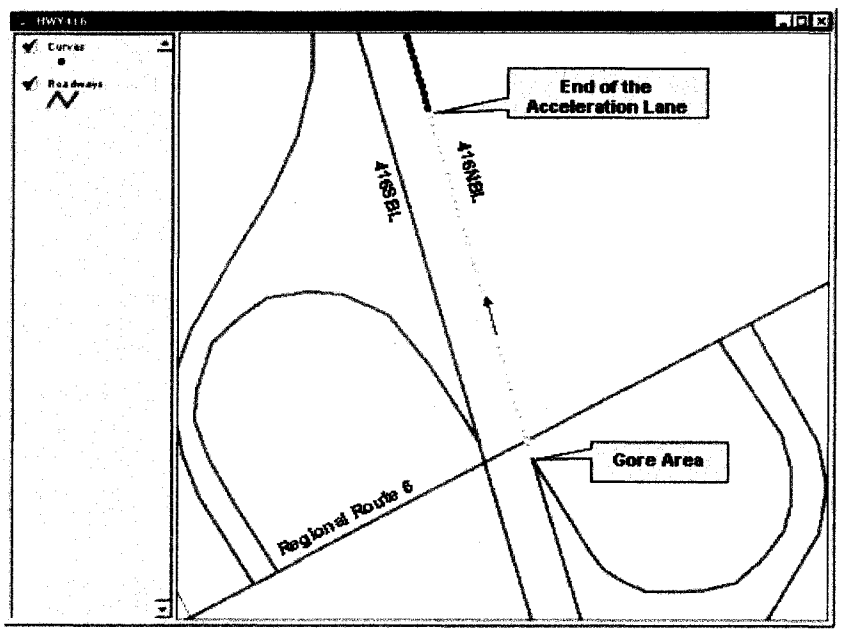


Figure 6-13: Acceleration Lane on Highway 416.

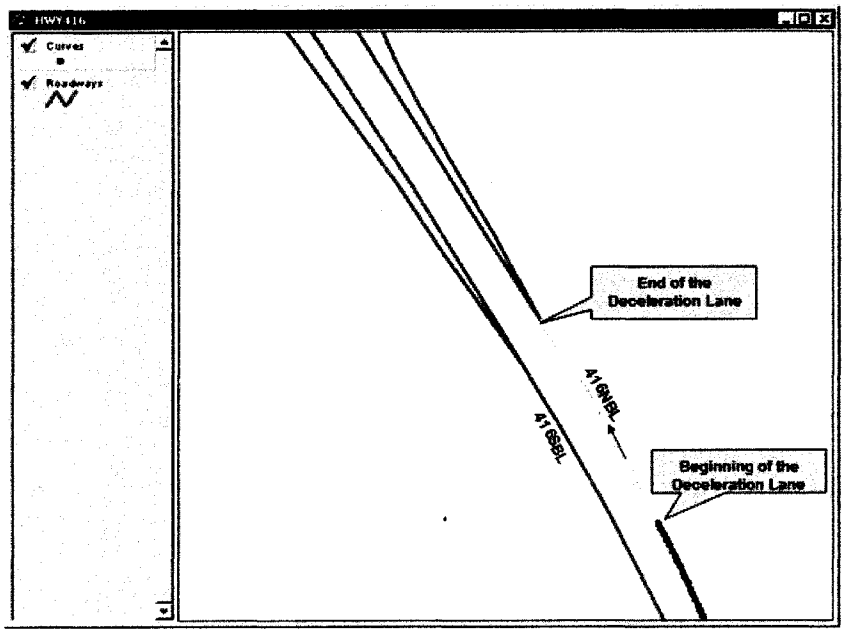


Figure 6-14: Deceleration Lane on Highway 416.

6.2.1 Data Reduction

The individual speed profile for each sample driver was plotted. Operating speeds both at the beginning and end of the acceleration/deceleration lane were extracted and the overall acceleration/deceleration rates were calculated accordingly to describe driver behaviour on the study section.

Driver speed profiles are unfortunately not available for the acceleration lane on Highway 416 and the deceleration lane on Highway 417, as previously explained. In addition, as shown in Figure 6-12, the deceleration lane on Highway 417 is a common speed change lane (*SCL*) shared by the entry traffic from the upstream interchange. Therefore, the beginning of the study section should start from the point at which the test vehicle shifted to this lane from the most right through lane. However, as mentioned

earlier, the shifting point was not marked. Therefore, the beginning of the study section could not be determined. Consequently, analyzing driver behaviour on this deceleration lane was not possible. Driver speed profiles for the acceleration lane on Highway 417 and the deceleration lane on Highway 416 are provided in Figures 6-15 and 6-16.

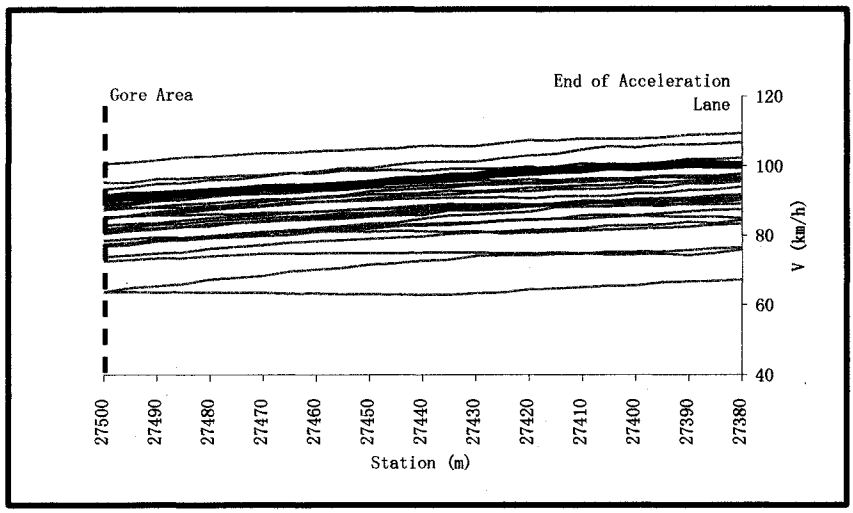


Figure 6-15: Speed Profiles for Acceleration Lane on Highway 417.

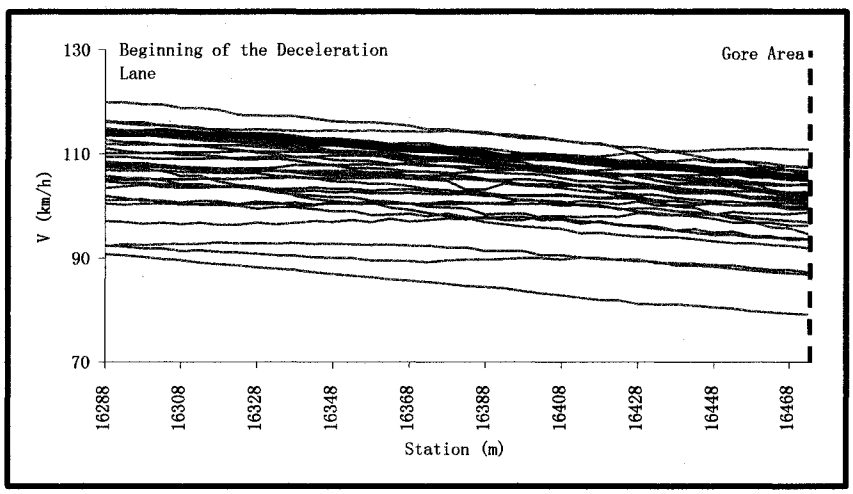


Figure 6-16: Speed Profiles for Deceleration Lane on Highway 416.

The 85th percentile speed at the end of acceleration/deceleration lane, the 85th percentile speed change value, overall acceleration/deceleration rates, and the posted speed on each freeway are presented in Table 6-5. The gore speeds for acceleration/deceleration lanes under investigation are 89.96 km/h (acceleration lane on Highway 417), 75.76 km/h (acceleration lane on Highway 416), and 106.37 km/h (deceleration lane on Highway 416) respectively, as presented in Tables 6-2 and 6-3.

Table 6-5: Speed Measures on Acceleration/Deceleration Lanes.

ID	V_d (km/h)	V_m (km/h)	a' (m/s^2)	d' (m/s^2)	*85MSR* (km/h)	*85MSI* (km/h)	V_p (km/h)
Acceleration Lane Highway 417	/	100.13	0.76	/	/	12.78	100
Acceleration Lane Highway 416	/	113.88	0.69	/	/	41.08	100
Deceleration Lane Highway 416	114.27	/	/	0.53	12.39	/	100

V_d: 85th percentile speed at the beginning of the deceleration lane

V_m: 85th percentile speed at the end of the acceleration lane

a': 85th percentile overall acceleration rate

d': 85th percentile overall deceleration rate

85MSR: Maximum speed reduction on the deceleration lane

85MSI Maximum speed increase on the acceleration lane

V_p: Posted speed

6.2.2 Driver Behaviour on Acceleration/Deceleration Lanes

Driver behaviour on the acceleration/deceleration lane is summarized as follows:

- Drivers were found to continuously accelerate on the acceleration lanes for both freeways with a similar acceleration rate. The maximum speed increase (41.08 km/h) on the acceleration lane of Highway 416 is greater than that (12.78 km/h) observed on Highway 417. This may result from a substantially longer acceleration lane (438 m) applied on Highway 416, compared to that (122 m) on Highway 417.
- A considerable operating speed disparity between the main traffic flows and the entry vehicle was observed on Highway 417. According to the results of speed study in this research (Table 5-16), the average 85th percentile speed on the main segment of Highway 417 reached 115 km/h, while the merging speed observed at the end of the acceleration lane was only 100.13 km/h (Table 6-5). Such a big speed discrepancy could lead to an unsafe merging manoeuvre and increase the likelihood of traffic collisions. On the other hand, the operating speed difference between the entry and through traffic on Highway 416 was relatively small, indicating a smooth and safe merging operation can be expected on the study section. (Tables 5-16 and 6-5).
- For deceleration manoeuvres on Highway 416, drivers were found to continuously reduce their speeds on the deceleration lane, with a moderate deceleration rate (0.53 m/s^2).

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

This study started with a comprehensive literature review concentrating on the previous research work related to design consistency, which is viewed as a promising and effective theory to address some shortcomings in the highway geometric design process. The selection of the design speed was firstly discussed in the literature review. Through a comparison between the traditional design speed approach in North American design guides and international design practices, it has been shown that the current North American design guidelines could bring about two major problems: (1) a large disparity between the selected design speed and the actual operating speed on a particular roadway element; and (2) a considerable operating speed variation between adjacent roadway elements. Both problems reflect the incompatibility between the roadway geometric features and the driver's expectation, and could create a critical situation that drivers are unable to handle, thus increasing the likelihood of traffic collisions.

The literature review then discussed countermeasures for the deficiencies arising from the adherence to the traditional design approach. Among the different measures for design consistency evaluation, the study of the operating speed method was emphasized. The previous research concerning operating speed prediction, such as the selection of study sites, data collection procedures, study methodology, and research findings were gathered and described in detail in the second part of the literature review. The pitfalls remaining in the reviewed research were spelled out. In addition, the study of the literature indicated that the current knowledge and practices of design consistency is still in a preliminary stage,

and there is a need to deepen and refine the current research. Based on the literature review, the main objective of this research was presented, which is to analyze driver speed behaviour in accordance with the most common road classes in Eastern Ontario.

The proposed experiment was then introduced in detail, covering the test route selection, the test driver's recruitment, the experiment equipment, and the data collection process. At the stage of database preparation, Arcview 3.2a – a GIS software used to construct the database - was introduced. Instead of using values from design or as-built drawings in most of the previous research, highway horizontal alignment parameters were extracted from the actual road centerline coordinates. Both the operating speed data and the horizontal alignment parameters were validated.

Following the database construction, identifying the study curves and quantifying driver speed behaviour were explained. The last step prior to the data analysis was to recognize the effects of possible factors, other than highway geometric features, on driver choice of speed. A 5-second time gap method was used to identify the speed observations under free-flow condition. The individual driver speed profile was studied to recognize speed observations with the influences of various traffic control devices at intersections in the vicinity of the study sites.

The data analysis was presented in two separate chapters. Operating speed and speed differential prediction models were developed for two-lane rural highways and urban/suburban roads, respectively. Driver speed behaviour on the acceleration and deceleration lanes of freeways, and those on the freeway ramps were analyzed in an exclusive chapter. The operating speed characteristics represented by the 85th percentile

maximum speed and minimum speed values, along with acceleration and deceleration behaviour, were described based upon the data analysis.

7.2 Findings of the Research

This research presented a comprehensive study of driver speed behaviour on the specific road class. Operating speed and speed differential prediction models were firstly established, using actual driving data. The findings of this research are concluded as follows:

- This research reveals that the effect of highway geometry on driver speed behaviour varies depending on the road class. Driver speed selections on two-lane rural highways and urban/suburban roads are highly correlated to the geometric features. Horizontal alignments on freeways have minor impacts on driver speed behaviour.
- On two-lane rural highways, operating speeds on the tangent section are affected by the tangent length. Operating speeds on the curved section are correlated to curvature parameters CCR_s, $Ln(r)$, or $1/r$. In addition, operating speeds at the end of circular curve can also be affected by the departure tangent length. Operating speed variations were observed within the limits of the curved section on most of the study curves, suggesting drivers do not maintain a constant speed, and the deceleration or acceleration could take place on the curved section. This finding coincides with that by Crisman et al (2005) in Italy. Therefore, the speed profile construction based on Leisch and Leisch (1977) assumption should be revised.

- On two-lane rural highways, the speed changes when the vehicle moves on a horizontal curve are governed by both curve parameters and the attached tangent lengths. This finding supports the importance of the relation of the adjacent elements on a roadway as stated in the work by Lamm et al (1988) and Lamm and Smith (1994). Furthermore, the developed speed differential models in this research include the departure tangent length (L_{DT}) as an independent variable contributing to *85MSR*. The inclusion of the departure tangent length in speed differential estimations implies that drivers would be more cautious to negotiate continuous curves than they do on isolated curves.

- ΔV_{85} is proven in this study to underestimate the speed reduction from the tangent to the curve, as demonstrated by many researchers, such as McFadden and Eleferiadou (2000), Misaghi (2003), and Park and Saccomanno (2005). It is also noticed that there is a large disparity between $\Delta_{85}V$ and *85MSR*, particularly on flatter curves, in this research.

- On urban/suburban roads, the posted speed and presence of intersections are two important factors that affect driver speed behaviour when traversing a horizontal curve. A higher posted speed would result in a greater operating speed along the curve on urban/suburban roadways. On the other hand, the operating speed can be reduced on the curve, where an intersection is in presence. In addition, the posted speed and presence of intersections were also found to contribute to speed differential estimations when negotiating a horizontal curve.

- Curvature parameters CCR_s and *DFC* were found to be the most significant variables affecting operating speeds as well as speed differential values along the curve on urban/suburban roads. Similar to two-lane rural highways, the departure tangent length plays an important role in affecting the value of *85MSR*, and was also found to influence *85MSI* estimation, while moving on a horizontal curve. In addition, the ratio of curve radii between the curve under consideration and its previous curve was found to have influence on operating speed estimations.

- The magnitude of the observed 85th percentile speed tends to decrease from suburban areas to urban areas. This finding suggests that the majority of driver populations will adopt a speed on a roadway facility, compatible with the posted speed, land development, and the population density. Thus, these factors should be taken into account when selecting a design speed for urban/suburban roadways.

- Operating speeds on freeways were observed to exceed the posted speed (100 km/h) by substantial amounts (>15 km/h). The same conclusion was found in the previous speed survey both in Canada and overseas. No statistically strong relationship was found in this study, relating driver speed behaviour to highway geometry on freeways.

- The length of the acceleration lane shows its importance in merging manoeuvres on highway interchanges in this study. Through a comparison of operating speeds at the end of the acceleration lane on both freeways under investigation, it is learned that a longer acceleration lane leads to a fully developed operating speed, resulting in a smaller operating speed disparity between the entry vehicle and the main traffic flows.

- Horizontal alignments play a significant role in affecting operating speed variations when moving on interchange ramps. Similar to the main segment of highways, generous ramp geometry could assist in alleviating operating speed variations on both entry and exit ramps.
- Driver acceleration/deceleration behaviour was also found to be correlated to the geometric features such as the length of the ramp, the length of the acceleration/deceleration lane, or the radius of the ramp curve. A high acceleration/deceleration rate was always observed at the location coupled with a severe ramp curve or relatively short lane length in this research.
- The acceleration and deceleration patterns were found to greatly vary among the investigated drivers. More than one acceleration and deceleration rates were commonly used by most of the drivers. This may take place when drivers were seeking a proper gap to merge into the main traffic, to change lanes, or to depart from the main segment of the freeway.

7.3 Recommendations for Future Study

Recommendations for future study in this area are presented below:

- Due to the limited study sites and driver sample, operating speed and speed differential models developed in this research were not validated. Thus, a large scale of operating speed investigations is needed to verify the recommended models in this study.

- This study only addresses driver speed behaviour for passenger cars. Further study for heavy trucks is necessary to cover a wider range of traffic compositions on the roadway facility.
- The actual operating speed data collected in this study could be used to validate the existing models developed in other research, particularly for those by Fitzpatrick el at (2000a), which are recommended to be included in the revised TAC design guides (1999), in order to verify their applicability for Canadian roads.
- As the test route selected in this study encompasses the areas where the terrain type is level, this research only accounts for the impacts of horizontal alignments on driver speed behaviour. For studies conducted in rolling or mountainous areas, it is wise to take into account the effects of roadway longitudinal alignments, as suggested by other researchers such as Hassan et al (2000).
- Other aspects of driver behaviour such as acceleration and deceleration characteristics corresponding to different road classes need to be examined along with the operating speed study in future work.
- The location where drivers change lanes is an important issue to study driver behaviour on the acceleration and deceleration lanes of freeway interchanges. The identification of those locations in future research is recommended by either investigating the reading of the steering from Corsa Acquisition System, or looking at the vehicle movement recorded by the video camera installed in the test vehicle.

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APPENDIX A: SPEED PROFILES ON STUDY CURVES

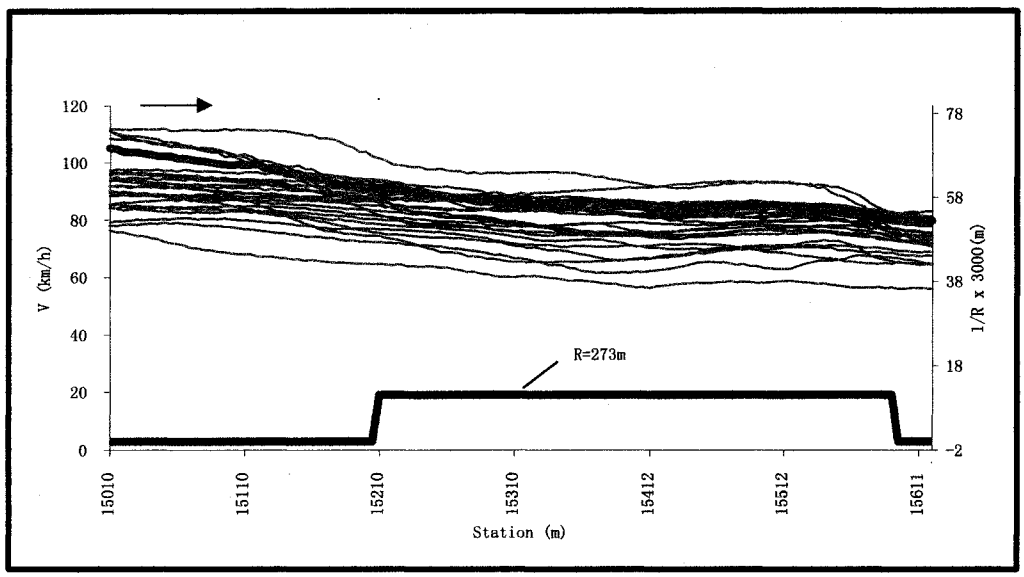

Figure A-1: Curve 3C4 on Regional Route 3 (Two-Lane Rural Highway).

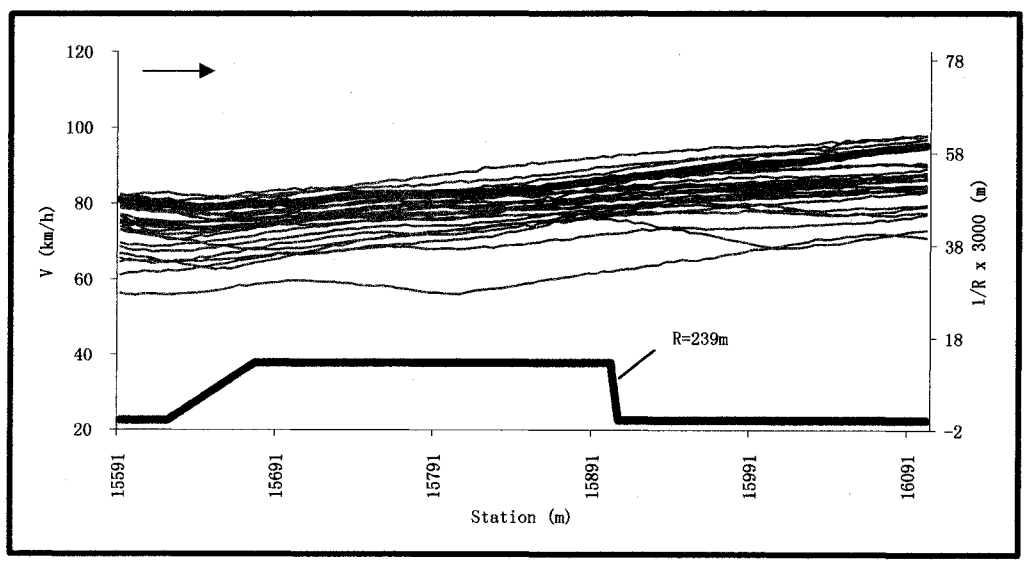

Figure A-2: Curve 3C5 on Regional Route 3 (Two-Lane Rural Highway).

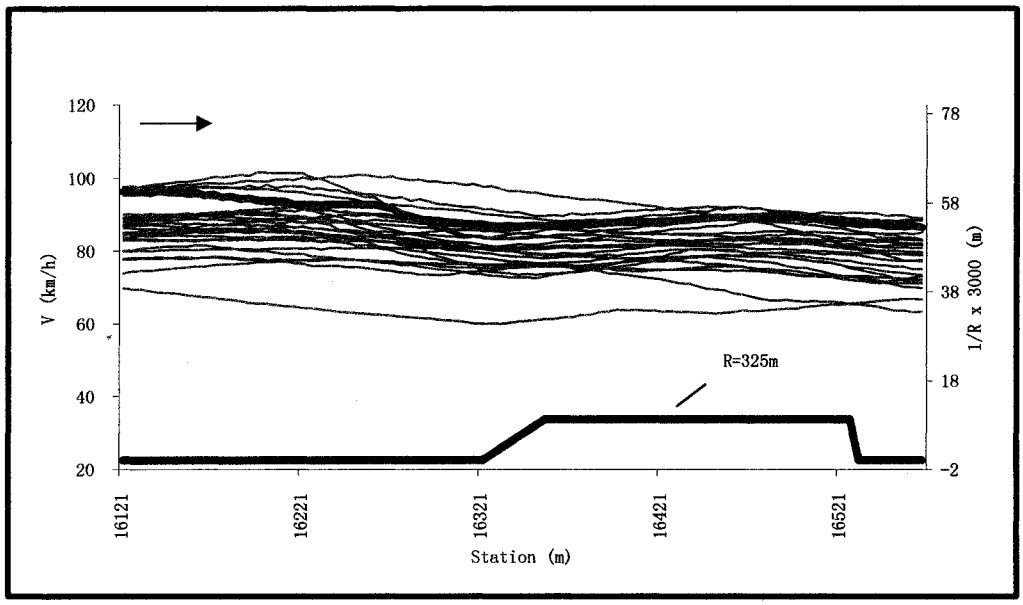

Figure A-3: Curve 3C6 on Regional Route 3 (Two-Lane Rural Highway).

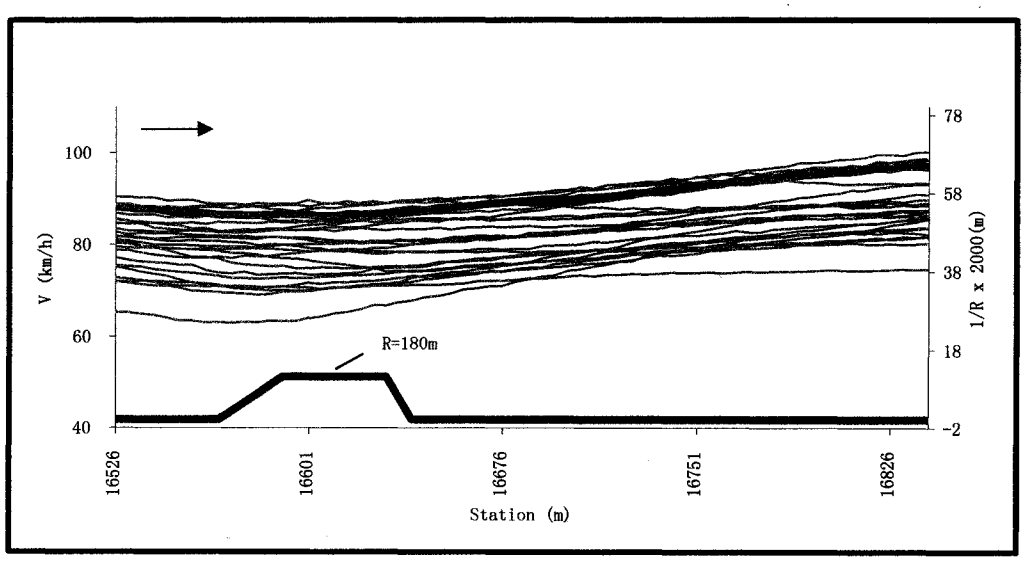

Figure A-4: Curve 3C7 on Regional Route 3 (Two-Lane Rural Highway).

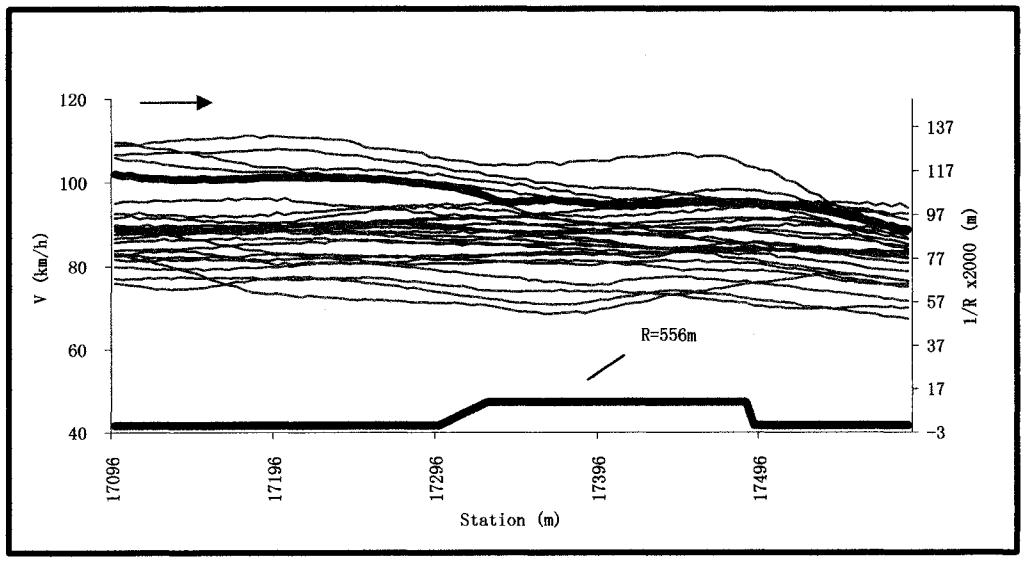

Figure A-5: Curve 3C8 on Regional Route 3 (Two-Lane Rural Highway).

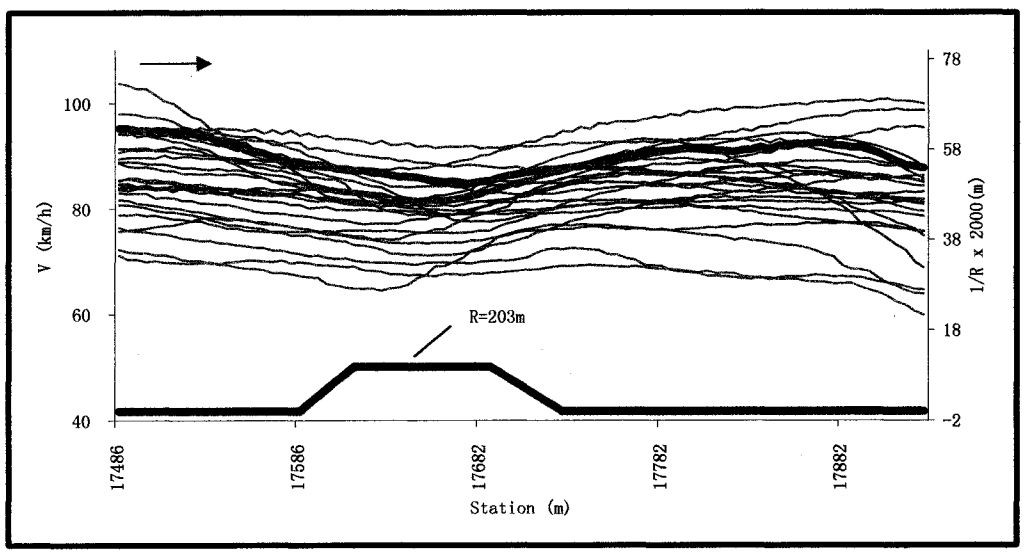

Figure A-6: Curve 3C9 on Regional Route 3 (Two-Lane Rural Highway).

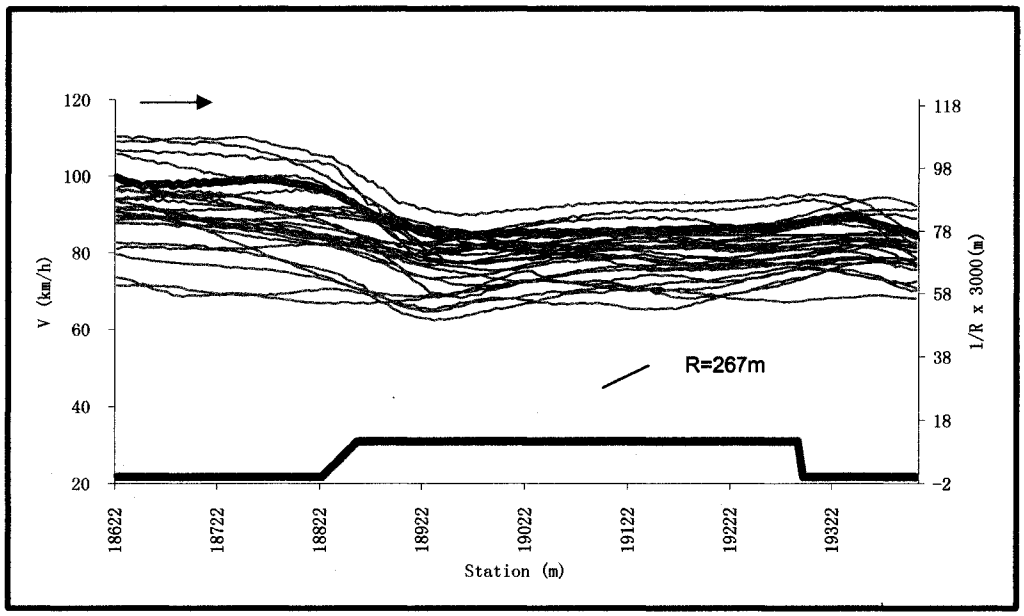

Figure A-7: Curve 3C10 on Regional Route 3 (Two-Lane Rural Highway).

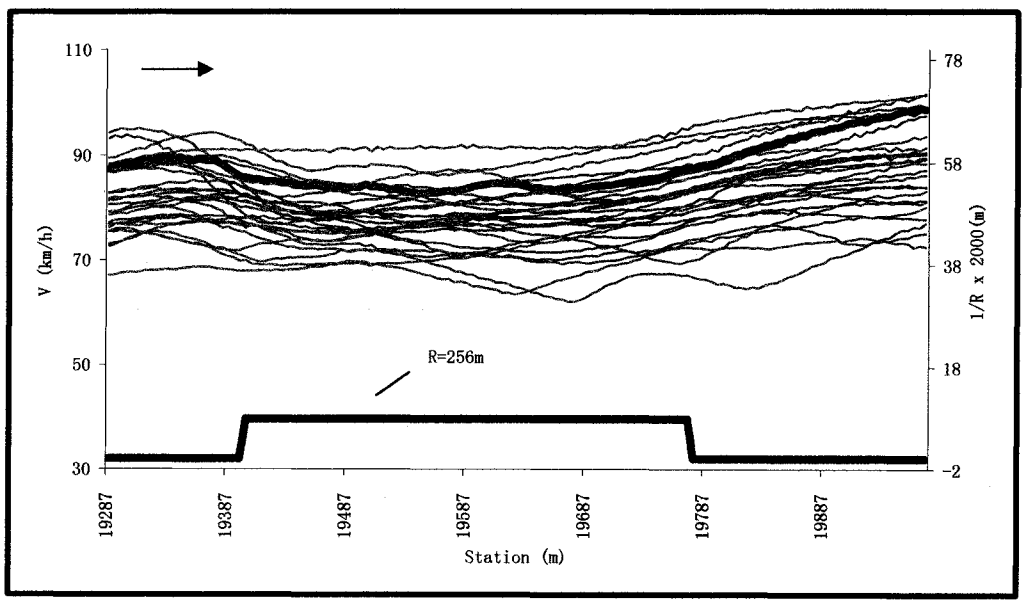

Figure A-8: Curve 3C11 on Regional Route 3 (Two-Lane Rural Highway).

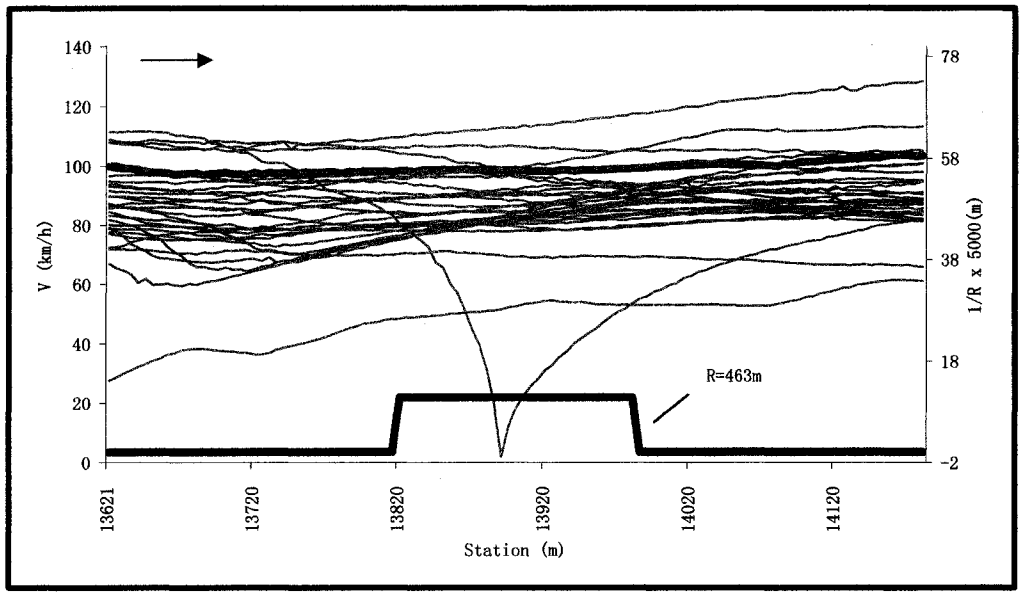

Figure A-9: Curve 6C1 on Regional Route 6 (Two-Lane Rural Highway).

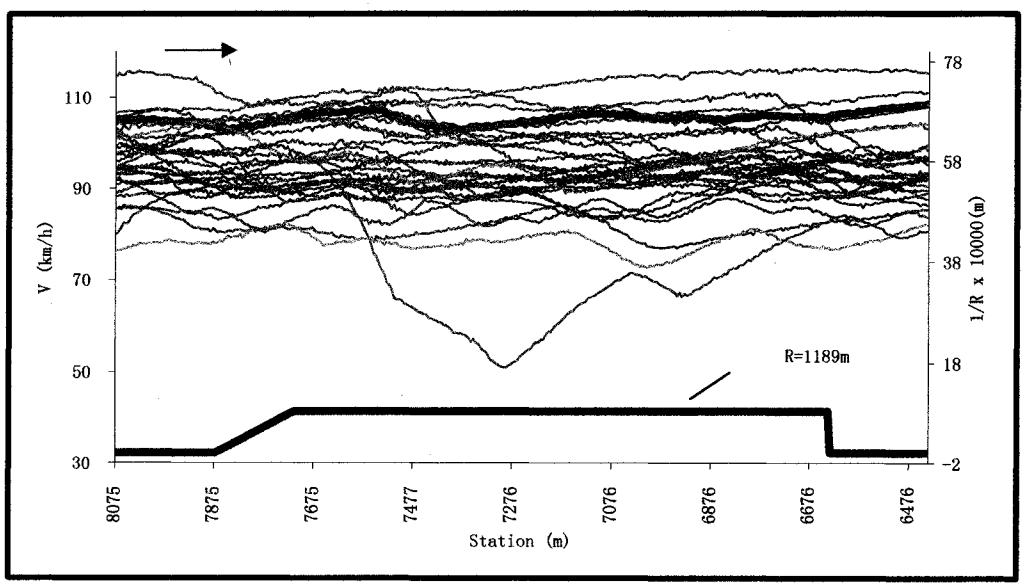

Figure A-10: Curve 7C1 on Provincial Route 7 (Two-Lane Rural Highway).

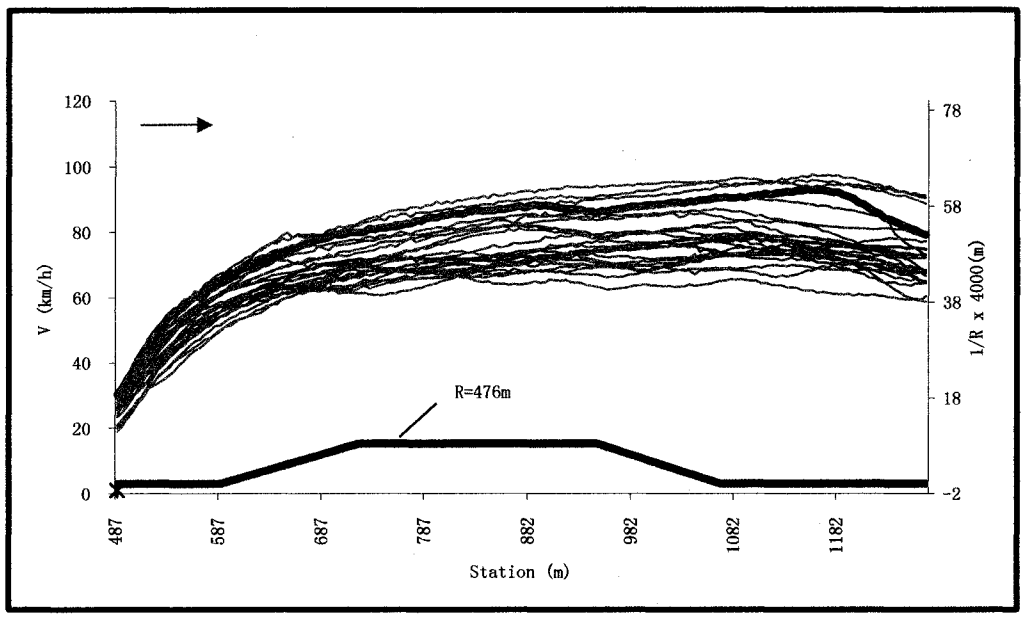

Figure A-11: Curve 12C1 on Regional Route 12 (Urban/Suburban Road).

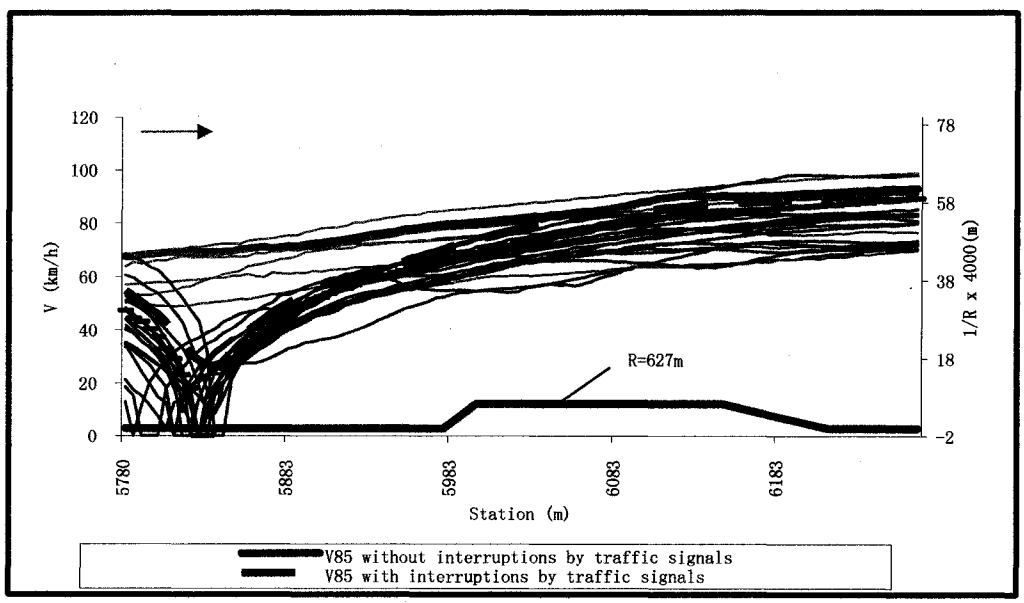

Figure A-12: Curve 12C2 on Regional Route 12 (Urban/Suburban Road).

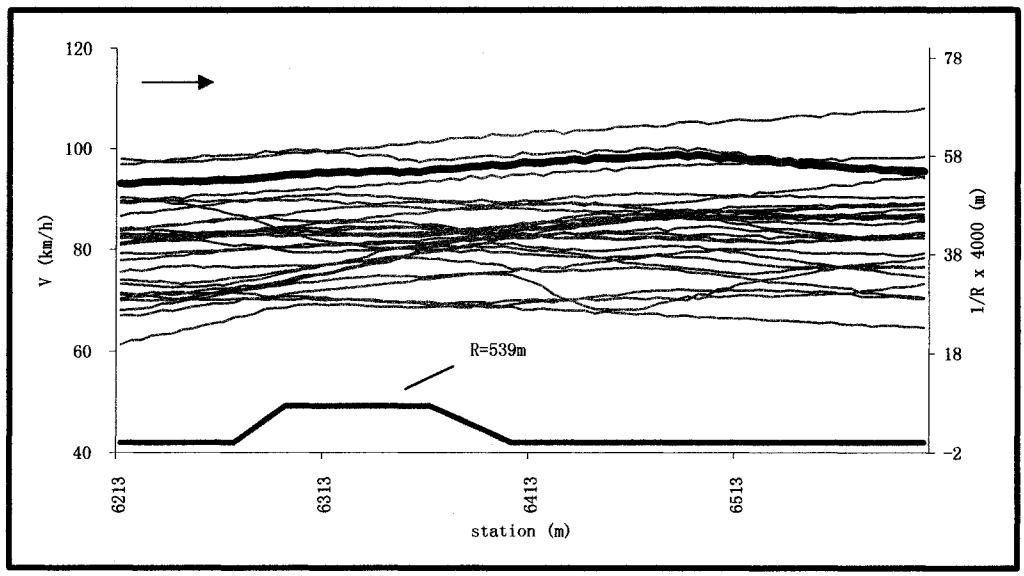

Figure A-13: Curve 12C3 on Regional Route 12 (Urban/Suburban Road).

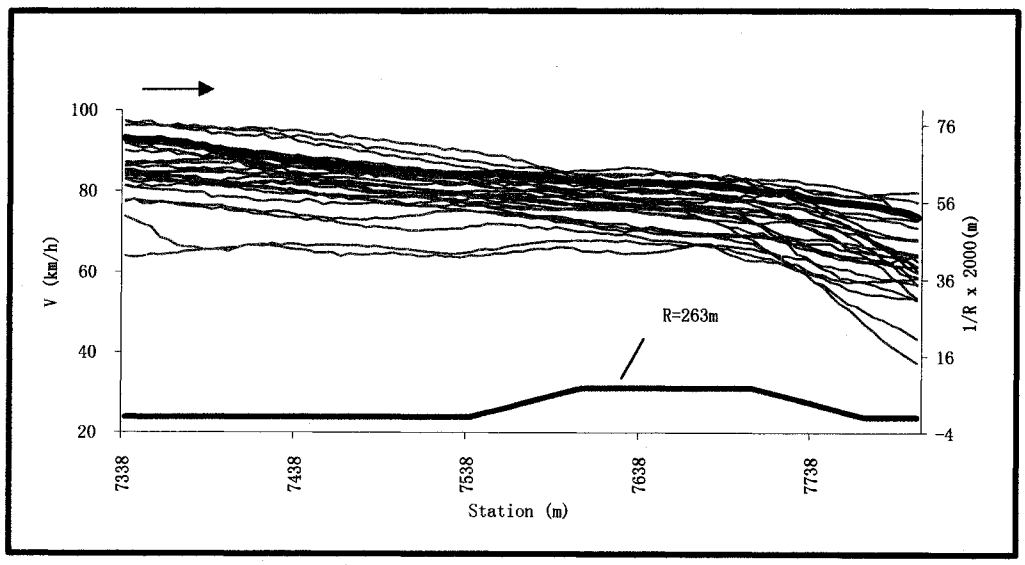

Figure A-14: Curve 12C4 on Regional Route 12 (Urban/Suburban Road).

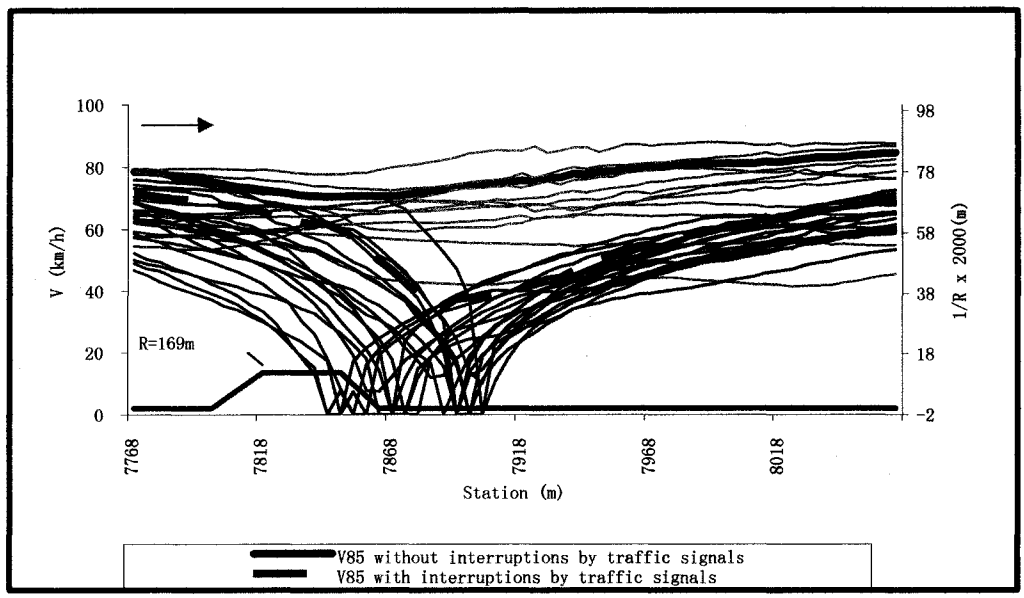

Figure A-15: Curve 12C5 on Regional Route 12 (Urban/Suburban Road).

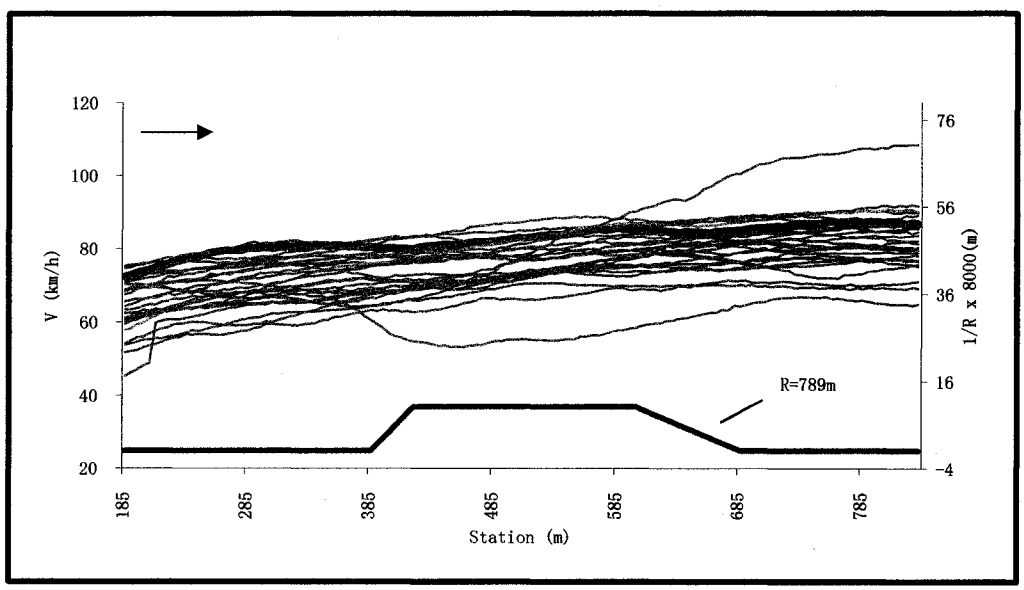

Figure A-16: Curve 73C1 on Regional Route 73 (Urban/Suburban Road).

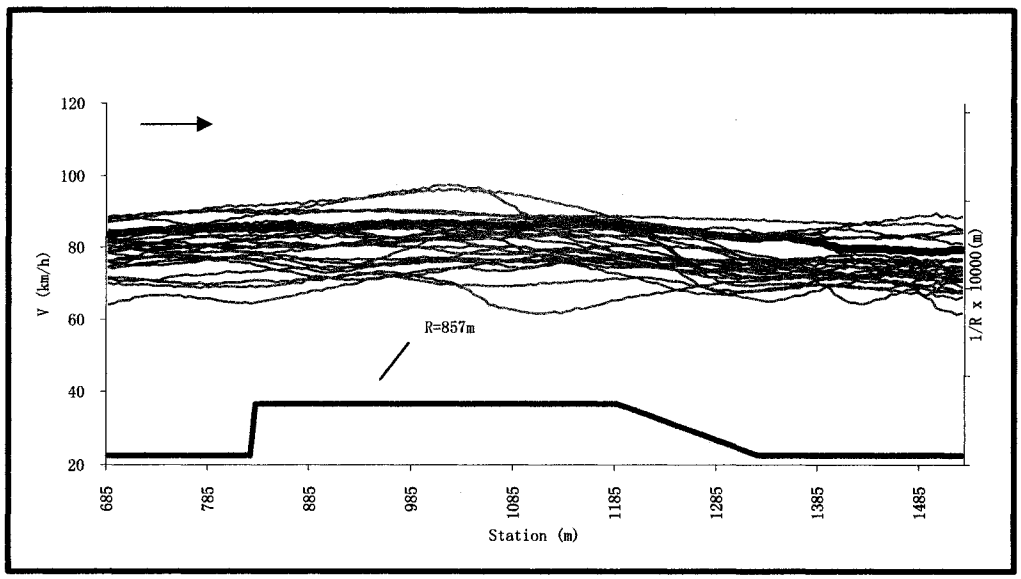

Figure A-17: Curve 73C2 on Regional Route 73 (Urban/Suburban Road).

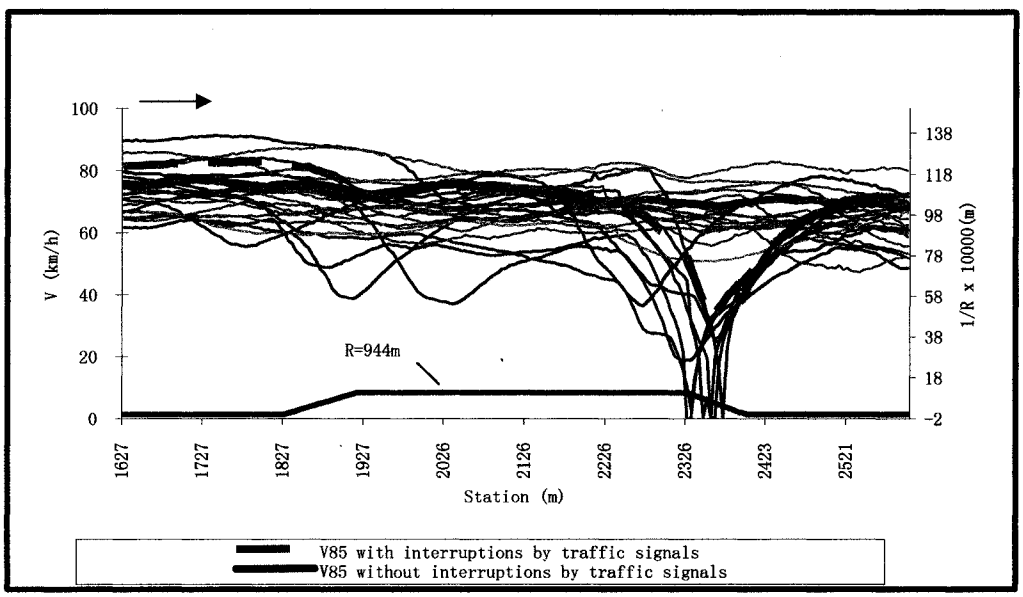

Figure A-18: Curve 73C3 on Regional Route 73 (Urban/Suburban Road).

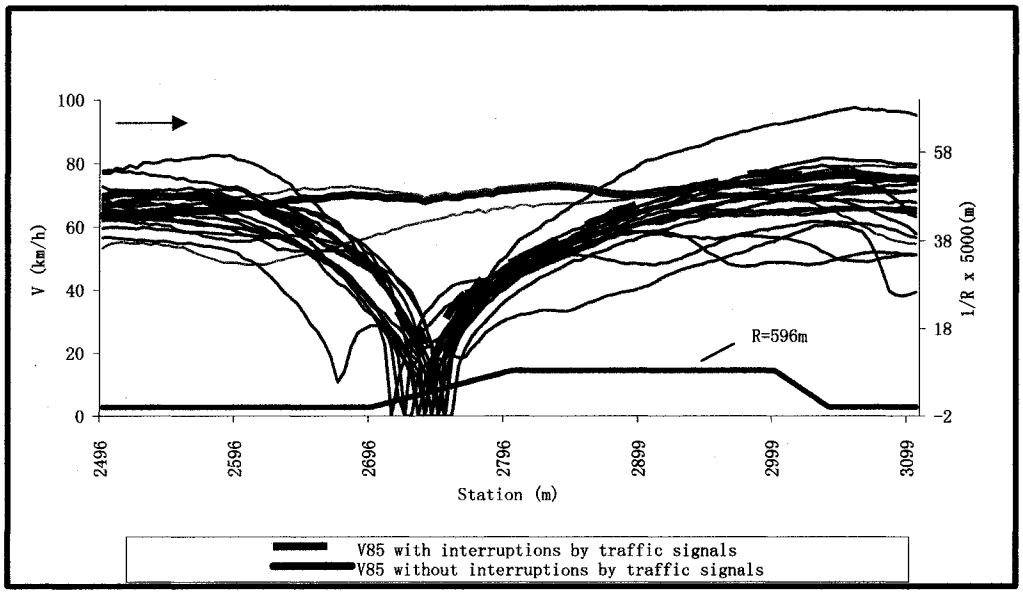

Figure A-19: Curve 73C4 on Regional Route 73 (Urban/Suburban Road).

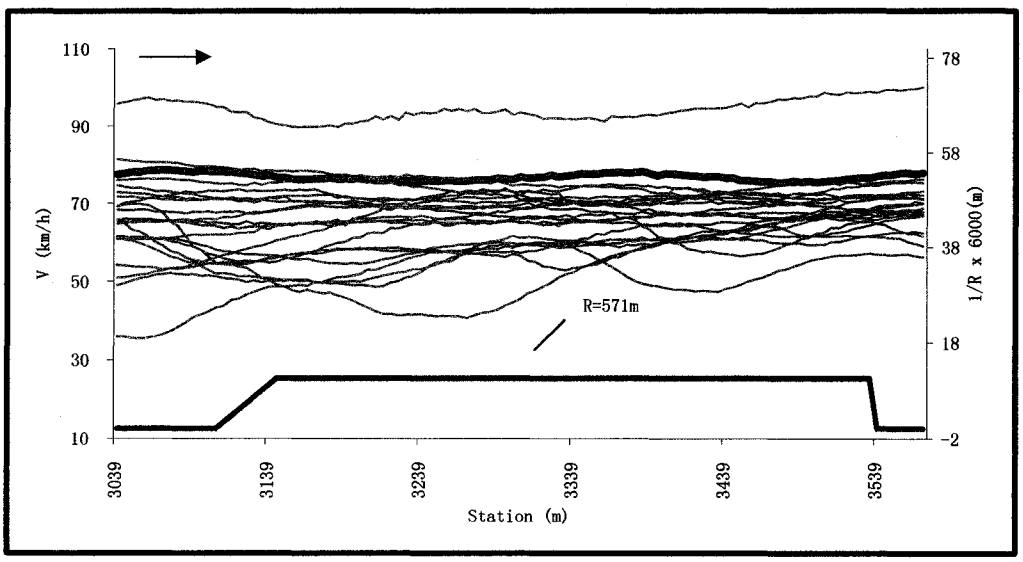

Figure A-20: Curve 73C5 on Regional Route 73 (Urban/Suburban Road).

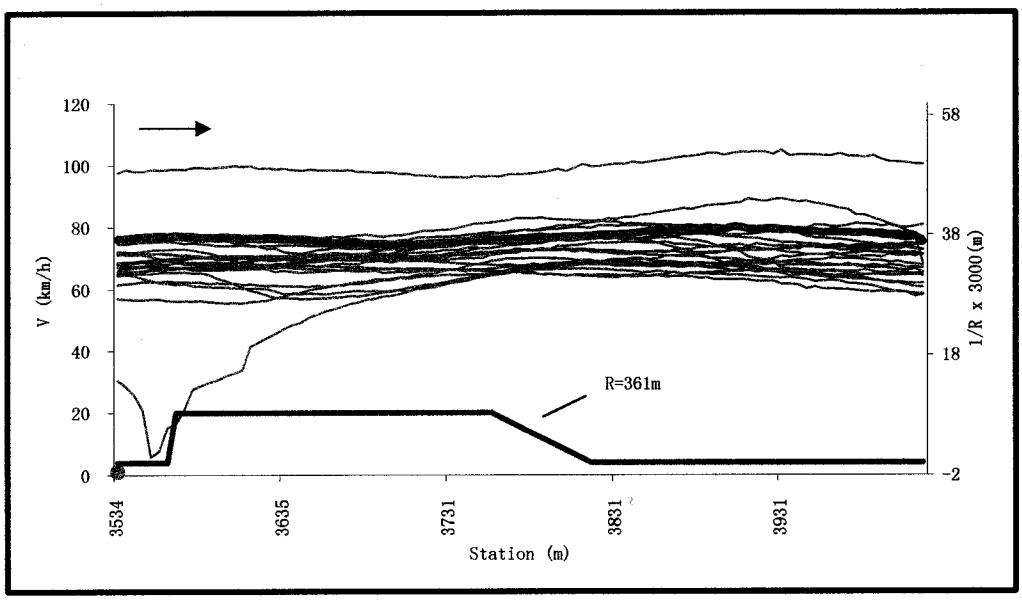

Figure A-21: Curve 73C6 on Regional Route 73 (Urban/Suburban Road).

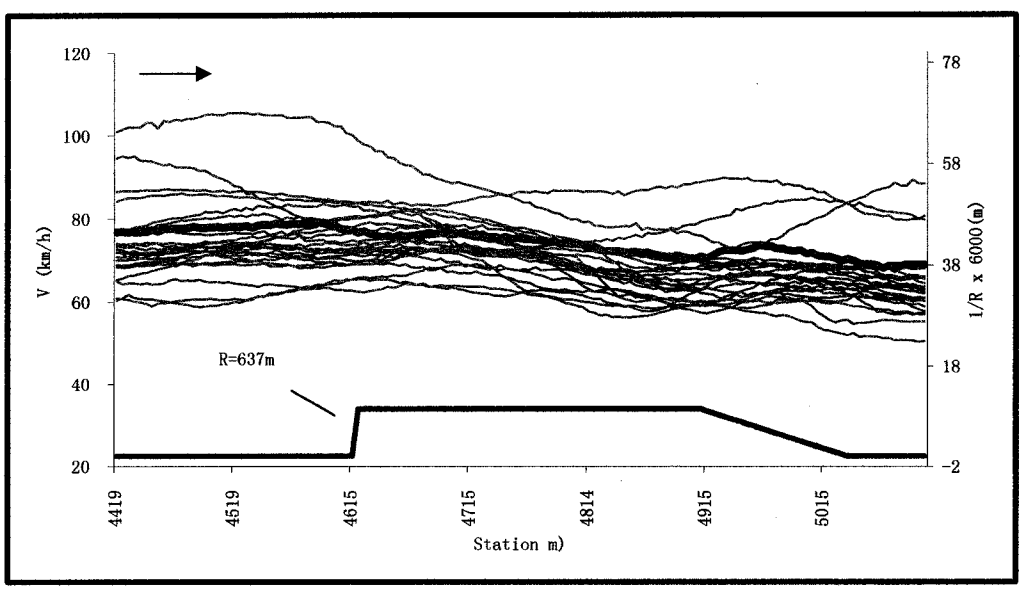

Figure A-22: Curve 73C7 on Regional Route 73 (Urban/Suburban Road).

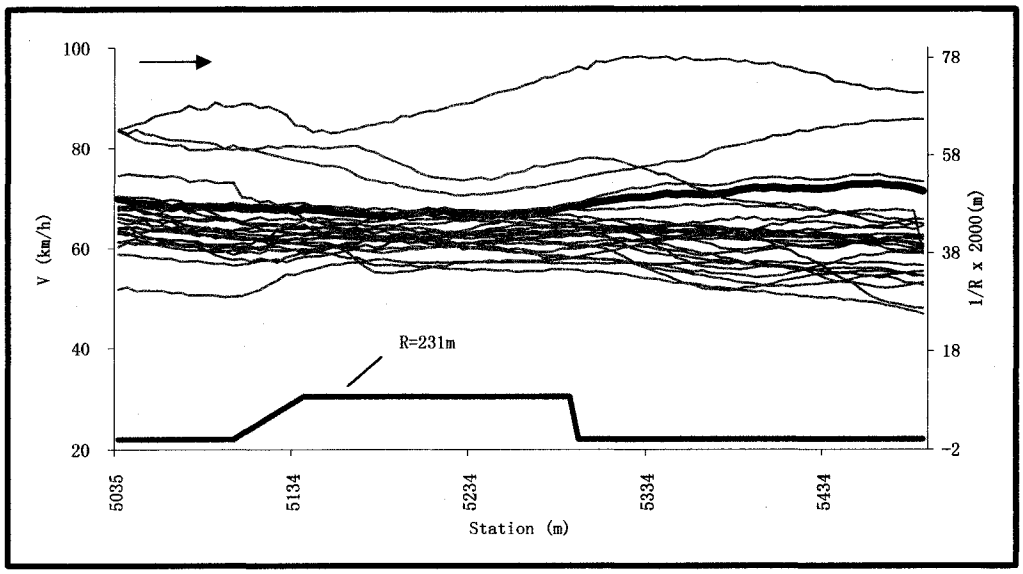

Figure A-23: Curve 73C8 on Regional Route 73 (Urban/Suburban Road).

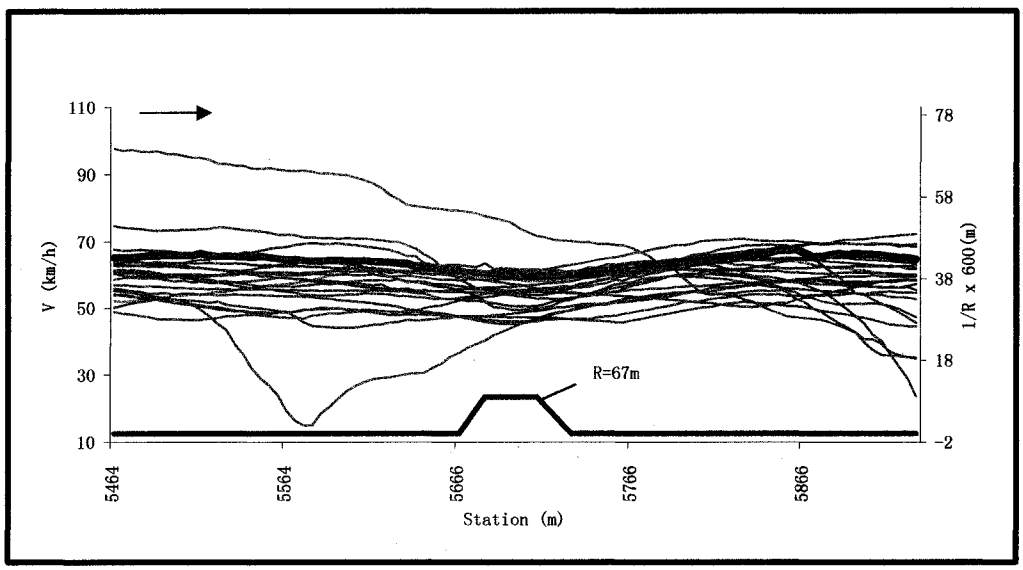

Figure A-24: Curve 73C9 on Regional Route 73 (Urban/Suburban Road).

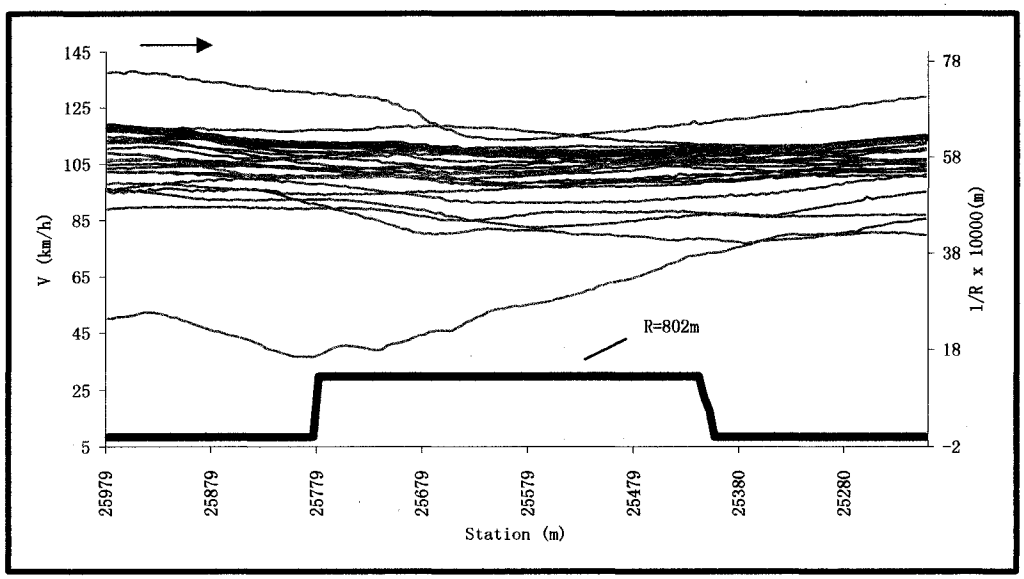

Figure A-25: Curve 417C1 on Highway 417 (Urban Freeway).

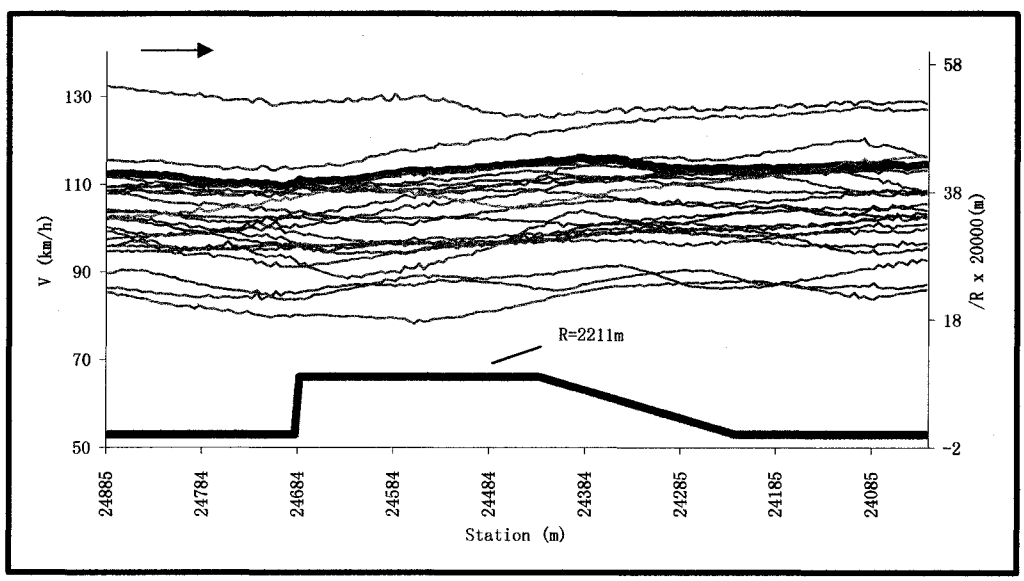

Figure A-26: Curve 417C2 on Highway 417 (Urban Freeway).

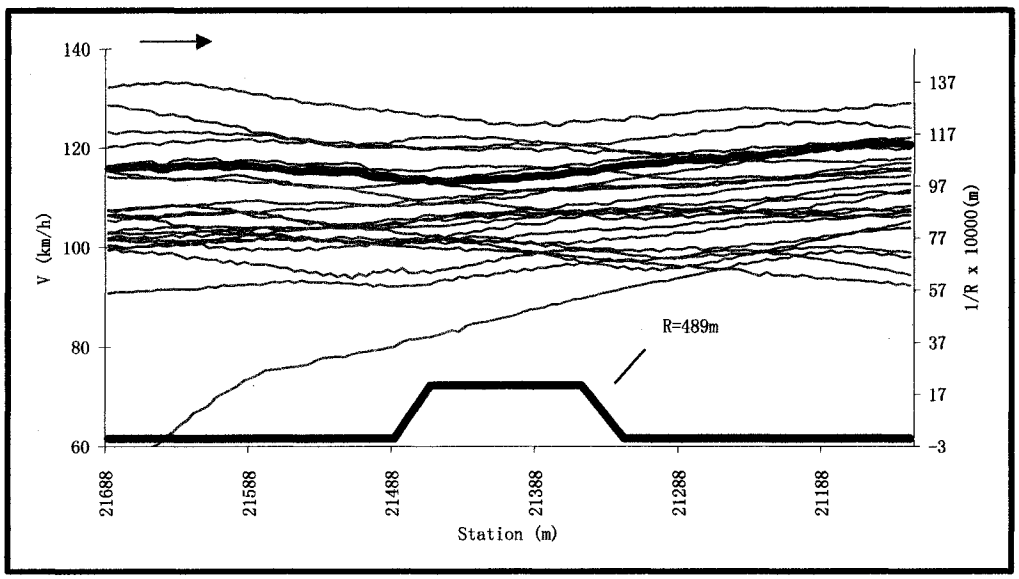

Figure A-27: Curve 417C3 on Highway 417 (Urban Freeway).

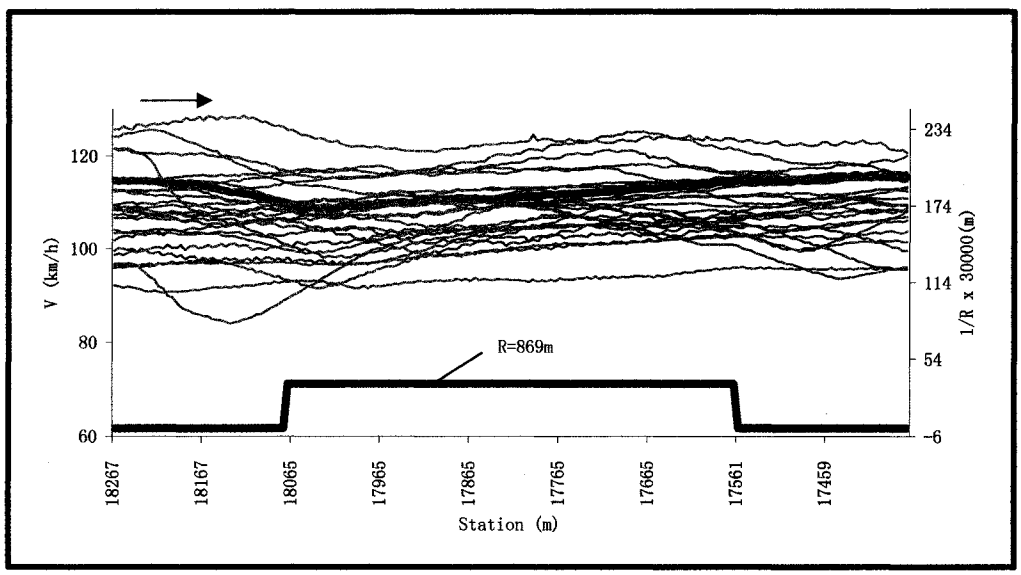

Figure A-28: Curve 417C4 on Highway 417 (Urban Freeway).

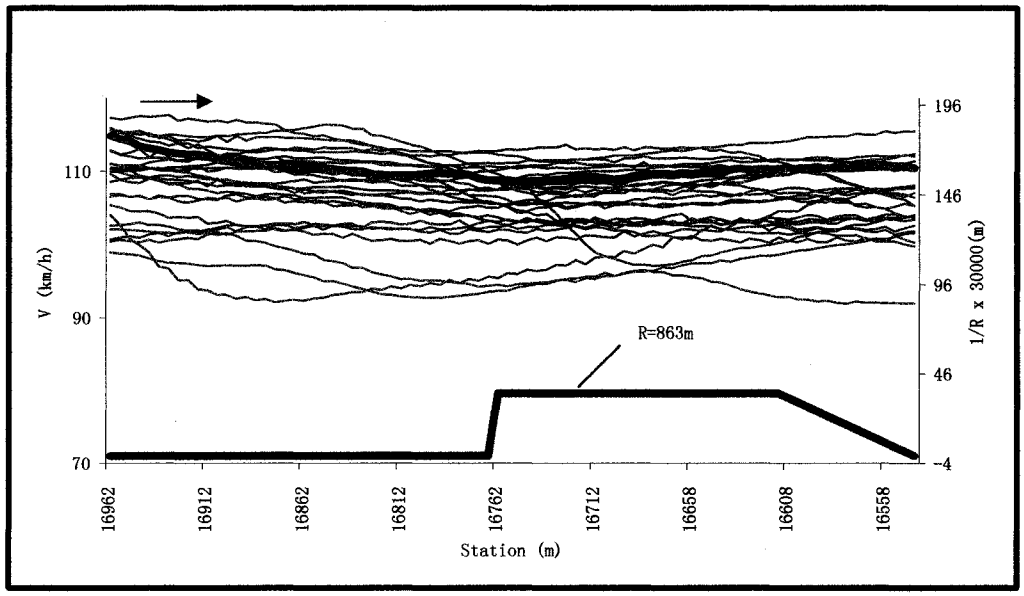

Figure A-29: Curve 417C5 on Highway 417 (Urban Freeway).

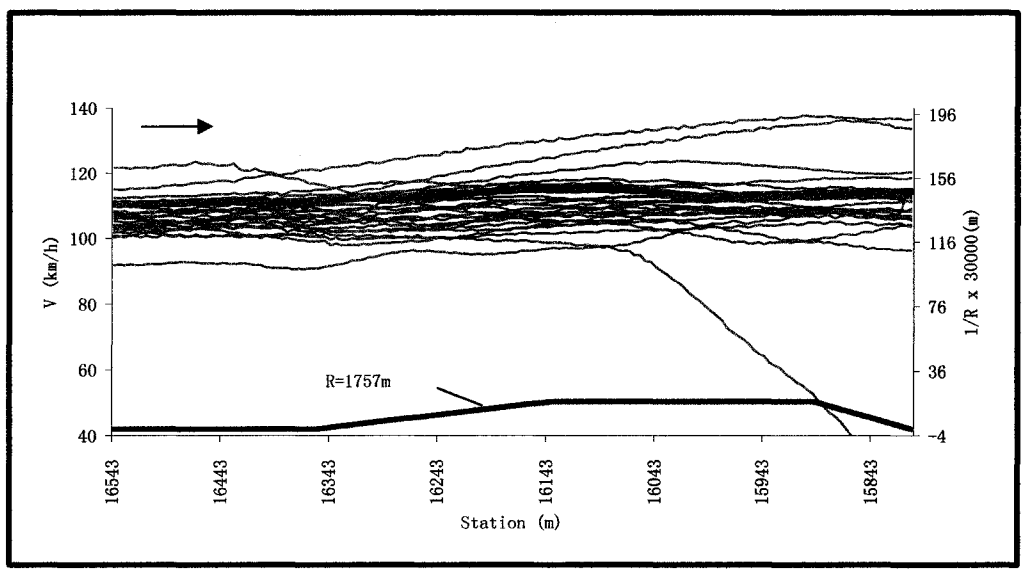

Figure A-30: Curve 417C6 on Highway 417 (Urban Freeway).

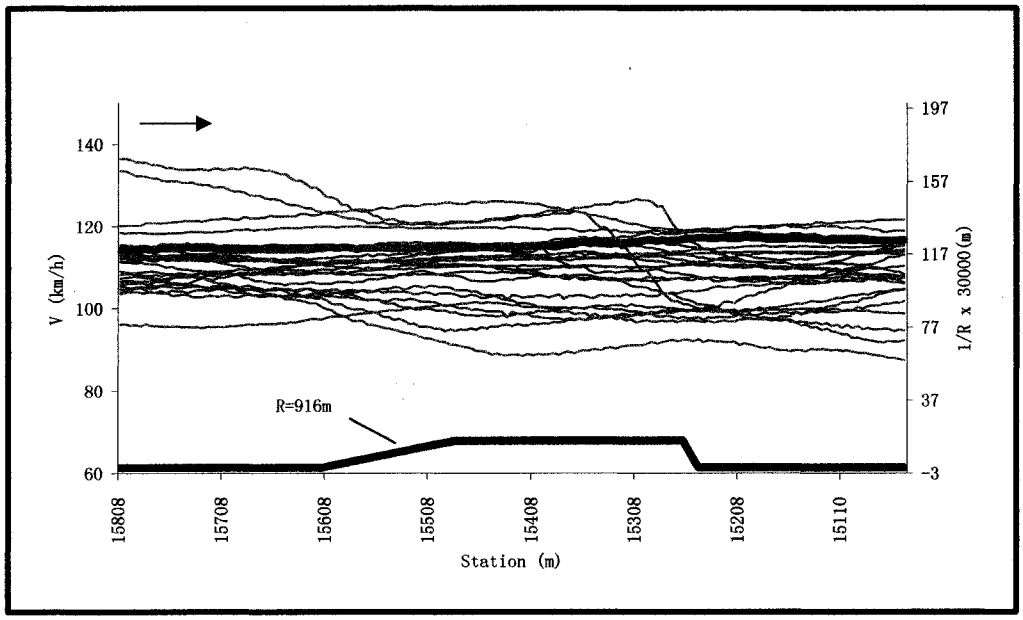

Figure A-31: Curve 417C7 on Highway 417 (Urban Freeway).

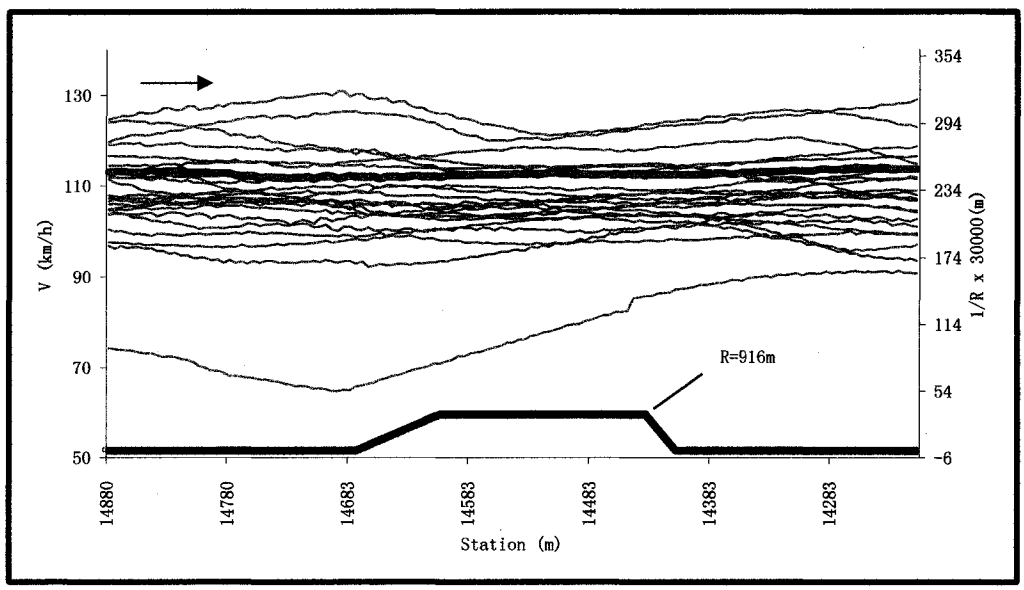

Figure A-32: Curve 417C8 on Highway 417 (Urban Freeway).

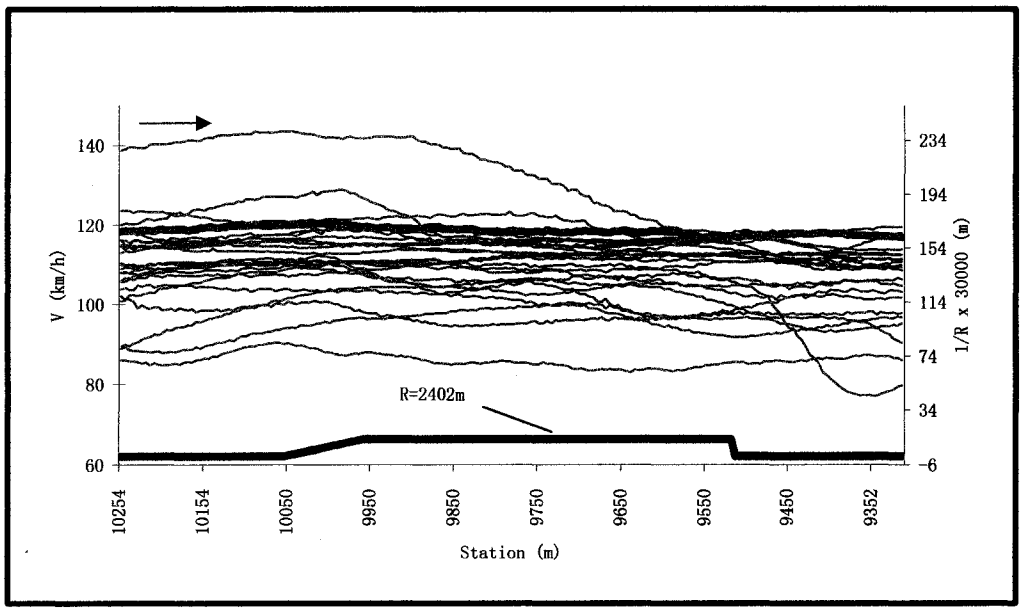

Figure A-33: Curve 417C9 on Highway 417 (Urban Freeway).

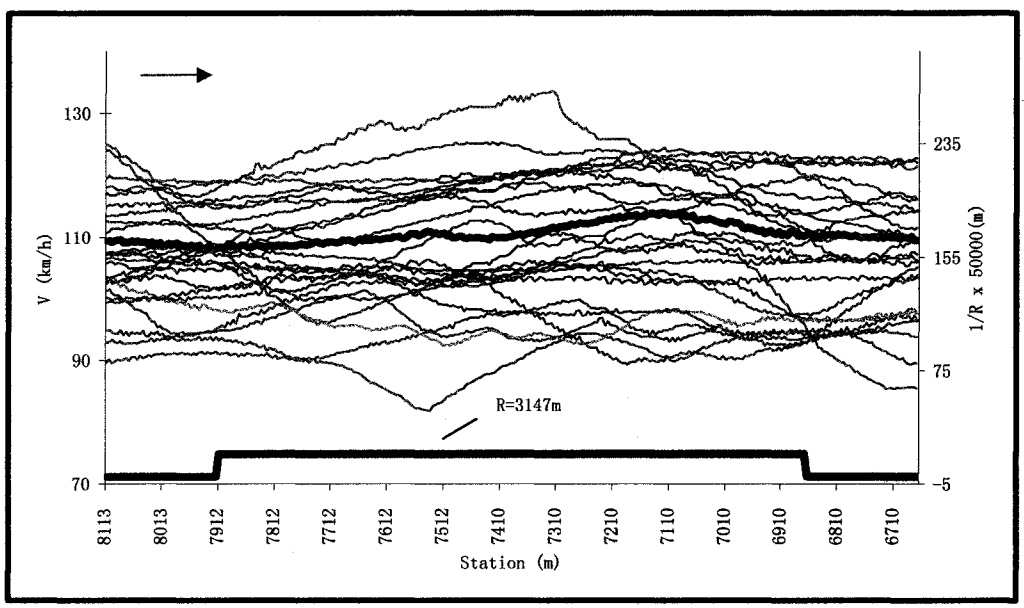

Figure A-34: Curve 417C10 on Highway 417 (Urban Freeway).

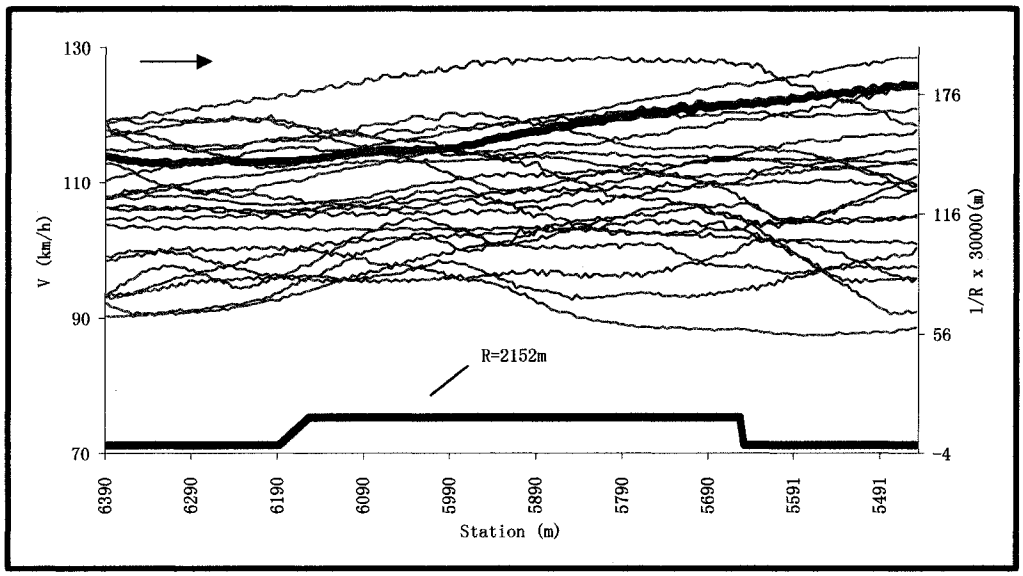

Figure A-35: Curve 417C11 on Highway 417 (Urban Freeway).

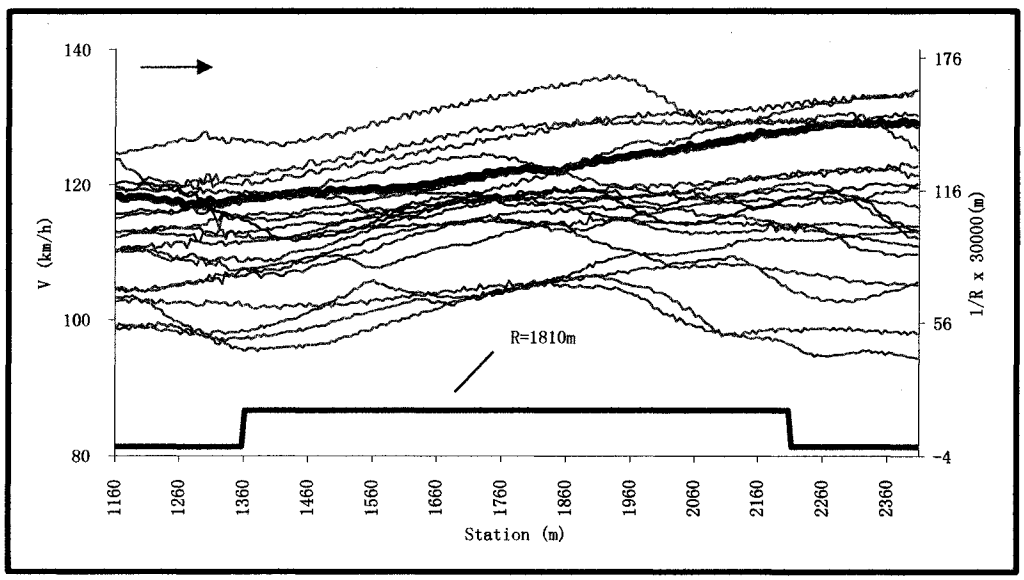

Figure A-36: Curve 416C1 on Highway 416 (Rural Freeway).

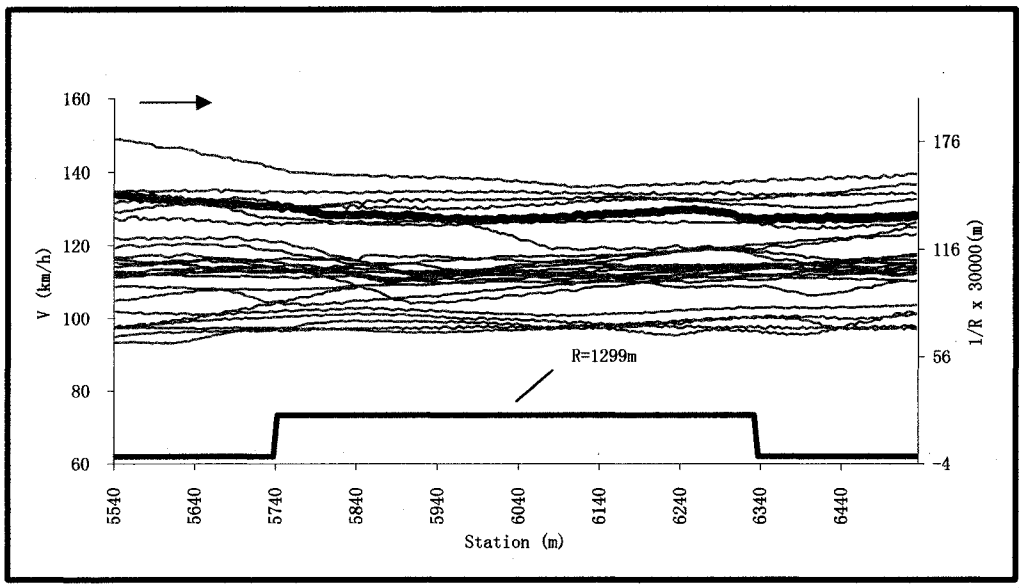

Figure A-37: Curve 416C2 on Highway 416 (Rural Freeway).

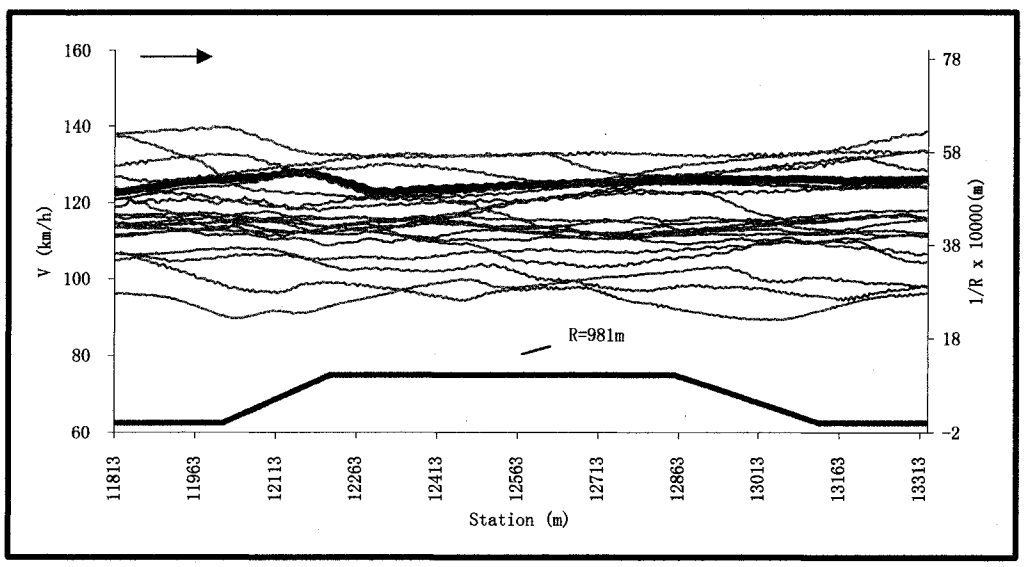

Figure A-38: Curve 416C3 on Highway 416 (Rural Freeway).

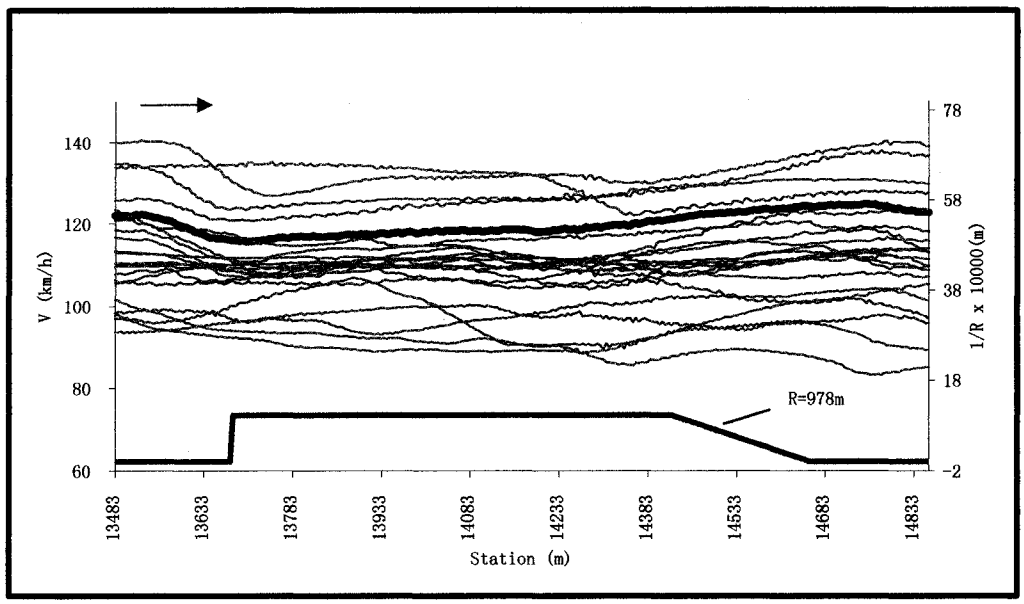

Figure A-39: Curve 416C4 on Highway 416 (Rural Freeway).

APPENDIX B: DEVELOPED MODELS AND SUMMARY OF STATISTICS FOR INDEPENDENT VARIABLES

Table B-1: Models for All Curves on Two-Lane Rural Highways.

Model No	Responder	Predictors	Coefficients	df	t	p-value	Adjusted R^2	SEE
1	V_{85_AT}	Constant L_{AT}	81.782 0.086	8	25.176 4.303	0.000 0.003	0.661	4.24
2	$V_{85\_PC/SC}$	Constant $L_n(r)$	22.973 11.690	8	1.740 5.163	0.120 0.001	0.740	3.84
3	$V_{85\_PC/SC}$	Constant CCR_s	108.132 -0.090	8	28.496 -4.839	0.000 0.001	0.714	4.03
4	V_{85_MC}	Constant CCR_s	108.357 -0.097	8	40.759 -7.510	0.000 0.000	0.860	2.83
5	$V_{85\_CS/PT}$	Constant $L_n(r)$	21.784 11.796	8	1.900 5.999	0.094 0.000	0.795	3.34
6	$V_{85\_CS/PT}$	Constant $L_n(r)$ L_{DT}	16.660 11.753 0.036	7	2.306 9.658 3.727	0.055 0.000 0.007	0.922	2.06
7	$V_{85\_CS/PT}$	Constant CCR_s	107.898 -0.091	8	33.627 -5.836	0.000 0.000	0.786	3.41
8	$V_{85\_CS/PT}$	Constant CCR_s L_{DT}	102.238 -0.092 0.039	7	48.164 -10.978 4.559	0.000 0.000 0.003	0.938	1.83
9	V_{85_DT}	Constant r^2	87.990 1.136E-05	8	42.645 3.073	0.000 0.015	0.484	5.61
10	V_{85_DT}	Constant r^2 L_{DT}	78.690 1.127E-05 0.066	7	34.723 4.730 4.676	0.000 0.002 0.002	0.857	2.95
11	ΔV_{85}	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*
12	$\Delta_{85}V_{(1-3)}$	Constant L_{DT}	16.521 -0.06	8	4.492 -2.674	0.002 0.028	0.406	4.80
13	*85MSR*	Constant L_{DT}	25.705 -0.088	8	6.038 -3.363	0.000 0.010	0.534	5.55
14	*85MSR*	Constant L_{DT} *DFC*	17.857 -0.080 7.324	7	4.028 -3.999 2.599	0.005 0.005 0.035	0.729	4.23
15	*85MSI*	Constant L_{DT}	-0.410 0.078	8	-0.156 4.862	0.880 0.001	0.715	3.42

Table B-2: Models for Independent Curves on Two-Lane Rural Highways.

Model No	Responder	Predictors	Coefficients	df	t	p-value	Adjusted R^2	SEE
1	V_{85_AT}	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*
2	$V_{85\_PC/SC}$	Constant *Ln(r)*	30.563 10.582	4	2.022 4.276	0.113 0.013	0.776	3.14
3	V_{85_MC}	Constant *(1/r)*	110.386 -6856.213	4	41.470 -7.090	0.000 0.002	0.908	2.48
4	$V_{85\_CS/PT}$	Constant *(1/r)*	111.404 -7360.698	4	32.031 -5.825	0.000 0.004	0.868	3.24
5	$V_{85\_CS/PT}$	Constant *(1/r)* L_{DT}	99.672 -5031.303 0.051	3	30.198 -6.191 4.048	0.000 0.008 0.027	0.973	1.47
6	V_{85_DT}	Constant L_{DT}	76.486 0.127	4	19.014 4.179	0.000 0.014	0.767	5.03
7	ΔV_{85}	Constant CCR_s	-5.326 0.073	4	-1.203 2.782	0.295 0.050	0.574	4.22
8	$\Delta_{85}V_{(1\text{-}3)}$	Constant CCR_s	-4.540 0.088	4	-1.214 3.989	0.291 0.016	0.749	3.56
9	*85MSR*	Constant CCR_s	-0.658 0.107	4	-0.149 4.104	0.889 0.015	0.760	4.22
10	*85MSI*	Constant L_{DT} *(1/r)*	-15.253 0.116 4000.429	3	-3.035 6.090 3.232	0.056 0.009 0.048	0.882	2.24

Table B-3: Models for Urban/Suburban Roads (Case 1).

Model No	Responder	Predictors	Coefficients	df	t	p-value	Adjusted R^2	SEE
1	V_{85_AT}	Constant V_p	23.686 0.807	5	1.634 3.858	0.163 0.012	0.698	5.48
2	$V_{85\_PC/SC}$	Constant *DFC* V_p	60.643 -28.819 0.54	4	3.856 -3.593 3.037	0.018 0.023 0.039	0.872	4.07
3	V_{85_MC}	Constant *DFC* V_p	54.738 -29.031 0.618	4	4.546 -4.727 4.541	0.010 0.009 0.010	0.931	3.12
4	V_{85_MC}	Constant *DFC* V_p CCR_s	55.859 -21.522 0.570 -0.014	3	8.429 -5.235 7.466 -3.200	0.004 0.014 0.005 0.049	0.979	1.71
5	$V_{85\_CS/PT}$	Constant *DFC* V_p	55.220 -30.113 0.613	4	4.328 -4.627 4.249	0.012 0.010 0.013	0.926	3.30
6	V_{85_DT}	Constant *DFC*	102.171 -37.630	5	17.406 -4.642	0.000 0.006	0.774	4.71
7	ΔV_{85}	Constant *DFC*	-10.671 17.309	5	-5.415 6.362	0.003 0.001	0.868	1.58
8	ΔV_{85}	Constant *DFC* L_{DT}	-6.991 15.963 -0.018	4	-5.125 11.268 -3.947	0.007 0.000 0.017	0.966	0.80
9	ΔV_{85}	Constant *DFC* L_{DT} V_p	-0.390 14.172 -0.021 -0.071	3	-0.365 28.830 15.019 -6.664	0.739 0.000 0.001 0.007	0.997	0.23
10	$\Delta_{85}V_{(1\text{-}3)}$	Constant *DFC*	-4.386 14.130	5	-1.791 4.177	0.133 0.009	0.733	1.96
11	*85MSR*	Constant L_{DT}	24.401 -0.080	5	6.916 -3.896	0.001 0.011	0.703	3.80
12	*85MSR*	Constant L_{DT} *DFC*	13.605 -0.070 13.352	4	4.022 -6.347 3.800	0.016 0.003 0.019	0.919	1.98
13	*85MSI*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*

Table B-4: Models for Urban/Suburban Roads (Case 2).

Model No	Responder	Predictors	Coefficients	df	t	p-value	Adjusted R^2	SEE
1	V_{85_AT}	Constant V_p INT	40.506 0.579 -7.944	9	4.730 4.629 -2.994	0.001 0.001 0.015	0.662	4.05
2	V_{85_AT}	Constant V_p INT r_1/r_2	53.432 0.483 -9.085 -3.806	8	8.665 6.008 -5.502 -3.994	0.000 0.000 0.001 0.004	0.873	2.48
3	$V_{85\_PC/SC}$	Constant r_1/r_2 INT V_p	58.397 -6.691 -10.741 0.458	8	10.158 -7.531 -6.977 6.113	0.000 0.000 0.000 0.000	0.927	2.32
4	V_{85_MC}	Constant r_1/r_2 V_p INT	53.358 -6.215 0.499 -8.262	8	6.276 -4.730 4.502 -3.629	0.004 0.001 0.002 0.007	0.841	3.43
5	$V_{85\_CS/PT}$	Constant CCR_s V_p	52.675 -0.026 0.400	9	4.140 -2.601 2.341	0.003 0.029 0.044	0.587	5.63
6	V_{85_DT}	Constant DFC	95.586 -31.126	10	17.758 -3.815	0.000 0.003	0.552	5.80
7	ΔV_{85}	Constant $Ln(r)$ L	40.141 -7.363 0.022	9	4.154 -3.803 2.436	0.002 0.004 0.038	0.553	2.92
8	$\Delta_{85}V_{(1-3)}$	Constant DFC	-3.788 16.153	10	-0.914 2.572	0.382 0.028	0.338	4.47
9	$85MSR$	Constant L_{AT}	4.541 0.072	10	0.970 2.308	0.355 0.044	0.282	7.78
10	$85MSI$	Constant INT V_p L_{AT}	30.441 10.886 -0.271 -0.029	8	5.733 6.631 -3.605 -2.869	0.000 0.000 0.007 0.021	0.803	2.43

Table B-5: Models for Urban/Suburban Roads (Case 3).

Model No	Responder	Predictors	Coefficients	df	t	p-value	Adjusted R^2	SEE
1	V_{85_AT}	Constant V_p	35.742 0.649	6	3.088 3.826	0.021 0.009	0.661	4.65
2	V_{85_AT}	Constant V_p r_1/r_2	49.709 0.546 -4.121	5	8.715 7.147 -5.179	0.000 0.001 0.004	0.936	2.02
3	$V_{85\_PC/SC}$	Constant $Ln(r)$ V_p	-2.239 8.080 0.486	5	-0.234 5.426 4.179	0.825 0.003 0.009	0.908	2.99
4	V_{85_MC}	Constant DFC	102.210 -36.817	6	13.091 -3.432	0.000 0.014	0.606	6.26
5	V_{85_MC}	Constant DFC V_p	58.589 -24.797 0.522	5	3.941 -3.152 3.114	0.011 0.025 0.026	0.839	4.00
6	V_{85_MC}	Constant DFC V_p r_1/r_2	53.710 -10.464 0.559 -4.495	4	16.445 -4.710 15.269 -10.097	0.000 0.009 0.000 0.001	0.992	0.87
7	V_{85_MC}	Constant DFC V_p r_1/r_2 $1/r$	53.760 -10.355 0.548 -3.530 -257.933	3	33.799 -9.567 30.364 -10.448 -3.724	0.000 0.002 0.000 0.002 0.034	0.998	0.42
8	V_{85_MC}	Constant DFC V_p r_1/r_2 $1/r$ L_{DT}	51.744 -8.423 0.557 -3.932 -260.500 0.005	2	109.719 -22.445 124.036 -39.895 -15.729 7.107	0.000 0.002 0.000 0.001 0.004 0.019	1.000	0.10
9	$V_{85\_CS/PT}$	Constant DFC V_p CCR_s	60.494 -14.150 0.435 -0.018	4	6.922 -2.548 4.399 -3.284	0.002 0.063 0.012 0.030	0.938	2.35
10	V_{85_DT}	Constant DFC	99.525 -34.274	6	18.872 -4.729	0.000 0.003	0.753	4.23
11	ΔV_{85}	Constant DFC	-5.910 12.861	6	-2.235 3.540	0.067 0.012	0.622	2.12
12	ΔV_{85}	Constant DFC L_{AT}	-6.280 9.418 0.023	5	-3.388 3.313 2.698	0.020 0.021 0.043	0.815	1.48
13	ΔV_{85}	Constant DFC L_{AT} r^2	-2.885 5.541 0.029 -5.240E-06	4	-1.677 2.385 4.802 -2.778	0.169 0.076 0.009 0.050	0.921	0.97

Table B-5: Models for Urban/Suburban Roads (Case 3, Cont'd).

14	ΔV_{85}	Constant *DFC* L_{AT} r^2 V_p	3.941 2.939 0.034 -5.140E-06 -0.083	3	2.172 2.432 11.545 -6.125 -4.150	0.118 0.093 0.001 0.009 0.025	0.984	0.43
15	$\Delta_{85}V_{(1-3)}$	Constant *DFC*	-5.046 16.434	6	-1.944 4.607	0.100 0.004	0.743	2.08
16	$\Delta_{85}V_{(1-3)}$	Constant *DFC* L_{AT}	-5.405 13.088 0.023	5	-2.928 4.622 2.632	0.033 0.006 0.046	0.871	1.48
17	*85MSR*	Constant L_{AT}	1.559 0.100	6	0.549 4.860	0.603 0.003	0.764	3.98
18	*85MSI*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*

Table B-6: Models for Freeways.

Model No	Responder	Predictors	Coefficients	df	t	p-value	Adjusted R^2	SEE
1	V_{85_AT}	Constant *DFC*	111.023 11.417	11	42.123 2.207	0.000 0.046	0.217	5.45
2	V_{85_DT}	Constant *L*	111.101 0.013	11	43.780 3.278	0.000 0.006	0.410	4.44
3	ΔV_{85}	Constant CCR_s	-3.583 0.082	11	-2.789 3.419	0.015 0.005	0.433	2.27
4	*85MSR*	Constant *DFC*	5.003 6.929	11	4.478 3.160	0.001 0.008	0.319	2.31