DEVELOPMENT OF THE INTERRELATIONSHIP BETWEEN DRIVER BEHAVIOUR AND HIGHWAY DESIGN AND SAFETY

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

Doctor of Philosophy
in Civil Engineering

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario

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The Doctor of Philosophy in Civil Engineering is a joint program with the University of
Ottawa, administered by the Ottawa-Carleton Institute for Civil Engineering

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ISBN: 978-0-494-43909-8

Our file Notre référence

ISBN: 978-0-494-43909-8

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ACKNOWLEDGEMENT

The author would like to express her deepest and sincerest appreciation to her two co-supervisors Professor Abd El Halim O. Abd El Halim and Professor Yasser Hassan. Professor Abd El Halim, Chair of Department of Civil and Environmental Engineering, has been more than just a supervisor. He is her role model for a successful researcher in the field and a cornerstone in her professional development. His valuable support and guidance throughout the completion of this research has been invaluable. His time, efforts, and patience are highly appreciated.

Professor Yasser Hassan has provided immense support, encouragement, and guidance throughout the thesis. His time, efforts, and patience are highly appreciated. Working under his supervision has helped her gain from his wide experience in the field. His constructive supervision had a remarkable influence on her research abilities as well as her professional development.

In addition, support by the Natural Sciences and Engineering Council (NSERC) is gratefully acknowledged. The author is also grateful to the financial support provided by the Transportation Association of Canada (TAC). The author is thankful for the support provided by Ontario Graduate Scholarship (OGS). Equipment used in this research was made available through funding by the Canada Foundation for Innovation (CFI) and Ontario Innovation Trust (OIT).

The author would also like to thank the staff members at the Civil and Environmental Engineering Laboratory: Ken McMartin, Stanley Conley, and Jim Whitehorne, whom have helped in setting up the test vehicle. In addition, the alignment

data required for this study was provided by Stephen Perkins, from the Mapping Department of the City of Ottawa and is gratefully acknowledged.

Finally, the author would like to thank the team members of this experiment, Bin Nie and Mostafa Said, who without their contribution and efforts, the experiment would not have been completed with such precision and in a timely manner. The author would also like to thank the volunteer drivers who drove the test vehicle and enabled the driver behaviour data collection.

DEDICATION

To who supported me with unconditional love and patience, my husband Mustafa Elkhedr Ahmed, I dedicate this thesis to you. I also dedicate it to my dearest son Muhammad, my mother Nagwa Al-Arabi, and my father Professor Galal Said.

Thank you.

ABSTRACT

The major goal of any transportation system is the safe and efficient movement of people and goods. In the case of the highway infrastructure, there are three components to this transportation system; the road, the vehicle, and the driver. The interaction between the characteristics of these three components controls the level of safety on roads. Therefore, the effective provision of safety on highways should aim jointly at these three components. There has been a great evolution in the characteristics of two of these components; the vehicle and the driver. However, geometric design of roads has not been updated enough to be integrated with these two components.

To date, geometric elements are designed based on assumptions and criteria studied and verified over 60 years ago. Therefore, the objective of this research is to relate driver behaviour to geometric characteristics of the highway and to safety. It is aimed at improving the current geometric guides to accommodate the driver behaviour in a quantitative manner. This is accomplished through focusing on the road user as a main element in highway design.

To fulfill this objective, an experiment was designed to collect driver behaviour data using an instrumented test vehicle. Thirty volunteers were recruited to drive the vehicle on a route consisting of different functional classes of highways. The data collected from these instruments along with the geometric alignment of the highways traversed formed a large database, which was used to evaluate the interaction between driver behaviour and geometric characteristics of the road. The models developed were in turn used to develop updates to the current geometric design guides. These driver behaviour models were then used to relate driver behaviour to safety on roads.

The importance of this research lies in its economic and social impacts. Economic impacts can be realized in terms of reduced property loss, large savings in terms of medical expenditure and insurance claims, and significant reduction in lost working days due to injury or loss of earning power due to death. The social benefits of improving highway safety, in terms of reduced fatalities and injuries are even more significant.

TABLE OF CONTENTS

Chapter 1. Introduction	1
1.1 Background.....	1
1.2 Problem Definition	5
1.3 Objectives	6
1.4 Research Plan	6
1.5 Thesis Outline.....	8
Chapter 2. Literature Review	10
2.1 Introduction	10
2.2 The Driving Task.....	11
2.2.1. Control.....	13
2.2.2. Guidance	14
2.2.3. Navigation	14
2.3 The Information System	14
2.3.1. Information handling.....	15
2.3.2. Primacy	17
2.3.3. Reaction Time	17
2.4 Driver Expectancy	18
2.4.1. A-Priori Expectancies	18
2.4.2. Ad-hoc Expectancies.....	19
2.5 Driver Errors.....	19
2.5.1. Errors due to Driver Deficiencies	20
2.5.2. Errors due to Design.....	20
2.6 Driver Workload and Mental Stress Concept.....	21

2.6.1. Definition of Workload	21
2.6.2. Yerkes-Dodson Law.....	22
2.6.3. Driver Workload Homeostasis	22
2.6.4. Methods of Measuring Workload	24
2.6.4.a. Subjective Methods.....	24
2.6.4.b. Physiological Methods.....	25
2.6.4.c. Visual Occlusion Method.....	26
2.6.4.d. Primary Task Performance	27
2.6.4.e. Secondary Task Performance.....	28
2.7 Current Practices of Horizontal Curve Design in North America.....	29
2.7.1. Curve Radius.....	29
2.7.2. Spiral Curves.....	32
2.7.2.a. Importance of Spiral Curves	32
2.7.2.b. Current Practice in North American Guides.....	33
2.8 Procedures for Evaluation of Geometric Design Consistency	35
2.8.1. Operating Speed	36
2.8.1.a. Operating Speed Safety Criteria.....	36
2.8.1.b. Models for Predicting Speeds on Curves.....	39
2.8.1.c. Models for Predicting Speeds on Tangents.....	44
2.8.1.d. Acceleration and Deceleration Rates	45
2.8.2. Vehicle Dynamics	49
2.9 Driver Workload as a Consistency Evaluation Criterion	51
2.9.1. Subjective Methods (Messer 1980).....	51

2.9.2. Physiological Method (Heger 1995)	54
2.9.3. Visual Occlusion Method (Krammes et al. 1995, Fitzpatrick et al. 1999).....	55
2.9.4. Primary Task Method (Cafiso et al. 2004, 2005).....	60
2.10 Relationship between Design Consistency Measures and Safety	62
2.10.1. Relating Different Consistency Measures and Safety	63
2.10.2. Relating Driver Workload and Collision Experience	69
2.11 Consideration of Design Consistency in North American Design Guides.....	73
2.12 Summary	77
Chapter 3. Research Methodology and Data Collection Process	78
3.1 Research Methodology	78
3.2 Preparation for Experiment	80
3.2.1. Preparation of the Test Vehicle.....	80
3.2.1.a. GPS Receivers.....	81
3.2.1.b. Corsa Box	83
3.2.1.c. Ball Bank Indicator	87
3.2.1.d. Laser Guns	89
3.2.1.e. Video Camera	89
3.2.2. Volunteer Recruitment	90
3.2.3. Test Route Selection and Choice of Control Points.....	91
3.3 Data Collection Process.....	93
3.3.1. Setting up at Control Station	93
3.3.2. Actual Test Runs.....	95
3.3.3. Downloading the Data.....	97
Chapter 4. Database Preparation and Methodology of Analysis	100

4.1 Introduction	100
4.2 Database Preparation and Synchronization	100
4.2.1. GPS Data	100
4.2.2. Corsa Box Data	102
4.2.3. Ball Banking Indicator (BBI) Data	103
4.2.4. Front and Back Laser Guns Data	105
4.2.5. Synchronization.....	106
4.3 Alignment Data.....	108
4.4 Methodology of Analysis	112
Chapter 5. Vehicle Path and Transition to Horizontal Curves.....	116
5.1 Vehicle Path.....	116
5.2 Driver Steering Behaviour.....	126
5.2.1. Steering Angle.....	126
5.2.2. Rate of Change of Steering Angle	128
5.3 Driver Behaviour and Spiral Curves	131
5.4 Quantification of Spiral Length Using Steering Behaviour	134
5.4.1. Modelling Approach	134
5.4.2. Entrance Steering Length.....	136
5.4.2.a. Conventional Modelling Approach.....	138
5.4.2.b. Variation Model Approach	141
5.4.2.c. Comparison of Modelling Approaches	143
5.4.3. Exit Steering Length	146
5.4.4. General Steering Length	149
5.5 Validation and Comparison with Current Practice in North American Guides	150

5.5.1. Driver Comfort.....	150
5.5.2. Aesthetics Control (Desirable Length).....	152
5.6 Validation and Comparison with Guides.....	155
5.7 Conclusions	160
Chapter 6. Driver Comfort Thresholds on Horizontal Curves.....	163
6.1 Introduction	163
6.2 Preparation of Data Set.....	165
6.2.1. Curves Used in Analysis	165
6.2.2. Effect of Body-Roll Angle	167
6.2.3. Calculation of Lateral Acceleration	168
6.2.4. Calculation of Speed Measures	170
6.3 Explanatory Variables	173
6.4 Preliminary Analysis: Effect of Curve Direction	176
6.4.1. Test 1	177
6.4.2. Test 2	180
6.5 Model Development	184
6.5.1. Modelling Approach	184
6.5.2. Model Development.....	185
6.6 Model Application and Curve Design	187
6.6.1. Comparison of Side Friction Demand.....	187
6.6.2. Inferred Curve Speed Using Developed Model	190
6.7 Conclusions	194
Chapter 7. Deceleration and Acceleration Behaviour for Two-Lane Rural Roads	196
7.1 Preparation of Data Set.....	197

7.1.1. Curves Used in Analysis	197
7.1.2. Calculation of Deceleration and Acceleration Rates Used in Analysis	199
7.2 Data Analysis.....	204
7.3 Model Development	208
7.3.1. Explanatory Variables.....	208
7.3.1.a. Deceleration Rate	209
7.3.1.b. Acceleration Rate.....	211
7.3.2. Modelling Approach	213
7.3.3. Model Development.....	214
7.3.3.a. Deceleration Rate	214
7.3.3.b. Acceleration Rate.....	217
7.4 Conclusions	220
Chapter 8. Relating Driver Behaviour to Safety Performance on Highway Horizontal Curves	222
8.1 Modelling Database.....	222
8.1.1. Geometric Alignment Data	224
8.1.2. Collision Data.....	225
8.2 Modelling Technique.....	226
8.2.1. Overview of Available Modelling Techniques	226
8.2.2. Exposure Term	230
8.2.3. Assessing Goodness of Fit	231
8.3 Application of Driver Behaviour Models to Studied Curves	233
8.3.1. Analysis of Rate of Change in Steering Angle	235
8.3.2. Analysis of Difference in Curvature	235

8.3.3. Analysis of 85 th Percentile Lateral Acceleration.....	237
8.3.4. Analysis of 85 th Percentile Maximum Deceleration Rate	238
8.3.5. Overall Safety Model based on Different Driver Behaviour Parameters	238
8.4 Sensitivity Analysis	240
8.5 Validation of Developed Safety Models	244
8.5.1. Data Used	244
8.5.2. Evaluation	247
8.5.3. Results of Evaluation	250
8.5.4. Conclusions	251
Chapter 9. Conclusions and Recommendations.....	253
9.1 Summary.....	253
9.2 Contributions	256
9.3 Recommendations for Future Research.....	259
References.....	261
Appendix A.....	271
Appendix B	274
Appendix C	290
Appendix D.....	307
Appendix E	309
Appendix F.....	320

LIST OF FIGURES

Figure 2.1: Relationship between Driver Behaviour and Highway Design.....	12
Figure 2.2: Yerkes-Dodson Law (Regenerated from Lamm et al. 1999)	23
Figure 2.3: Average collision rates by level of consistency (Figure reproduced with permission of TRB from Krammes and Glascock 1992).....	70
Figure 2.4: Evaluation of Effective Workload (Figure reproduced with permission of TRB from Wooldridge 1994).....	72
Figure 2.5: Evaluation of Yaw (Figure reproduced with permission of TRB, from Wooldridge 1994).	73
Figure 2.6: Mean Collision rate versus mean speed difference between geometric elements (Used with permission of the Transportation Association of Canada, www.tac-atc.ca (1999)).....	77
Figure 3.1: Research Methodology.....	79
Figure 3.2: The Test Vehicle.	82
Figure 3.3: A View of the Instruments Installed Inside the Test Vehicle.....	82
Figure 3.4: Setup for the Corsa Box.	84
Figure 3.5: The Speed Sensor Installed on one of the Test Vehicle's Tires.....	85
Figure 3.6: String Potentiometers Installed below the Dashboard for Measuring Steering Wheel, Throttle, and Braking Pad Positions.	87
Figure 3.7: Ball Bank Indicator Installed in the Test Vehicle.	88
Figure 3.8: The Laser Gun mounted at the Back of the Test Vehicle using a Tripod.	90
Figure 3.9: Test Route and Control Stations Positions.....	92
Figure 3.10: Control Station No. 1 on Regional Road 5.....	94

Figure 3.11: Static GPS Equipment Set Up at the Control Station.....	94
Figure 3.12: Rover GPS Equipment Set Up on the Test Vehicle.....	95
Figure 3.13: Questionnaire for Volunteers.	98
Figure 3.14: Methodology of Downloading Data from Equipment to Laptop.....	99
Figure 4.1: Results View: Local Coordinates of Vehicle Trajectory at 0.2 sec. Interval.	102
Figure 4.2: A Display of the Data Retrieved by Corsa Box.	103
Figure 4.3: Text Format of Data Downloaded from Corsa Box.....	104
Figure 4.4: Downloaded Data from Ball Bank Indicator Opened in Excel Spreadsheet.....	105
Figure 4.5: Downloaded Data from Front Laser Gun Opened in Excel Spreadsheet... ..	106
Figure 4.6: Example of synchronized data in spreadsheet format.	108
Figure 4.7: Alignment, Speed and Driver's View for Part of RR3.....	114
Figure 4.8: Types of Analyses using Driver Behaviour Data Collected.....	115
Figure 5.1: Vehicle Path for Different Curves.....	119
Figure 5.2: Relationship between Curve Radii and $\Delta Curve$ for All Data Points.....	122
Figure 5.3: Relationship between Curve Radii and $\Delta Curve$ without Outliers.....	123
Figure 5.4: Relationship between Curve Length and $\Delta Curve$ for All Curves.	125
Figure 5.5: Relationship between Deflection Angle ($^{\circ}$) and $\Delta Curve$ for All Curves. ..	125
Figure 5.6: Examples of Steering Paths on Curves.....	127
Figure 5.7: Relation between Curve Radii and Maximum RSA.	130
Figure 5.8: Relation between Spiral Length and Steering Length.....	133
Figure 5.9: Relation between 85 th Percentile and Average Entrance Steering Length and Curve Radius Using Conventional Model.	140

Figure 5.10: Relation between 85 th Percentile and Average Entrance Steering Length and Curve Radius Using the Variation Model.....	144
Figure 5.11: Comparison between Conventional and Variation Modelling.....	145
Figure 5.12: Comparison Between the 85 th Percentile Desirable Steering Length at Entrance and Exit of Horizontal Curves on Two-Lane Highways	148
Figure 5.13: 85 th Percentile and Average Steering Time with Radius.....	154
Figure 5.14: Desirable Spiral Length Compared to Current North American Control Criteria.....	158
Figure 5.15: Comparison of Desirable Steering Length and Spiral Length Based on New Recommended Comfort Control Values.....	162
Figure 6.1: An Example of Lateral Acceleration Profile for Two Drivers (Curve ID: 3c4, $R= 267$ m).....	169
Figure 6.2: Scatter Plot between Approach Speed and Lateral Acceleration.	174
Figure 6.3: Scatter Plot between Speed Reduction and Lateral Acceleration.	175
Figure 6.4: Comparison between Measured Lateral Acceleration and Calculated Side Friction Factor.	176
Figure 6.5: Values Read by Corsa Box for Same Inclinations with Different Directions.	180
Figure 6.6: Relation of the Observed Mean Lateral Acceleration and that Predicted by Model.	187
Figure 6.7: Comparison of Predicted Lateral Acceleration with Side Friction Factor of Design Guides and Developed Model.....	190

Figure 6.8: Comparison between the Designed Speed and the Inferred Speed using Measured Driver Comfort Thresholds.	191
Figure 6.9: Inferred Curve Speeds for Different Approach Speeds and Superelevations.	193
Figure 7.1: Speed Profiles for Free-Flow Drivers on Curve 6-416c1.....	198
Figure 7.2: Speed Profile for Free Flow Drivers on Curve 416-12c2.	199
Figure 7.3: Horizontal Alignment, Deceleration Rate, Throttle, and Speed Profiles for Driver 18 on Curve 3c4.....	200
Figure 7.4: Relation between Calculated Deceleration Rate and Measured Deceleration Rate for Each Driver on Each Curve.....	203
Figure 7.5: Relationship between Radius and 85 th Percentile Deceleration Rate.....	210
Figure 7.6: Relationship between Radius and 85 th Percentile Maximum Deceleration Rate.	211
Figure 7.7: Relationship between Radius and 85 th Percentile Average Acceleration Rate.	212
Figure 7.8: Relationship between Radius and 85 th Percentile Maximum Acceleration Rate.	213
Figure 7.9: Relation of the Observed and Model Predicted Values of d_{85} and d_{max_85} ..	216
Figure 7.10: Relation of the Observed and Model Predicted Values of a_{85} and a_{max_85} ..	219
Figure 8.1: Collision Frequency Distribution.	226
Figure 8.2: Sensitivity of Safety to RSA	242
Figure 8.3: Sensitivity of Safety to $\Delta Curve$	243
Figure 8.4: Sensitivity of Safety to a_{L85}	243

Figure 8.5: Sensitivity of Safety to d_{85}	244
Figure 8.6: Evaluation of Safety Performance based on (a) Developed Models and (b) IHSDM and Ng and Sayed (2004).	248

LIST OF TABLES

Table 2.1: Consistency Evaluation Based on Operating Speed.....	37
Table 2.2: Regression Models for Operating Speed Backgrounds for Two-Lane Rural Roads (Lamm et al. 1999, Misaghi and Hassan 2004).	40
Table 2.3: Speed Models Developed by Nie and Hassan (2007).	44
Table 2.4: Acceleration and Deceleration Rates for Different Curve Radii (Fitzpatrick and Collins 2000).....	47
Table 2.5: Consistency Evaluation Based on Vehicle Stability.....	50
Table 2.6: Level of Consistency Bases on Workload Value (Messer 1980).	53
Table 2.7: Models Showing the Relationship between Each Design Consistency Measure to Road Safety (Ng and Sayed 2004).....	67
Table 2.8: Models for Consistency Evaluation Measures (Awatta et al. 2006).	68
Table 3.1: Coordinates of Control Points	83
Table 3.2: Distances Traversed on Each Highway.	92
Table 4.1: Comparison of Characteristics Obtained from MTO Plans and That Obtained Using “Roadfit”.....	110
Table 4.2: Geometric Characteristics of Curves used in Analysis.....	111
Table 5.1: Calculated Values of $\Delta Curve$ for All Curves.	121
Table 5.2: Model for $\Delta Curve$ on Horizontal Curves.....	123
Table 5.3: Values of RSA for All Curves in Analysis.....	129
Table 5.4: Model for RSA on Horizontal Curves.	130
Table 5.5: Steering Behaviour Measures for Entrance on Curves.....	137

Table 5.6: Descriptive Statistics of the Characteristics of Study Curves.	138
Table 5.7: Entrance Steering Length Models for Two-lane Highways and Freeways .	139
Table 5.8: Variation Models for Entrance Steering Lengths for Two-lane Highways and Freeways.	142
Table 5.9: Steering Behaviour Measures for Exits on Curves.....	146
Table 5.10: Models for Exit Steering Lengths for Two-lane Highways and Freeways	147
Table 5.11: Models for Both Entrance and Exit Steering Lengths for Two-lane Highways and Freeways.....	150
Table 5.12: Models of Steering Time with Curve Radii for Two-lane Highways and Freeways	153
Table 5.13: Parameters Used in Validation of Results and Comparison with Design Guides	156
Table 5.14: Recommended Corrections to the Design Speeds and Rate of Change of Lateral Acceleration in the Current Geometric Design Guides.	161
Table 6.1: Characteristics for Curves Used in Analysis.	166
Table 6.2: Lateral Acceleration and Speed Characteristics of Curves Used in Analysis	171
Table 6.3: Descriptive Statistics of the Characteristics of Study Curves.	172
Table 6.4: Mean Lateral Accelerations and Standard Deviations for Each Curve for Control Test 2.....	181
Table 6.5: Test for Equality of Means between Right and Left Turn Curves of Control Test.....	182
Table 6.6: Model for Driver Comfort on Horizontal Curves.....	186
Table 6.7: Values of Radii and Design Speeds Used for Comparing $f_{s,d}$	189

Table 7.1: Geometric Characteristics of Curves Used in Analysis.....	198
Table 7.2: Data Used for Analysis of Deceleration and Acceleration Rates.	205
Table 7.3: Descriptive Statistics of the Characteristics of Study Curves.	206
Table 7.4: Models for Average and Maximum Deceleration Rates on Horizontal Curves.	215
Table 7.5: Models for Average and Maximum Acceleration Rates on Horizontal Curves.	219
Table 8.1: Summary of Selected Highways.....	223
Table 8.2: Descriptive Statistics of the Characteristics of the Study Curves.....	225
Table 8.3: Models Relating Driver Behaviour Parameters to Collision Experience. ...	236
Table 8.4: Pearson's Correlation Coefficient Matrix.....	239
Table 8.5: Overall Safety Model Relating Driver Behaviour Parameters to Collision Experience.....	239
Table 8.6: Values of δ and <i>CRF</i> Factors for Each Driver Behaviour Parameter.....	241
Table 8.7: Geometric Characteristics of Fictitious Alignment Used (Hassan et al. 2001).	246

CHAPTER 1. INTRODUCTION

1.1 Background

The major goal of any transportation system is the safe and efficient movement of people and goods. In the case of the highway system, there are three main components; the road, the vehicle, and the driver. The interaction between the characteristics of the three components controls the level of safety on roads. Therefore, effective provision of safety on highways could be achieved by aiming jointly at these three components. To achieve this, it is the responsibility of the road designer to design roads that reduce the chance of human errors to prevent collisions and reduce its severity if a collision does occur. There has been a great evolution in the characteristics of two of these components; the vehicle and the driver. However, geometric design of roads has not been updated enough to incorporate the evolution of these two components.

In Canada, and North America in general, transportation systems have evolved through the need of linking the East with the West for exploration and trade. It started by canoe on the rivers in the summer and sleighs on the frozen surface of the rivers in the winter. Narrow trails connecting rivers were traversed by horse-led wagons until roadways were established to minimize travel on waterways. Roadways started as earth roads to serve wagon coaches, and eventually bicycles in the late 1800s. Then it evolved to stone and gravel paved roads, and finally to paved roads (Drader 1996).

In Canada today, the road network consists of more than 1.4 million two-lane equivalent kilometres. Thirty five percent of the network is paved, and another 57% has gravel surface with surface treatment. Only 8% remains unsurfaced (Transport Canada 2004).

Evolution of vehicle characteristics started in 1893, when the first gasoline-powered “motor wagon” was operated in the United States. The new vehicle lacked any brakes and was brought to a stop by simply driving it into a curb (Bellis 2004). In the early 1900s, automobiles evolved enough to be driven safely, but were a privilege that can only be enjoyed by the very rich. Common transportation was still achieved using horse wagons or railroads for longer trips. By 1908, Henry Ford introduced the first low price motorized vehicle, “Model T”. Its high efficiency and popularity created a breakthrough in automobile ownership, where over 16.5 million were sold. Needless to say, this created pressure on the different levels of government to build better roads (Cooper 2002). Since then, vehicle technology has experienced a great evolution, and vehicle characteristics have continually improved.

Driver characteristics progressed with the advancements of vehicles as well as with the evolution of the surrounding environment. Historically, most people travelled no more than a few ten kilometres from their homes in their entire lives (Wikipedia 2006). The presence of automobiles increased the vehicle-kilometres travelled resulting in a transformation of the society such that those who do not travel long distances were a minority. Driving has become part of the everyday culture of humans, and there is a growing dependency on automobiles to the extent that most households in North America own more than one (Wikipedia 2006).

Human characteristics in general have evolved changing the driver behaviour. The more conservative, “no rush” attitude of the past has changed to a more risk-taking and sometimes more aggressive driving behaviour. This was triggered by the fast rate of the present life style, revolution in technology, information availability, personal needs, and

educational levels. The number of drivers has increased and more drivers are younger than in the past.

No doubt, this has resulted in a great evolution in travelling habits. The drivers with little experience with long distance driving on rural highways and drive with relatively lower speeds than the present times have progressed to drivers with stabilized driving experiences, and who expect better roads where they can drive with consistent high speeds. Drivers now have a motivational goal of arriving to their destination as fast as they can and to maintain their chosen speed as long as possible. This could result in some instances that the speed choice is not compatible with the current situation, for example a sharp curve, but with the situation the driver had just passed, for example a long tangent.

To satisfy this evolution in the characteristics of the road user and vehicle technology, it is important to provide progressive geometric design. However, the North American highway design guides as American Association of State Highway and Transportation Officials' Green Book (AASHTO 2004) and The Transportation Association of Canada's Geometric Design Guide for Canadian Roads (TAC 1999) that were based mostly on characteristics of vehicles designed in the 1930's and 1940's have not evolved enough to keep pace with modern vehicle design and driving behaviour. Very few studies have been carried out to update the fundamentals and criteria in the design guides to consider the on-going developments in vehicles and driving behaviour.

The approach used in the North American design guides to ensure the design safety is widely referred to as the design-speed concept through the selection of one design speed and basing the highway geometric features on that speed. For example, the selected design speed is used to determine the minimum radius of horizontal curve or minimum

length of vertical curve, and it is encouraged to adopt larger values. However, a weakness of this concept is the fact that it does not explicitly consider the speeds with which drivers travel through tangents or larger radii curves (Fitzpatrick et al. 2000). Hence, the operating speeds of drivers can be higher than the design speeds for certain geometric features such as smooth curves or long tangents, and this will generate inconsistent sections on the highway.

On the other hand, in the design guides of many European countries, such as in the German and Swiss guides, the operating speed concept is adopted. In this method, successive elements are designed such that the difference in operating speed, or the 85th percentile speed, between these elements is kept within a tolerable range; thus, ensuring consistency.

Design consistency is the degree to which the geometric features are designed so as to avoid critical manoeuvres by the drivers that could lead to collisions (Gibreel et al. 1997). It could also be defined as the degree to which the geometric features of the highway conform to driver expectations (Al-Masaeid et al. 1995). Driver expectations are the result of repeated driving experience on the highway. A design inconsistency such as an unexpected sudden sharp curve after a series of smooth curves could surprise drivers and could cause serious errors, thus increasing the risk of collisions. For this reason, methods have been developed for the evaluation of design consistency to ensure the compliance of geometric design to the driver expectations. It is hoped that collision risks would decrease through these methods creating safer roads. Therefore, there has been an increased awareness among engineers to the need to gain a clear and quantitative understanding of driver behaviour and its relation to geometric design.

1.2 Problem Definition

Through a general review of the literature, it was found that relatively few researchers have studied driver behaviour qualitatively. Moreover, studying the relationship of driver behaviour to geometric design of highways or safety in a quantitative manner has been a missing link. Previous studies focused on qualitative measures, such as “workload”, to identify inconsistent sections. Fewer studies related these qualitative measures to safety in terms of collision experience.

To date, there has been no effort to use quantitative indicators of driving behaviour as operating speeds, acceleration rates, steering angles, and vehicle path to design geometric features of the highway. Instead, geometric elements are designed based on a “chosen design speed” where drivers may or may not adopt. The design is based on assumptions and criteria studied and verified over 60 years ago. Therefore, there is a need for a better quantitative understanding of driver behaviour and its interaction with the different geometric elements and different classes of roads.

More specifically, the following are the concerns of this research:

1. Assumptions made to design road elements in the current guides may no longer be valid due to the evolutions in both vehicle technology and driving behaviour. For example, friction values of minimum horizontal curvature are based on driver comfort thresholds that may require revision.
2. Although the guides describe the road user as one of the three elements of the transportation system, drivers are considered in a qualitative manner where there is a description of their characteristics without incorporating these characteristics into the design of the road elements.

3. Quantitative measures for driver behaviour need to be developed and related to geometric characteristics of the road. These measures should also be related to safety to ensure their validity in assessing the geometric design.

1.3 Objectives

Consequently, the overall objective of this research is to integrate the interaction between the three identified variables through updating the current geometric design of horizontal curves to accommodate the driver behaviour in a quantitative manner. This will be accomplished through focusing on the road user as a main element in highway design in order to create a consistent geometric design.

The more specific objectives are:

1. Relating different driver behaviour parameters to horizontal curve geometry on different types of highways, and
2. Investigating the relationship between driver behaviour parameters and safety of the road.

1.4 Research Plan

Although there is qualitative information available from literature on driver interaction with geometric features, mostly two-lane two-way highways, a major effort is still required for a comprehensive quantitative analysis of how drivers behave on horizontal curves of different road classes. The research up to date in this area is very young and limited and is open to wide experimentation and analysis.

Therefore, an experiment was designed to collect driver behaviour data including driver's choice of speed, steering position, acceleration rates, vehicle path, and throttle and braking positions. These data along with the geometric alignment of the highways

traversed in the experiment form a huge database. This database was used to evaluate driver behaviour and analyze the interaction between it and geometric design and safety in terms of collision experience.

An advantage of this experiment over other methods of investigation is that it collects reliable data under actual driving conditions and on many classes of roads. This is opposed to experiments performed on simulators or on special test routes designed for only the test vehicle to drive on. In addition, most experiments studying driver behaviours were based on subjective measures such as questionnaires, ratings, or secondary task performances as will be explained in the literature review.

The realization of the objectives of this research was through the following plan:

1. The first phase was to complete a literature review on driver behaviour including the relationship between highway design and human factors.
2. The second phase was the design and preparation for the experiment. It included:
 - a. Design and planning of the experimental study.
 - b. Approval of Carleton University Research Ethics Board.
 - c. Installation of the measuring equipment on the instrumented vehicle, already available at Carleton University. These instruments were for measuring driver behaviour.
 - d. Recruiting volunteers for the experiment and training them on the procedure.
 - e. Selection of the test route and control points. The alignment was composed of both urban and rural freeways and two-lane two-way highways.
3. The third phase involved the actual experiment. It was concerned with collecting reliable data about driver behaviour along the test route's alignment.

4. The fourth phase was the preparation of the database used for analysis. This database included the driver behaviour data collected during the experiment and the geometric alignment data of the roads traversed.
5. The fifth phase, which required the most time, involved examining, analyzing, and modelling the interaction between the road's alignment and the human behaviour derived from these experiments.
6. Finally, the main outcome of this research was a complete revision of different aspects of horizontal curve design, and a methodology for considering driver behaviour in these aspects. This resulted in recommended updates in the current geometric design of horizontal curves in order to accommodate driver behaviour in a more accurate and quantitative manner.

The experiment was performed during the summer and fall months, which are favourable conditions for executing the experiment. These are also the months where drivers would drive less conservatively causing the geometric features of the road to be a direct influence on driver behaviour.

1.5 Thesis Outline

Following is an outline of the remaining chapters that will be covered in the thesis:

- Chapter 2 is a comprehensive review of previous literature focusing on the driver side as a major component of the highway geometric design. A description of driver characteristics as explained in previous research is given. Also, different consistency measures and relation between these measures and safety is highlighted.
- Chapter 3 is devoted to the description of the research methodology and the data collection process for driver behaviour along with the alignment data.

- Chapter 4 mainly focuses on the preparation of the database used for the analysis. This database included driver behaviour data downloaded during the experiment and the geometric features of the roads traversed in the experiment.
- Chapter 5 describes the driver path, investigates the steering behaviour of driver, and finally models the desirable spiral lengths based on the vehicle path.
- Chapter 6 examines the driver comfort thresholds used in the current geometric design guides and compares it with the collected driver behaviour data. A model is developed relating the lateral acceleration to speeds on horizontal curves. Finally, a comparison between the current geometric guides and the developed model is illustrated.
- Chapter 7 shows the model developed relating both the average and maximum deceleration rates entering the horizontal curve and acceleration rates exiting the horizontal curve to the geometric characteristics of the curve. A comparison is also made with the current practices.
- Chapter 8 demonstrates the relationship between the developed driver behaviour models in previous chapters of this research and safety.
- Finally, Chapter 9 presents a summary showing the main conclusions and findings of this research. In addition, recommendations for future research work in order to further implement the results are presented.

CHAPTER 2. LITERATURE REVIEW

The approach of this research is to study and analyze driver behaviour given the geometric characteristics of the highways, and to relate driving behaviour to collision experiences on highways. Thus, the main topics covered in the literature review include the driver characteristics, including the physiological and mental characteristics. The workload concept will be introduced with the different methodologies for measuring it. The current practices in the North American geometric design guides for horizontal curve design will be reviewed. Detecting geometric inconsistencies will be studied through different approaches namely vehicle operation (speed and vehicle stability) and driver workload. Finally, models describing the relationship between different design consistency measures and collision experience will be reviewed. Due to the wide range of driver behaviour aspects covered in this research, the thesis is structured such that this chapter is limited to the background and general knowledge of driver behaviour, in addition to the current practices in geometric design. At the beginning of each chapter, for each driver behaviour aspect studied, previous research done in this specific area is presented.

2.1 Introduction

Over the years, consideration of human factors in highway and traffic engineering has changed from merely focusing on the mechanistic side of the design and the relationship between the vehicle and the road to designing with the road users in mind in order to conform to their expectations (Kanellaidis 1996). Initially, the understanding of driver behaviour and characteristics was through research findings from social sciences.

However, engineers need to have a clear quantitative understanding of the road drivers; their behaviour, performance, characteristics, and interaction with the highway environment. This way the designers can provide a safer road environment, which reduces the probability of driver error and provides for a safer operation (Dimitropoulos and Kanellaidis 1995, Kanellaidis 1996).

Good coordination between the driver, the road, and the vehicle allows for a comprehensive evaluation of the interrelationships of traffic safety and these three components (Lamm et al. 1999). The relationship between highway design and the driver behaviour is best explained through the framework shown in Figure 2.1.

The highway features including the geometrics, roadside environment, traffic control devices, weather, and traffic conditions are the inputs to the driving task. This information is processed given the driver's physiological characteristics including reaction time, visual abilities, and fatigue level. These characteristics form the mental characteristics such as the expectancy level, attention level, and workload capacity. The result is how the driver will behave, translated through the vehicle operations characterized by the speed, tangential and lateral acceleration rates, and lateral placement.

Therefore, one of the main factors that affect driving behaviour is the road geometry. This interaction between road geometry and driving behaviour should be studied to find points of deficiency in geometric design and increase safety on highways. This way a better conformance to driving behaviour could be achieved.

2.2 The Driving Task

Before discussing the driver behaviour as a criterion for designing and evaluating the consistency of highways, a brief description of the driving task, the information

Figure 2.1: Relationship between Driver Behaviour and Highway Design.

system, and how drivers handle this information given their mental and physiological characteristics will be given. Expectancy of drivers and their perception and reaction capabilities influence the driver's performance. Consequently, these characteristics along with vehicle performance will influence the driver's choice of speed (TAC 1999). Examples of understanding driver's performance are knowledge of driving tasks, vehicle control, safe and efficient guidance through roads and traffic, and proper navigation through routes.

To understand what drivers expect on the highway, it is important to appreciate the tasks that are required from them. Drivers receive a vast array of inputs, which are mainly visual. These inputs are compared with the information already processed in drivers' minds. Based on the relevant information, they make decisions and execute the appropriate control response in order to achieve mobility with safety.

There are three performance levels for the driving task: control, guidance, and navigation. The three levels form a hierarchy of information handling complexity. The control level is relatively simple and with time could be performed automatically. The guidance and navigation levels are of increased complexity and require more processing times to take decisions (Alexander and Lunenfeld 1986, Lamm et al. 1999, AASHTO 2004).

2.2.1. Control

Control involves the interaction of drivers with the vehicle. Drivers control the vehicle through steering, accelerating, and braking. They receive continuous feedback on how well they are controlling the vehicle through the vehicle's response and its displays. Through this feedback, drivers make little adjustments to their control. Once mastered,

most control activities are performed automatically with little complexity (Alexander and Lunenfeld 1986, Lamm et al. 1999).

2.2.2. Guidance

Guidance refers to the driver's role in maintaining a safe speed and correct path in accordance with the roadway and traffic conditions. The driver receives information from the geometric characteristics of the highway, hazards, traffic speeds, relative positions between vehicles and between the vehicle and shoulders, and traffic control devices. Guidance requires judgment, estimation, and prediction where drivers evaluate the constraints imposed on them through the dynamic and continuously changing environment and translate it into control actions to maintain their safe speed and path (Alexander and Lunenfeld 1986, Lamm et al. 1999). Guidance activities include lane placement and road following, car following, passing manoeuvres, merging, lane changing, avoidance of pedestrians, and response to traffic control devices (AASHTO 2004).

2.2.3. Navigation

The navigation level is the most complex level in the driving task; however it has the least impact on safety. It includes a pre-trip phase and an in-trip phase. The pre-trip phase involves the planning and route selection of the trip. Sources may be maps or verbal instructions. The in-trip phase involves following the planned route. Information sources include guide signs and landmarks.

2.3 The Information System

Drivers are required to carry out several individual tasks simultaneously, which requires smooth, efficient handling, and integration of information (Lamm et al. 1999,

AASHTO 2004). Each input providing information to drivers is part of the information system. Drivers use their senses, usually sight sense, to pick up this information.

Through the description of the three different driving task levels in the previous section, it is found that there are a number of different sources of information. These sources are either formal or informal. Formal sources are traffic control devices that provide guidance and navigation information. They include regulatory, warning, and guide signs or markings. Informal sources of information include the geometric characteristics of the roadway and the roadside and traffic conditions. Drivers could also detect information through other senses such as hearing horns, emergency vehicle sirens, and engine noise. They can feel the road roughness by the vibration of the steering wheel and can feel raised pavement markers. They can smell burning wires in the vehicle. They can also sense changes in acceleration, pitch, and yaw of their vehicles (Alexander and Lunenfeld 1986, AASHTO 2004).

2.3.1. Information handling

Alexandra and Lunenfeld (1986) provided a thorough analysis on driver information handling, which is reviewed in this section. Drivers perform several tasks simultaneously such as looking for information sources, making decisions, and performing control actions, all this in a changing and dynamic environment. Therefore, the information provided to drivers should be in their field of view. It should capture their attention and be available in a usable format so as not to increase the complexity of the driving task (Lamm et al. 1999, AASHTO 2004).

Consideration should be given to drivers' behaviour when receiving information. They are "serial" processors of information in that they are able to handle one source of

information at a time rather than “parallel” ones. Therefore when handling information they balance by “juggling” the sources of information (Lamm et al. 1999).

The process of receiving and handling information includes signal search, detection, recognition, and use of information. When searching for information, drivers scan the continuously changing environment. Then, they detect available information in short glances and attend to it until recognized. If the information is needed, it is perceived and used in a feedback process to help make decisions and perform control actions. When information sources compete, unneeded information is discarded, and relevant information that is not required instantly is stored in the short-term memory for rapid access later. If this stored information is not used in a short time or repeated, it is usually forgotten. However, if the short-term information is reinforced, it is transferred to the long-term memory for future use.

This process of receiving and handling information structures the expectancies of the driver. Relevant information is immediately attended to, and is processed by comparison with the long-term storage that formed expectancies of drivers. On that basis, decisions are made and control actions are taken, and during the driving task, this process is continuously repeated.

At times, the driving task may be complex and demanding depending on the driver's familiarity with the alignment, traffic conditions, time pressure, and surrounding environment. When drivers become overloaded, they may get confused and miss important pieces of information. At other times, driving may be simple and undemanding, causing the driver to lose concentration resulting in a risk of collision. Therefore, the key to a safe and sufficient driving task performance is the error-free

information handling. A steady pace of information processing demand should be provided where there are no “peaks”, “valleys”, or “spikes”. Peaks are where too much information needs to be handled, valleys are where too little information needs handling, and spikes are where there is a sudden high demand of information processing following a low demand of information handling (Lamm et al. 1999).

2.3.2. Primacy

Primacy is a concept that describes the relative importance of the three levels of driving tasks based on safety criterion. The consequences of driver error on safety and system performance are higher at the control and guidance levels than at the navigation level. Errors in the prior levels may lead to collisions, while errors in the latter level may lead to inefficient traffic flow and are less likely to lead to collisions. Therefore, when providing information, the design should be made so as to draw drivers’ attention to safety-related design elements first. Drivers should process information related to control level first, then guidance level, then finally navigation level tasks (Alexander and Lunenfeld 1986, Lamm et al. 1999).

2.3.3. Reaction Time

Reaction time is the time it takes for the driver to detect information, identify and understand it, decide on the action to take in response, and initiate the action. This time depends on the mental capacity of the driver, alertness, and expectancy, and hence varies from driver to driver. Naturally, complex and unexpected decisions with several alternatives may take longer times than simple and anticipated ones. In addition, the longer the reaction time, the less time available to attend to other sources of information,

and thus the higher the chances for error (Alexander and Lunenfeld 1986, Lamm et al. 1999, TAC 1999, AASHTO 2004).

2.4 Driver Expectancy

Driver expectancy describes how ready drivers are to respond to situations and circumstances in the driving environment and perform routine driving tasks (Glennon and Harwood 1978, Messer 1980). It affects all task levels (control, guidance, and navigation) and influences the speed and accuracy of information processing (Lamm et al. 1999). It is a function of drivers' immediate memory of the highway they are currently driving and their driving experience in general. When expectancy is realized, drivers' performance tends to be error free. However, when violated, drivers may need longer response times and higher workload, and consequently improper driver behaviour (Messer 1980).

Therefore, when designing a highway, the geometrics, traffic operations, and traffic control devices should conform to drivers' expectancies to avoid longer reaction times, confusion, improper response, or driver error. Unusual designs should be avoided especially at transitions between one design element and another. There are two types of expectancies. The first is a-priori expectancies or long-term expectancies. The second is ad-hoc expectancies or short-term expectancies.

2.4.1. A-Priori Expectancies

A-priori expectancies are long-term expectancies which are a result of lifetime experience and habits in driving (Lamm et al. 1999). Similar situations and the successful responses to them are stored in the driver's memory. When needed, this knowledge is retrieved and drivers use it to perceive and handle information and hence modify their speeds and responses (AASHTO 2004). An example would be that most freeway

interchanges have exits at the right side of the road; therefore, drivers expect to exit from the right. Examples of violations to a-priori expectancies would be an intersection hidden by a crest vertical curve or a series of gentle horizontal curves followed by a sharp one (Lamm et al. 1999).

2.4.2. Ad-hoc Expectancies

Ad-hoc expectancies are short-term expectancies that happen in response to site-specific situations. As drivers traverse an unfamiliar road, the geometrics of the highway, the traffic control devices, and traffic patterns structure the ad-hoc expectancy of drivers. An example would be when placing warning signs before relatively sharp curves. The ad-hoc expectancy structured would be that before similar curves there will be warning signs.

It is the role of the designer to consider both a-priori and ad-hoc expectancies, and to prevent any violations of drivers' expectancies through the use of consistent design elements and uniform traffic control devices. It is important to realize that designs that are not in accordance to drivers' anticipation would lead to driver error and there will be a higher potential for a collision.

2.5 Driver Errors

As explained earlier in Section 2.3, when information-processing demands are high resulting in increased driver's workload, the chances of improper driver performance and error increase (AASHTO 2004). There are a variety of reasons for driver errors. These could be reasons beyond the control of highway designers or within their control (Alexander and Lunenfeld 1986, Lamm et al. 1999).

2.5.1. Errors due to Driver Deficiencies

Many errors are caused by driver deficiencies. An example would be the inability to recover from a skidding situation resulting from insufficient experience. Another example would be improper risk taking in gap acceptance during passing manoeuvres. There may also be errors due to the alcohol, fatigue, or drugs (Alexander and Lunenfeld 1986, Lamm et al. 1999, AASHTO 2004).

2.5.2. Errors due to Design

Reasons that are within the control of designers include:

- Expectancy violations.
- Overloading or underloading drivers.
- Ambiguous, deficient, confusing, or missing information.
- Misplaced, blocked, or obscured information displays.

These deficiencies may lead drivers to miss or not process traffic control information, and it is the designer's responsibility to eliminate these sources of errors (Alexander and Lunenfeld 1986, Lamm et al. 1999).

When drivers have to perform several highly demanding tasks simultaneously and under time pressure, chances of committing errors increase. This usually happens at urban sites where decision points are close and there are intensive land use, complex design features, and heavy traffic. These situations cause driver overload and confusion resulting in improper driver behaviour due to the misunderstanding of the driving situation.

The opposite case would be at rural sites where the decision points are widely spaced and there are sparse land use, smooth alignment, and light traffic. This minimal

demand for information and decision-making may result in driver inattention. Therefore, error could be caused by driver underload, and drivers may fail to detect and react to unexpectedly encountered design elements (AASHTO 2004).

2.6 Driver Workload and Mental Stress Concept

As shown in the framework explained earlier in Figure 2.1, the amount of information imposed on drivers during a particular driving task is regulated by the vehicle's speed and the highway geometric features, including the horizontal and vertical alignment, cross section characteristics, and amount of information signs on roadside. Drivers' capacity to process this information depends on the available preprocessed information (expectancies) and their capabilities as a driver, depending on their mental state (fatigue or attentive) and driving experience.

The driving tasks impose workload on drivers. These tasks may vary in frequency and difficulty, and therefore the workload imposed may vary. In addition, workload varies according to drivers' capabilities and their expectations. A highway with inconsistent design may violate expectations of drivers and impose higher workloads on them. Therefore, due to workload's impact on driver behaviour, it is important to understand the concept of workload and incorporate it in geometric design and in evaluation of design consistency of highways.

2.6.1. Definition of Workload

The link between highway design and driver behaviour is workload (Messer 1980). It was defined by Senders et al. (1967) as the amount of effort the human operator exerts to perform a task, independent of the performance of the task itself. Knowles (1963) defined it as the answer of two questions: "How much attention is required?" and "How

well will the operator be able to perform additional tasks?” These definitions appear suitable for the driving environment, where there are different tasks performed simultaneously, and each task requires a portion of driver’s attention.

Messer (1980) specifically defined workload for driving situations as the “time rate at which drivers must perform a given amount of work or driving tasks.” This rate increases at occurrence of unexpected geometric features. It also increases with the geometric complexity of the highway, speed increase, and decrease of available sight distances. Krammes et al. (1995) defined workload as the “proportion of the total driving time that drivers need to look at the roadway.” According to that, the visual occlusion method was used to measure workload which will be explained in Section 2.6.4.

2.6.2. Yerkes-Dodson Law

The Yerkes-Dodson Law shown in Figure 2.2 explains the different levels and implications of workload. At low workload levels, driver performance is also low because of inattentiveness, and information could be missed. As the workload demand on drivers increases, driver performance also increases until drivers reach a maximum mental capacity. After this point, any additional workload demand and increase in the amount of information to be processed will lower driver performance and may lead to error. Therefore, it is expected that the probability of a collision increases in the areas where the workload demand on drivers is extremely high or extremely low, or there is a sudden increase in workload demand (Wooldridge 1994, Heger 1995, Lamm et al. 1999).

2.6.3. Driver Workload Homeostasis

Driver workload homeostasis is the tendency of drivers to maintain workload at a comfortable range. This is realized by varying their speed, modifying their attention to

information sources, or modifying their participation in nonessential activities. In other words, drivers try to increase the workload imposed on them the same way they try to decrease it at other times (Wooldridge 1994, Heger 1995, Lamm et al. 1999).

Figure 2.2: Yerkes-Dodson Law (Regenerated from Lamm et al. 1999)

Reflecting to workload homeostasis effect on safety, on a highway with a gentle alignment of long tangents and mild curves, drivers increase their speed to raise the workload imposed on them. The first sharp curve they meet violates their expectancy which therefore increases the workload on them. Drivers are susceptible to error and there is a great potential for collisions. However, if the alignment had successive curves with relatively equal radii, drivers would expect the sharpness of the curves and would adjust their speed accordingly. Therefore, inconsistent designs violate workload homeostasis (Lamm et al. 1999).

Factors that lead to increased workload on drivers are (Kanellaidis 1996):

- Violations of driver expectancies, whether a priori or ad-hoc expectancies,
- Traffic flow characteristics,
- Density of roadside signs, and
- Use of in-vehicle telematics equipment.

2.6.4. Methods of Measuring Workload

Many studies have been undertaken to find suitable procedures to assess the mental workload imposed on drivers. These methods could generally be divided into five categories; subjective methods, physiological methods, secondary task performance, visual occlusion, and primary task performance.

2.6.4.a. Subjective Methods

A subjective method is a technique of obtaining the operator's opinion through the use of rating scales. Not only is this technique very easy to perform, the ratings correspond to drivers' judgment of the difficulties faced in the driving task. If they say that they are loaded, then this indicates that drivers are loaded no matter what the driving behaviour measures indicate (Gopher and Braune 1984). It is also a nonintrusive technique where this technique is usually performed after the driving task.

This technique however is associated with problems where drivers may confuse mental and physical workload while rating and not adequately rate the various outputs during the driving task. In addition, adaptation may result in ratings that are higher or lower than normal. Ratings may change as drivers gain experience with a particular difficult task. Subjective ratings may also be affected by experience, abilities, or fatigue (Hicks and Wierwille 1979).

Messer (1980) used this technique to evaluate driver workload. A model was formed based on the assumption that the road is the source of information for drivers and that it imposes workload on them. The more complex the geometric features, the higher the workload; and it can increase significantly when drivers encounter unexpected or unusual combinations or geometric features.

In this technique, ratings were based on the severity of the roadway elements that could be encountered on a highway. A seven-point rating scale was developed where 0 represents no problem and 6 represents a critical problem. Average criticality ratings for nine basic geometric features were developed based on driver expectancy. These features were horizontal curves, crest vertical curves, bridges, divided-highway transitions, lane drops, intersections, railroad grade crossings, shoulder-width changes, and lane-width reductions. The average operating speed was 93 km/h.

The criticality ratings for these nine basic features were then further used to estimate the range of criticality ratings for specific cases that may exist. These calculated expectancy ratings were defined as workload ratings and were used to evaluate the consistency of the geometric design of rural highways (Messer 1980). This will be further discussed in Section 2.9.1.

2.6.4.b. Physiological Methods

Various methods are used to record physiological processes for measuring mental workload. These methods include, for example, recording the eye movements of the driver, recording heart rate and heart rate variability, brain activity, skin conductance reaction, and respiration rate. These methods are used in correlation with the driver's control operations such as measuring the brake pressure and movements of the steering

wheel, speed, and acceleration. For the physiological methods, a low level of activation means a low performance level, a moderate level reflects a high quality of performance, and a high level of activation would imply a probability of overloading the driver with information (Wildervanck et al. 1978, Hicks and Wierwille 1979, Lamm et al. 1999).

From these test methods, using the heart rate variability and its correlation with driver performance has shown greater improvement over the other physiological tests and is most frequently used. Heart rate measures provide an overall index of arousal and physical work associated with the driving tasks. Heart rate measures could be considered as nonintrusive (Wildervanck et al. 1978, Hicks and Wierwille 1979, Wierwille and Eggemeier 1993, Lamm et al. 1999).

There are different measures of eye activity such as eye-blink latency, eye-blink duration, and eye-blink rate. There is a relationship between blink rate and visual workload. As the workload increases the blink rate decreases. Similarly, as the workload increases, the blink duration decreases. Eye blink activity could be recorded using electrooculographic (EOG) procedures which require placement of electrodes above and below the eye. This could be intrusive to the driving task, although according to Wierwille and Eggemeier (1993) it is considered as relatively nonintrusive.

2.6.4.c. Visual Occlusion Method

As explained earlier, subjective ratings are difficult to validate and therefore, its credibility is limited. The visual occlusion method, which is an objective method, was first used by Senders et al. (1967) where they were attempting to measure the attentional demand of automobile driving. The method was used by developing a special helmet equipped with an occluding visor that dropped in front of the driver's sight involuntarily.

The result of the study was that the less frequent the observations or the shorter the time intervals of observations, the slower the speed of the driver. Similarly, the more frequent and longer the observations are, the faster the speed. Farber and Gallagher (1972) used a similar concept, where a visual interruption instrument was used to measure the sensitivity of drivers to degraded visibility conditions in steering and control tasks.

Based on Senders et al. (1967) approach, Krammes et al. (1995) developed a visual occlusion method for measuring driver workload. In this method, drivers voluntarily occlude their vision, where they open their eyes only when they think it is essential to extract information for the guidance task. The mental workload is measured by the amount of time drivers are unwilling to have their vision occluded over a fixed length of roadway given a constant speed. Therefore, the lower the information processing demands for the guidance task, the longer drivers will voluntarily keep their vision occluded. Likewise, the higher the information processing demands, the shorter drivers will keep their vision occluded since they need to look at the highway longer, indicating high mental workload.

2.6.4.d. Primary Task Performance

This method is a direct measure technique, where the driving performance is an indicator of the mental workload imposed on the driver. That is, how driving is affected by the driving environment such as traffic characteristics, lane width, and most importantly consistency of the geometric features. It is expected that as the workload increases beyond a critical value, the speed or accuracy of performance decreases. The performance could be measured by the frequency of using the driving pedals, or the fine or gross movement of the steering wheel. An example would be when the driver is

fatigued, fine movements of the steering wheel tend to decrease, and larger movements are repeated (Wildervanck et al. 1978, Wierwille and Eggemeier 1993).

A clear advantage of this method is that it is nonintrusive. In addition, when using more than one measure, more reliable results could be obtained in assessing workload. Lamm et al. (1999) also discussed that the primary task performance method can only differentiate between overload and non-overload conditions; however, it cannot provide relative measures of workload in the non-overload condition.

2.6.4.e. Secondary Task Performance

The main idea of this method is to have drivers perform a secondary task while driving. If they are able to show a good performance on the secondary task, this implies that the primary task is producing little loading on the driver. As the load by the primary task increases, the secondary task performance degrades. The measure of the workload is expressed in terms of loading scores derived from the secondary task performance scores. One of the characteristics of the secondary task is that it should not interfere with or disturb the primary task performance. This is difficult, as some degree of interaction is always expected. The task should be simple and require very little learning and show little variability between drivers. This could be achieved by practicing the driver on the secondary task alone before testing. The secondary task should be self-pacing, where the rate is determined by the driver. Examples of secondary tasks may be solving simple arithmetic problems, tapping with a certain constant rhythm, a digit display mounted on the hood of the vehicle, delayed digit recall, or random digit generation (Knowles 1963, Wildervanck et al. 1978, Hicks and Wierwille 1979, Zeitlin 1995).

Whatever the method used for measuring workload, it should fulfill certain properties. The measure should be sensitive to changes in workload, and should measure workload accurately and consistently between subjects. There should be little or, even better, no interference with the primary task, or else the results may reflect the combined workload of both the measurement technique and the primary. The technique should be diagnostic in that it is able to determine the cause of workload. For more confidence of the results, it is always better to use a combination of different methods to arrive at a weighted combination of values to predict mental workload task. However care should be taken such that the measures do not interfere with each other. For example, a secondary task measure should not be used in combination with a primary task measure (Wierwille and Eggemeier 1993, Lamm et al. 1999, Fitzpatrick et al. 1999).

2.7 Current Practices of Horizontal Curve Design in North America

2.7.1. Curve Radius

When a vehicle negotiates a curve with constant radius and at a constant speed, a centripetal acceleration acts on the vehicle towards the center of the curvature with a value of (v^2/R) . The resulting force is counterbalanced by the side friction between the tires and pavement surface and/or by the weight component of the vehicle due to superelevation. The minimum radius is controlled by the value of the selected design speed, its corresponding maximum design side friction coefficient, and the superelevation rate. The relationship between these four variables as presented in the Green Book is widely known as the point mass formula shown in Equation 2.1.

$$R_{min} = \frac{V^2}{127(e + f_{s,d})} \quad (2.1)$$

Where

R = curve radius (m);

V = curve design speed (km/h);

e = superelevation rate; and

$f_{s,d}$ = coefficient of side friction demand.

Enough side friction should be available (supplied) between the tires and pavement surface to prevent skidding of the vehicle. If the effects of the vehicle body roll were ignored, the centripetal (or lateral) acceleration felt by the driver would be equivalent to the side friction demand. Values of the side friction coefficient demanded by the vehicle are set to the threshold of driver comfort, which is considerably lower than the threshold for skidding. It is assumed that upon reaching this comfort threshold, drivers would reduce their speed due to the feeling of discomfort, thus affording a sufficient margin of safety against skidding (AASHTO 2004, TAC 1999).

The values of side friction, however, are based mostly on early research work in the 1930's (Barnett 1936, Moyer and Berry 1940). Barnett's study (1936) concluded that an acceptable side friction coefficient was 0.16 for speeds between 30 km/h and 100 km/h. The values suggested by Moyer and Berry (1940) ranged from 0.21 at speeds as low as 30 km/h, 0.18 for speeds between 40 and 50 km/h, and 0.15 for speeds up to 80 km/h. In addition, Bonneson analyzed data collected from horizontal curves of different U.S. states. The results of this study were in general agreement with the values of the Green Book for high-speed design and the findings were incorporated in the Green Book 2004 edition (AASHTO 2004, Bonneson 1999, Bonneson 2000).

The model developed by Bonneson (1999, 2000) for passenger cars, shown in Equation 2.2, is based on the hypothesis that drivers' choice of side friction demand is based on a desire to achieve a combination of both safety and efficiency of driving.

$$f_d = 0.256 - 0.0022V_a + (0.0133 - 0.0074I_{TR})(V_a - V_c) \quad (2.2)$$

Where:

f_d = 85th percentile side friction demand factor;

V_a = 85th percentile approach speed (km/h);

V_c = 85th percentile curve speed (km/h); and

I_{TR} = dummy variable (=1 for ramps, 0 for curves on two-lane, two-way highways).

The model reflects the drivers' desire to drive at a higher level of comfort at higher approach speeds, as reflected by the lower side friction demands. It also reflects the drivers' willingness to tolerate higher centripetal forces, as reflected by the higher side friction demands, to minimize the extent of speed reduction on relatively sharp curves. Bonneson's model is used in the new revised edition of AASHTO (2004) to determine the maximum allowable side friction based on a maximum allowable speed reduction, which in turn is used for the design of horizontal curves. It is worth noting that this model was developed using calculated values of side friction from the point mass formula (Equation 2.1), and the only parameters that reflected actual driving behaviour were the approach and curve speeds.

Other limitations exist in the point mass formula. First, the model assumes that drivers will traverse the curve with a constant speed and with a curvature equal to that of the design curvature. This is not necessarily valid; as studies have shown that vehicle

speeds vary through the curve (Bonneson 1999). Second, the model approximates the vehicle to a point mass without considering the difference of frictional forces between the inner, outer, front, and rear tires. Third, the model is applicable to all vehicle types whether passenger cars or trucks although the size, number of tires, tire characteristics, and suspension characteristics are different.

Another major limitation of the equation is that it does not explicitly consider the vehicle rollover threshold. The rigid-vehicle model suggests that the lateral acceleration necessary to reach the rollover threshold of a passenger car exceeds the cornering capabilities of the tires leading to the case where the car spins before rolling over. However, modern trucks have the center of gravity higher and therefore they tend to rollover before sliding making it possible to reach the rollover threshold within the friction limits of the tire (Easa and Dabbour 2003).

2.7.2. Spiral Curves

2.7.2.a. Importance of Spiral Curves

Many studies have claimed the ineffectiveness of the presence of spiral curves and recommended that design codes should no longer recommend using them since they can deceive the drivers into maintaining an excessive speed (Stewart and Chudworth 1990, Tom 1995). It was also shown that curves with no spirals had lower collision rates (Tom 1995). Perco (2005) found that the natural steering time on curves did not differ between curves with or without spiral transitions. The average time was found to be between 2.23 and 2.25 s. It was suggested that the advantage of the spiral transition lies in that the speed reduction would end before the curve begins. Therefore, there would be less demand for side friction on the curve and hence a greater longitudinal friction would be

available on the curve. On the other hand, the braking on curves without spiral transitions occurs along the circular arc where the demand for side friction is maximum. A recommendation was therefore to compare the alignment curvature with the real vehicle path to confirm the advantage of the spiral.

2.7.2.b. Current Practice in North American Guides

The Green Book defines a minimum spiral length based on driver comfort, minimum lateral shift control, and sufficient length to provide superelevation. It also defines a maximum length based on maximum lateral shift, and a desirable length based on the natural steering time adopted by most drivers. The Canadian TAC guide recommends a spiral length of:

$$L_s = A^2/R \quad (2.3)$$

Where A is the spiral parameter which depends on the design speed. This parameter considers three criteria; driver comfort through gradual increase in lateral acceleration, sufficient distance to provide superelevation, and highway aesthetics defined by the natural steering time. An overview of each of these controls is presented.

1. Driver Comfort

Driver comfort is provided by gradually increasing lateral acceleration as the vehicle enters the curve. The length of spiral curve is controlled by the rate of change of lateral acceleration (c) and the design speed of the curve (V) by the following equation:

$$L_s = \frac{0.0215V^3}{Rc} \quad (2.4)$$

The Green Book recommends using a maximum value of c of 1.2 m/sec^3 . The use of lower values will result in longer, more “comfortable” spiral lengths; however they will not represent the minimum length consistent with driver comfort (Green Book 2004). On the other hand, the TAC guide recommends a range of $0.3\text{-}0.9 \text{ m/sec}^3$, and uses an average value of 0.6 m/sec^3 .

2. *Superelevation Control*

The spiral length is a suitable distance to provide superelevation transition. Therefore, the minimum spiral length should provide a distance to limit the relative slope between the pavement’s outer edge and the profile control line.

$$L_s = \frac{100we}{2s} \quad (2.5)$$

Where w is the width of pavement and s is the relative slope in percentage.

3. *Control of Lateral Shift*

AASHTO (2004) states that the lateral shift control defines the minimum and maximum length of spiral curves. The minimum length is based on a minimum lateral offset (p) of 0.20 m between the tangent and curve to ensure that the spiral curve is long enough for a shift in the lateral position within the driving lane. The maximum length is based on a maximum p of 1.0 m to prevent misconception when drivers underestimate the sharpness of the approaching curve due to the long spiral length. The length of spiral curve based on lateral shift of the vehicle’s path follows Equation 2.6.

$$L_s = \sqrt{24pR} \quad (2.6)$$

4. *Aesthetics Control (Desirable Length)*

Finally, to realize a visually pleasant alignment, it is recommended that the length of spiral be such that the driving time on the spiral is at least 2 s. That is:

$$L_s = 2 \times V \quad (2.7)$$

This is also the desirable length defined by the Green Book as the natural steering length.

2.8 Procedures for Evaluation of Geometric Design Consistency

Inconsistency may arise due to the influence of a number of factors (Messer 1980, Krammes 1994, Kanellaidis 1996):

- Changes with time in the design guides causing adjacent sections constructed at different times to be designed with different guides.
- The wide ranges of allowable design values by the guides due to the application of the design speed concept, which results in the unrestricted use of above-minimum values.
- The application of the design-speed concept to horizontal and vertical curves and not to tangents
- Rehabilitation of portions of highways without an overall review of the design.

In the following sections a brief summary will be given on the different procedures used for evaluating geometric design consistency. The most common procedures could be classified into operating speed, vehicle dynamics, alignment indices, and workload. Parameters that reflect driving behaviour, and hence will be covered in this review, include speed, friction factors between tires and pavement, and workload. The first two

procedures will be discussed in this section. The following main section will be devoted to using workload as a criterion for evaluating geometric design consistency.

2.8.1. Operating Speed

Operating speed is a common measure of evaluating geometric design consistency since it is the most visible indicator of the conformance of the design to the driver's expectations. Drivers will usually slow down when the roadway violates their expectancy, and will drive at higher speeds when they do not feel overloaded. The operating speed is the 85th percentile speed or the speed below which 85% of the vehicles travel. It is obtained from the frequency distribution of spot speeds of a sample of vehicles on the road.

2.8.1.a. Operating Speed Safety Criteria

There are two main criteria for evaluating safety on highways based on speed consistency suggested by Lamm et al. (1988):

- **Safety Criterion I: Design Speed Consistency**, which is based on the difference between the operating and design speeds on a single curve ($V_{85} - V_d$).
- **Safety Criterion II: Operating Speed Consistency**, which is based on the operating speed difference between two successive curves or between a tangent and a successive curve.

Table 2.1 shows a classification system for differentiating good, fair, and poor design for evaluating design consistency on highways (Lamm et al. 1988). Higher occurrence of these violations may lead to higher probability of collisions (Krammes et al. 1995).

In order to calculate the speed reduction between two successive elements, two approaches may be used (Ottesen and Krammes 2000):

- Calculating 85th percentile speed on the curve and the preceding independent tangent, or on two successive curves separated by nonindependent tangent. The 85th percentile of the speed reduction would be the difference between these two values $\Delta V_{85} = |V_{85i+1} - V_{85i}|$, or
- Estimating the maximum speed reduction from tangent to curve or two successive curves for each driver in a sample, then calculating the 85th percentile of the operating speed reduction (85MSR).

Table 2.1: Consistency Evaluation Based on Operating Speed.

Evaluation	Safety Criterion I: Design Consistency	Safety Criterion II: Operating Speed Consistency
Good	$V_{85i} - V_d \leq 10 \text{ km/h}$	$\Delta V_{85} \leq 10 \text{ km/h}, \Delta DC \leq 5^\circ$
Fair	$10 < V_{85i} - V_d \leq 20 \text{ km/h}$	$10 < \Delta V_{85} \leq 20 \text{ km/h}, 5^\circ < \Delta DC \leq 10^\circ$
Poor	$V_{85i} - V_d > 20 \text{ km/h}$	$\Delta V_{85} > 20 \text{ km/h}, \Delta DC > 10^\circ$
Note: V_{85i} and V_{85i+1} = operating speeds on elements i and $i+1$; $\Delta V_{85} = V_{85i+1} - V_{85i} $; DC =degree of curve; ΔDC = difference in degree of curve.		

Independent tangents are long tangents to the extent that the vehicles are able to accelerate to and decelerate from higher speeds up to free flow speeds (V_{85max}). The non-independent tangents are short tangents such that the vehicles do not have sufficient length to reach high speeds before entering the next curve.

Most research use the first approach as it is more difficult to account for the factors that affect the speeds on the approach tangent such as the preceding curve's sharpness and the length of the tangent. However, McFadden and Elefteriadou (2000) questioned the efficiency of the use of the 85th percentile speed to evaluate consistency through

subtracting V_{85} on two successive elements. They found that it underestimates the actual speed reduction since it is based on independent speed distributions at each location. Therefore, the consistency evaluation using this measure would overrate the consistency level of this alignment.

Instead, they adopted the second approach and investigated the use of the 85th percentile maximum speed reduction (85MSR), determined by using each driver's speed profile from the approach tangent to the horizontal curve and calculating the maximum speed reduction each curve experiences. After sorting the MSR, the 85th percentile value is determined (85MSR) and is used in evaluating consistency. It was found that the 85MSR is on average two times larger than ΔV_{85} . They also developed two models to predict 85MSR as a function of the geometric design elements. However, these models cannot produce speed profiles, and the reduction of speed while entering the curve and increase in speed while exiting the curve may occur over different lengths. Further studies based on this approach were conducted by Misaghi and Hassan (2005), and will be discussed in the following section.

The two safety criteria described earlier were based on collision experiences at the sites selected in Lamm et al.'s (1988) study. It was shown that for horizontal curves in the "fair" design range, the collision rates were twice as high as those that were in the "good" design range. For curves in the "poor" design range, the collision rates were four times as those in the "good" design range.

Although Lamm et al. (1988) specified a maximum desirable speed difference of 20 km/h for Safety Criteria I and II, other studies recommended different values. For

example, Leisch and Leisch (1977) recommended speed differences of 15 km/h in average vehicle running speed or design speed between two successive elements.

It should also be noted that the analysis of these safety criteria was based on linear regression. This is subject to criticism since collisions are rare, discrete, nonnegative, and random events. Presently, the main statistical approaches used for modelling collision frequencies are Poisson and negative binomial regression. In addition, using collision rates is questioned since it is based on a false assumption that there is a linear relationship between traffic volume and collisions. Instead, collision frequency should be used to measure safety (Lawless 1987, Hassan 2004). Finally, Safety Criteria I and II adopt a step form rather than a continuous form, where there is no difference in the safety performance between a ΔV_{85} of 10.1 km/h and a ΔV_{85} of 20 km/h (Hassan 2004).

2.8.1.b. Models for Predicting Speeds on Curves

Several models have been developed for estimating the predicted operating speed in terms of different alignment parameters such as curvature change rate on a single curve (CCR_s), degree of curve (DC), length of curve (L), or deflection angle (Δ). Most of these models were developed on horizontal curves of highways where the grade is less than 5%, and where there are no vertical curves or intersections. New models were developed taking into account grades on highways and vertical curves (Fitzpatrick and Collins 2000, Fitzpatrick and Wooldridge 2001, Easa 2003). A review of several regression equations that have been developed to model the 85th percentile speeds on horizontal curves are given in Table 2.2.

Table 2.2: Regression Models for Operating Speed Backgrounds for Two-Lane Rural Roads (Lamm et al. 1999, Misaghi and Hassan 2004).

1. Germany (Lamm et al. 1993)		
$V_{85} = \frac{10^6}{8270 + 7.2CCR_s}$		$R^2 = 0.73$
2. Greece (Lamm et al. 1999)		
$V_{85} = \frac{10^6}{10150.1 + 8.529CCR_s}$		$R^2 = 0.81$
3. United States, New York (Lamm et al. 1995)		
$V_{85} = 93.85 - 1.82D = 93.85 - 0.05CCR_s$		$R^2 = 0.81$
4. Germany (RAS 1984)		
$V_{85} = 60 + 39.7e^{-3.98 \times 10^{-3} CCR_s}$		N/A
5. United States (Ottesen and Krammes 1994)		
$V_{85} = 103.04 - 1.94D = 103.04 - 0.053CCR_s$		$R^2 = 0.81$
6. France (SETRA/DLI 1986)		
$V_{85} = \frac{102}{1 + 346(CCR_s / 637000)^{1.5}}$		N/A
7. Australia (McLean 1978)		
$V_{85} = 101.2 - 1.56D = 101.2 - 0.043CCR_s$		$R^2 = 0.87$
8. Lebanon (Choueiri et al. 1995)		
$V_{85} = 91.03 - 2.06D = 91.03 - 0.056CCR_s$		$R^2 = 0.63$
9. Canada (Morall and Talarico 1994)		
$V_{85} = e^{(4.561 - 5.86 \times 10^{-3} DC)} = e^{(4.561 - 5.27 \times 10^{-4} CCR_s)}$		$R^2 = 0.68$
10. United States (Otteson and Krammes 2000)		
$V_{85} = 102.44 - 1.57D + 0.0037L - 3.274 \times 10^{-3} DL$		$R^2 = 0.81$
11. United States (Krammes et al. 1995)		
$V_{85} = 41.62 - 1.29D + 0.0049L - 0.12 DL + 0.95V_t$		$R^2 = 0.90$
12. Greece (Kanellaidis et al. 1990)		
$V_{85} = 129.88 - 623.1/\sqrt{R}$		$R^2 = 0.78$
13. United States (Islam and Seneviratne 1994)		
$V_{85} = 95.41 - 1.48D - 0.012D^2$		$R^2 = 0.99$
$V_{85} = 103.03 - 2.41D - 0.029D^2$		$R^2 = 0.98$
$V_{85} = 96.11 - 1.07D$		$R^2 = 0.98$
For (PC), (MC), and (PT) respectively		
14. United States (Fitzpatrick and Collins 2000)		
$V_{85} = 104.82 - 3574.51/R$	For grades < 4%	$R^2 = 0.92$

15. Puerto Rico (Medina and Tarko 2007)	
$V_{85} = 51.973 + 0.003SD - 2.639RES - 2.296DC$ $+ 7.748SE - 0.624SE^2$	$R^2 = 0.92$
Note: V_{85} = operating speeds on elements; D =degree of curve ($^{\circ}$); CCR_s = curvature change rate on a single curve; L =length of curve (m); V_t = speed on approach tangent (km/h); R =radius of curve (m); PC =point of curve, MC = middle of curve, PT =point of tangent; SD = Sight Distance (ft), RES = residential development indicator variable, equal to 1 if the segment has 10 or more residential driveways per mile; equal to 0 otherwise; SE = superelevation rate (%).	

In Canada, there are four models for predicting the operating speed on curves developed by Morall and Talarico (1994), Gibreel et al. (2001), Misaghi and Hassan (2004), and Nie and Hassan (2007). The model by Morall and Talarico (1994) collected speed data using a radar gun for nine horizontal curves on two-lane two-way highways in Alberta. The main concerns of this model are that it was developed using a small sample size. In addition, speeds measured using radar guns are questionable since they may result in lower operating speeds. This is because drivers usually lower their speeds when perceiving speed measuring devices ahead of them. Also, the shooting angle of the radar gun affects the speed values measured (Hassan 2004). In addition, the model was developed for simple horizontal curves only without studying the effect of spirals, compound curves, or combination of horizontal and vertical curves. The model developed by Morall and Talarico is shown in Table 2.2.

On the other hand, Gibreel et al. (2001) showed that operating speeds on a combined alignment of horizontal and vertical curve are significantly different from the predicted speeds using models developed for horizontal curves only. Therefore, they developed models considering the effect of combined alignment of horizontal and vertical curve. The speed data were collected using radar guns for combined horizontal curves and vertical curves from 38 sites in Ontario. The speed data were measured at five

different points along the curve to account for speed variability within a curve. Through validation of the model, it was found that the difference of the observed and predicted speeds did not exceed 2.5%, verifying that 3D models are more accurate than 2D models.

Misaghi and Hassan (2005) developed a model for operating speed (V_{85}) and operating speed differential ($\Delta_{85}V$) using 20 curve sites. The value of $\Delta_{85}V$ represents the 85th percentile value of the speed differential between advance tangent and curve for individual drivers. Automatic traffic counters/classifiers were used for collection of the speed data and a ball bank indicator was used for collection of superelevation data. The traffic counters were installed at the middle of the curve and at the approach and departure tangents for a 24-hour period, which were proved to be more accurate than radar guns.

For operating speed prediction, a model was developed after removing four outliers from the data. Regression analysis yielded two models:

$$V_{85MC} = 91.85 + 9.81 \times 10^{-3} R \quad R^2=0.464 \quad (2.8)$$

$$V_{85MC} = 94.30 + 8.67 \times 10^{-6} R^2 \quad R^2=0.524 \quad (2.9)$$

As for the speed differential model ($\Delta_{85}V$), the following model was developed:

$$\begin{aligned} \Delta_{85}V = & -198.74 + 21.42(\sqrt{V_T}) + 0.11DFC - 4.55SW \\ & - 5.36(\text{curve_dir}) + 1.30G + 4.22(\text{drv_flag}) \end{aligned} \quad R^2=0.889 \quad (2.10)$$

Where:

V_T = operating speed on tangent;

DFC = deflection angle of curve (degrees);

- SW = shoulder width;
 $curve_dir$ = curve direction;
 G = longitudinal grade; and
 drv_flag = presence of a driveway

The model could be further simplified to include the two most significant variables, operating speed on tangent and deflection angle:

$$\Delta_{85}V = -168.1 + 17.419\sqrt{V_T} + 0.087DFC \quad R^2=0.634 \quad (2.11)$$

By observing Equation 2.10, one finds that the speed differential depends on the deflection angle. That is the greater the deflection angle, the longer the curve, and the higher the speed reduction. This contradicts the assumption of most previous studies where the drivers maintain a constant speed on the curve. Also, the model suggests a larger speed reduction on left-turns than on right-turns of the same curve. This is a very important point, which will be further studied and verified in this research.

Nie and Hassan (2007) also developed models for operating speeds at the beginning, middle, and end of horizontal curves, as well as operating speed differential using curve sites in Ottawa located on the same test route as this research. The developed models are shown in Table 2.3.

Hassan (2004) had concerns on the applicability of operating speed consistency. The first concern was the reliability of the prediction of the operating speed due to the fact that there are numerous models that use different predictors even in the same country. The second is that most of the models limit their applicability to horizontal curves with no intersections or changes in number of lanes. This may be hard to realize especially in densely populated areas where most curves have close intersections. In

addition, some studies do not report the procedure for data collection or the sample size which are needed to check the validity of the model. Another concern would be the method of data collection, which could be manual such as radar guns. Human error could be associated with these methods, and drivers could reduce their speed voluntarily due to the presence of speed measuring devices. In addition, the attempt of the observer to hide from drivers causes the observer to measure speed far from the road, increasing the errors in speed measurement. This was avoided in the study by Misaghi and Hassan (2004).

Table 2.3: Speed Models Developed by Nie and Hassan (2007).

No.	Model	df	Adjusted R^2	SEE
(a) All Curves.				
1	$V_{85\ AT} = 81.782 + 0.086L_{AT}$	8	0.661	4.24
2	$V_{85\ BC} = 108.132 - 0.090CCR$	8	0.714	4.03
3	$V_{85\ MC} = 108.357 - 0.097CCR$	8	0.860	2.83
4	$V_{85\ EC} = 102.238 - 0.092CCR + 0.039L_{DT}$	7	0.938	1.83
5	$V_{85\ DT} = 78.690 + 0.00001127r^2 + 0.066L_{DT}$	7	0.857	2.95
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
(b) Curves with Independent Approach Tangent.				
8	$V_{85\ BC} = 30.563 + 10.582\ln(r)$	4	0.776	3.14
9	$V_{85\ MC} = 110.386 - 6856.213(1/r)$	4	0.908	2.48
10	$V_{85\ EC} = 111.404 - 7360.698(1/r)$	4	0.868	3.24
11	$V_{85\ DT} = 76.486 + 0.127L_{DT}$	4	0.767	5.03
12	$\Delta V_{85} = -5.326 + 0.073CCR$	4	0.574	4.22
13	$\Delta_{85}V = -4.540 + 0.088CCR$	4	0.749	3.56
14	$85MSR = -0.658 + 0.107CCR$	4	0.760	4.22

df = degrees of freedom; Adjusted R^2 = adjusted coefficient of determination; and SEE = standard error of the estimate.

2.8.1.c. Models for Predicting Speeds on Tangents

Speeds on the tangents largely depend on the length of the tangent. There are two types of tangents, independent and non-independent. The operating speeds on non-independent tangents are taken similar to that on preceding and succeeding curves (Lamm et al. 1988).

The desired speed on independent tangents may be an assumed value, for example a speed limit as in Switzerland (Ottesen and Krammes 2000), or an estimated value, as the intercept of the regression equations for speeds on curves (Lamm et al. 1999). In the model by Krammes et al. (1995) it was found that the desired speed on long tangents is independent from the characteristics of the preceding and succeeding curves, and therefore the 85th percentile speed of all tangents in their study was 97.9 km/h at a speed limit of 88.5 km/h. Other studies found it to range from 94 to 104 km/h and suggested a value of 100 km/h as a good estimate for two-lane rural highways with a speed limit of 90 km/h (Fitzpatrick and Collins 2000). In addition, there are models for operating speeds on tangents that depend on the radii of the preceding and succeeding curves and on the tangent length (Polus et al. 2000). However, these models had coefficients of determination as low as 0.2 and as high as 0.838, but with very low number of sites tested. In addition, the results rendered speeds of 90 km/h when the preceding and successive radii are small, and up to 105 km/h when the tangents were very long which are not far from the speeds assumed or obtained from the operating speeds on curve models.

In Misaghi and Hassan (2004), the operating speed on approach tangents was found to be 103 km/h for independent tangents, and 95.7 km/h for non-independent tangents. This was based on the observed 85th percentile speeds on the approach tangents.

2.8.1.d. Acceleration and Deceleration Rates

The deceleration and acceleration rates on horizontal curves have been the focus of many studies since the time speed profiles have been used as a means of evaluating design consistency for horizontal curves. In the 1970s, the Swiss standards incorporated

the use of speed profile models for the evaluation of design consistency through the prediction of speed reduction of successive horizontal curves. In this method, a constant rate of 0.8 m/s^2 was used for the acceleration and deceleration (Swiss 1981, Lamm et al. 1986, Fitzpatrick et al. 2000, Crisman et al. 2007). In the 1970s, Leisch and Leisch (1977) introduced the use of speed profiles for evaluation of design consistency on rural roads in the United States. In their method, due to the insufficient data on deceleration and acceleration of vehicles on curves, the values used were empirical rates based on those used in the design of intersections and interchanges in the AASHO Geometric Design Policies (1965).

Later in the 1980s, Lamm et al. (1988) recommended using a constant rate of 0.85 m/s^2 for both the acceleration and deceleration rate based on car following tests. Collins and Krammes (1996) later evaluated this assumption using 10 curves with radii ranging from 175 m to 437 m. The speed was measured through seven infrared photoelectric sensors placed at regular intervals along the approach tangent and half of the horizontal curve length. The acceleration and deceleration rates were calculated by dividing the difference of the square of operating speeds at both ends of each interval by the length of the interval, and the average rate was taken. The deceleration rate was found to range from -0.54 m/s^2 to 1.19 m/s^2 , with a mean value of 0.53 m/s^2 . The negative sign for the deceleration rate implies an acceleration rate. It was concluded that the average deceleration rate did not differ significantly from 0.85 m/s^2 . The acceleration rate ranged from 0.12 m/s^2 to 0.52 m/s^2 , with an average rate of 0.29 m/s^2 . In this case, it was concluded that the average acceleration rate did differ significantly from 0.85 m/s^2 .

Fitzpatrick et al. (2000) conducted an extensive research on speed prediction models for two-lane rural highways, where the acceleration and deceleration rates were investigated. Speed data were collected using Lidar guns for 21 curves with radii ranging from 175 m to 873 m. Their study concluded that using a constant value of acceleration or deceleration rate of 0.85 m/s^2 should be revised, as they are significantly different from that value at 5% level of significance. They suggested that only curves with radii less than 250 m have deceleration rates as high as 0.85 m/s^2 . They developed a group of models for these rates that depended on the curve radii as shown in Table 2.4. Their models were the first to relate deceleration and acceleration rates to the geometric characteristics of the curve. It is worth noting here that all the previous studies assumed that the deceleration and acceleration of vehicles occur entirely on the tangents or transition curves preceding and following the curve. In case of compound curves, it is more realistic that the acceleration or deceleration is performed on the flatter curve.

Table 2.4: Acceleration and Deceleration Rates for Different Curve Radii (Fitzpatrick and Collins 2000).

Radius (m)	Acceleration rate (m/s^2)	Radius	Deceleration Rate
$R > 875$	0.00	$R > 436$	0
$436 < R < 875$	0.21	$175 < R < 436$	$-0.6794 + 295.14/R$
$250 < R < 436$	0.43	$R < 175$	1.00
$175 < R < 250$	0.54		

In addition, Cafiso et al. (2005) found ranges for acceleration and deceleration rates on horizontal curves. An experiment was designed where a test vehicle was used to collect field data measurements, which were used to determine design inconsistencies on existing two-lane rural highways. In performing the experiment, a sample of drivers all aged between 24 and 32 with equal driving experience and homogenous physiological parameters and driving reactions were selected. The test courses were selected from the

local two-lane rural road network in Italy. Each test course was from two to seven kilometres long. Measures taken to represent driver behaviour were longitudinal speed (V_x), and longitudinal (a_x) and lateral (a_y) accelerations. The test courses were between two and seven kilometres long, free from effect of intersections, and with low AADT in order to achieve free flow conditions of driving.

The test courses were evaluated by the safety evaluation procedures developed by Lamm et al. (1999) using Safety Criteria I, II, and III explained earlier in this chapter. The following were the findings of the study:

- The longitudinal acceleration profile showed a maximum deceleration from tangent to curves of less than 0.5 m/s^2 on curves rated as “good” based on Safety Criterion II.
- Deceleration values of about 1 m/s^2 were obtained on curves rated as “fair” based on Safety Criterion II.
- Deceleration values of about 2 m/s^2 were observed on curves rated as “poor” based on Safety Criterion II. It was found that this value is achieved through braking manoeuvre and not only by the release of the gas pedal, which means that drivers were forced to adapt their speed to the curvature.
- On the acceleration phase from curve to tangent, speed gradients were more gradual with a range of $0.5 - 0.9 \text{ m/s}^2$.
- The maximum value of deceleration was reached at the beginning of curves and acceleration starts near the middle of the curve.

A recent study by Medina and Tarko (2007) developed a family of four models describing the location and length of speed variation before and after horizontal curves.

They studied 10 curves and found that 66% of the deceleration and 72% of the acceleration take place on the tangents preceding and following the horizontal curve respectively. They also found that the mean deceleration and acceleration rates were 0.732 m/s^2 and 0.488 m/s^2 respectively for a 16.1 km/h speed change.

Another study by Crisman et al. (2007) investigated the speed reduction on tangents preceding 18 horizontal curves. They found that adopting a constant value for the deceleration rate was not conforming to actual driving conditions. Two models were developed that relate the deceleration rate with the curve radius, and the deceleration rate to the approach speed and the speed reduction. In their study, it was assumed that all the deceleration occurs on the tangent preceding the curve.

$$dm_{V85} = 1.7568 - 0.2217 \ln(R) \quad R^2=0.74 \quad (2.12)$$

$$dm_{V85} = 1.4223 - 0.0106V85_{appr} + 0.0146(V85_{appr} - V85_{min}) \quad R^2=0.66 \quad (2.13)$$

Where dm_{V85} is the mean deceleration rate calculated using the 85th percentile speeds at both ends of the deceleration distance, assuming a uniform deceleration rate along the deceleration distance. $V85_{appr}$ is the 85th percentile approach speed and $V85_{min}$ is the 85th percentile of minimum speed on the curve section.

2.8.2. Vehicle Dynamics

Vehicle stability on curves is an important issue to ensure dynamic safety that could be compromised by selecting improper design elements and horizontal and vertical alignment sequences. The point mass formula was discussed previously in Section 2.7.1. Lamm et al. (1999) recommended Safety Criterion III shown in Table 2.5 to evaluate the vehicle stability on horizontal curves given the difference (Δf_R) between the assumed side

friction (f_R) and that demanded by the vehicle (f_{RD}). The difference determines the level of consistency on the curve, whether Good, Fair, or Poor. Safety Criterion III was developed similarly as Safety Criterion I and II mentioned in Section 2.7.1. Therefore, the same concerns regarding its development approach are applicable in its case.

Table 2.5: Consistency Evaluation Based on Vehicle Stability.

Evaluation	Safety Criterion III: Driving Dynamic Consistency
Good	$\Delta f_R = f_R - f_{RD} \geq 0.01$
Fair	$0.01 > \Delta f_R \geq -0.04$
Poor	$\Delta f_R < -0.04$
Note: $\Delta f_R = f_R - f_{RD}$ where f_R and f_{RD} = side friction assumed and demand on element i	

The side friction demand is most commonly determined through the point mass formula using the operating speed as shown previously in Equation 2.1. The side friction assumed is determined through Equation 2.14 developed by Lamm et al. (1999):

$$f_R = 0.25 - 2.04 \times 10^{-3} V_D + 0.63 \times 10^{-5} V_D^2 \quad \text{for flat terrains} \quad (2.14)$$

$$f_R = 0.22 - 1.79 \times 10^{-3} V_D + 0.58 \times 10^{-5} V_D^2 \quad \text{for mountainous terrains} \quad (2.15)$$

Where f_R is the side friction assumed and V_D is the design speed for a highway section.

The study by Cafiso et al. (2005) explained in 2.8.1.d found that on curves with large radii ($R > 300\text{m}$) or on curvilinear alignments, lateral accelerations are less than 0.2 g. On sharp curves after long tangents, lateral accelerations reach values of 0.3 g, showing violation of driver expectancy.

2.9 Driver Workload as a Consistency Evaluation Criterion

From Safety Criteria I, II, and III explained in the previous section, the operating speed consistency (Safety Criterion II) is believed to have the strongest link with human factors research (Kanellaidis (1996)). This approach in design consistency could be classified as a designer-side assessment since it requires measurable data indicating the operational (V_{85}) and physical (R) characteristics of the driving environment. Another approach is the evaluation of design consistency based on the inferred driver workload. This could be called user-side assessment since the data obtained are from drivers themselves whether from actual driving situations, experimental settings, and/or interviews (Kanellaidis 1996).

Kanellaidis (1996) pointed out the advantages of the behavioural approach, which is applied in the user-side assessment method, as first, driver workload is directly linked to drivers' level of performance, while the operating speed consistency is indirectly related to it. Second, in the user-side assessment, the issue of driver familiarity and how it affects expectancy is better described and implemented than in the designer-side assessment. The main limitation of implementing user-side methods is the difficulty of setting acceptable procedures for extracting design features ratings. This may be due to the variation of design guides in different countries, driving habits and a priori expectancies, resulting in different applicable ratings.

2.9.1. Subjective Methods (Messer 1980)

In Section 2.6.4, the different methods of measuring workload were explained. The subjective method was thoroughly investigated by Messer (1980). Messer's model was first to introduce driving workload as a criterion for consistency evaluation. The model

took into consideration the previous features as well as the feature itself, implying that ad-hoc expectations are formed from experiences while driving. In addition, the model considers features such as intersections, narrow bridges, lane width changes, and so on as other consistency evaluation methods.

The rationale behind the development of Messer's model was previously explained. The model used for estimating the driver workload is described as:

$$WL_n = R_f \times U \times E \times S + C \times WL_\ell \quad (2.16)$$

Where:

WL_n = workload value for feature n ;

R_f = workload potential rating for feature n ;

U = driver unfamiliarity factor;

E = feature expectation factor;

S = sight distance factor;

C = feature carryover factor; and

WL_ℓ = workload value for the preceding feature.

Regarding the feature itself, the workload potential rating (R_f) is based on subjective ratings. The unfamiliarity factor (U) adjusts the workload value by considering the percentage of unfamiliar drivers on the highway. As the percentage increases, the probability of drivers being surprised by relatively unexpected geometric features increases. The feature expectation factor (E) reduces the workload value if the prior feature is similar to the current one, taking into account the ad-hoc expectancy formed while driving on the highway. The sight-distance factor (S) increases as the sight distance to the feature decreases, resulting in a higher workload. This is because as the sight

distance decreases, the available time for the driver to process information decreases, and therefore, the workload increases. Numerical values of each factor are given in figures and tables.

To account for the preceding feature, a carryover factor is multiplied by the preceding workload value. As the distance between the two features decreases, the contribution of the preceding one increases. This is because there is less time to shed the information associated with the prior feature, and process new information for the present feature. Using the workload value obtained (WL_n) a level of consistency is determined using Table 2.6.

Table 2.6: Level of Consistency Bases on Workload Value (Messer 1980).

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

It should be mentioned that one of the main differences between this method and the operating speed consistency method is that the latter method assumes that when two consecutive curves are sharp, the speed difference between the two curves is small, and therefore does not violate consistency. In Messer's model, on the other hand, this would result in higher workload values which increase the level of inconsistency. It should be noted, however, that one of the main disadvantages of this model is the fact that it was based on subjective ratings making it difficult to validate.

2.9.2. Physiological Method (Heger 1995)

Heger (1995) developed a model to quantify mental workload using physiological measures such as blink rates, electromyographic, electrodermal, and electrocardiographic response measurements. The measurements were analyzed given information on the geometric features, speeds, accelerations, and subjective measures. A stepwise evaluation of the multiple response systems was used to determine an overall index of driver mental workload. After adjusting the index by weight functions to account for the influence of different psychological parameters, it was used for measuring driver workload values and furthermore evaluating geometric design consistency as shown in the following equation.

$$I_{s,i} = \frac{\sum_{p=0..n}^p [z_{i,p} CDOTw(z_{i,p})]}{\sum_{p=0..n}^p [z_{95,p} CDOTw(z_{95\%,p})]} CDOT100 \quad (2.17)$$

Where:

$I_{s,i}$ = Standardized Driver Mental Workload Index of a road feature i ;

$Z_{i,p}$ = z-standardized value of the parameter p and the road feature i ;

$Z_{95,p}$ = 95th percentile of the parameter p and the road feature i ;

$w(Z_{i,p})$ = Weighted function dependent on the z-standardized value of p ; and

$w(Z_{95,p})$ = Weighted function dependent on 95th percentile value of p .

The study showed that high speeds on sharp curves result in high workload levels (increased galvanic skin reactions, decreased eye blink rates, and decreased heart rate variance). Therefore, drivers reduce their speeds on the curve to reduce the workload levels. The same behaviour was shown on other inconsistent features such as lane narrowing. In addition, most physiological parameters increase after the inconsistent

design feature, causing an increase in the workload drivers are subjected to, and therefore resulting in an increase in the possibility of collisions. When analyzing the collision experience in the study area, it was found that high portions of collisions occur on tangents after passing the inconsistent design feature. Subjective ratings were used for further validation of the physiological measurements. Details on the methods of subjective ratings were not given. The research showed that the results of both measures were consistent.

One of the main advantages of this study is its correlation between operating speeds, physiological measures, subjective measures, and the geometric characteristics of horizontal curves. However, it is a complex procedure, requiring many physiological measurements.

2.9.3. Visual Occlusion Method (Krammes et al. 1995, Fitzpatrick et al. 1999)

As previously explained in Section 2.6.4, Krammes et al. (1995) used the visual occlusion method to measure workload and evaluate consistency of highways. The test involved developing a regression model for driver workload in terms of visual demand. In this study, 24 volunteers were asked to traverse six curves on a test-track. Each curve was traversed six times. The individual workload observations using the visual occlusion method were calculated using Equation 2.18. This observation measures the percentage of time the driver is observing the roadway.

$$WL_i = \frac{0.5}{t_2 - t_1} \quad (2.18)$$

Where

WL_i = workload over time interval i from t_1 to t_2 ;

t_2 = time of current information request (s);

t_1 = time of previous information request (s); and

0.5 = time increment during which subject has vision (s).

A statistical model was developed relating driver workload to degree of curvature resulting in the following equation:

$$WL = 0.193 + 0.016 * D \quad (2.19)$$

Results of the study showed that as the degree of curvature increases the workload on the driver increases. Workload values are low at the beginning of the approach tangent, increase toward the end of the approach tangent, peak near the beginning of the horizontal curve, decrease again along the curve, and level off at the end of the curve.

The main conclusion of this method was that the visual occlusion is a reasonable objective method for obtaining driver workload values on horizontal curves. However, it does not explicitly evaluate consistency, and it does not consider geometric features other than horizontal curves. In addition, results of the study were obtained under test conditions and not under real life traffic conditions making the results questionable.

Fitzpatrick et al. (1999) continued Krammes et al.'s (1995) research. In this study, two sets of tests were performed, simulator tests and on-road test. The simulator test was similar to the test-track test explained earlier, where six curves were traversed six times by 24 drivers. The on-road test was generally for evaluating the workload measures resulting from the test-track test and included six volunteers traversing five curves with four repetitions only. For the on-road test, several precautions were taken to avoid any potential safety hazards due to the visually occluded drivers.

The research by Fitzpatrick et al. (1999) studied the effect of curve sequence, separation distance between curves, radius, and deflection angle using visual occlusion and subjective ratings. For the visual occlusion procedure, drivers wore a Liquid Crystal Display (LCD) visor that was opaque except when the driver requested a glimpse. The subjective rating ranged from 1 (very easy) to 10 (impossible). To ensure consistency between drivers, descriptive terms and anchor definitions were provided with the subjective ratings.

Curve characteristics were varied to study the effect of radius and deflection angle on workload. Curve sequences were examined to assess workload carryover and driver expectancy. The study excluded the use of heart rate data since it was found unsuitable for short-term, transient tasks like driving through horizontal curves. The general pattern of the visual demand on a curve had the same trend explained earlier in Section 2.9.3.

Both the visual occlusion and subjective rating methods found a linear relationship between visual demand and the reciprocal of curve radius. For sharper curves, more visual information is required to process information and control the vehicle. It was also found that visual demand did not vary considerably with the deflection angle. In addition, results of the occlusion test found that the length of curve does not affect the visual demand. However, subjective ratings showed that drivers found longer curves more difficult to traverse. This may be because the length of curve does not increase the visual demand, but increases the time interval it is experienced.

Statistical analysis of the results showed discrepancies in the visual demand on successive curves, where different runs resulted in different responses for successive curves. In general, successive curves did not greatly affect visual demand. Further

research was recommended on this point. An analysis of the driver's eye fixation was also developed. Drivers tend to look ahead searching for points where the curve begins, and generally ignore edge markings in the near field. Generally, when the curve radius was large, drivers preferred to look to the inside of the curve. For sharper curves, drivers preferred to look to the outside of the curve, and this was more common on left curves.

The collected data were used in a regression model to relate average workload on curves to the reciprocal of the curve for drivers both unfamiliar and familiar to the highway. Since the test was repeated six times, unfamiliar drivers' visual demand was obtained from the first run, and familiar drivers' visual demand was obtained from the remaining five runs. The resulting models were:

$$VD_{LU} = 0.173 + \frac{43.0}{R} \quad R^2 = 0.711 \quad (2.20)$$

$$VD_{LF} = 0.198 + \frac{29.2}{R} \quad R^2 = 0.526 \quad (2.21)$$

Where:

VD_{LU} = visual demand of unfamiliar drivers; and

VD_{LF} = visual demand of familiar drivers.

Hassan et al. (2001) pointed out that, based on statistics, regression models are valid only within the range of tested values. Therefore, these models are valid for curve radii within the range evaluated in the study. The study included only five curves of radii between 146 m and 349 m, which should be the range within which the models can be used. Hassan et al. (2001) further examined the equations and noted that for curves exceeding a radius of 552 m the visual demand for familiar drivers becomes higher than that for unfamiliar drivers which is unreasonable.

Furthermore, this model is based on only five unique radii. Therefore, the large number of observations used in the regression model (134 observations in run 1 and 670 observations in runs 2-6) may artificially increase the coefficient of determination (R^2) due to the increased degrees of freedom. In addition, the visual demand model does not explicitly relate driver workload to consistency evaluation.

Easa and He (2006) evaluated driver visual demand on combined horizontal and vertical alignments using a driving simulator. A sample of fifteen volunteers drove in the simulator, and visual demand data were analyzed. The models given in the following equations show that the visual demand varied significantly with the inverse of the horizontal curve radius and the inverse of the vertical curvatures. In addition, the visual demand also varied with the driver's age.

$$\begin{aligned} VDF = & 0.1668 + 28.6502RINV + 1.2826KINV * C_type(C) \\ & + 0.9592KINV * C_type(S) + 0.0032 Age \quad R^2 = 0.80 \end{aligned} \quad (2.22)$$

$$\begin{aligned} VDH = & 0.1685 + 37.2199RINV + 1.0282KINV * C_type(C) \\ & + 0.6457KINV * C_type(S) + 0.0029 Age \quad R^2 = 0.57 \end{aligned} \quad (2.23)$$

$$Log(VD30) = -1.5148 + 85.3096RINV + 0.0084 Age \quad (PChisq/DF) = 0.01 \quad (2.24)$$

Where:

$RINV$ = inverse of horizontal curve radius (m^{-1});

$KINV$ = inverse of K (K is the absolute value for crest or sag vertical curves);

$C_type(C)$ and $C_type(S)$ = nominal character variables which are defined as follows: (1) When the 3D curve involves a crest vertical curve, $C_type(C) = 1$

and $C_type(S)=0$; (2) when the 3D curve involves a sag vertical curve, $C_type(C)=0$ and $C_type(S)=1$; and (3) when there is no vertical curve (only a flat horizontal curve exists), $C_type(C)=0$ and $C_type(S)=0$. ; and Age = subject age (years).

2.9.4. Primary Task Method (Cafiso et al. 2004, 2005)

Cafiso et al. (2004, 2005) used the primary task method where the driving performance is the main indicator of the mental workload imposed on the driver. Field data were collected using an instrumented vehicle called Driver Instrumented Vehicle Acquisition System (DIVAS), travelling under normal traffic conditions. The dynamic data obtained were vehicle speed, vertical acceleration (a_z) and lateral acceleration (a_y), car trajectory, driver's visual field, and yaw angle. They were used to reflect driver behaviour and road-vehicle interaction. In addition, the physiological parameters were used to show the variability in driver behaviour while driving. Signals of electrocardiogram (*ECG*), electrooculogram (*EOG*), electrodermal activity (*EDA*), and electromyography (*EMG*) were recorded. Instruments on the car included a GPS receiver, speed and acceleration sensors, a video camera, and physiological measures system. In addition, the geometric features of the road being traversed such as alignment, roadside environment, cross section, and available sight distance were recorded.

For evaluation of driver behaviour, the study suggested using the following parameters:

1. Dynamic Data:

- The speed profile

- The relationship between skid friction available to the demanded skid resistance of test vehicle (a_x/g) could be studied.
- The ratio (a_y/V_x^2), which represents the curvature ($1/R$) of the vehicle trajectory, could be used to compare the vehicle's actual trajectory to the horizontal alignment.
- To evaluate the driver comfort on horizontal curves, the variation of lateral acceleration over time (a_y/t) was suggested to be measured to analyze the rate of gain of radial acceleration C .

2. Human Data:

- It was suggested to use the *ECG* signals for measuring the heart rate (number of beats per minute) and heart period (duration between beats).
- The *EMG* could also be used to measure facial reactions of pleasant and unpleasant stimuli
- The *EOG* could be used to measure the blink rate and blink duration.
- The *EDA* could be used to measure the change in response to emotional and cognitive activity.

The human data obtained in this initial study were compared with the existing geometric features on the highway. The initial results showed changes in physiological reactions of the driver corresponding to variations in mental work. Based on that, increase in workload levels would mean inconsistency of the design. The study concluded that there is considerable influence of horizontal alignment and sight distance on both dynamic data and human data.

The research was further continued (Cafiso et al. 2005) where the field data measurements were used to determine design inconsistencies on existing two-lane rural highways. The experiment and its results in regards to acceleration and deceleration rates were presented earlier in Section 2.8.1.d.

A clear advantage of this method is that it is nonintrusive. In addition, when using more than one measure, more reliable results could be obtained in evaluating geometric design consistency by assessing workload as a criterion. The study is still in progress to show correlation between driving behaviour and geometric consistency.

2.10 Relationship between Design Consistency Measures and Safety

Highway safety is a major concern to highway designers along with efficiency and cost effectiveness. The interaction between the characteristics of the three components of the transportation system; the road, the vehicle, and the driver controls the level of safety on roads. In addition to the three main components of the transportation system, the surrounding conditions in terms of traffic and weather conditions also affect the level of safety on the roads. Through modelling the effect of these components on safety, the interrelationship between the collision frequency and the factors affecting collisions could be determined. This in turn would be used in reducing collisions and increasing safety on the long run through changing the factors that increase collision experience using cost effective means (Miaou et al. 1998).

Many models in literature have been developed that relate the five factors explained above to safety on roads. The models were developed for two-lane rural highways, urban highways, multi-lane freeways, and intersections. For each type of model, different factors were studied such as geometric characteristics of road segments or intersections

(lane width, shoulder width, curve radius, pavement type, vertical grades...), weather conditions (daytime, night time, raining, fog, snow...), or traffic conditions (no control intersections, traffic signs, traffic signals...). Collisions were also modeled as one group regardless of the severity or in groups depending on their severity (property damage only, injury, or fatality). Following is a review of the results of previous research which quantifies the relationship between different measures of design consistency and safety.

2.10.1. Relating Different Consistency Measures and Safety

Speed is believed to have the greatest impact on traffic safety (Lamm et al. 1999), and there is a great risk of collisions as the difference in operating speed between one element and the other increases. This increased variation may occur on transition from a tangent to a curve or between two curves with different radii. Therefore, Lamm et al. (1999) introduced recommendations for the difference of degree of curve (ΔDC) between successive elements as was shown earlier in Table 2.1. From the table, it is shown that curves with degree of curve (ΔDC) less than 5° fall in the good design, while ΔDC between 5° and 10° fall in fair design, and ΔDC greater than 10° after independent tangents fall in poor design. The three levels of consistency ratings were based on observation of data in regards to safety. It was found that for ΔDC between 5° and 10° , the mean accident rate was twice to three times the mean collision rate for curves with ΔDC less than 5° . It was also found that curves with ΔDC greater than 10° the mean collision rate was five to eight times the mean collision rate for curves with ΔDC less than 5° (Lamm et al. 1999, Lamm et al. 2003).

In addition, unbalanced curve radii ratios on compound curves could have unfavourable effects on collision measures. Lamm et al. (1999) also found that transition

curves may increase safety on curves because friction demands increase gradually. However, this finding was significant only for curves up to radii of 200m.

As mentioned earlier, the analysis by Lamm was based on collision rates which is recently a subject of criticism because it assumes a linear relationship between traffic volume and safety. In addition, linear regression is not suitable for collision analysis because collisions are rare, discrete, nonnegative, and random events (Lawless 1987, Hassan 2004).

Anderson et al. (1999) recommended an explicit relationship between speed reduction and collisions on horizontal curves based on road and traffic data of 176 two-lane highway sites in six states in the United States. These sections were composed of 5287 curves. The Safety Criterion II was verified using the traffic data of the 5287 curves. It was found that the average collision rate was highest for the horizontal curves in the poor design range and lowest for the horizontal curves in the good design range. Explicitly quantitative relationships were then developed between safety and operating speed reduction, average radius, ratio of maximum radius to minimum radius, average rate of vertical curvature, and ratio of individual curve radius to average radius. The relationships were based on Poisson, negative binomial, and lognormal regression analysis. The relationship between collision frequency, exposure and speed reduction was investigated. The following regression equations were obtained:

$$Y = e^{(-7.1977)} AADT^{0.9224} L^{0.8419} e^{(0.0662\Delta V/85)} \quad R^2 = 0.195 \quad (2.25)$$

$$Y = e^{(-0.8571)} MVKT e^{(0.0780\Delta V/85)} \quad R^2 = 0.156 \quad (2.26)$$

$$Y = e^{(-5.932)} AADT^{0.8265} L^{0.7727} e^{(-0.3873CRR)} \quad R^2 = 0.196 \quad (2.27)$$

Where:

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

$AADT$ = average annual daily traffic;

L = curve length, km;

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

ΔV_{85} = speed reduction on horizontal curve from adjacent tangent or curve (km/h);

and

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

The models developed showed low correlation which is the case with regression relationships between geometric design elements and safety because of the complex nature of collision occurrence (Anderson et al. 1999).

Anderson and Krammes (2000) developed a linear regression model between mean operating speed reduction and mean collision rate (Acc_Rate) shown in Equation 2.16. The study indicated a statistically significant relation between the two variables, and recommended using it in evaluation of design consistency. The model has an uncommonly high R^2 due to grouping the curve sites into speed reduction intervals.

$$Acc_Rate = 0.54 + 0.27 (\text{mean } \Delta V_{85}) \quad R^2=0.93 \quad (2.28)$$

Ng and Sayed (2004) also investigated and quantified the relationship between design consistency and road safety in terms of expected collision occurrence based on 319 horizontal curves and 511 tangents in British Columbia, Canada. They first studied the effect of individual design consistency measures (namely ΔV_{85} , $V_{85}-V_d$, Δf_R , $CRRs$, VD_{LU} , and VD_{LF}) on road safety shown in Table 2.7. Then two quantitative relationships between collision frequency and different consistency measures that were found to be statistically significant were developed.

The first model included (ΔV_{85} , $V_{85}-V_d$, Δf_R) and is applicable for horizontal curves. The second included ΔV_{85} and Δf_R and is applicable for both horizontal curves and tangent sections. The two models enable the designer to predict the safety impact of improving design consistency in terms of collision reduction. The two model equations are:

$$Acc/5yrs = \exp(-3.369) L^{0.8858} V^{0.5841} \times \exp [0.0049(V_{85}-V_d) + 0.0253 \Delta V_{85} - 1.177 \Delta f_R] \quad (2.29)$$

$$Acc/5yrs = \exp(-2.338) L^{1.092} V^{0.4629} \times \exp[IC \times (0.022 \times \Delta V_{85} - 1.189 \Delta f_R)] \quad (2.30)$$

Where:

- $Acc/5yrs$ = number of collisions on the curve during a 5- year period;
- V = average annual daily traffic (vpd);
- L = horizontal curve length (km);
- $\Delta f_R = f_R - f_{RD};$
- f_R = side friction supply;
 $= 0.22 - 1.79 \times 10^{-3} V_d + 0.56 \times 10^{-5} V_d^2;$ (Lamm et al. 1999)
- f_{RD} = side friction demand;
 $= V_{85}^2 / 127 R - e;$ and
- IC = Dummy variable (0 for tangents and 1 for horizontal curves).

What makes these models more distinct from Anderson et al. (1999) is they have the advantage of incorporating the effect of vehicle stability and the difference between the operating and design speed. In addition, collision data were based on local data of Canadian roads. Validation of Ng and Sayed (2004)'s model was done using fictitious alignments, and through comparison with the IHSDM safety model.

Awatta et al. (2006) developed regression models quantifying the relationships between individual consistency measures to collision experience for 70 curves on two-lane rural highways in Eastern Ontario. The models were developed through negative

binomial regression analysis. Statistically significant models were developed based on ΔV_{85} , $\Delta_{85}V$, Δf_R , CRR , and VD_{LU} as shown in Table 2.8.

Table 2.7: Models Showing the Relationship between Each Design Consistency Measure to Road Safety (Ng and Sayed 2004).

No.	Model form	t -ratio	κ	Pearson χ^2 (χ^2 test) Standard deviation
Difference between operating and design speed ($V_{85} - V_d$)				
1	Accidents per 5 years = $\exp(-3.380) L^{0.8920}$ $\times V^{0.5913} \exp[0.009091(V_{85} - V_d)]$	a_0 -1.904	1.539	293.24
		a_1 6.511		(357.39)
		a_2 2.784		341.26
		a_3 1.985		
Speed reduction (ΔV_{85})				
2	Accidents per 5 years = $\exp(-3.796)$ $\times L^{0.8874} V^{0.5847} \exp(0.04828 \Delta V_{85})$	a_0 -2.072	1.533	289.19
		a_1 6.482		(357.39)
		a_2 2.742		341.26
		a_3 2.043		
Difference between side friction demanded and assumed (Δf_R)				
3	Accidents per 5 years = $\exp(-3.303)$ $\times L^{0.8733} V^{0.5680} \exp(-2.194 \Delta f_R)$	a_0 -1.851	1.521	294.01
		a_1 6.412		(357.39)
		a_2 2.672		341.27
		a_3 -1.986		
Ratio of the radius of individual section to the average radius of the alignment (CRR)				
4	Accidents per 5 years = $\exp(-3.159)$ $\times L^{0.8898} V^{0.5906} \exp(-0.3606 CRR)$	a_0 -1.791	1.541	294.06
		a_1 6.514		(357.39)
		a_2 2.785		341.27
		a_3 -2.016		
Visual demand of unfamiliar drivers (VD_{LU})				
5	Accidents per 5 years = $\exp(-4.297)$ $\times L^{0.8866} V^{0.5831} \exp(3.076 VD_{LU})$	a_0 -2.231	1.531	295.34
		a_1 6.476		(357.39)
		a_2 2.735		341.23
		a_3 2.040		
Visual demand of familiar drivers (VD_{LF})				
6	Accidents per 5 years = $\exp(-4.679)$ $\times L^{0.8873} V^{0.5841} \exp(4.566 VD_{LF})$	a_0 -2.323	1.533	295.58
		a_1 6.481		(357.39)
		a_2 2.740		341.26
		a_3 2.027		

All models experienced acceptable levels of goodness of fit and the over-dispersion. One of the main advantages of these models is that it was based on actual speed data. Another main advantage is that the models were continuous functions providing a flexible and quantitative tool for evaluating the safety benefits of design improvements and can be used for safety– explicit highway design.

Table 2.8: Models for Consistency Evaluation Measures (Awatta et al. 2006).

Parameter	Estimate	<i>p</i> -value	D^m χ^2 (Test)	D^m/DF χ^2/DF R^2_α	α
(a) Design speed consistency: $Y = \exp(b_0 + b_1(EXPO) + b_2(V_{85} - V_L))$					
b_0	-0.9709	0.0002	66.6446	0.9947	0.1948
b_1	0.4047	<0.0001	78.3297	1.1691	
b_2	0.0571	0.0014	(86.6501)	0.9403	
(b) Operating speed consistency, ΔV_{85} : $Y = \exp(b_0 + b_1(EXPO) + b_2(\Delta V_{85}))$					
b_0	-2.9874	<0.0001	67.1407	1.0021	0.1293
b_1	0.4275	<0.0001	81.2878	1.2133	
b_2	0.2829	0.0010	(86.6501)	0.9607	
(c) Operating speed consistency, $\Delta_{85}V$: $Y = \exp(b_0 + b_1(EXPO) + b_2(\Delta_{85}V))$					
b_0	-3.3287	0.0008	59.7649 80.8651 (86.6501)	0.8920	0.4278
b_1	0.2035	0.0282		1.2069	
b_2	0.1885	0.0257		0.6502	
(d) Vehicle stability consistency: $Y = \exp(b_0 + b_1(EXPO) + b_2(\Delta f_R))$					
b_0	-0.8660	0.0015	64.4910	0.9626	0.2181
b_1	0.3526	<0.0001	76.1580	1.1367	
b_2	-16.1188	0.0014	(86.6501)	0.8957	
(e) Alignment indices consistency: $Y = \exp(b_0 + b_1(EXPO) + b_2(CRR))$					
b_0	1.6425	0.0081	57.1032	0.8523	<0.0001
b_1	0.3318	<0.0001	70.6395	1.0543	
b_2	-3.3682	<0.0001	(86.6501)	0.9836	
(f) Driver workload consistency: $Y = \exp(b_0 + b_1(EXPO)) \times VD_{LU}^{b_2}$					
b_0	5.3212	0.0207	62.2858	0.9296	0.2717
b_1	0.3740	<0.0001	75.7663	1.1308	
b_2	4.9046	0.0048	(86.6501)	0.8034	

An attempt was further made to develop an overall safety – based consistency model including a combination of the different consistency measures. However, the regression analysis resulted in counter-intuitive variable signs and statistically

insignificant results due to the high multicollinearity existing between the predictor variables. Hassan (2004) explained this showing how all consistency measures are correlated since they include some form of either of the terms for operating speed or radius, which in turn are also correlated to each other. Therefore, all criteria are strongly correlated, making it difficult to find an overall safety-based consistency model.

2.10.2. Relating Driver Workload and Collision Experience

Krammes and Glascock (1992) studied the influence of geometric inconsistencies on collision experience. They used driver workload ratings based on Messer (1980) as a measure of geometric inconsistency and reported a statistical relationship between workload measures and collision experience on five two-lane rural highways in Texas. To perform this analysis, the workload value of Messer's procedure was modified when features overlap at sections of the road in order to associate a unique workload value and a collision rate. This effective workload value (EWL_i) was selected for each uniform section i with length l_i where it was equal to the highest of the workload values for the features that overlapped. Workload values of tangents were given a baseline value of zero.

Two levels of analysis were performed. A microscopic analysis evaluated the statistical relationship between collision rates and EWL_i value for individual geometric features. In addition, a macroscopic analysis was used to evaluate the statistical relationship between the collision rates and mean EWL_i value for each highway direction. Results of the microscopic analysis are shown in Figure 2.3, which suggests a strong relationship between collision rate and levels of consistency as defined by Messer (1980).

Figure 2.3: Average collision rates by level of consistency (Figure reproduced with permission of TRB from Krammes and Glascock 1992).

For the macroscopic analysis, a regression analysis was performed to relate the mean EWL_i value to the overall collision rate for extended highway sections. The mean EWL_i value was calculated by the following equation:

$$\mu_{EWL} = \frac{\sum (l_i)(EWL_i)}{\sum l_i} \quad (2.31)$$

Where μ_{EWL} = mean effective workload value for the extended highway section

The best fitting model was a quadratic equation of the form:

$$ACCR = 4.95 - 10.6179(\mu_{EWL}) + 6.8088 (\mu_{EWL})^2 \quad R^2 = 0.88 \quad (2.32)$$

Where ACCR= average collision rate for one direction of an extended highway section

This equation was obtained through 10 data points representing values for two directions of five highways. The quadratic shape of the relation shows that at low values of average

workload, collision rates increase and at high values of workloads, the collision rates also increase. This is in accordance with the Yerkes-Dodson Law.

The main conclusions of this study were that Messer's (1980) procedure of measuring driver workloads may be a good indicator of collision experience on two-lane highways. Also, the EWL_i profile could be a useful graphical tool for highlighting sections with potential problems.

In another study, Wooldridge (1994) examined the influence of design inconsistency on driver error and hence probability of collisions. It was hypothesized that extremely high or low workloads are associated with high collision rates. This would include also sudden increases or decreases in workload. The hypothesis was based on Yerkes-Dodson Law. To prove this hypothesis, Messer's (1980) driver workload procedure was applied on selected portions of 19 highways in Texas. The workload ratings were then compared with collision records for these road segments.

Initially, the influence of a priori expectancy was determined by grouping the studied segments within the six levels of consistency. For each group the number of collisions per 10^8 vehicle miles was determined. The results in Figure 2.4 show that the collision rates are higher in features with effective workloads greater than six ($LOC F$) compared to the other levels of consistency.

Another examination was performed to study ad-hoc expectancy by defining the yaw of the workload on individual features. Yaw was used to represent the difference between the moving average workload and a specific feature's workload. The following equation shows its calculation:

$$\text{Yaw} = (\text{workload rating of feature}) - (\text{moving average workload}) \quad (2.33)$$

Then, the roadway segments having the same yaw categories were grouped together, and for each group the collision rates were determined. Figure 2.5 shows the results. The collision rates are dramatically higher for segments with yaw values greater than or equal to 4; however, for other groups, the collision rates are not effectively different.

Figure 2.4: Evaluation of Effective Workload (Figure reproduced with permission of TRB from Wooldridge 1994).

Figure 2.5: Evaluation of Yaw (Figure reproduced with permission of TRB, from Wooldridge 1994).

This proves the hypothesis that high collision rates result from extremely high workload levels or high sudden increase in workloads. However, no evidence was found that low workload levels or yaws are accompanied by high collision rates. The study suggested that the reason may be that the workload levels were not low enough since the highway segments passed through rolling terrain. One of the additional main conclusions of the study was that Messer's (1980) driver workload procedure was a practical method for assessing design consistency. The main drawback of Krammes and Glascock (1992) and Wooldridge (1994) studies is that they are both based on collision rates, whose disadvantages were explained previously.

2.11 Consideration of Design Consistency in North American Design

Guides

In order to realize the goal of a transportation system of safety, efficiency, and economy, it is required to minimize the gap between the needs and expectations of the

user and the offered designed system (Kanellaidis 1996). This is not clear enough in North American design guides (TAC 1999, AASHTO 2004), and designers should think beyond the clear-cut requirements for the discrete elements of the geometric design, and examine the overall driving environment. Engineers tend to conform to available funds and minimum design levels, and forget the main component of the system, which is the user.

Although AASHTO (2004) has established the principle of designing with the user in mind, it is not explicitly reflected in the design of specific elements. There exists a thorough review of driver characteristics and human factors in geometric design, which was shown in Section 2.2 through 2.5. However, no quantitative methods are given to aid the designers in considering the human factors in their design.

The approach used in the North American design guides to ensure the design safety is widely referred to as the design-speed concept through the selection of one design speed and basing the highway geometric features on that speed. For example, the selected design speed is used to determine the minimum radius of horizontal curve or minimum length of vertical curve, and it is encouraged to adopt larger values.

However, a weakness of this concept is the fact that it does not explicitly consider the speeds that drivers adopt on tangents or larger radii curves (Fitzpatrick et al. 2000). Hence, the operating speeds of drivers can be higher than the design speeds for certain geometric features such as smooth curves or long tangents, and this will generate inconsistent sections on the highway. On the other hand, although the idea of geometric design consistency is encouraged in these guides, there are no procedures given to ensure consistency.

As for the Canadian TAC Guide (1999) and its proposed revision (Easa 2002) the design consistency subject is explicitly discussed where there are two categories of suggested measures for evaluation of design consistency; local and system consistency measures. Local consistency measures are within individual elements and include design speed margin consistency and vehicle stability design consistency. As for the system consistency measures, four principles are suggested for evaluating design consistency: cross section consistency, operating speed consistency, alignment geometry consistency, and driver workload consistency.

The cross section consistency encourages adopting consistent dimensions such as lane and shoulder widths. The designer is encouraged to avoid creating incompatibilities between the cross section and its horizontal and vertical alignment. As for the operating speed consistency, it is introduced in a quantitative approach, and operating speed uniformity is encouraged through avoiding two types of speed variation:

- Varying the operating speeds on successive features along the alignment, since the more frequent the speed variations, the higher the probability of collision
- Drivers travelling considerably slower or faster than the average traffic speed which increases the probability of being involved in a collision.

The speed prediction model developed by Krammes et al. (1995) shown in Equation 2.34 is recommended in TAC (1999) to ensure the uniformity of the operating speeds. Also, a value for acceleration and deceleration rates is given (0.85 m/s^2). Speed profiles could be drawn, and the designer should avoid increased speed reductions. TAC (1995) provided a figure from Krammes et al. (1995) showing the difference between the mean speed reduction and the mean collision rate shown in Figure 2.6, which is based on

Equation 2.34. It is also stated that this figure should be used with caution and that local data should be used when available. In the revision by Easa (2002), consideration of combination of horizontal and vertical alignments was encouraged. The speed prediction models and speed profile models developed by Fitzpatrick and Collins (2000) were recommended. The relationship of mean speed reduction to mean collision rate was based on studies by Anderson et al. (1999). It is worth noting that both these studies were based on American data.

$$V_{85} = 102.44 - 1.57D + 0.0037L - 3.274 \times 10^{-3} DLR^2 = 0.81 \quad (2.34)$$

As for workload measures, it is encouraged not to design roads so as to drop the workload too low or raise it too high so as collision rates do not increase. However, the guide states that the methods used to quantify driver workload till now have not been developed or validated to the point where it can be used for quantitative evaluation. In the revision, the study by Fitzpatrick et al. (1999) relating the visual demand of familiar and unfamiliar drivers to curve radius was mentioned. As for vehicle stability measures, it was only mentioned in the proposed revision (Easa 2002), where the study by Lamm et al. (1999) was reviewed. Although TAC Guide (1999) and its revision (Easa 2002) have started adopting the evaluation of design consistency concept, it is still in its preliminary stages. The drawbacks of the different consistency measures and relation of these measures and safety should be reviewed and updated based on models developed using local Canadian data.

Figure 2.6: Mean Collision rate versus mean speed difference between geometric elements (Used with permission of the Transportation Association of Canada, www.tac-atc.ca (1999)).

2.12 Summary

The previous review of North American guides showed that they have established the principle of designing with the user in mind; however, it is not explicitly reflected in the design of specific elements. In addition, although the TAC (1999) guide included a section for evaluation of design consistency, the models used are not based on local data. There were studies that developed models for design consistency using Canadian data, but most of them recommended further verification using more curve sites. Therefore, this study will provide updates to the current geometric design guides with the road user as the main focus. In doing so, a thorough assessment of the drawbacks explained in the literature will be undertaken, and interrelationships between the driver and geometric design of highways and safety will be evaluated.

CHAPTER 3. RESEARCH METHODOLOGY AND DATA COLLECTION PROCESS

As previously discussed in Chapter 1, this study aims at analyzing driving behaviour given the geometric characteristics of the highway. From reviewing the literature, it was found that the potential criteria for describing driving behaviour could be operating speed, lateral and longitudinal acceleration rates, side friction between the vehicle's tires and road pavement, vehicle path, and steering behaviour.

In this chapter, the research methodology will be presented. In addition, a thorough description of the preparation of the experimental procedure will be described.

3.1 Research Methodology

The steps pursued in this study could be summarized as shown in Figure 3.1. It also shows how the thesis will be laid out. In the remaining parts of this chapter, the preparation for the experiment as well as the data collection process will be discussed. The preparation for the experiment made up the first component in this research, which is composed of Preparation of Test Vehicle, Volunteer Recruitment, Test Route Selection, and Choice of Control Points. These steps were completed during the period of March 2005 to August 2005.

The second component of the research was the actual execution of the experiment which took around a month and half during the months of August and September of 2005. It involved setting up the GPS receiver at the control station, the actual test drive, and downloading the data recorded to a laptop for later synchronization.

Figure 3.1: Research Methodology.

The third component involved the preparation of all elements of the research database for further analysis. This database included the data collected from the test runs and data on the geometric alignment of the highways obtained from the Ontario Ministry of Transportation (MTO) and City of Ottawa.

The fourth component was the analysis of the data obtained. Six main types of analysis are proposed as shown in Figure 3.1.

From the results obtained through the previously described analyses, the fifth component was a summary of the potential improvements that could be made to the current North American geometric design procedures. It is hoped that these results will help improve highway safety especially in Canada.

3.2 Preparation for Experiment

Since this research involves human subjects, a prior approval from *the Tri-Council Policy Statement: Ethical Conduct for Research Involving Humans* and the *Carleton University Policies and Procedures for the Ethical Conduct of Research* are essential. To obtain this approval, a proposal was prepared describing the significance of this research in terms of improving highway safety especially in Canada. In addition, a brief description of the methodology, data collection procedure, and description of the volunteers who will be driving the vehicle were given. The anonymity of the participants was ensured and the possible risks imposed on them were clarified. Finally, an approval for the research was given for a year starting April 2005.

3.2.1. Preparation of the Test Vehicle

The test vehicle used in this study was a Ford Windstar minivan owned by Carleton University and used by the Department of Civil and Environmental Engineering in

research related to highway design and traffic studies. Figure 3.2 shows the test vehicle. The vehicle could be easily adapted to install different types of measuring equipment as shown in Figure 3.3. The seat beside the driver was removed and a board was installed in its place with angles that could hold the various instruments. Following is a description of the instruments installed in the test vehicle.

3.2.1.a. GPS Receivers

The GPS receivers are important to record the vehicle's exact path during the experiment. The type of GPS system used in this study was SR 520. The method used for recording the vehicle's path is the differential kinematic GPS survey method. For this method, two sets of GPS receivers were used. The first one was the rover receiver used to record the vehicle's path, and its antenna was installed on top of the test vehicle to record the Northing, Easting, and elevation of the test vehicle during the test run (Figure 3.2). The other was a static receiver installed on a control station. A control station is a point with known coordinates chosen such that it was close to the center of the test route.

Using the differential kinematic GPS survey method minimizes the errors that could result from satellite readings and increases the reliability of the collected data. Since most of the GPS observation errors are common for both the static and roving receivers, they can be reduced in the data post-processing, thus producing higher accuracies. For the purpose of this experiment, the GPS receivers were set to record 5 readings/ second. Data recorded included Easting, Northing, Elevation, and time stamp of each reading.

Figure 3.2: The Test Vehicle.

Figure 3.3: A View of the Instruments Installed Inside the Test Vehicle.

The GPS equipment also included PC cards to record the data and download on the laptop for post processing. The data post processing was performed using a software, “SKI-Pro”, to download the vehicle’s path during the run. A detailed description of the methodology of setting up the GPS Receivers will be given later in this chapter.

Two control stations were used during the whole experimentation duration depending on the availability of the stations since they were used simultaneously by the City of Ottawa for mapping. Data on the control stations including their locations and their coordinates were obtained from the City of Ottawa’s Surveys and Mapping Division for the first station and from the website sponsored by the Geomatics Division Office of the Ontario Ministry of Natural Resources (www.mnr.gov.on.ca/cosine) for the second station. The coordinates of the two stations are as shown in Table 3.1.

Table 3.1: Coordinates of Control Points

Control Station	Source	Name	Coordinates in UTM units
1	City of Ottawa	1554	Easting: 434942.8857 Northing: 5005610.0650 Elevation: 60.955 m
2	Ontario Ministry of Natural Resources	01919760710	Easting: 434095.2148 Northing: 5004884.6889 Elevation: 62.0 m

3.2.1.b. Corsa Box

The Corsa Box is a data acquisition box that records data from various sensors connected to different vehicle parts. These data are stored in the data acquisition box with a time stamp with a sampling rate of 5 readings/ second. The data acquisition box was

controlled through a Driver Control Box which has an on/off switch that controlled the timing for recording data. The Driver Control Box was controlled by the person recording the data. Later, the data were downloaded through an RS232 serial cable to the computer for analysis. The data acquisition box was placed horizontally on a board in the test vehicle. Figure 3.4 shows the setup of the Corsa Box. The following were measured using the Corsa Box:

Figure 3.4: Setup for the Corsa Box.

1. Instantaneous Speed

The instantaneous speed was measured in km/h using a speed sensor. The speed sensor was calibrated such that readings were recorded at a time interval of 5 readings/second. The speed sensor included four magnets that were installed on one of the tires of the test vehicle. Then, a speed sensor was fixed in front of the magnets such that it switches on whenever a magnet passes by it. This output signal was transmitted through a

cable that connects the sensor to the data acquisition box. Figure 3.5 shows the speed sensor installed on one of the tires of the test vehicle.

Figure 3.5: The Speed Sensor Installed on one of the Test Vehicle's Tires.

2. Lateral and Longitudinal Accelerations

Lateral and longitudinal accelerations were measured internally using accelerometers built in the data acquisition box. The accelerations were measured in units of g (9.81 m/sec^2). The data acquisition box was configured to record 5 readings/ second.

3. Steering, Throttle, and Braking Pad Positions

The steering, throttle, and braking pad positions were measured using string potentiometers which are generally used to measure relative displacements. The readings were recorded based on how far the spring is pulled out of the housing of the potentiometer. The three potentiometers were mounted inside the dashboard such that strings were hooked to the moving part. Figure 3.6 shows the three sensors coming out of the dashboard, while the potentiometers themselves were hidden inside the dashboard.

For the steering wheel, the string was wrapped around the steering shaft and the sensor was mounted next to the steering column. As the wheel turns, the shaft would wind and unwind the string. Readings were read every 0.2 second, and the units were in percentage; 0% corresponds to a straight position, $\pm 100\%$ correspond to a full revolution to the right or left respectively. The percentage could be converted to angle values ranging from 0° at the straight position to $\pm 360^\circ$ when the steering wheel makes a full revolution to the right or left respectively.

For the throttle and brake positions, the corresponding strings were also wrapped around the throttle shaft and brake shaft respectively, similar to the steering wheel. The units were in percentage; 0% corresponds to zero force on the throttle or brake pad, and 100% corresponds to maximum pressure on the throttle or brake pad.

Figure 3.6: String Potentiometers Installed below the Dashboard for Measuring Steering Wheel, Throttle, and Braking Pad Positions.

3.2.1.c. Ball Bank Indicator

The ball bank indicator is a device used to measure the lateral acceleration on the curves in degrees. It indicates the side force the driver feels while negotiating curves. It was included in the test because it is the main device used to develop the horizontal curve design criteria in current design guides through measuring the side force felt by the driver feels while negotiating curves. The ball bank indicator used in this study was digital, with a screen giving the value of lateral acceleration in units of degrees. This output is recorded with a time stamp using an RS232 serial cable connecting between the ball bank indicator and the laptop. Figure 3.7 shows the ball bank indicator installed in the test vehicle.

The idea behind the ball bank indicator is that as the driver negotiates the horizontal curve, a centrifugal force acts on the vehicle with a value of (mv^2/R) . This force is counterbalanced by the weight component of the vehicle due to superelevation of the

pavement and by the side friction between the vehicle and the pavement surface. This is governed by the point mass formula as explained earlier in Chapter 2:

$$\frac{v^2}{gR} = e + f_s \quad (3.1)$$

Where:

v^2/gR = centrifugal component;

v = Speed of the vehicle (m/s);

R = radius of the horizontal curve (m);

g = acceleration due to gravity = 9.81 m/s²;

e = superelevation; and

f_s = side friction between the tires and the pavement surface.

Figure 3.7: Ball Bank Indicator Installed in the Test Vehicle.

Drivers feel a push or force acting on them while driving on the curve. This force is equivalent to the side friction value or the centrifugal force minus the weight component of the vehicle due to the superelevation. Converting forces into unitless values by dividing by (mg) , the angle given by the ball bank indicator would be:

$$(\alpha) = \frac{V^2}{gR} - e \quad (3.2)$$

This formula considers the vehicle as a rigid body which is not true. Considering body roll (ρ) , the angle given by the ball bank indicator would be:

$$(\alpha) = \frac{V^2}{gR} - e + \rho \quad (3.3)$$

3.2.1.d. Laser Guns

The vehicle was also equipped with two laser guns. One was at the front as was shown in Figure 3.3, and one at the back as shown in Figure 3.8. The laser guns were used to measure the distance (m) and speed difference (km/h) between the test vehicle and the other vehicles in front and behind it. The speed difference can only be measured if it was greater than 15 km/h. These values were obtained with a time stamp. Measuring the distance between the vehicles was important to understand if the other vehicles represented a constraint to the test vehicle's choice of speed.

3.2.1.e. Video Camera

As shown in Figure 3.3, a video camera was also mounted beside the driver in order to record the driver's view during the test course. This is important in order to have a

permanent record of the outside events that might have affected on the driver's behaviour.

Figure 3.8: The Laser Gun mounted at the Back of the Test Vehicle using a Tripod.

3.2.2. Volunteer Recruitment

Thirty volunteers were recruited for this experiment. The sample size was chosen as 30 to obtain a normal distribution. The volunteers were asked to fill out a questionnaire after the experiment to show their general driving experience and their experience during this test. Most of the volunteers were in the mid-age group ranging from around 20-50 years of age. They were undergraduate students, graduate students, friends of the team members, and professors. Results of the filled questionnaires, as well as the experiment, showed that the sample contained different driving behaviours ranging from slightly cautious, moderately cautious, extremely cautious, and slightly aggressive. In addition,

the sample contained various driving experiences ranging from less than 5 to more than 16 years of driving experience.

The main task of the volunteers was to drive the same way they drive on the road, and they were not asked to do specific manoeuvres. All volunteers were G-licensed drivers. The task involved no confusion and drivers were faced with the same usual risks of everyday's driving. The drivers were given explanation and instructions on the procedure before starting the experiment and elements of the experiment were made clear to them. The experiment was performed during the summer and fall months so that the drivers are not exposed to the more dangerous conditions of snow or freezing ice, which could be perceived as potential danger.

3.2.3. Test Route Selection and Choice of Control Points

The route chosen for the experiment has a total length of around 110 km. Table 3.2 shows each highway, its type whether freeway or two-lane two-way, the distance traversed on it, and the number of curves in the alignment. Figure 3.9 shows a plan of the route.

As shown from Figure 3.9, the test route was chosen such that it would consist of a Freeway in an urban area (Highway 417), a Freeway in a rural area (Highway 416), two-lane two-way highways in rural areas (Highway 7, RR3, and RR6), and two-lane two-way highways in urban areas (RR 12 and RR73).

As mentioned earlier, the control stations were chosen at points near the center of the test route to minimize the errors of the moving GPS receiver, and increase the accuracy of the readings. The following section will discuss the data collection process.

Table 3.2: Distances Traversed on Each Highway.

Highway	Highway type	Segment length (km)	Total number of curves
Highway 417	Freeway	24.9	11
Highway 7	Two-lane two-way	11.4	3
Regional Road 3	Two-lane two-way	22.8	11
Regional Road 6	Two-lane two-way	19.1	2
Highway 416	Freeway	16.5	6
Regional Road 12	Two-lane two-way	8.5	5
Regional Road 73	Two-lane two-way	6.8	9

Figure 3.9: Test Route and Control Stations Positions.

3.3 Data Collection Process

As described in Figure 3.1, the second component of the research was the actual execution of the experiment. This took place in a month and half during the months of August and September. The data collection process required three team members. Two runs were normally performed per day at 10:00 a.m. and 12:30 p.m. The data collection process for each working day involved set up of the GPS receiver at the control station, the actual test run, and downloading the data recorded to a laptop for later synchronization.

3.3.1. Setting up at Control Station

The team members, composed of the applicant of this research, a Master's degree student, and an undergraduate student, would usually meet at around 8:00 a.m. The test vehicle was driven to one of the control stations shown in Figure 3.9 previously. Most of the runs were made using control station No. 1.

Figure 3.10 shows the control station and Figure 3.11 shows the static GPS equipment installed on it. The equipment consisted of a tripod, a GPS receiver, and an antenna. The antenna collected coordinate data for the control station from satellites during the run. Using these data, along with the known coordinates of the control station, errors of the rover receiver could be reduced in the post processing of the GPS data.

Figure 3.10: Control Station No. 1 on Regional Road 5.

Figure 3.11: Static GPS Equipment Set Up at the Control Station.

The undergraduate student was left at the control station to prevent any theft or interference with the equipment and to control the starting time of the equipment to coincide with the starting time of the test run.

Figure 3.12 shows the set up of the Rover GPS equipment. The GPS receiver was set on the board that carried also the rest of the equipment, and the antenna was mounted on top of the test vehicle using a magnet.

Figure 3.12: Rover GPS Equipment Set Up on the Test Vehicle.

3.3.2. Actual Test Runs

The remaining two team members would then get back to Carleton University to start the test run with the chosen volunteer at around 10:00 a.m. The volunteer was given an idea about the experiment and was familiarized with the equipment, the test car, and

the route that will be taken. The driver then signed a consent form, shown in Appendix A, that he/she agreed to participate in this test and understood that the risks of driving the vehicle on the highways were similar to everyday's driving risks. The driver also agreed to disclose the data for the purpose of the research, and filled in a form to add them to the insurance policy of the vehicle.

The tests started from Carleton University, however recording started from the entrance ramp 121 on Highway 417. One of the team members started the video camera before the entrance ramp to start viewing the driver's view. Then, the Corsa Box and the ball bank indicator were started simultaneously to ensure data synchronization. This equipment then worked without further interference from the member except marking events using the Corsa Box marker at points where the vehicle made a turn on a new highway or where there were work zones. The same team member operated the Front Laser Gun. The other team member guided the driver through the route and controlled the Back Laser Gun. It should be noted that other than guiding the drivers on the route, the two team members did not instruct the volunteers on how to drive, as it was important to record the actual driving behaviour without the team members nor the equipment imposing any affect on them.

The run took an average of 90 minutes, but it depended mainly on the traffic conditions, and on the driving behaviour whether aggressive, moderate, or cautious. The data collection ended at the intersection of Regional Road 73 (Prince of Wales Drive) and Hog's Back Road. The drivers continued their drive till Carleton University. After the run was finished the volunteers were asked to fill out a questionnaire asking about general

driving behaviour of the driver, and their driving experience in the test. Figure 3.13 shows a copy of the questionnaire.

3.3.3. Downloading the Data

Usually, a second run would be carried out after the initial one starting at 12:30 p.m., depending on the availability of volunteers on that day. After the two runs were over, the two team members drove back to the control station to collect the static GPS equipment and the third team member. The fieldwork ended at around 3:30 p.m. and the team members would return back to Carleton University to return the test equipment and park the vehicle. Figure 3.14 shows a schematic of how the data from the equipment were downloaded to the laptop. All data recorded on PCMCIA Cards from the static and rover GPS receivers and the two laser guns were downloaded to the laptop. In addition, the data from the Corsa Box were downloaded using the serial input RS232. The video recording was downloaded through an IEEE 1394 cable.

The following chapter presents a thorough description of the third component of this research, Database Preparation.

Carleton University
Department of Civil and Environmental Engineering

QUESTIONNAIRE

for experimental investigation of

Evaluation of Driver Behavior and its Effect on Highway Design and Safety

Before Driving

Please check the answer that best represents you:

- Education : ☐ University ☐ College ☐ High School
- Profession :
- Gender : ☐ Male ☐ Female
- Age : ☐ <20 ☐ 20-25 ☐ 26-40 ☐ 41-60 ☐ >60
- Corrective eye : ☐ Yes ☐ No
- Driving experience : ☐ <5 years ☐ 5-10 ☐ 11-15 ☐ 16-20 ☐ >20
- Driving education : ☐ Driving School ☐ Self-Learning
- Type of vehicle you normally drive:
- ☐ Sub-compact Sedan ☐ Compact Sedan ☐ Midsize Sedan
- ☐ MiniVan ☐ Station Wagon ☐ Convertibles
- Driving frequency : ☐ Daily ☐ Often ☐ Rarely ☐ Never
(On two-lane rural highway)
- Duration of average trip: ☐ <1 hour ☐ 1-2 ☐ 2-3 ☐ 3-5 ☐ >5
(On two-lane rural highway)
- How would you evaluate your driving performance?
- ☐ Extremely Cautious ☐ Slightly Cautious ☐ Moderately Cautious
- ☐ Slightly Aggressive ☐ Extremely Aggressive
- How would you describe your familiarity with passing maneuvers whenever possible?
- ☐ I always pass ☐ I usually pass ☐ I sometimes pass
- ☐ I almost never pass ☐ I never pass

After Driving

On a scale of 1 – 5 (1 = very low... 5 = very high), rate the following:

– Quality of the handling (feeling) of the vehicle:	1 2 3 4 5
– Quality of the driving task:	1 2 3 4 5
– Degree of familiarity with the route:	1 2 3 4 5
– Level of comfort with the experiment:	1 2 3 4 5

Figure 3.13: Questionnaire for Volunteers.

Figure 3.14: Methodology of Downloading Data from Equipment to Laptop.

CHAPTER 4. DATABASE PREPARATION AND METHODOLOGY OF ANALYSIS

4.1 Introduction

As described in Figure 3.1 of Chapter 3, the third component of the research was the preparation of the data collected from the equipment along with data obtained on alignment characteristics for further analysis. Database preparation included downloading data from each equipment, processing them into a common format for all equipment, and finally the synchronization of the data.

4.2 Database Preparation and Synchronization

Post processing of each instrument is explained in the following points:

4.2.1. GPS Data

The GPS data from the static and rover receivers were downloaded through the PCMCIA cards from the GPS receivers. The data were post processed using the software “SKI-Pro” (Static-Kinematic - Professional). This software minimizes the errors that could result from satellite readings by means of Differential Kinematic GPS survey. Most of the GPS observation errors are common for both static and roving receivers; therefore, post-processing can reduce these errors and produce higher accuracies (Leica Geosystems 2002).

The accuracy level of the data recorded from the GPS receivers in raw format was $\pm 100\text{m}$ (Leica Geosystems 2002). After post-processing, the points were stored with their coordinates and their ambiguity status. The ambiguity status is an indicator of the accuracy status of the points stored through kinematic survey. The ambiguity status is

either “yes”, “no” or “?”. Ambiguity status “yes” means that all ambiguities are resolved and that the accuracy of the point is in centimetre level. Ambiguity status “?” means that no attempt was made to resolve the ambiguities. Ambiguities could be resolved for baselines (i.e. distance between control station and rover station) less than 20 km by default. Distances greater than 20 km may result in unresolved ambiguity status and reduced accuracy.

For the project in hand, the maximum distance between the control station and the test vehicle was at the starting ramp of HWY 417 which was 25 km. Distance of 20 km or less was achieved 4 km after the start of the test route on HWY 417. No other location for the control station could give a distance less than 25 km, since the station chosen was almost in the center of the test route. However, by comparing coordinates of all drivers with each other and also with the coordinates of the centreline of the highways obtained from City of Ottawa, it was found that the Northing and Easting coordinates were accurate to 3-10 mm, which is the level of accuracy after post-processing. Ambiguities, if present, were in the height readings, which are not in scope of this study.

Ski-Pro was also used to transfer the coordinates from Cartesian coordinates (X, Y, and Z) to local grid coordinates. The local coordinate system chosen for this study was WGS 1984 ellipsoid, and UTM 18 map projection. The processed data were then displayed in a form of a report as shown in Figure 4.1 which could be saved as a text file for further analysis in a spreadsheet.

Results	Point Id	Epoch	Stored Status	Ambiguity	Type	Easting	Northing	Height	Posn. + Hgt. Ch.	
	06990825_1420636	08/25/2005 10:20:54	Yes	yes	Moving	445460.5163	5025926.4789	31.2745		0.0141
	06990825_1420638	08/25/2005 10:20:54	Yes	yes	Moving	445460.5183	5025926.4858	31.2726		0.0327
	06990825_1420640	08/25/2005 10:20:54	Yes	yes	Moving	445460.5125	5025926.4840	31.2640		0.0214
	06990825_1420642	08/25/2005 10:20:55	Yes	yes	Moving	445460.5085	5025926.4831	31.2710		0.0293
	06990825_1420644	08/25/2005 10:20:55	Yes	yes	Moving	445460.5072	5025926.4851	31.2793		0.0216
	06990825_1420646	08/25/2005 10:20:55	Yes	yes	Moving	445460.5066	5025926.4866	31.2670		0.0249
	06990825_1420648	08/25/2005 10:20:55	Yes	yes	Moving	445460.5112	5025926.4825	31.2793		0.0246
	06990825_1420650	08/25/2005 10:20:55	Yes	yes	Moving	445460.5117	5025926.4847	31.2804		0.0319
	06990825_1420652	08/25/2005 10:20:56	Yes	yes	Moving	445460.5125	5025926.4878	31.2766		0.0409
	06990825_1420654	08/25/2005 10:20:56	Yes	yes	Moving	445460.5117	5025926.4899	31.2790		0.0388
	06990825_1420656	08/25/2005 10:20:56	Yes	yes	Moving	445460.5082	5025926.4938	31.2797		0.0313
	06990825_1420658	08/25/2005 10:20:56	Yes	yes	Moving	445460.5092	5025926.4875	31.2709		0.0229
	06990825_1420660	08/25/2005 10:20:56	Yes	yes	Moving	445460.5152	5025926.4872	31.2845		0.0249
	06990825_1420662	08/25/2005 10:20:57	Yes	yes	Moving	445460.5133	5025926.4791	31.2818		0.0417
	06990825_1420664	08/25/2005 10:20:57	Yes	yes	Moving	445460.5084	5025926.4826	31.2841		0.0311
	06990825_1420666	08/25/2005 10:20:57	Yes	yes	Moving	445460.5105	5025926.4902	31.2694		0.0322
	06990825_1420668	08/25/2005 10:20:57	Yes	yes	Moving	445460.5079	5025926.4804	31.2787		0.0311
	06990825_1420670	08/25/2005 10:20:57	Yes	yes	Moving	445460.5084	5025926.4894	31.2819		0.0529
	06990825_1420672	08/25/2005 10:20:59	Yes	yes	Moving	445460.5095	5025926.4934	31.2663		0.0430
	06990825_1420674	08/25/2005 10:20:58	Yes	yes	Moving	445460.5135	5025926.4942	31.2730		0.0238
	06990825_1420676	08/25/2005 10:20:58	Yes	yes	Moving	445460.5098	5025926.4963	31.2719		0.0296
	06990825_1420678	08/25/2005 10:20:58	Yes	yes	Moving	445460.5131	5025926.4980	31.2811		0.0343
	06990825_1420680	08/25/2005 10:20:58	Yes	yes	Moving	445460.5089	5025926.4917	31.2909		0.0321
	06990825_1420682	08/25/2005 10:20:59	Yes	yes	Moving	445460.5113	5025926.4932	31.2830		0.0193
	06990825_1420684	08/25/2005 10:20:59	Yes	yes	Moving	445460.5142	5025926.4964	31.2809		0.0272
	06990825_1420686	08/25/2005 10:20:59	Yes	yes	Moving	445460.5153	5025926.4927	31.2722		0.0171
	06990825_1420688	08/25/2005 10:20:59	Yes	yes	Moving	445460.5143	5025926.4965	31.2642		0.0169
	06990825_1420690	08/25/2005 10:20:59	Yes	yes	Moving	445460.5068	5025926.4953	31.2563		0.0308
	06990825_1420692	08/25/2005 10:21:00	Yes	yes	Moving	445460.5081	5025926.4968	31.2737		0.0166
	06990825_1420694	08/25/2005 10:21:00	Yes	yes	Moving	445460.5079	5025926.4912	31.2615		0.0235
	06990825_1420696	08/25/2005 10:21:00	Yes	yes	Moving	445460.5131	5025926.4897	31.2729		0.0102
	06990825_1420698	08/25/2005 10:21:00	Yes	yes	Moving	445460.5103	5025926.4884	31.2691		0.0261
	06990825_1421000	08/25/2005 10:21:00	Yes	yes	Moving	445460.5165	5025926.4851	31.2950		0.0272
	06990825_1421002	08/25/2005 10:21:01	Yes	yes	Moving	445460.5137	5025926.4668	31.2721		0.0353
	06990825_1421004	08/25/2005 10:21:01	Yes	yes	Moving	445460.5130	5025926.4950	31.2673		0.0274
	06990825_1421006	08/25/2005 10:21:01	Yes	yes	Moving	445460.5135	5025926.4918	31.2710		0.0241
	06990825_1421008	08/25/2005 10:21:01	Yes	yes	Moving	445460.5140	5025926.4899	31.2728		0.0241

Figure 4.1: Results View: Local Coordinates of Vehicle Trajectory at 0.2 sec. Interval.

4.2.2. Corsa Box Data

As explained earlier, the Corsa Box provides readings for the speed (V), longitudinal acceleration (a_x), lateral accelerations (a_y), steering (SP), throttle (TP), and braking positions (BP). The data were stored in the data acquisition box, and were downloaded through an RS232 serial cable to the laptop. The software named “Corsa for Windows” was used to process these data. From the Corsa main menu, the data could be displayed through the “Display” option. Figure 4.2 shows an example for Driver (2). The horizontal axis is the total time of the test, and a number of vertical axes are presented in stacked windows. From top to bottom, these windows display the data for lateral acceleration, longitudinal acceleration, speed, braking position, steering position, and throttle position.

The data could also be viewed in text format as shown in Figure 4.3. These data were exported as a text file to be viewed in spreadsheet format for further analysis. As

shown in the figure, the starting time was not a time in format of hours, minutes, and seconds. Instead, it started from time 0.0 and increased in 0.2 seconds intervals.

Figure 4.2: A Display of the Data Retrieved by Corsa Box.

4.2.3. Ball Banking Indicator (BBI) Data

The data from the ball bank indicator were downloaded through an RS232 serial cable. A software, namely “Advanced Serial Data Logger Standard”, was used to add a time stamp to the downloaded data. The downloaded data were saved as a text file, and could be retrieved using a spreadsheet. Figure 4.4 shows an example of downloaded data from the ball bank indicator (BBI). On average, the BBI could take readings with a rate of 4 readings/ second. The time stamps displayed using the software are to the seconds level. Later during post processing in an Excel spreadsheet, these time stamps were given

distinct counts so that no two timestamps would be similar. This could be seen in Figure 4.6, in the column “Time_count.”

Time	Lateral	Long	Speed	Brake	Steering	Throttle
00.000	0.032	1.063	19.347	0.572	0.538	35.429
00.200	0.032	0.167	40.411	0.572	0.430	33.429
00.400	0.032	0.178	41.359	0.572	0.000	30.571
00.600	0.072	0.169	41.752	0.429	-2.903	30.000
00.800	0.067	0.158	41.605	0.572	-3.333	29.714
01.000	0.017	0.151	44.084	0.572	-4.194	25.714
01.200	-0.012	0.137	43.879	0.572	-6.559	20.857
01.400	-0.039	0.098	44.658	0.572	-10.108	21.143
01.600	-0.081	0.094	44.333	0.572	-13.441	25.429
01.800	-0.140	0.075	44.190	0.572	-12.608	29.714
02.000	-0.150	0.091	44.553	0.572	-12.258	30.571
02.200	-0.129	0.114	45.150	0.572	-12.258	31.143
02.400	-0.124	0.116	45.553	0.429	-12.258	34.571
02.600	-0.117	0.121	46.031	0.572	-11.398	37.429
02.800	-0.107	0.123	46.474	0.572	-10.323	38.286
03.000	-0.098	0.121	46.960	0.572	-9.032	39.429
03.200	-0.091	0.119	48.048	0.572	-7.097	42.000
03.400	-0.062	0.120	48.593	0.572	-6.237	44.286
03.600	-0.067	0.144	49.227	0.572	-6.022	45.429
03.800	-0.076	0.128	49.707	0.572	-3.871	45.714
04.000	-0.034	0.116	51.052	0.572	-2.366	46.571
04.200	-0.001	0.139	51.428	0.572	-1.935	46.571
04.400	0.001	0.132	51.980	0.572	-0.323	46.571
04.600	0.017	0.130	52.692	0.572	0.323	46.857
04.800	0.029	0.132	54.035	0.572	1.075	46.571
05.000	0.041	0.128	54.472	0.572	1.720	46.571
05.200	0.040	0.123	54.980	0.572	2.366	46.571
05.400	0.069	0.126	55.810	0.572	3.118	46.571
05.600	0.080	0.121	56.597	0.572	3.441	46.571
05.800	0.093	0.126	57.111	0.572	3.656	46.571
06.000	0.103	0.128	57.917	0.572	3.656	46.571
06.200	0.096	0.126	58.747	0.572	3.763	46.571
06.400	0.084	0.132	59.544	0.572	3.978	46.571
06.600	0.105	0.130	60.133	0.572	4.516	46.571
06.800	0.112	0.112	60.831	0.572	4.409	46.206

Figure 4.3: Text Format of Data Downloaded from Corsa Box.

Although the methodology of downloading ball bank indicator readings and the synchronization with the other types of equipment proved to be feasible, it was not possible to download ball bank indicator for the whole test run for all drivers. Software issues caused the downloaded data to terminate at a certain point, thus ball bank indicator data were not complete for all drivers. However, results from the Corsa Box were sufficient to obtain data for lateral acceleration values.

	A	B	C	D	E	F
1	Time	BBI-value				
2	10:39:07	-4.23				
3	10:39:08	-7.07				
4	10:39:08	-12.06				
5	10:39:08	-10.88				
6	10:39:08	-9.11				
7	10:39:09	-8.95				
8	10:39:09	-8.51				
9	10:39:09	-7.42				
10	10:39:09	-7.66				
11	10:39:10	-7.04				
12	10:39:10	-9.22				
13	10:39:10	-3.64				
14	10:39:10	-2.42				
15	10:39:11	-3.25				

Figure 4.4: Downloaded Data from Ball Bank Indicator Opened in Excel Spreadsheet.

4.2.4. Front and Back Laser Guns Data

Data from the front and back laser guns were recorded on PCMCIA cards, and then were downloaded to the laptop. The data were saved as text file and could be opened using any spreadsheet program. An example for the front laser gun is shown in Figure 4.5. As shown from the figure, for every reading there is a time stamp, as well as the distance from the test vehicle to the vehicle in front (metres), and a recording of the relative speed between the test vehicle and the vehicle in front of it (km/h) as long as the relative speed was greater than 15 km/h. If the relative speed was less than 15 km/h, the relative speed was not recorded. Since the laser guns cannot be programmed to take automatic readings, the trigger of the laser gun was shot as long as there was a vehicle in front or behind the test vehicle. Therefore, there was no constant time interval between the readings.

	A	B	C	D
1	Time	Dist. From front car	Relative Speed(Front)	
2	11:41:35	49.61	24.8	
3	11:41:37	37.6	18.3	
4	11:47:11	58.62	32.8	
5	11:48:40	145.8	-15.4	
6	11:51:28	113.27	49.8	
7	10:40:55	26.37		
8	10:41:20	62.96		
9	10:41:44	48.1		
10	10:43:25	34.3		
11	10:43:58	61.46		
12	10:47:09	35.88		
13	10:47:40	121.62	-16.7	
14	10:47:46	24.77	-15	
15	10:47:46	24.97	-15	
16	10:47:46	25.19	-15	
17	10:47:46	25.39	-15	

Figure 4.5: Downloaded Data from Front Laser Gun Opened in Excel Spreadsheet.

4.2.5. Synchronization

Figure 3.14 showed a schematic of the different equipment used to collect data on the driver behaviour and their connection scheme. Each equipment was calibrated to record 5 readings/ second, except the front and back laser guns. Every equipment showed its output along with a timestamp, except for the Corsa Box which showed its data at time intervals of 0.2 seconds starting from 0.0 seconds. Therefore, to ensure complete synchronization, all clocks from all devices were set to the same exact timing before every run.

Using all data, a database for each driver was built. The database included the following:

1. Vehicle's path (Northing and Easting) obtained from the GPS receivers.
2. Vehicle's speed, lateral acceleration, longitudinal acceleration, steering, throttle, and braking positions obtained from the Corsa Box.
3. The distance and speed difference between the test vehicle and the vehicles in front of and behind it from the front and back laser guns.

The data from the different equipment were integrated through the interface of ArcView GIS 3.2 software. This was facilitated through importing the spreadsheet file for each equipment to ArcView GIS 3.2, then joining the tables using the common field, which was the time stamp. This way, for each driver, a data point was recorded every 0.2 second on the driver's behaviour as collected from each piece of equipment explained earlier.

In addition, the success of synchronization was further investigated by comparing the speed data obtained from the Corsa Box with a calculated speed obtained by dividing the distance travelled as recorded by the GPS receivers by the time interval between each two readings (0.2 seconds). It was ensured that the difference between the two speeds for most readings does not exceed 1-2 km/h. Therefore, it was guaranteed that data collected from all equipment for the thirty drivers were in complete synchronization. An example of the synchronized data is shown in Figure 4.6.

Time	Count	Time_count	V	V(2)	ax	ay	Throttle	Steering	Point_id	Easting	Northing	Bbl_value
10:39:26	2	10:39:26.2	57.11	55.72	0.081	0.035	46.571	3.656	06990825	444946.0816	5028161.947	2.58
10:39:26	3	10:39:26.3	57.92	55.73	0.086	0.035	46.571	3.656	06990825	444943.6458	5028160.036	2.29
10:39:26	4	10:39:26.4	58.75	56.17	0.1	0.04	46.571	3.763	06990825	444941.1724	5028158.133	2.1
10:39:26	5	10:39:26.5	59.54	58.07	0.088	0.038	46.571	3.978	06990825	444938.5152	5028156.304	2.53
10:39:27	1	10:39:27.1	60.13	59.09	0.086	0.033	46.571	4.516	06990825	444935.7743	5028154.497	2.4
10:39:27	2	10:39:27.2	60.83	58.68	0.093	0.031	46.286	4.409	06990825	444933.063	5028152.687	2.21
10:39:27	3	10:39:27.3	61.25	60.04	0.098	0.035	46	4.409	06990825	444930.28	5028150.848	1.38
10:39:27	4	10:39:27.4	62.09	60.34	0.088	0.026	46	4.409	06990825	444927.4436	5028149.061	0.67
10:39:27	5	10:39:27.5	62.55	60.56	0.083	0.024	46.286	4.516	06990825	444924.6012	5028147.261	-0.38
10:39:28	1	10:39:28.1	63.11	58.55	0.072	0.01	46.286	4.301	06990825	444922.3543	5028144.909	
10:39:28	2	10:39:28.2	63.73	62.17	0.06	0.005	46.571	4.301	06990825	444919.3809	5028143.152	0.07
10:39:28	3	10:39:28.3	64.23	62.1	0.055	0.007	46.571	3.871	06990825	444916.3965	5028141.421	0.11
10:39:28	4	10:39:28.4	64.8	62.81	0.043	0	46.286	3.548	06990825	444913.3204	5028139.774	0.07
10:39:28	5	10:39:28.5	65.43	65.4	0.043	-0	46.286	3.333	06990825	444909.6944	5028140.003	0.19
10:39:29	1	10:39:29.1	66	65.21	0.043	0.003	46.571	2.796	06990825	444906.4292	5028138.434	
10:39:29	2	10:39:29.2	66.45	65.22	0.045	0.01	46.571	2.688	06990825	444903.1635	5028136.865	-0.49
10:39:29	3	10:39:29.3	67.01	65.94	0.038	0.037	46.571	2.796	06990825	444899.8397	5028135.325	-0.61
10:39:29	4	10:39:29.4	67.61	66.51	0.043	0.031	46.571	2.688	06990825	444896.4823	5028133.782	-1.65
10:39:29	5	10:39:29.5	68.35	67.05	0.029	0.026	46.571	2.688	06990825	444893.0755	5028132.276	0.35
10:39:30	1	10:39:30.1	68.72	67.71	0.043	0.028	46.571	2.688	06990825	444889.6283	5028130.771	
10:39:30	2	10:39:30.2	69.15	68.3	0.027	0.033	46.857	2.581	06990825	444886.1321	5028129.296	1.3
10:39:30	3	10:39:30.3	70	68.8	0.034	0.044	46.857	2.473	06990825	444882.6008	5028127.834	1.17
10:39:30	4	10:39:30.4	70.57	69.25	0.06	0.042	46.571	2.473	06990825	444879.036	5028126.387	0.73
10:39:30	5	10:39:30.5	71	69.96	0.062	0.042	46.571	2.258	06990825	444875.4228	5028124.955	-0.14

Figure 4.6: Example of synchronized data in spreadsheet format.

In addition to synchronization of the data, ArcView GIS 3.2 is a practical way of displaying the data collected from the different equipment. Through it, the user can view the study area, the centreline of the highways traversed in the test route, as well as the vehicle's path. The driver behaviour data were linked to the centreline of the highways through a spatial joint which is one of the capabilities of Arcview 3.2. Therefore, for every 5 m on the centreline of the highway, a driving data point was available. Analysis of the data could be performed either through ArcView GIS 3.2 or through exporting it to Excel.

4.3 Alignment Data

One of the major components for the data required for analysis was the horizontal alignment of the highways traversed. As explained earlier in Chapter 3, six highways were studied. These highways were either provincial highways or regional roads. Highways 417, 7, and 416 were provincial highways. Data for these highways were

obtained from the Ontario Ministry of Transportation in the form of design plans. The characteristics determined from these plans were the radius of curve, degree of curve, length of curve, beginning of curve, end of curve, and the spiral length and parameters if present.

In addition, the City of Ottawa provided data on the Northing and Easting coordinates of all roads in Ottawa in the form of GIS shape files that could be viewed using ArcView. Therefore, data required for the Regional Roads (RR3, RR6, RR12, and RR73) could be obtained from this database. These coordinates were fitted to tangent sections, curves, and spiral lengths using an extension called “Roadfit” developed by Imran et al. (2005). This extension was written for Arcview using the programming language “Avenue”. The extension uses the shape files of roads in point format and calculates the geometric characteristics of the horizontal curves including radius, length, coordinates of center of curve, and spiral lengths. In the study, the extension was verified using GPS data for the horizontal alignments of Highways 31 and 41 and was proven successful.

For the alignment data in hand, the “Roadfit” extension was verified for the curve characteristics of Highways 7, 416, and 417. Table 4.1 shows a comparison of the characteristics obtained from the plans given by the MTO and that obtained using the extension in terms of the curve radii, lengths, and spiral lengths. The extension was able to determine the radii with values very close to the actual alignment. Although there are differences in the values, the extension was able to a great extent to calculate radii close to the actual values. In terms of the curve and spiral lengths, although the values are not identical, the sum of calculated curve lengths and spiral lengths are very close between

the values calculated using the extension and the actual alignment. Therefore, it was verified that the extension could be applied on all highways in the study.

Table 4.1: Comparison of Characteristics Obtained from MTO Plans and That Obtained Using “Roadfit”.

417 Curve ID	Plans			Road fit extension			
	R (m)	Lc (m)	Ls(m)	R (m)	Lc (m)	Entry Ls(m)	Exit Ls(m)
1	698.55	-	-	801.66	359.60	-	15.00
2	2555.672	-	-	2211.64	250.00	-	204.39
3	698.55	202.875	-	488.74	104.99	25.00	30.00
4	873.188	469.727	-	869.31	499.98	-	-
5	1600	325.418	58.25	863.21	148.18	-	70.00
6	2000	491.776	86.612	1756.90	240.00	220.00	90.00
7	2000	358.521	66.612	2045.95	220.00	130.00	15.00
8	873.188	143.413	60.96	916.02	170.00	70.00	25.00
9	2413.109	-	-	2402.16	440.00	95.00	-
416 Curve ID	Plans			Road fit extension			
	R (m)	Lc (m)	Ls(m)	R (m)	Lc (m)	Entry Ls(m)	Exit Ls(m)
1	1800	699.619	68.056	1810.15	844.99	-	-
2	1000	214.939	67.6	1299.08	589.97	-	-
3	1000	918.341	67.6	981.37	639.95	200.00	270.00
4	1000	940.337	67.6	978.12	739.98		235.00
7 Curve ID	Plans			Road fit extension			
	R (m)	Lc (m)	Ls(m)	R (m)	Lc (m)	Entry Ls(m)	Exit Ls(m)
1	1200	1106.252	75	1167.95	1073.56	25.00	273.53
Ramps Curve ID	Plans			Road fit extension			
	R (m)	Lc (m)	Ls(m)	R (m)	Lc (m)	Entry Ls(m)	Exit Ls(m)
417-7c1	475	94.049	84.211	447.83	104.99	35.00	50.00
417-7c2	700	1020.654	57.143	548.22	384.09	185.00	244.87
416_1	150	81.83	48	151.42	94.97	20.00	20.00
416_2	90	50	54.44	98	49.95	25.00	30

Curve ID is composed of the name of the highway and curve number on this highway (417c1 means the first curve on Highway 417), for ramps the Curve ID is composed of two parts, the first is the highway the ramp is exiting and the second is the highway it is entering,

Table 4.2 shows the geometric characteristics of the different curves on the highways studied. The curves are classified into isolated curves, reverse curves, and curves on two-lane two-way highways or freeways.

Table 4.2: Geometric Characteristics of Curves used in Analysis.

	Curve ID	R (m)	Direction	Lc (m)	Entry Ls (m)	Exit Ls (m)
Ramps	417-7c1	447.83	RT	104.99	35.00	50.00
	417-7c2	548.22	LT	384.09	184.99	244.87
	416-12c1	151.42	RT	94.97	20.00	20.00
	416-12c2	98.00	LT	49.95	25.00	30.00
	6-416c1	71.62	RT	119.85	54.90	84.79
Isolated Curves	Two-lane Highways	6c1	RT	159.97	-	-
		7c1	RT	1073.56	25.00	273.53
		73c8	RT	149.97	40.00	-
		73c9	LT	29.99	-	-
		12c1	RT	225.83	134.98	119.99
		73c1	RT	179.98	35.00	84.99
		73c2	LT	354.99	-	139.99
		73c3	RT	418.82	90.00	61.77
		73c7	RT	289.02	-	125.78
	Freeways	417c1	LT	359.60	-	15.00
		417c2	RT	250.00	-	204.39
		417c3	RT	104.99	25.00	30.00
		417c4	RT	499.98	-	-
		417c8	LT	170.00	70.00	25.00
		417c9	LT	440.00	95.00	-
		417c10	RT	781.57	-	-
		417c11	LT	502.49	35.00	-
		416c1	LT	844.99	-	-
		416c2	RT	589.97	-	-
		416c3	LT	639.95	199.99	269.99
		416c4	RT	739.98	-	234.99
Reverse Curves	Two-lane Highways	3c4	RT	381.54	-	-
		3c5	LT	224.95	54.98	-
		3c6	LT	169.97	35.00	-
		3c7	RT	39.99	25.00	10.00
		3c8	RT	159.99	30.00	-
		3c9	LT	70.97	30.00	40.00
		3c10	RT	429.90	34.99	-
		3c11	LT	369.94	-	-
		12c2	LT	149.99	20.00	65.00
		12c3	RT	70.00	25.00	40.00
		12c4	RT	99.98	65.00	65.00
		12c5	LT	29.99	19.98	14.99
		73c5	RT	390.15	39.48	-
		73c6	LT	186.90	-	59.97
	Freeways	417c5	LT	148.18	-	70.00
		417c6	RT	240.00	220.00	90.00
		417c7	LT	220.00	130.00	15.00

Curve ID is composed of the name of the highway and curve number on this highway (417c1 means the first curve on Highway 417), for ramps the Curve ID is composed of two parts, the first is the highway the ramp is exiting and the second is the highway it is entering, LT: left turn curve, RT: right turn curve, and Δ =Deflection angle.

As previously mentioned, the driver behaviour data were linked to the centreline of the highways through a spatial joint which is one of the capabilities of Arcview 3.2. For every five metres on the centreline of the highway, a driving data point was available. Therefore, driver behaviour data could be examined, analyzed, and modelled given the road geometry, traffic conditions, vehicle dynamics, and consistency ratings of the highways.

4.4 Methodology of Analysis

To highlight the significance of the collected data, Figure 4.7 shows part of the collected data of driver behaviour for one driver on a section of 5.5 km on Regional Road 3. The road is a two-lane two-way rural highway with a speed limit of 80 km/h. The horizontal axis represents the distance and the first and second axes from the bottom represent the curve radii (m) and speed (km/h) respectively. With a closer look, it is obvious that the vehicle's speed decreases with decreasing radii values from 95 km/h on long independent tangents and curve radius of 740 m to 87 km/h on curve radius of 335 m to 77 km/h on curve radius of 250 m. Given the operating speed values and lateral acceleration rates of all 30 drivers of the sample, these data could be related to the geometric characteristics of the highway.

Evidently, parts of the highway where the driver's speed is affected by other vehicles or by traffic control devices should be extracted. An example of that is shown in Figure 4.7. The driver's speed was significantly reduced to 55 km/h at a certain point on Regional Road 3. By taking a look at the driver's view from the recorded video it turns out that the reduced speed is due to the presence of another vehicle in front of the test vehicle which slowed down to turn left at an intersection with another road.

Therefore, driver behaviour data in terms of speed, lateral and longitudinal acceleration rates, vehicle path, steering angles, and braking and throttle position obtained from the experiment were examined and analyzed given the road geometry and traffic conditions. Each of the six research components that will fulfill the research objective and the data required for each type of analysis are shown in Figure 4.8.

Figure 4.7: Alignment, Speed and Driver's View for Part of RR3.

Figure 4.8: Types of Analyses using Driver Behaviour Data Collected

CHAPTER 5. VEHICLE PATH AND TRANSITION TO HORIZONTAL CURVES

In this chapter, driver behaviour is studied in a quantitative manner in terms of vehicle path and steering behaviour. Analysis of vehicle path is used to determine the difference between the vehicle path on a curve and the curve's actual alignment for both two-lane highways and freeways. The driver's steering behaviour is then investigated in order to find the trend of rate of change of steering angle for the different types of roads and different curve radii. Finally, the steering behaviour is used in modelling the desirable spiral lengths for horizontal curves. In addition, the desirable spiral lengths were compared with the different controls of spiral length found in the North American design guides (AASHTO 2004, TAC 1999) as discussed in Chapter 2. Furthermore, recommended values for rate of change of lateral acceleration and spiral parameters are given (Said et al. 2006, Said et al. 2007, Said et al. 2009).

5.1 Vehicle Path

The parameter that best describes the vehicle path on horizontal curves is the instantaneous curvature ($1/R$). It can be calculated for every three points on the driver's path using the simple mathematical formulation of a circle's radius based on three points on the circumference as follows (Math Forum 2000).

$$x_0 = \frac{\begin{vmatrix} x_1^2 + y_1^2 & y_1 & 1 \\ x_2^2 + y_2^2 & y_2 & 1 \\ x_3^2 + y_3^2 & y_3 & 1 \end{vmatrix}}{2 \times \begin{vmatrix} x_1 & y_1 & 1 \\ x_2 & y_2 & 1 \\ x_3 & y_3 & 1 \end{vmatrix}}, \quad y_0 = -\frac{\begin{vmatrix} x_1 & x_1^2 + y_1^2 & 1 \\ x_2 & x_2^2 + y_2^2 & 1 \\ x_3 & x_3^2 + y_3^2 & 1 \end{vmatrix}}{2 \times \begin{vmatrix} x_1 & y_1 & 1 \\ x_2 & y_2 & 1 \\ x_3 & y_3 & 1 \end{vmatrix}} \quad (5.1)$$

$$R = \sqrt{(x_1 - x_0)^2 + (y_1 - y_0)^2} \quad (5.2)$$

Where (x_1, y_1) , (x_2, y_2) , and (x_3, y_3) = coordinates of the three points on the circumference of a circle; (x_0, y_0) = coordinates of the center of the circle; and R = circle's radius.

This method of calculating the instantaneous curvature of the vehicle's path is a different approach from that used in Cafiso et al. (2004, 2005). In their study, the instantaneous curvature ($1/R$) was calculated using the lateral acceleration rate as shown in Equation 5.3.

$$\frac{1}{R} = \frac{a_y}{V_x^2} \quad (5.3)$$

Where a_y is the lateral acceleration felt by the vehicle on the horizontal curve, and V_x is the longitudinal speed of the vehicle.

The main drawback of their method lies in that the lateral accelerations obtained from accelerometers include the effect of body roll of the vehicle. Even though many research works have justified neglecting the value of body roll, the two methods do not result in similar values of curvature. Therefore, calculating the instantaneous curvature from the actual vehicle path is more accurate.

The curvature adopted by drivers while traversing each curve in the experiment was first analyzed visually using the profile of the 85th percentile instantaneous curvature of the drivers' sample on each curve in the study database and comparing it to the alignment's curvature extracted from the surveying data as explained earlier in Section 4.3. Figure 5.1 presents examples of these profiles for three specific curves. The profiles

for all curves on the route are presented in Appendix B. The figure shows that drivers follow a spiral transition path at the beginning and end of the curve, regardless of the presence of spirals. It also shows that for curves with spirals, the spiral transition of the drivers is not necessarily equal to that of the actual alignment, which will be further illustrated in the following section. Generally, drivers would steer their vehicles after perceiving the road's curvature, which is more evident on the circular curve than the spiral. Therefore, by the beginning of the curve, the curvature of the vehicle path would not have reached the curvature of the actual alignment. To compensate for this situation, the vehicle path would require a greater curvature in the middle of the curve than the alignment's actual curvature. This is called curve-cutting behaviour, and was also reported in other studies (Johnston 1983, Glennon and Weaver 1971, Shinar et al. 1974). This explanation may be strengthened by the fact that the difference in curvature between the vehicle path and the alignment was greater for the second curve, Figure 5.1 (b), which is shorter than the first curve, Figure 5.1 (a), and thus smaller distance is available to correct the vehicle path.

The third curve, Figure 5.1 (c), is considered a special case for the curves analyzed in the experiment, where the drivers perceived the curve early and started changing the vehicle path before the beginning of the curve. This could be explained by the long tangent preceding the curve radius which was greater than 500 m, allowing for a long sight distance ahead of the curve, and the small deflection angle of 20° , which further helped the drivers follow the alignment's curvature.

(a) Curve with no Spirals (Curve ID: 3c11; $R=256$ m; $L_c=370$ m).

(b) Curve with Spirals (Curve ID: 3c8; $R=556$ m; $L_c=160$ m; $L_{s1}=30$ m; $L_{s2}=0$).

(c) Instantaneous Curvature Lower than Actual Alignment (Curve ID: 6c1; $R=463$ m; $L_c=160$ m).

Figure 5.1: Vehicle Path for Different Curves.

To further investigate the curvature adopted by drivers on different curve radii on both two-lane highways and freeways, the difference between vehicle path curvature and actual alignment curvature ($\Delta Curve$) was calculated as shown in Equation 5.4.

$$\Delta Curve = \left(\frac{1}{R} \right)_{VP_{85}} - \left(\frac{1}{R} \right)_C \quad (5.4)$$

Where $\left(\frac{1}{R} \right)_{VP_{85}}$ = the 85th percentile vehicle path curvature at the point of maximum curvature for a certain curve, and $\left(\frac{1}{R} \right)_C$ = the alignment's curvature.

The calculated values of $\Delta Curve$ are shown in Table 5.1. The relation between $\Delta Curve$ and the actual radius (R) is also shown in Figure 5.2, where the general trend of the relationship between the two parameters is that as R decreases, $\Delta Curve$ increases. Although the relationship is as expected, it raises a safety concern for sharper curves, where the difference is higher, since a greater $\Delta Curve$ implies that drivers are experiencing higher lateral accelerations than that designed for. This will be further discussed in Chapters 6 and 8.

The figure also shows that there were five points which can be considered as outliers. Four of these points (Points 1 – 4 in the figure) correspond to the second curve of reverse curves with deflection angles less than or equal to 20°. The curve IDs of these points are 3c7, 3c9, 12c3, and 12c5.

Table 5.1: Calculated Values of $\Delta Curve$ for All Curves.

	Curve ID	R (m)	Lc (m)	Deflection (°)	$(1/R)_{VP_{.85}}$ (1/m)	$(1/R)_C$ (1/m)	$\Delta Curve$ (1/m)
Freeways	416c1	1810.15	844.99	26.74	0.00100	0.00055	0.00045
	416c2	1299.08	589.97	26.01	0.00158	0.00077	0.00081
	416c3	981.37	639.95	37.35	0.00130	0.00102	0.00028
	416c4	978.12	739.98	43.33	0.00154	0.00102	0.00052
	417c1	801.66	359.60	25.69	0.00241	0.00125	0.00116
	417c2	2211.64	250.00	6.47	0.00074	0.00045	0.00028
	417c3	488.74	104.99	12.30	0.00278	0.00205	0.00073
	417c4	869.31	499.98	32.94	0.00219	0.00115	0.00104
	417c5	1494.59	148.18	25.41	0.00110	0.00067	0.00043
	417c6*	1756.90	240.00	7.82	0.00083	0.00057	0.00026
	417c7*	2045.95	220.00	6.16	0.00100	0.00049	0.00051
	417c8	916.02	170.00	10.63	0.00166	0.00109	0.00057
	417c9	2402.16	440.00	10.49	0.00072	0.00042	0.00030
	417c10	3147.22	781.57	14.22	0.00060	0.00032	0.00028
	417c11	2151.94	502.49	13.37	0.00075	0.00047	0.00029
Rural Two-lane Highways	3c4	273.32	381.54	79.95	0.00551	0.00366	0.00185
	3c5*	239.17	224.95	53.87	0.00548	0.00418	0.00130
	3c6	325.05	169.97	29.95	0.00406	0.00308	0.00099
	3c7*	180.17	39.99	12.71	0.00601	0.00555	0.00046
	3c8	555.89	159.99	16.48	0.00305	0.00180	0.00126
	3c9*	203.35	70.97	19.99	0.00497	0.00492	0.00005
	3c10	267.25	429.90	92.13	0.00537	0.00374	0.00163
	3c11*	255.94	369.94	82.78	0.00535	0.00391	0.00144
	6c1	463.05	159.97	19.79	0.00220	0.00216	0.00004
	7c1	1167.95	1073.56	52.64	0.00180	0.00086	0.00094
Urban/Sub-Urban Two-lane Highways	12c1	475.66	225.83	27.19	0.00348	0.00210	0.00138
	12c2	627.34	149.99	13.69	0.00306	0.00159	0.00146
	12c3*	538.62	70.00	7.44	0.00221	0.00186	0.00036
	12c4	263.39	99.98	21.74	0.00456	0.00380	0.00076
	12c5*	168.83	29.99	10.17	0.00596	0.00592	0.00004
	73c1	789.47	179.98	13.06	0.00224	0.00127	0.00097
	73c2	856.69	354.99	23.73	0.00186	0.00117	0.00069
	73c4	595.62	198.71	19.11	0.00250	0.00168	0.00082
	73c5	571.42	390.15	39.10	0.00273	0.00175	0.00098
	73c6*	360.70	186.90	29.68	0.00490	0.00277	0.00213
	73c7	637.28	289.02	25.97	0.00277	0.00157	0.00120
	73c8	230.55	149.97	37.26	0.00657	0.00434	0.00223
	73c9	86.00	144.97	73.26	0.01385	0.01163	0.00222

* Second curves of reverse curves

Figure 5.2: Relationship between Curve Radii and Δ_{Curve} for All Data Points.

A potential reason why these points did not follow the general trend is that having driven on the first curve of the reverse curve pair, drivers had greater expectations of the alignment of the second curve, especially the easily visible second curves with deflection angles less than 20° . This makes the vehicle's path for the second curve with small deflection angles very close to that of the actual alignment, and therefore the difference between the two is very low.

The fifth outlier is the curve previously presented in Figure 5.1, part (c), curve ID 6C1, and introduced as a special case. For this curve, the 85th percentile vehicle path was very close to the alignment's curvature, and the difference was minimal for the reason explained earlier. This curve has a long tangent before it, allowing for a long sight distance ahead of the curve, in addition to a small deflection angle.

Using linear regression, the best relationship between $\Delta Curve$ and R after removing the five potential outliers was found to be logarithmic as shown in Figure 5.3 and in Equation 5.5. Table 5.2 shows the model characteristics.

$$\Delta Curve = -0.00057 \ln(R) + 0.0047 \quad (5.5)$$

Figure 5.3: Relationship between Curve Radii and $\Delta Curve$ without Outliers.

Table 5.2: Model for $\Delta Curve$ on Horizontal Curves.

Variables	Coefficients	SEE	<i>t</i> -statistic	<i>p</i> -value
Constant	0.0047	<0.0001	10.59	< 0.0001
$\ln R$	-0.00057	<0.0001	-8.48	< 0.0001
Model diagnostics: Adjusted $R^2=0.689$, Degrees of freedom=32				
$F= 71.96$				

It is noticed that a similar trend exists for both two-lane highways and freeways, and that generally higher differences in curvature are observed on sharper curves. Therefore, this relationship is more pronounced on two-lane highways.

An attempt was made to find the general trend between $\Delta Curve$ and other geometric characteristics of the curve as the curve length and the deflection angle. The relations of these parameters to $\Delta Curve$ are shown in Figure 5.4 and Figure 5.5 respectively. Figure 5.4 suggested the existence of a general trend between the curve length and difference in curvature, where as the curve length increases $\Delta Curve$ decreases. The trend also indicates that for smaller curve lengths the value of the curve difference is very sensitive, while as the length exceeds 500 m the difference appears to be smaller and constant. This is because drivers have a longer length on the curve to adjust the vehicle's path to alignment. Several outliers however remain, for example when the curve length is less than 200 m. These are the same curves that were outliers for the previous relationship between $\Delta Curve$ and curve radius. The correlation coefficient of $\Delta Curve$ with the curve length was $r=-0.1$ with the presence of the five outliers, and $r=-0.333$ with the removal of these curves from the analysis.

For the relationship between $\Delta Curve$ and the deflection angle, Figure 5.5 shows that as the deflection angle increases $\Delta Curve$ increases. Although the relationship between $\Delta Curve$ with the curve radius is not very strong, it shows that as the deflection angle increases, the difficulty of driver to follow the actual alignment curvature increases as well.

Figure 5.4: Relationship between Curve Length and $\Delta Curve$ for All Curves.

Figure 5.5: Relationship between Deflection Angle (°) and $\Delta Curve$ for All Curves.

In summary, the model described in Equation 5.5 showed that for sharper curves, there were higher differences in curvature between the vehicle path and actual alignment. This could be a safety concern, and hence further investigation was carried out using the steering behaviour data in this chapter, and lateral acceleration data in Chapter 6. In Chapter 8, the relation between $\Delta Curve$ and safety in terms of collision frequency was also investigated.

5.2 Driver Steering Behaviour

5.2.1. Steering Angle

Studying the steering behaviour should allow a comparison between the vehicle path and the built alignment. This should in turn facilitate improving the design of horizontal curves through determining the spiral length that conforms to the drivers' steering behaviour. In this context, the steering behaviour is quantified here using the steering angle, which is the angle the steering wheel makes with the straight position. Theoretically, this angle ranges from 0° at the straight position to $\pm 360^\circ$ with the sign convention indicating positive angles for right turns, and vice versa.

Figure 5.6 shows the profile of driver steering behaviour in terms of the 85th percentile values on the same curves shown in Figure 5.1. The profile of the steering angle is expectedly very close to that of the vehicle path for the curves shown and for all curves studied. Thus, this represented a further verification of the consistency of the different pieces of equipment used in the data collection and the synchronization of the data. It also verifies that drivers, in approaching horizontal curves, change their behaviour gradually to follow a natural spiral-curve-spiral path.

(a) Curve with no Spirals (Curve ID: 3c11; $R=256$ m; $L_c=370$ m).

(b) Curve with Spirals (Curve ID: 3c8; $R=556$ m; $L_c=160$ m; $L_{s1}=30$ m; $L_{s2}=0$).

(c) Curve with No Spirals (Curve ID: 6c1; $R=463$ m; $L_c=160$ m).

Figure 5.6: Examples of Steering Paths on Curves

5.2.2. Rate of Change of Steering Angle

While driving on the road, drivers receive information through their perception of the geometric characteristics ahead, and control their vehicle accordingly. Misconception of the alignment ahead or violating driver's expectations may lead to extra workload on the driver. One of the parameters that could describe driver workload is the rate of change of steering angle (*RSA*) while negotiating curves. It is calculated as:

$$RSA = \frac{(SA_n - SA_{n-1})}{0.2} \text{ degrees/ second} \quad (5.6)$$

Where SA_n , SA_{n-1} represent two consecutive readings of steering angles; and 0.2 represents the time interval in seconds between two consecutive readings.

RSA in this study was calculated as the 85th percentile value for the 30 drivers, where the value corresponding to each driver is the maximum rate observed on a specific curve. It should be noted that in analyzing the *RSA*, the ramp curves followed the same trend as the horizontal curves on two-lane highways and freeways. Therefore, they were included in the analysis. Values of *RSA* are shown in Table 5.3.

The plotted results in Figure 5.7 show a strong correlation between the curve radius and *RSA*, where *RSA* increases as the curve radius decreases, thus implying greater driver workload on sharp curves. A model was developed using the data shown, and the results are shown in Equation 5.7 and in Table 5.4

$$RSA = 7.129 + 2947.282 / R \quad (5.7)$$

Table 5.3: Values of *RSA* for All Curves in Analysis.

	Curve ID	R (m)	Lc (m)	Deflection (°)	<i>RSA</i> (°)
Freeways	416c1	1810.15	844.99	26.74	7
	416c2	1299.08	589.97	26.01	8
	416c3	981.37	639.95	37.35	8
	416c4	978.12	739.98	43.33	9
	417c1	801.66	359.60	25.69	12
	417c2	2211.64	250.00	6.47	10
	417c3	488.74	104.99	12.30	11
	417c4	869.31	499.98	32.94	8
	417c5	1494.59	148.18	25.41	5.12
	417c6*	1756.90	240.00	7.82	8
	417c7*	2045.95	220.00	6.16	6
	417c8	916.02	170.00	10.63	11
	417c9	2402.16	440.00	10.49	8
	417c10	3147.22	781.57	14.22	7
	417c11	2151.94	502.49	13.37	6
Rural Two-lane Highways	3c4	273.32	381.54	79.95	19.9
	3c5*	239.17	224.95	53.87	24.97
	3c6	325.05	169.97	29.95	14.81
	3c7*	180.17	39.99	12.71	18.68
	3c8	555.89	159.99	16.48	12
	3c9*	203.35	70.97	19.99	21.87
	3c10	267.25	429.90	92.13	20
	3c11*	255.94	369.94	82.78	26
	6c1	463.05	159.97	19.79	15
	7c1	1167.95	1073.56	52.64	9
Urban/Sub-Urban Two-lane Highways	12c1	475.66	225.83	27.19	17
	12c2	627.34	149.99	13.69	19
	12c3*	538.62	70.00	7.44	13
	12c4	263.39	99.98	21.74	17
	12c5*	168.83	29.99	10.17	25
	73c1	789.47	179.98	13.06	11
	73c2	856.69	354.99	23.73	11
	73c3	943.67	418.82	25.42	16
	73c4	595.62	198.71	19.11	17
	73c5	571.42	390.15	39.10	15
	73c6*	360.70	186.90	29.68	14
	73c7	637.28	289.02	25.97	10
	73c8	230.55	149.97	37.26	22
	73c9	86.00	144.97	73.26	45
Freeway Ramps	417-7c1	447.83	104.99	13.43	10
	417-7c2	548.22	384.09	40.13	14
	416-12c1	151.42	94.97	35.92	20
	416-12c2	98.00	49.95	29.19	35
	6-416c1	71.62	119.85	95.84	48

*Second curve of pair of reverse curves

Figure 5.7: Relation between Curve Radii and Maximum RSA.

Table 5.4: Model for RSA on Horizontal Curves.

Variables	Coefficients	SEE	<i>t</i> -statistic	<i>p</i> -value
Constant	7.129	0.606	11.758	< 0.0001
1/ <i>R</i>	2947.282	148.642	19.828	< 0.0001
Model diagnostics: $R^2_{adj.} = 0.901$, Degrees of freedom=43, $F = 393.151$				

Given the power relationship in the figure, concerns become greater for curves with radii less than 200 m where *RSA* increases to more than 27 degrees/sec for curve radius of 150 metres and 37 degrees/sec for curve radius of 100 metres. This is compared to *RSA* of 9 to 10 degrees/sec for curve radii greater than 1000 m. For radii between 200 and 1000 m, *RSA* ranged from 10 to 22 degrees/sec. This shows that radii less than 200 m could impose significantly higher workloads on drivers. The strong relation between the radius and *RSA* should be analyzed further as a potential contributor to increased

workload and thus increased collision frequency. The relationship of *RSA* with safety will be demonstrated in Chapter 8.

5.3 Driver Behaviour and Spiral Curves

Many studies have claimed the ineffectiveness of the presence of spiral curves and recommended that design codes should no longer recommend using them since they can deceive the drivers into maintaining an excessive speed (Stewart and Chudworth 1990, Tom 1995). Tom (1995) also showed that curves with no spirals had lower collision rates. Perco (2005) found that the natural steering time on curves, which was found to average between 2.23 and 2.25 s, did not differ between curves with or without spiral transitions. It was suggested that the advantage of the spiral transition lies in that the speed reduction would end before the curve begins. That is, the driver brakes along the spiral curve which has a lower curvature than the circular curve. Therefore, there would be less demand for side friction and greater longitudinal friction would be available for speed reduction on the spiral curve. On the other hand, on curves without spiral transitions, at least part of the speed reduction would occur along the circular arc, where the demand for side friction is maximum, thus reducing the available longitudinal friction for braking. To confirm this advantage of spiral curves, the study recommended a comparison between the alignment curvature with the real vehicle path. Perco (2005) also stated that the advantage of spiral curves diminishes if the spiral was too long, which would affect the driver's curve perception, and consequently could affect the path and speed adopted at the curve entrance. In conclusion, Perco (2005) recommended that the most desirable spiral length, which provides the advantages previously explained compared to curves with no spirals, is one that equals the distance traveled during natural steering time at

operating speed. This recommendation was also mentioned by Bonneson (1999), and is adopted in this chapter.

In this context, the term steering length is introduced as the length the driver takes to adjust the vehicle path from a tangent section to the curve section and is used to compare the vehicle path to the built alignment and to determine the desirable spiral length. For each curve in the test route, the steering length was measured using the steering angle profile for each of the 30 drivers in the test sample. Then the 85th percentile was calculated and compared to the actual spiral length of the alignment. Figure 5.8 shows the relationship between the 85th percentile steering length and the actual alignment spiral length for both freeways and two-lane highways on the test route. Regardless of the presence of the spiral and its length, the steering length ranged from 80 to 170 m for freeways and from 60 to 150 m for two-lane highways, which means that the lengths of spirals of the actual alignment were different from those adopted by 85% of the drivers. Since it is important to provide alignments conforming to the natural vehicle path, a next step was to model driver steering behaviour and to compare it to that of the current design guides. The comparison between the two would help in finding spiral lengths that conform to the natural steering behaviour, and which would no longer deceive the drivers. This would be an improved approach compared to not using spirals at all due to their importance, as previously mentioned.

(a) Two-Lane Highways.

(b) Freeways.

Figure 5.8: Relation between Spiral Length and Steering Length.

5.4 Quantification of Spiral Length Using Steering Behaviour

5.4.1. Modelling Approach

Two different approaches were considered in the attempts to model the steering length. The first approach, which is commonly used in highway design, was to model the 85th percentile of steering length. Most of the existing driver behaviour models specify the 85th percentile as a critical percentile recommended for design purposes. For example, when performing an evaluation of the design consistency of a geometric alignment, the operating speed of the vehicles is taken as the 85th percentile speed.

However, recent studies argued that it is also important to consider other percentiles to provide a greater margin of safety (Bonneson 2000, Medina and Tarko 2005). Therefore, the second approach was to model the mean steering length and its dispersion. This latter approach was introduced by Medina and Tarko (2005), and is gaining wider acceptance in geometric design of highways because of its ability to predict the overall distribution rather than one statistic such as the 85th percentile. Another advantage of this approach is that it can determine the factors that affect both the mean steering length and its dispersion.

The procedure is referred to as the ordinary least squares-panel data (OLS-PD). Panel data is usually used in econometrics and represents a number of observations over time on a number of cross-sectional units. For the purpose of this study, and similar traffic applications, the panel is created by determining percentile steering lengths from the 5th to the 95th percentiles in increments of five for each site. Then, each of the explanatory variables is multiplied by the Z_p value corresponding to this particular percentile. This method improves the efficiency of modelling the parameter, and reduces

multicollinearity between explanatory variables since the degrees of freedom increase due to the higher number of observations (Medina and Tarko 2005, 2007).

The general model form can be written as:

$$L_{ST,ip} = L_{ST,iAVG} + Z_p \cdot L_{ST,iSTDEV} + \epsilon = \sum_j b_j \cdot X_{ij} + \sum_k c_k \cdot (Z_p \cdot X_{ik}) + \epsilon \quad (5.8)$$

In this form, $L_{ST,ip}$ represents the steering length at any percentile (p) at a certain curve (i). Assuming a normal distribution, $L_{ST,ip}$ is calculated by adding the average steering length at that curve ($L_{ST,iAVG}$) to the product of the standardized normal variable (Z_p) and the standard deviation of the steering length at that curve ($L_{ST,iSTDEV}$). For example if the mean steering length was to be calculated, the Z_p value used would be $Z_{50}=0$, and for the 85th percentile $Z_{85}=1.036$. Then, each of the explanatory variables is multiplied by the Z_p value corresponding to this particular percentile. In the regression form, in the right-hand side of Equation 5.8, $L_{ST,iAVG}$ is modelled as the sum of the products of each relevant explanatory variable (X_{ij}) by its regression coefficient (b_j) and the dispersion is modelled as the sum of the products of each relevant explanatory variable (X_{ik}) by its regression coefficient (c_j) by Z_p (Medina and Tarko 2005).

Medina and Tarko (2005) used this modelling approach, which will be referred to as variation model approach hereon, and found it to be a very suitable method to model speed behaviour on tangents and curves and to distinguish the mean speed factors from the speed dispersion factors. They also used this approach to model the transition speeds at the entrance and exit of a horizontal curves (Medina and Tarko 2007). Similar approaches have also been used in medical studies. Morgenstern et al. (2002) modelled the mean total body water as a function of patient characteristics, and the standard

deviation of the individual observations was considered constant. Morrell et al. (1997) modelled the mean of hearing thresholds and the standard deviation separately.

The objective of using both model approaches (85th percentile approach and the variation modelling approach) was to compare between their results to justify the use of the new approach. To do this, the two approaches will be used to model the entrance steering length. The results of modelling the 85th percentile and the 50th percentile will first be presented, followed by the results of modelling using the variation modelling approach.

5.4.2. Entrance Steering Length

For each horizontal curve in the study, the natural steering profile represented by the steering angle was plotted individually for each of the 30 volunteer drivers, which is shown in Appendix C. The steering length for each driver was then calculated from the profile as shown previously in Figure 5.6 (a) as an example. Then, for each curve, the steering length adopted by 85% of the 30 drivers at the curve entrance ($L_{85_ST_EN}$) and the average steering length ($L_{AVG_ST_EN}$) were determined. These data are shown in Table 5.5, and a summary of descriptive statistics is shown in Table 5.6. The curves used in this analysis were isolated curves and the first curve of reverse curves. The second curves of reverse curves were not included in the analysis of desirable spiral length since the transition on a reverse curve is different from the transition from a tangent to curve.

Table 5.6 shows that the analysis covers a wide range of radii from 72 m to 944 m for two-lane highways and from 489 m to 3147 m for freeways. It should be noted that three of the curves were in a work zone area. However, the steering lengths were found to follow the same trend of steering length as the remaining curves, and therefore they were

Table 5.5: Steering Behaviour Measures for Entrance on Curves.

Curve ID	R (m)	$L_{85_ST_EN}$	$L_{AVG_ST_EN}$	$L_{15_ST_EN}$	85 th Percentile Steering Speed	85 th Percentile Steering Time	Average Steering Time	15 th Percentile Steering Time
(a) Two-lane Highways								
3c4	273.32	131.50	107.33	86.750	91.30	5.57	4.54	3.66
3c6	325.05	138.25	110.93	82.000	88.59	5.80	4.93	3.75
3c8	555.89	115.00	86	65.000	94.67	4.86	3.54	2.72
3c10	267.25	130.00	98.67	65.000	87.43	5.58	4.39	3.09
6c1	463.05	124.50	88.86	70.000	96.91	5.26	3.76	2.60
12c1	475.66	105.00	76.21	45.000	67.45	6.68	4.66	2.50
12c2	656.34	98.25	73.66	50.000	81.08	5.41	3.88	2.57
12c4	263.39	145.00	123.3	95.000	84.84	7.29	5.72	4.22
73c1	789.47	72.00	47.59	35.000	77.61	3.38	2.41	1.57
73c2	856.69	72.20	55.75	42.250	88.34	3.18	2.47	1.89
73c3	943.67	64.00	42.51	25.000	87.55	2.67	1.84	1.16
73c4	595.62	86.36	67.51	43.750	70.16	5.23	3.97	3.02
73c7	637.28	85.75	65.20	34.750	77.03	4.63	3.46	2.58
73c9	86.00	102.81	79.21	54.730	61.42	5.65	4.64	3.78
6-416c1	71.62	65.00	51.50	35.000	51.10	5.40	3.91	2.62
416-12c1	151.42	100.00	81.43	66.750	77.83	5.18	4.05	3.23
(b) Freeways								
417c1	801.66	124.75	94.07	50.500	110.44	3.18	2.63	1.78
417c2	2211.64	99.25	61.40	40.000	111.73	2.44	1.77	1.18
417c3	488.74	167.50	130.38	97.500	112.85	5.54	4.02	3.09
417c4	869.31	160.00	114.10	79.250	116.33	4.45	3.45	2.65
417c5	863.21	124.00	102.24	65.000	112.26	4.04	3.52	2.38
417c8	916.02	133.76	100.62	62.500	114.52	4.87	3.39	2.05
417c9	2402.16	101.00	80.94	61.000	116.37	3.33	2.60	1.96
417c10	3147.22	84.07	64.89	50.558	109.22	2.54	2.02	1.47
417c11	2151.94	85.00	68.21	40.250	113.27	2.82	1.98	1.09
416c1	1810.15	117.75	99.29	54.750	118.82	3.69	3.38	2.91
416c2	1299.08	119.75	99.46	70.500	128.57	4.01	3.07	2.35
416c3	981.37	144.00	102.10	70.000	127.48	4.72	3.67	2.36
416c4	978.12	151.25	108.67	75.000	116.13	5.22	3.80	2.71

Table 5.6: Descriptive Statistics of the Characteristics of Study Curves.

Curve Characteristics	Minimum	Maximum	Range	Mean	Standard Deviation
(a) Two-lane Highways and Ramps.					
R (m)	71.62	943.67	872.05	463.23	272.31
L_{s_EN} (m)	20	185	165	48.78	41.47
L_{s_EX} (m)	10	273.53	263.53	85.04	73.58
$L_{ST_EN_AVG}$ (m)	42.51	123.30	80.79	78.48	23.51
$L_{ST_EX_AVG}$ (m)	42.51	100.56	58.05	75.55	19.34
V_{ST_85} (m)	51.10	96.91	45.81	79.78	12.29
t_{ST_AVG} (s)	1.84	4.93	3.09	3.69	0.94
(b) Freeways.					
R (m)	488.74	3147.22	2658.48	1514.9	763.07
L_{s_EN} (m)	25	220	195	110	76.67
L_{s_EX} (m)	15	270	255	106.04	102.26
$L_{ST_EN_AVG}$ (m)	61.40	130.38	68.98	94.34	20.25
$L_{ST_EX_AVG}$ (m)	58.79	127.82	69.03	93.15	23.59
V_{ST_85} (m)	109.22	128.57	19.35	116.00	5.96
t_{ST_AVG} (s)	1.77	4.02	2.24	3.02	0.75

kept in the analysis of steering lengths to increase the degrees of freedom. Separate models were developed for two-lane highways and freeways as it was found that they do not follow the same behaviour as will be presented shortly.

5.4.2.a. Conventional Modelling Approach

Significant relations were found for the 85th percentile and average steering length for two-lane highways as shown in Equations 5.9 and 5.10 and for freeways as shown in Equations 5.11 and 5.12. The most significant parameter that explained the variability in steering length was the curve radius. The statistical details of these models are shown in Table 5.7. All models are statistically significant at 0.05 p -level.

Table 5.7: Entrance Steering Length Models for Two-lane Highways and Freeways

Variables	Coefficients	SEE	t-statistic	p-value
(a) Two-lane Highways and Ramps.				
85 th Percentile Entrance Steering Length ($L_{85_ST_EN}$)				
(Constant)	36	14.001	2.571	0.024
R^3	8.39E-007	0.000	4.647	0.001
R^2	-0.001493	0.000	-5.446	< 0.0001
R	0.6976	.119	5.845	< 0.0001
Model diagnostics: Adjusted $R^2=0.824$, $DF=15$, $RMS=11.2$ m				
Average Entrance Steering Length ($L_{AVG_ST_EN}$)				
(Constant)	27.486	13.240	2.076	0.060
R^3	7.21E-007	.000	4.227	0.001
R^2	-.001	.000	-4.886	< 0.0001
R	.576	.113	5.108	< 0.0001
Model diagnostics: Adjusted $R^2=0.80$, $DF=15$, $RMS=10.57$ m				
(b) Freeways.				
85 th Percentile Entrance Steering Length ($L_{85_ST_EN}=a \times e^{bR}$)				
(a)	174.59	10.672	16.360	< 0.0001
e^{bR}	-.0003	0.000	-6.758	< 0.0001
Model diagnostics: Adjusted $R^2=0.79$, $DF=12$, $RMS=0.11$ m				
Average Entrance Steering Length ($L_{AVG_ST_EN}=a \times e^{bR}$)				
(a)	132.325	9.146	14.469	< 0.0001
e^{bR}	-.00025	0.000	-5.922	< 0.0001
Model diagnostics: Adjusted $R^2=0.74$, $DF=12$, $RMS=0.12$ m; Where a and b are constants.				

For two-lane highways:

$$L_{85_ST_EN} = 8.39E-07R^3 - 0.0015R^2 + 0.6976R + 36.0 \quad (5.9)$$

$$L_{AVG_ST_EN} = 7.21E-07R^3 - 0.00127R^2 + 0.5765R + 27.5 \quad (5.10)$$

For freeways:

$$L_{85_ST_EN} = 174.59 e^{-0.0003R} \quad (5.11)$$

$$L_{AVG_ST_EN} = 132.32 e^{-0.00025R} \quad (5.12)$$

Figure 5.9 shows the relations for both two-lane highways and freeways. For small radii on two-lane highways, the operating speeds are low and increase rapidly with the increase in curve radius causing the steering length to increase at a relatively rapid rate.

(a) Two- Lane Highways.

(b) Freeways.

Figure 5.9: Relation between 85th Percentile and Average Entrance Steering Length and Curve Radius Using Conventional Model.

As the radius increases, the rate of increase of curve operating speed decreases, while the steering angle required to negotiate the curve decreases. Therefore, the steering length required decreases with the increase in radius. The turning point in the relationship is found here to be around a radius of 270 m. It should be noted that the model should not be used for R greater than 840 m, which is close to the maximum radius in the analysis of 944 m as previously mentioned, since the cubic relation of the regression model deflect upwards after this point.

Comparing the steering lengths on freeways and two-lane highways shows that drivers do not adopt the same steering behaviour on both types of roads. For example, for a radius of 500 m, on two-lane highways, drivers would adopt a length of 120 m whereas on freeways, the length would be around 160 m. This difference is expected as the geometric characteristics of the freeway is not the dominant factor affecting the driver behaviour on freeways. Examples of other factors that affect the driving conditions on freeways are larger traffic volumes on freeways, even under free flow conditions, as well as the higher speed environment.

5.4.2.b. Variation Model Approach

The conventional model approach was used to compare the results with the variation model approach. As explained previously, for this approach the average steering length and standard deviation at each curve were determined. A model was then developed to quantify the entrance steering length at any percentile in terms of the geometric characteristics of the curves represented by the curve radius.

The regression analysis was performed using the software SPSS v15. Significant relations for both two-lane highways and freeways were found as shown in Equations

5.13 and 5.14 respectively. In addition, the statistics indicate the reliability of this model in predicting the steering length given the curve radius as shown in Table 5.8. The model diagnostics show a high coefficient of determination of 89.4 % and a low root-mean-square error of 9.97 m. In addition, each coefficient has a high t -statistic with p -values less than 0.0001 for most coefficients indicating the significant contribution of each variable to the model. Similarly, for freeways, the coefficient of determination is high with a value of 90%, and all coefficients have high t -statistics with p -values less than 0.0001.

For two-lane highways:

$$L_{ST_EN,P} = 27.486 + 0.5765R - 0.001267R^2 + 7.27 \times 10^{-7} R^3 + 8.08Z_p + 0.147Z_p R - 0.00031Z_p R^2 + 1.75 \times 10^{-7} Z_p R^3 \quad (5.13)$$

For freeways:

$$L_{ST_EN,P} = 126.04 - 0.0218R + 43.682Z_p - 0.00694Z_p R \quad (5.14)$$

Table 5.8: Variation Models for Entrance Steering Lengths for Two-lane Highways and Freeways.

Variables	Coefficients	SEE	t-statistic	p-value
(a) Two-lane Highways and Ramps.				
(Constant)	27.486	2.866	9.590	< 0.0001
R^3	7.21E-007	0.000	19.524	< 0.0001
R^2	-0.001267	0.000	-22.569	< 0.0001
R	0.5765	0.024	23.593	< 0.0001
Z_p	8.080	3.320	2.434	0.016
$Z_p R^3$	1.75E-007	0.000	4.082	< 0.0001
$Z_p R^2$	-0.00031	0.000	-4.777	< 0.0001
$Z_p R$	0.147	0.028	5.195	< 0.0001
Model diagnostics: Adjusted $R^2=0.894$, $DF=303$, $RMS= 10.0$ m				
(b) Freeways.				
(Constant)	126.04	1.497	84.209	< 0.0001
Z_p	43.682	1.733	25.199	< 0.0001
R	-.0218	0.001	-24.004	< 0.0001
$Z_p R$	-.00694	0.001	-6.599	< 0.0001
Adjusted $R^2=0.90$, $DF=246$, $RMS = 11$ m				

In addition, Figure 5.10 shows the relations for both two-lane highways and freeways. For both types of highways, the relation has a very similar trend to that explained previously in the conventional modelling approach.

5.4.2.c. Comparison of Modelling Approaches

Finally, a comparison was performed between the two modelling approaches to validate the new approach. To perform this, the 85th percentile and average values of desirable spiral were calculated using both modelling approaches. For the conventional model, Equations 5.9 and 5.10 are used for the 85th percentile and average values respectively. To obtain the 85th percentile value using the variation model, Z_p was substituted with 1.036 in Equation 5.14, which is the standardized normal variable at 85th percentile. The result is shown in Equation 5.15. Similarly, for the average values, Z_p was substituted with zero, resulting in Equation 5.16.

$$L_{ST_EN,85} = 35.857 + 0.7288R - 0.00159R^2 + 9.02 \times 10^{-7} R^3 \quad (5.15)$$

$$L_{ST_EN,50} = 27.486 + 0.5765R - 0.001267R^2 + 7.27 \times 10^{-7} R^3 \quad (5.16)$$

Figure 5.11 shows the results of the comparison. It is clear that the values using the two modelling approaches are very similar, and almost equal. Thus, this verifies the validity of using the variation modelling approach in regression analysis, given its previously mentioned advantages. Therefore, for the remainder of the analyses, regression using the variation model will be presented.

(a) Two-Lane Highways.

(b) Freeways.

Figure 5.10: Relation between 85th Percentile and Average Entrance Steering Length and Curve Radius Using the Variation Model.

(a) Two-Lane Highways.

(b) Freeways.

Figure 5.11: Comparison between Conventional and Variation Modelling Approaches

5.4.3. Exit Steering Length

Similarly, a regression analysis was performed to model the steering behaviour at curve exits. The models were based on isolated curves and the second curve of reverse curves with no intersections in the vicinity as shown in Table 5.9. The descriptive statistics were shown earlier in Table 5.6. Significant relations were found for both two-lane highways and freeways as shown in Table 5.10. The models are shown in Equations 5.17 and 5.18 for the two-lane highways and freeways respectively.

Table 5.9: Steering Behaviour Measures for Exits on Curves

Curve ID	R (m)	$L_{85_ST_EX}$ (m)	$L_{AVG_ST_EX}$ (m)
(a) Two-lane Highways			
3c5	239.17	119.00	97.87
3c7	180.17	125.00	100.56
3c9	203.35	106.00	86.97
3c11	255.94	110.00	88.08
6c1	463.05	93.25	71.00
12c1	475.66	103.25	81.67
12c3	538.62	90.00	67.33
12c5	168.83	121.00	85.37
73c1	789.47	70.00	55.86
73c2	856.69	68.25	45.00
73c3	943.67	64.00	42.51
73c6	360.70	115.00	85.00
73c8	230.55	121.75	97.12
73c9	86.00	95.50	78.40
6-416c1	71.62	75.00	50.50
(a) Freeways			
417c1	801.66	149.75	122.50
417c2	2211.64	95.00	68.50
417c3	488.74	138.00	110.69
417c4	869.31	170.60	127.82
417c7	2045.95	108.47	76.11
417c8	916.02	115.00	89.48
417c9	2402.16	86.37	58.79
417c10	3147.22	84.17	67.62
416c1	1810.15	95.00	74.00
416c2	1299.08	138.50	107.17
416c3	981.37	123.50	103.91
416c4	978.12	139.00	111.21

Table 5.10: Models for Exit Steering Lengths for Two-lane Highways and Freeways

Variables	Coefficients	SEE	t-statistic	p-value
(a) Two-lane Highways and Ramps.				
(Constant)	42.468	2.980	14.250	< 0.0001
R^3	5.55E-007	0.000	12.950	< 0.0001
R^2	-0.00094	0.000	-14.852	< 0.0001
R	0.39947	0.026	15.175	< 0.0001
Z_p	41.489	3.452	12.020	< 0.0001
$Z_p R^3$	-1.19E-007	0.000	-2.405	0.017
$Z_p R^2$	0.0002	0.000	2.771	0.006
$Z_p R$	-0.11292	0.030	-3.704	< 0.0001
Model diagnostics: Adjusted $R^2=0.89$, $DF=284$, $RMS= 9.7$ m				
(b) Freeways.				
(Constant)	130.581	1.383	71.055	< 0.0001
R	-0.025	0.001	-22.956	< 0.0001
Z_p	35.251	2.128	16.562	< 0.0001
$Z_p R$	-0.003	0.001	-2.285	0.023
Model diagnostics: Adjusted $R^2=0.87$, $DF=227$, $RMS= 12.9$ m				

For two-lane highways:

$$L_{ST_EX,P} = 42.468 + 0.3995R - 0.00094R^2 + 5.55 \times 10^{-7} R^3 + 41.489Z_p - 0.11292Z_p R + 0.0002Z_p R^2 - 1.19 \times 10^{-7} Z_p R^3 \quad (5.17)$$

For freeways:

$$L_{ST_EX,P} = 130.581 - 0.025R + 35.251Z_p - 0.003Z_p R \quad (5.18)$$

Both models are statistically significant at 0.05 p -level and have high coefficients of determination. Although the range of radii covered in the analysis for the two-lane highways was from 72 m to 947 m, the developed model, shown in Equations 5.17, should not be used for R greater than 900 m as the cubic relation deflects upwards after that point.

Figure 5.12 shows the plotted model for the 85th percentile steering length for both the entrance and exits of horizontal curves on two-lane highways. It is shown from the

(a) Two-Lane Highways.

(b) Freeways.

Figure 5.12: Comparison Between the 85th Percentile Desirable Steering Length at Entrance and Exit of Horizontal Curves on Two-Lane Highways

figure that the exit steering length for two-lane highways follows the same trend as the entrance steering length but with shorter lengths. As for freeways, the steering lengths for curve entrance and exit are almost the same. Due to the similarity of the entrance and exit steering behaviour, and due to the practicality of using the same spiral length for both entrance and exit of a specific curve, a regression analysis was performed after combining the entrance and exit steering lengths.

5.4.4. General Steering Length

The resulting models are statistically significant at 0.05 p -level, and are presented in Table 5.11. As these models provide a compromise between drivers' behaviour at curve entrance and exit, the coefficients of determination decreased slightly compared to the corresponding separate models for entrance and exit spirals. However, it is recommended to use the models shown in Table 5.11 as the desirable spiral length for both entrances and exits for two-lane highways and freeways respectively in order to have one length representing the steering length for both the entrance and exit of a curve. It is important to note that these models are valid for the range of $70 \text{ m} < R < 900 \text{ m}$ for two-lane highways and $490 \text{ m} < R < 3145 \text{ m}$ for freeways.

For two-lane highways:

$$L_{ST,p} = 35.449 + 0.472R - 0.001R^2 + 5.97 \times 10^{-7} R^3 + 26.536Z_p - 1.23 \times 10^{-5} Z_p R^2 \quad (5.19)$$

For freeways:

$$L_{ST,p} = 128.207 - 0.023R + 39.7181Z_p - 0.005Z_p R \quad (5.20)$$

Table 5.11: Models for Both Entrance and Exit Steering Lengths for Two-lane Highways and Freeways

Variables	Coefficients	SEE	t-statistic	p-value
(a) Two-lane Highways and Ramps.				
(Constant)	35.449	2.510	11.760	< 0.0001
Z_p	26.536	0.791	33.566	< 0.0001
R^3	5.97E-007	0.000	19.124	< 0.0001
R	0.472	0.022	24.215	< 0.0001
R^2	-0.001	0.000	-22.599	< 0.0001
$Z_p R^2$	-1.23E-005	0.000	-5.804	< 0.0001
Model diagnostics: Adjusted $R^2=0.86$, $DF=588$, $RMS= 12.1$ m				
(b) Freeways.				
(Constant)	128.207	1.188	107.889	< 0.0001
R	-0.023	0.001	-32.758	< 0.0001
Z_p	39.718	1.376	28.859	< 0.0001
$Z_p R$	-0.005	0.001	-6.081	< 0.0001
Model diagnostics: Adjusted $R^2=0.89$, $DF=474$, $RMS = 12.1$ m				

5.5 Validation and Comparison with Current Practice in North

American Guides

To complete the analysis, a closer look was taken at the current practice of the North American guides (AASHTO 2004, TAC 1999). A complete review of the current practice of both the Green Book and the Canadian design guide was previously presented in Section 2.7.2.b of Chapter 2. For the section in hand, a brief comparison will first be made between the recommendations of the design guides and the findings of this research. Following that, a comparison between the desirable steering length adopted by the drivers and the values recommended in the guides will be presented.

5.5.1. Driver Comfort

As previously explained in Section 2.7.2.b, drivers are provided with comfort by limiting the rate of increase in lateral acceleration as the vehicle enters the curve. The

length of spiral curve is controlled by this rate (c) and the design speed of the curve (V) by Equation 2.4:

This criterion was further investigated using the collected experiment data. The value of c was measured for the 30 drivers on the entrance steering length of the curves of both two-lane highways and freeways. It was calculated for each driver using the following equation:

$$c = \frac{\Delta a_L}{t_{ST}} = \frac{\Delta a_L}{\left(L_{ST} / 0.278 V_{ST} \right)} \quad (5.21)$$

Where:

Δa_L = difference in lateral acceleration between the beginning and end of the entrance steering length as obtained from the steering angle profile for each driver;

t_{ST} = steering time (s);

L_{ST} = steering length (m); and

V_{ST} = speed at the middle of the steering length (km/h).

The lateral acceleration was obtained from the Corsa Box data, while the steering time was calculated for each driver by dividing the steering length by the steering speed. Since the steering time is affected by the value of steering speed, only free flow speeds were considered. This is unlike the steering length, where all drivers were considered, whether the vehicle followed free flow conditions or not. This is because, the vehicle speed is affected by surrounding traffic and free flow conditions, while the vehicle path is not. A minimum time headway of 5 s, as calculated from the distance measurement by the front laser gun, was used to identify free flow conditions. The value of 5 s is recommended by previous studies (Fitzpatrick et al. 2000).

In addition, curves on Highway 12 (12c1, 12c2, and 12c4) showed significantly higher steering times due to the presence of work zones, and therefore were not considered in the analysis of c . In general, for two-lane highways the value of 85th percentiles of c ranged between 0.1 and 0.75 m/sec³ for the largest and smallest radii respectively. As for freeways, the values of 85th percentiles of c ranged from 0.2-0.6 m/sec³, regardless of curve radii. These results suggest that the range given in the Canadian design guide is valid.

A regression analysis was performed using the 85th percentile values of c as the dependent variable and R , $1/R$, 85th percentile steering speed, and 85th percentile steering length, as descriptive variables. The only parameter that gave a statistically significant relation for two-lane highways with c was R as shown in the following equation.

$$c_{85} = 0.724e^{-0.0011R}; \quad R^2_{adj.}=0.61; DF=12 \quad (5.22)$$

Expectedly, the equation shows that as the radius increases, the rate of change in lateral acceleration experienced by the drivers decreases. The coefficient of determination of the relation is moderate, suggesting that there is a significant trend between the two variables. This, in turn, implies the need to adopt variable values for c in the design rather than a constant one. As for freeways, there was no significant relationship between c and R . As explained previously, geometry is not a critical factor on highways.

5.5.2. Aesthetics Control (Desirable Length)

For a visually pleasant alignment, it is recommended that the length of spiral be such that the driving time on the spiral is at least 2 seconds. This is also the desirable length defined by the Green Book as the natural steering length.

The 85th percentile, average, and 15th percentile steering times were shown in Table 5.5. The 85th percentile steering time represents the slower drivers, and the 15th percentile represents the faster drivers. A regression analysis was performed for the steering time using the variation model approach. Table 5.12 shows the results of the regression analysis for two-lane highways and freeways. The steering time has a very similar trend to that of the steering length, where it increases at first for sharp curves, then decreases with increase in radius as shown in Figure 5.13

Table 5.12: Models of Steering Time with Curve Radii for Two-lane Highways and Freeways

Variables	Coefficients	SEE	t-statistic	p-value
(a) Two-lane Highways and Ramps.				
(Constant)	3.757	0.103	36.614	< 0.0001
R	0.006	0.001	6.884	< 0.0001
R^2	-1.53E-005	0.000	-6.896	< 0.0001
R^3	6.79E-009	0.000	4.572	< 0.0001
Z_p	0.900	0.078	11.567	< 0.0001
$Z_p R$	0.002	0.000	4.664	< 0.0001
$Z_p R^2$	-2.20E-006	0.000	-5.771	< 0.0001
Adjusted $R^2=0.94$, DF=227, RMS = 0.34 s				
(b) Freeways.				
(Constant)	4.218	0.071	59.038	< 0.0001
Z_p	1.551	0.083	18.739	< 0.0001
R	-0.00085	0.000	-17.219	< 0.0001
$Z_p R$	-0.0003	0.000	-5.232	< 0.0001
Adjusted $R^2=0.86$, DF=227, RMS = 0.46 s

(a) Two- Lane Highways.

(b) Freeways

Figure 5.13: 85th Percentile and Average Steering Time with Radius

In the TAC guide, the aesthetic control depends on a minimum spiral length for the least required steering time in order to realize a visually pleasant alignment. Therefore, this requirement does not relate to driver behaviour. On the other hand, in the Green Book, the desirable spiral length is defined as the natural steering length of the drivers, which is taken as the travel distance for 2.0 s. However, the model in Table 5.12 shows that the steering time of drivers depends on R . The relation is statistically significant at 0.05 p -level and has a high coefficient of determination of 94%. In addition, the average value of the 85th percentile steering time, which represents the slower drivers, for the curves in the study was 4.8 s. On the other hand, the average of the 15th percentile steering time, which represents the faster drivers, for all curves was 2.7 s. The average of the average values was 3.7 s.

From the collected data it is shown that 85% of the drivers do not follow a 2.0 s steering time. The value is more likely based on faster drivers (close to the 15th percentile steering time). Therefore, the results of the study suggest that the 2.0 s value used in the Green Book should be re-examined.

5.6 Validation and Comparison with Guides

As explained earlier, geometric features are not the dominant factor on freeways. Therefore, values of desirable spiral length are more critical on two-lane highways. Thus, in this analysis, the comparison with the guides will be done using only the relation for the two-lane highways. Table 5.13 shows the desirable spiral length (85th percentile steering length) as calculated using Equation 5.19 for a range of radii from $70\text{m} < R < 900\text{m}$. The desirable spiral length is compared to the different recommended minimum and maximum spiral lengths from the different control criteria defined by the Green Book

Table 5.13: Parameters Used in Validation of Results and Comparison with Design Guides

R	L_{MS_ST}	L_{AVG_ST}	Design Speed	Comfort ($c=0.6$)	Comfort ($c=1.2$)	Min. Lateral Shift	Max. Lateral Shift	Aesthetics	e	s	Superelevation Control	A	A^2/R
70	79.40	61.88	50	63.94	31.97	18.33	40.99	27.80	0.08	0.65	46.15	65	60.36
90	89.25	69.63	50	49.73	24.87	20.78	46.48	27.80	0.08	0.65	46.15	65	46.94
120	102.01	79.66	60	64.45	32.23	24.00	53.67	33.36	0.08	0.6	50.00	85	60.21
180	112.51	87.88	70	68.23	34.12	29.39	65.73	38.92	0.08	0.55	54.55	110	67.22
240	120.88	94.41	80	76.39	38.20	33.94	75.89	44.48	0.08	0.55	54.55	135	65.10
270	127.28	99.35	80	67.90	33.95	36.00	80.50	44.48	0.08	0.51	58.82	135	57.87
300	131.85	102.83	90	87.01	43.51	37.95	84.85	50.04	0.08	0.47	63.83	160	85.33
330	134.73	104.97	90	79.10	39.55	39.80	88.99	50.04	0.08	0.47	63.83	160	77.58
360	136.08	105.87	90	72.51	36.26	41.57	92.95	50.04	0.08	0.47	63.83	160	71.11
390	136.05	105.67	100	91.82	45.91	43.27	96.75	55.60	0.08	0.44	68.18	190	92.56
420	134.77	104.47	100	85.26	42.63	44.90	100.40	55.60	0.08	0.44	68.18	190	85.95
480	132.41	102.39	100	74.60	37.30	48.00	107.33	55.60	0.08	0.44	68.18	190	75.21
510	129.09	99.55	100	70.21	35.11	49.48	110.63	55.60	0.08	0.44	68.18	190	70.78
540	124.98	96.06	110	88.26	44.13	50.91	113.84	61.16	0.08	0.41	73.17	220	89.63
600	120.22	92.05	110	79.43	39.72	53.67	120.00	61.16	0.077	0.41	70.43	220	80.67
630	114.95	68.17	110	75.65	37.83	54.99	122.96	61.16	0.077	0.41	70.43	220	76.83
690	109.32	59.01	110	69.07	34.54	57.55	128.69	61.16	0.077	0.41	70.43	220	70.14
720	103.49	54.99	110	66.20	33.10	58.79	131.45	61.16	0.072	0.41	65.85	220	67.22
780	97.59	48.62	110	61.10	30.55	61.19	136.82	61.16	0.066	0.41	60.37	220	62.05
810	91.77	46.52	110	58.84	29.42	62.35	139.43	61.16	0.066	0.41	60.37	220	59.75

and TAC using Equations (2.3, 2.4, 2.5, 2.6, and 2.7), which were presented earlier in Chapter 2.

The values of the design controls are calculated and shown in Table 5.13 and the plots are shown in Figure 5.14 (a) for the Green Book controls and Figure 5.14 (b) for the Canadian design guide controls. The design speeds used in the comparison are the maximum design speeds for these radii as specified by the guides ranging from 50 to 110 km/h. The minimum length for comfort control is calculated using Equation 2.4 where V is the design speed for each corresponding radius, and c was taken as 1.2 m/sec^3 for the Green Book and as 0.6 m/sec^3 for the Canadian design guide. The superelevation control length was calculated using Equation 2.5, taking superelevation (e) and relative slope of pavement (s) as recommended by the design guides for each corresponding radius and design speed and as shown in the table. The minimum and maximum lateral shift control were calculated using Equation 2.6, taking p as 0.2 m for the minimum lateral shift and 1.0 m for the maximum lateral shift. Finally, TAC guide recommends a desirable spiral length based on the spiral parameter (A) as shown in Equation 2.3.

From Figure 5.14, it is obvious that the desirable spiral length represented by the drivers steering behaviour follows the same trend as the minimum comfort control, where the steering length increases with increase in radius in the lower range of radii, then decreases with increase in radii in the upper range. This proves the validity of the model, since the comfort control is also a third degree equation and follows the same trend. However, the desirable spiral length, which is based on 85th percentile values, exceeds all minimum controls. More importantly, the difference between the minimum controls and

(a) The Green Book Control Criteria

(b) The Canadian Design guide Control Criteria.

Figure 5.14: Desirable Spiral Length Compared to Current North American Control Criteria

the desirable length is large, which is especially critical in the lower range of radii, which leads to safety concerns.

It is believed from comparing the results of this study to the results of the guides and also previous studies (such as Perco 2006) that the difference between the lengths of spirals is due to one main reason. The reason is that the spiral length obtained in previous studies is calculated from other measured parameters rather than directly measuring the steering length.

It should be noted that the Canadian guide recommends using larger spiral curves than the minimum wherever possible in the interests of safety, comfort, and aesthetics. However, the Green Book provides a maximum spiral length based on maximum lateral shift control. Referring back to Figure 5.14, 85% of the drivers adopt steering lengths greater than that length. This indicates that there is a gap between actual driving behaviour and the design guide.

Another concern is the value of c taken in the Green Book which is 1.2 m/sec^3 . This value is higher than the value found by 85% of the drivers for sharp curves. The highest value observed was 0.75 m/sec^3 for the sharpest curve of $R= 86 \text{ m}$. Moreover, using constant comfort values for c for a wide range of radii is proven to be unrealistic. On the other hand, the value taken by the Canadian guide ($c=0.6 \text{ m/sec}^3$) is closer to driver behaviour especially in the range of smoother curves ($R> 500 \text{ m}$).

Therefore, to design spiral curves conforming to driver behaviour the following recommendations should be considered:

1. Using Equation 5.19 derived in this research as the desirable spiral curve length for two-lane highways and Equation 5.20 for freeways. These equations are based on desired steering lengths based on actual driving conditions.
2. The following corrections in Table 5.14 could be made to the design guides in order to follow the steering behaviour model in Equation 5.19. The table shows recommended values for c and A based on the steering behaviour. Furthermore, Figure 5.15 shows the calculated spiral length based on Equation (2.4) and using the recommended values of A from Table 5.14. It is proven that that when using the recommended values by this study for c and A , the equations conform very well to the steering behaviour.

5.7 Conclusions

This chapter presented a procedure for studying driver behaviour in terms of vehicle path and steering path. It was found that there is a difference between the actual alignment and the alignment that drivers adopt. In addition, the trend of change in steering behaviour was analyzed for a range of radii for both two-lane highways and freeways. Using these investigations, it was recommended that the steering behaviour of drivers should be used to find desirable spiral lengths that conform to driving behaviour. In doing so, a newly applied variation model was used for finding desirable spiral lengths on horizontal curves. Based on the results of this study, the spiral lengths follow a cubic relationship with the same trend as the comfort control in design guides. However, when comparing the desirable spiral lengths with the current control values in the design guides, it was found that the desirable lengths are greater than minimum values, which raises safety issues. It was found that discrepancies between the design and the actual

Table 5.14: Recommended Corrections to the Design Speeds and Rate of Change of Lateral Acceleration in the Current Geometric Design Guides.

R	Design Speed (km/h)	c (m/s ³)			Spiral Parameter (A)	
		Current in AASHTO	Current in TAC	Study Recommendation	Current in TAC	Study Recommendation
70	50	1.2	0.6	0.40	65	80
90	50	1.2	0.6	0.30	65	95
120	60	1.2	0.6	0.35	85	115
150	60	1.2	0.6	0.30	85	130
180	70	1.2	0.6	0.35	110	145
210	70	1.2	0.6	0.30	110	160
240	80	1.2	0.6	0.35	135	175
270	80	1.2	0.6	0.35	135	185
300	90	1.2	0.6	0.40	160	195
330	90	1.2	0.6	0.35	160	205
360	90	1.2	0.6	0.35	160	215
390	100	1.2	0.6	0.45	190	220
420	100	1.2	0.6	0.45	190	225
450	100	1.2	0.6	0.40	190	235
480	100	1.2	0.6	0.40	190	235
510	100	1.2	0.6	0.40	190	235
540	110	1.2	0.6	0.50	220	235
570	110	1.2	0.6	0.50	220	235
600	110	1.2	0.6	0.50	220	235
630	110	1.2	0.6	0.50	220	235
660	110	1.2	0.6	0.50	220	235
690	110	1.2	0.6	0.50	220	235
720	110	1.2	0.6	0.50	220	235
750	110	1.2	0.6	0.50	220	235
780	110	1.2	0.6	0.50	220	235
810	110	1.2	0.6	0.50	220	235

Figure 5.15: Comparison of Desirable Steering Length and Spiral Length Based on New Recommended Comfort Control Values

driving behaviour could be attributed to the current practice in design guides which take a constant value for rate of change of lateral acceleration. However, through this study, it was found that rate of change of lateral acceleration depends on the radius of the curve, with maximum values found at smaller radii. The maximum value of rate of change of lateral acceleration was 0.75 m/sec^3 . Recommendations for corrections are made to the design guides in order to follow Equation 5.15 which considers 85th percentile desirable steering length. It is also recommended to incorporate driver behaviour to the control criteria of design guides in order to provide highways designed with the driver in mind.

CHAPTER 6. DRIVER COMFORT THRESHOLDS ON HORIZONTAL CURVES

This chapter is concerned with the investigation of the relationship between the driver comfort thresholds used in the design of horizontal curves and both the speed characteristics adopted by the driver and the horizontal curve elements. In performing this analysis, the lateral acceleration profiles for each driver on each curve were used to determine a representative lateral acceleration. In addition, several control tests were performed to validate the measured lateral acceleration values. A regression analysis was then performed to find the relationship between the measured lateral accelerations and speed characteristics on the curves. Finally, the suggested driver comfort thresholds were compared to the values adopted in the North American design guide (Said et al. 2008).

6.1 Introduction

In Chapter 2, a thorough review of the point mass formula along with a criticism was presented. Among the main concerns and motivations for the research in hand is that the values of driver comfort thresholds were based on experiments performed back in the 1930's (Barnett 1936, Moyer and Berry 1940). Barnett's study (1936) concluded that an acceptable side friction coefficient was 0.16 for speeds between 30 km/h and 100 km/h. The values suggested by Moyer and Berry (1940) ranged from 0.21 at speeds as low as 30 km/h, 0.18 for speeds between 40 and 50 km/h, and 0.15 for speeds up to 80 km/h. Finally, Bonneson analyzed data collected from horizontal curves in different U.S. states. The results were in general agreement with the values of the Green Book for high-speed design and his study was incorporated in the Green Book 2004 edition (AASHTO 2004,

Bonneson 1999, Bonneson 2000). The model developed by Bonneson (1999, 2000) was shown previously in Equation 2.2

It is therefore the objective of this chapter to develop a model that quantifies the upper limit of driver comfort thresholds based on actual road driving data. Previous studies have shown that the comfort threshold describes the desire of the drivers to minimize their speed reduction as they move from the approach tangent to the curve (Bonneson 1999, Bonneson 2000, Chowdhury et al. 1991). As a result, it was suggested that drivers tend to accept greater side friction demands when facing greater speed reductions (Chowdhury et al. 1991). This implies that drivers may end up negotiating curves at higher speeds and higher side friction than what they were designed for, which can raise two concerns:

- The margin of safety between the side friction demanded ($f_{s,d}$) and that supplied at the interaction between the tires and the road is reduced, and therefore the risk of skidding increases.
- Drivers' operating speeds exceed the design speed, causing inconsistency in the design of highways, which is another safety concern.

To provide designers with a tool to address these concerns, this chapter presents a side friction demand prediction model for horizontal curves on two-lane rural highways and ramps. The advantage of the model developed in this study is that it is based on measured lateral acceleration values under actual driving conditions. Implications to the design are presented using a comparison between the lateral acceleration values calculated from the model and the side friction demand values in the Green Book.

Another comparison is also presented for the inferred design speeds, as surrogate to curve operating speeds, using different approaches to estimate the coefficient of side friction.

6.2 Preparation of Data Set

6.2.1. Curves Used in Analysis

For the purpose of the analysis of driver comfort threshold, horizontal curves on two-lane highways and ramps will be considered. This is because these are the types of facilities where the risk of skidding or rollover due to excess side friction demand with respect to that supplied is of concern. It is worth mentioning that 82 % of the Canadian road network is two-lane two-way highways, and that 67% of all collisions in Canada occurred on rural highways in 2003 (Road Safety 2006). These statistics suggest that these roads do deserve more attention. In addition, since 50% of fatal collisions were single-vehicle collisions (Road Safety 2006), more attention is also needed to vehicle stability especially on horizontal curves.

For the analysis of driver comfort thresholds, only free flow speeds were used, as this would be the speed representing the driver's choice based on the surrounding geometric characteristics of the road, rather than the traffic around the test vehicle. As explained in Chapter 3, the front laser gun was used to measure the spacing between the test vehicle and the vehicle in front of it and in turn to identify free flow conditions. Minimum time headway of five seconds could be considered as a free-flow condition as recommended by previous research work (Fitzpatrick et al. 2000a). The time gap between the successive vehicles was measured by dividing the distance between the two vehicles by the speed of the test vehicle. The distance was recorded by triggering the Lidar gun when the test vehicle was following another vehicle. In addition to the distance, the Lidar

gun also records a time stamp that was used to match it with the operating speed data of the test vehicle. As this procedure was followed throughout the test route, free flow conditions were determined for each curve. Table 6.1 shows the total number of drivers experiencing free-flow conditions on each curve. When performing the regression analysis using the variation model, curves with drivers under free-flow conditions less than or equal to six were eliminated from the analysis based on recommendations by other studies such as Bonneson (2000). Therefore, curve IDs 73c1 and 73c8 were removed from the analysis.

Table 6.1: Characteristics for Curves Used in Analysis.

Curve ID	R (m)	Dir	L_c (m)	N
(a) Rural Two-Lane Highways				
3c4	273.32	RT	381.54	28
3c5	239.17	LT	224.95	26
3c6	325.05	LT	169.97	27
3c7	180.17	RT	39.99	24
3c8	555.89	RT	159.99	27
3c9	203.35	LT	70.97	27
3c10	267.25	RT	429.90	28
3c11	255.94	LT	369.94	27
6c1	463.05	RT	159.97	21
(b) Ramps				
416-12c1	151.42	RT	94.97	30
416-12c2	98.00	LT	49.95	30
417-7c1	447.83	RT	104.99	29
417-7c2	548.22	LT	384.09	29
6-416c1	71.62	RT	119.85	30
(c) Urban/Sub-urban Two-Lane Highways				
12c3	538.62	RT	70.00	11
12c4	263.39	RT	99.98	14
73c1	789.47	RT	179.98	5
73c2	856.69	LT	354.99	11
73c5	571.42	RT	390.15	7
73c6	360.70	LT	186.90	8
73c7	637.28	RT	289.02	14
73c8	230.55	RT	149.97	4
73c9	86.00	LT	29.99	12

In addition to that, curves that are in the vicinity of an intersection were removed from the analysis. This applied to curve IDs 3c1, 3c2, 3c3, 12c1, 12c2, 12c5, 73c3, and 73c4. On these curves the intersections were either on the approach tangent, the curve section, or the departure tangent. Due to the presence of traffic control devices on these intersections, the speeds of drivers, and thus the lateral accelerations were affected. That is to say, the drivers' choice of speed was not due to the geometric characteristics of the road, but rather due to the effect of traffic control devices.

6.2.2. Effect of Body-Roll Angle

Referring back to the point mass formula, the lateral acceleration demanded by the vehicle is represented by (V^2/R) , while the lateral acceleration supplied is represented by $((e+f_s) g)$. Since it is believed that drivers perceive discomfort before the threshold of skidding or rollover, the side friction demanded is taken as the driver comfort threshold, which is measured using the accelerometer. That is, the accelerometer reading gives the value of lateral acceleration the driver feels in excess of $(V^2/R - eg)$, which is equivalent to the side friction demanded by the drivers. However, due to the body roll angle, the accelerometer reading is actually the side friction demanded plus the lateral acceleration due to body roll as shown in Equation 6.1.

$$a_L = \frac{V^2}{R} - eg + \rho g = f_{s,d} + \rho g \quad (6.1)$$

Where a_L is the lateral acceleration reading by accelerometer, and $f_{s,d}$ is the side friction demanded by the driver. However, a comprehensive study by Carlson and Mason (1999), found that "body-roll does not significantly influence the readings provided by the ball bank indicator" when using typical passenger cars to determine safe speeds on horizontal curves. This result could be considered applicable also for accelerometer

readings. In addition, Moyer and Berry (1936) recommended that the body-roll angle should be neglected since even the extreme effect of body-roll for different vehicle models would affect the ball bank indicator by 1 degree. This is equivalent to an error of around 2 km/h. It should be noted that Moyer and Berry's study was made in 1936, where the characteristics of the vehicles were different, and body-roll angles expectantly would be higher than for vehicles nowadays.

Therefore, the body-roll angle was neglected in this investigation such that the values of lateral acceleration represent the side friction demanded by the driver. Therefore, the two terms of lateral acceleration and side friction are used in the thesis synonymously.

6.2.3. Calculation of Lateral Acceleration

As explained previously in Chapter 3, among other items in the dataset for each driver were the continuous and synchronized lateral acceleration and speed profiles. These profiles were cross-referenced with the geometry of the road and vehicle path. An average lateral acceleration was then calculated for each driver on each curve, and was used to represent the driver comfort threshold on this curve. Figure 6.1 shows two cases of different lateral acceleration profiles. For part (a), Driver 3 on curve 3c4 had experienced a high lateral acceleration at the beginning of the curve, and then the acceleration fluctuated around a central average value. Part (b) is the profile for Driver 4 on the same curve 3c4. The lateral acceleration also spiked to a maximum value at the beginning of the curve, and then fluctuated around a central value. Both of these trends were similar for all drivers on the study curves. It is important to note the high initial lateral acceleration at the beginning of the curve. The initial spike in some cases, or the

(a) Driver3

(b) Driver 4

Figure 6.1: An Example of Lateral Acceleration Profile for Two Drivers (Curve ID: 3c4, $R=267$ m).

higher acceleration values than the rest of the profile for others, indicates that the drivers initially entered the curves with a higher speed, and then when they felt discomfort due to the high lateral acceleration, they lowered their speed to “a comfortable zone.”

Based on this observation, it was found that the best representative value for the level of comfort on a horizontal curve should be the central value that the lateral acceleration was fluctuating around. This value was calculated as the average lateral acceleration on the horizontal curve after excluding the extreme values at the beginning of the curve. To remove judgment in identifying the extreme values and acceptable fluctuation around the mean, a Matlab function, shown in Appendix D, was created to identify the extreme values as outliers to a normally distributed sample. A z-score of 1.96, which corresponds to a 5% level of significance, was set as the threshold for outliers. Once the extreme values were removed from the profile of lateral acceleration, a new dataset of lateral acceleration was created for each curve where each point in this dataset corresponded to the mean lateral acceleration for a specific driver on this curve. This dataset was then used to calculate the average lateral acceleration (a_{L_AVG}), 85th percentile lateral acceleration (a_{L_85}), and standard deviation (a_{L_SD}) on each curve. Table 6.2 shows these values for the study curves. In addition, the descriptive statistics, presented in Table 6.3, show that there was a significant spread in the lateral acceleration tolerated by the drivers depending on the facility type and curve parameters.

6.2.4. Calculation of Speed Measures

One of the expected parameters affecting driver comfort threshold on a horizontal curve is the approach speed (V_A). The approach speed was taken as the maximum speed on the tangent preceding the curve. The length of tangent taken was the straight segment

Table 6.2: Lateral Acceleration and Speed Characteristics of Curves Used in Analysis

Curve ID	R (m)	a_{L_AVG}	a_{L_85}	a_{L_STDEV}	V_{AT_AVG}	V_{AT_85}	V_{c_AVG}	V_{c_85}
(a) Rural Two-Lane Highways								
3c4	273.32	0.19	0.22	0.04	93.53	107.35	78.06	85.13
3c5	239.17	0.10	0.13	0.03	79.23	83.95	77.85	82.46
3c6	325.05	0.07	0.10	0.03	88.09	97.63	81.63	88.00
3c7	180.17	0.19	0.23	0.04	83.81	90.33	79.74	87.66
3c8	555.89	0.13	0.16	0.03	91.33	105.05	86.76	95.50
3c9	203.35	0.09	0.12	0.03	87.56	95.95	80.62	85.44
3c10	267.25	0.20	0.24	0.04	92.41	103.68	79.60	84.96
3c11	255.94	0.12	0.16	0.04	83.49	91.35	78.03	85.01
6c1	463.05	0.08	0.11	0.03	88.42	106.17	87.26	99.46
(b) Ramps								
416-12c1	151.42	0.20	0.25	0.04	99.71	107.04	65.80	71.16
416-12c2	98.00	0.13	0.17	0.03	68.31	72.45	60.95	65.68
417-7c1	447.83	0.08	0.11	0.03	102.65	110.96	87.32	95.10
417-7c2	548.22	0.05	0.08	0.03	91.96	99.03	89.00	97.59
6-416c1	71.62	0.30	0.35	0.05	52.81	57.18	54.63	58.27
(c) Urban/Sub-urban Two-Lane Highways								
12c3	538.62	0.09	0.11	0.02	87.42	98.11	87.94	98.86
12c4	263.39	0.16	0.19	0.03	86.44	95.83	76.32	83.40
73c1	789.47	0.06	0.07	0.01	80.36	83.65	80.70	84.60
73c2	856.69	0.03	0.04	0.01	82.73	88.20	81.97	87.64
73c5	571.42	0.08	0.10	0.03	72.82	92.64	68.78	88.78
73c6	360.70	0.02	0.04	0.01	69.44	77.03	69.18	74.96
73c7	637.28	0.06	0.08	0.02	71.57	80.84	67.73	74.43
73c8	230.55	0.12	0.13	0.00	63.98	66.17	62.15	63.83
73c9	86.00	0.11	0.13	0.03	61.41	68.01	54.37	58.88

200 m before the beginning of the curve. This distance was based on the study by Fitzpatrick et al. (2000) that showed that the speed along the approach tangent starts to decrease due to the effect of a horizontal curve at a point in the 200 m tangent segment preceding the beginning of the curve. In cases where the approach tangent was less than 200 m, the maximum speed on the whole tangent section was taken. Having determined V_A for each driver, the average approach speed (V_{A_AVG}) and the 85th percentile approach speed (V_{A_85}) were calculated for each curve. A summary of the descriptive statistics of the approach speed measures is also shown in Table 6.3.

Table 6.3: Descriptive Statistics of the Characteristics of Study Curves.

Curve Characteristics	Minimum	Maximum	Range	Mean	Standard Deviation
<i>(a) Rural Roads.</i>					
R (m)	180.17	555.89	375.72	307.02	124.18
a_{L_Avg} (g)	0.07	0.20	0.13	0.13	0.05
a_{L_85} (g)	0.10	0.24	0.13	0.16	0.05
V_{A_AVG} (km/h)	79.23	93.53	14.31	87.54	4.67
V_{A_85} (km/h)	83.95	107.35	23.40	97.94	8.22
V_{C_AVG} (km/h)	77.85	87.26	9.41	81.06	3.60
V_{C_85} (km/h)	82.46	99.46	17.00	88.18	5.60
ΔV_{85} (km/h)	1.49	22.22	9.76	6.85	20.73
<i>(b) Urban/ Suburban Roads.</i>					
R (m)	86.00	856.69	770.69	481.57	262.87
a_{L_Avg} (g)	0.02	0.16	0.14	0.08	0.04
a_{L_85} (g)	0.10	0.24	0.13	0.16	0.05
V_{A_AVG} (km/h)	61.41	87.42	26.01	75.13	9.53
V_{A_85} (km/h)	66.17	98.11	31.94	83.39	11.51
V_{C_AVG} (km/h)	54.37	87.94	33.57	72.13	10.56
V_{C_85} (km/h)	58.88	98.86	39.99	79.49	12.68
ΔV_{85} (km/h)	-0.95	12.43	3.90	4.59	13.38
<i>(c) Ramps.</i>					
R (m)	71.62	548.22	476.60	263.42	218.98
a_{L_Avg} (g)	0.05	0.30	0.25	0.15	0.10
a_{L_85} (g)	0.10	0.24	0.13	0.16	0.05
V_{A_AVG} (km/h)	52.81	102.65	49.85	83.09	21.64
V_{A_85} (km/h)	57.18	110.96	53.78	89.33	23.42
V_{C_AVG} (km/h)	54.63	89.00	34.37	71.54	15.69
V_{C_85} (km/h)	58.27	97.59	39.32	77.56	17.77
ΔV_{85} (km/h)	-1.09	35.88	11.77	14.96	36.97

R = radius; a_{L_Avg} = average lateral acceleration; a_{L_85} = 85th percentile lateral acceleration; V_{A_AVG} = average approach tangent speed; V_{A_85} = 85th percentile approach tangent speed; V_{C_AVG} = average curve speed; V_{C_85} = 85th percentile curve speed; and $\Delta V_{85} = V_{A_85} - V_{C_85}$.

The speed of the curve (V_C) was taken as the average speed of the effective curve section, which was the same section used to calculate the lateral acceleration adopted for each driver and where the fluctuation in speed in this section was minimal. There were two potential approaches in describing the speed reduction from the tangent to the curve. The first was describing the speed reduction as the difference in speed between the speed environment of the road represented by the 85th percentile approach tangent speed and the

85th percentile curve speed ($\Delta V_{85} = V_{A_{85}} - V_{C_{85}}$). The second approach was to take the 85th percentile of the speed difference between the tangent and the curve, known as the 85th percentile maximum speed reduction (85MSR). Only ΔV_{85} gave significant results at 5% level of significance, therefore, its results will be presented.

6.3 Explanatory Variables

Although the speed limit of rural roads in the study was 80 km/h, the 85th percentile approach speed ($V_{A_{85}}$) ranged from 84 to 107 km/h. Similarly, the curve speeds ($V_{C_{85}}$) ranged from 82 to 100 km/h, with a wide range of speed reduction from 1.5 to 22 km/h. For ramps, the range of $V_{A_{85}}$ was from 57 to 110 km/h. Cases where the speed increased throughout the curve were removed since these indicate that the corresponding lateral acceleration did not reflect driver comfort threshold affected by curve geometry. These cases were very limited and included only 10 incidents out of a total of 217 cases.

$V_{A_{85}}$ was chosen as an explanatory variable in the regression analysis as a surrogate to the speed environment adopted by the drivers before having to change their speed to negotiate the curve. Figure 6.2 shows the relationship between $V_{A_{85}}$ and $a_{L_{85}}$ for all curves on the different road types and turning directions. Three observations could be concluded from this figure. The initial impression of the graph is that as $V_{A_{85}}$ increases, $a_{L_{85}}$ also increases, which is opposite to the premise in the design guides. However, a more thorough and closer examination shows that if each type of road is examined separately (rural versus urban versus ramps), then for each road type, $a_{L_{85}}$ decreases as $V_{A_{85}}$ increases. Another trend found from the relationship is that for the same road facility, right turns have higher $a_{L_{85}}$ than left turns. This will be investigated further in the following section. Another observation is that there is a difference in the values of

$a_{L_{85}}$ for different road types. There also seems to be a scatter in the data points, which is partly due to varying curve speed reductions.

Figure 6.2: Scatter Plot between Approach Speed and Lateral Acceleration.

Figure 6.3 shows the relationship between ΔV_{85} and $a_{L_{85}}$. The main trend found in the figure, is that as ΔV_{85} increases, $a_{L_{85}}$ tolerated by the drivers increases. Such a trend conforms to the findings of previous studies that drivers may accept greater discomfort in order to resist higher speed reductions (Bonneson 1999, Bonneson 2000). As noted previously, the scatter in the figure is due to the difference in approach speed, in addition to the effect of road type and turning direction. It should also be noted here that each of the two previous figures shows the effect of a single parameter on the lateral acceleration.

When the two parameters were included together in the regression analysis presented in the following section, the reading with $a_{L_{85}}$ of 0.34 m/s^2 in both Figure 6.2 and Figure 6.3 would no longer be an outlier.

Figure 6.3: Scatter Plot between Speed Reduction and Lateral Acceleration.

Figure 6.4 shows another comparison made between the lateral acceleration (a_L) adopted by drivers and the corresponding side friction factor ($f_{s,d}$) calculated by the point mass formula for the right turn curves on rural roads. In the figure, $f_{s,d}$ for each driver was calculated using the mean speed of that driver on the same part of the curve used to estimate a_L as explained earlier. The figure reveals that the calculated $f_{s,d}$ is a good representative of actual driver comfort thresholds for curves where ΔV_{85} is minimal (3.76 and 6.75 km/h), as in Curves 6c1 and 3c7. However, for Curves 3c4, 3c8, and 3c10, ΔV_{85}

values are 9.55, 18.71, and 22.22 km/h, respectively, and the measured a_L are significantly greater than the calculated $f_{s,d}$. This implies that driver comfort thresholds, or lateral acceleration (a_L) adopted by drivers, are higher than the corresponding side friction factor at high speed reductions.

Figure 6.4: Comparison between Measured Lateral Acceleration and Calculated Side Friction Factor.

6.4 Preliminary Analysis: Effect of Curve Direction

Preliminary analysis of the collected data of the experiment showed a statistically significant difference between the right and left turn curves. This is also similar to observations of previous studies. Felipe and Navin (1998) found that for sharp curves ($R=16$ and 26 m), the speeds of test drivers on left turn curves were significantly higher than

on right turn curves at a 5% level of significance. They explained that this could be because when drivers are on the left-turn curve, they are on the side of the inside lane, which helps them rely on the delineation of the inside lane, thus increasing their confidence level on left turn curves and in turn their speed would increase. Moreover, Misaghi and Hassan (2005) also noted the effect of the curve direction on the 85th percentile speed reduction on a curve. They reported that for right turn curves the speed reduction was significantly greater than left turn curves.

When translating this conclusion to lateral acceleration, drivers would be expected to experience higher lateral accelerations on right turns than on left turns, thus forcing them to adopt greater speed reductions. Syed and Hassan (2005) investigated this trend and hypothesized that the reason behind the difference is the reaction of the vehicle's suspension system to the uneven weight distribution during data collection. Therefore, it was essential to confirm the validity of this observation in this research. To do this, two control tests were performed.

6.4.1. Test 1

The first control test was performed to investigate the finding of differences in lateral acceleration values between the right and left turn curves. The objective of this control test was to examine whether the accelerometer has a manufacturer error that would cause it to record higher lateral acceleration values in a certain direction. This control test was performed with the Corsa Box containing the accelerometer placed on a stationary surface. The following steps were performed:

1. The Corsa Box was placed on a flat surface, which will be referred to as zero position, for a short period of time,

2. Then, the Corsa Box was raised to a certain measured height,
3. It was then returned to the zero position, after which the other side of the Corsa Box was raised to the same height again.
4. It was then returned to the zero position,
5. The Corsa Box was rotated 180°, and then the same procedure followed in steps 1 to 4 was repeated.

This test was repeated four times. The results of the tests are found in Figure 6.5 and are referred to as trials 1 to 4, with the value of lateral acceleration shown for each position for the Corsa Box. The numbers from 1 to 5 on the figure refer to the step number explained earlier. The figures in general prove that the lateral acceleration value did not change significantly for the different positions explained. The maximum difference between the lateral acceleration was 0.017 g, which is the difference between 0.904 g and 0.887 g and represents a difference of 1.88%.

(a) Trial 1

(b) Trial 2

(c) Trial 3

(d) Trial 4

Figure 6.5: Values Read by Corsa Box for Same Inclinations with Different Directions.

6.4.2. Test 2

Another control test was performed to verify that there is a significant difference between the lateral acceleration values collected on right and left turns under controlled driving conditions. The control test was performed in April 2007 on Regional Road 3 for eight curves (Curves 3c4 to 3c11). The vehicle used for this control test was a sedan vehicle different from the minivan vehicle used in the main experiment to ensure that the body roll of the minivan was not the reason for this difference. In the control test, four runs were made on the route with a constant speed of 70 km/h which was judged as a safe speed on all curves including the sharp ones. The objective of having a constant speed was to remove the effect of the curve direction on the choice of speed.

The first two runs were in opposite directions with the accelerometer facing a certain direction. The following two runs were similar to the first two runs but the accelerometer was rotated 180°. Therefore, four sets of lateral acceleration measurements were available for each curve: (1) right turn, (2) left turn, (3) right turn with the accelerometer rotated 180°, and (4) left turn with the accelerometer rotated 180°. The mean and standard deviation lateral acceleration experienced by the driver on each curve for the four runs are shown in Table 6.4.

Table 6.4: Mean Lateral Accelerations and Standard Deviations for Each Curve for Control Test 2.

	Run 1		Run 2		Run 3		Run 4	
Curve ID	Mean a_L (g)	Standard Deviation	Mean a_L (g)	Standard Deviation	Mean a_L (g)	Standard Deviation	Mean a_L (g)	Standard Deviation
3c4	0.1279	0.0393	0.1255	0.030582	0.0942	0.0430	0.1316	0.0497
3c5	0.1229	0.0125	0.0905	0.0263	0.1344	0.0425	0.0860	0.0185
3c6	0.0780	0.0163	0.0530	0.0141	0.0733	0.0267	0.0568	0.0173
3c7	0.1062	0.0046	0.1534	0.0375	0.0659	0.0084	0.1233	0.0230
3c8	0.0329	0.0250	0.0579	0.0185	0.0237	0.0205	0.0600	0.0306
3c9	0.0940	0.0271	0.0739	0.0255	0.1146	0.1808	0.0697	0.0209
3c10	0.0905	0.0219	0.1267	0.0187	0.0673	0.0306	0.1290	0.0298
3c11	0.1184	0.0357	0.1024	0.0279	0.1529	0.0280	0.1014	0.0312

The significance of the difference between the lateral acceleration data for the four runs was tested using five sets of the t -test of two independent samples at 5% level of significance. The results of the five tests are shown in Table 6.5. The mean difference for each set of test is the difference between the mean lateral acceleration for the pair of runs being compared. For example, for curve ID 3c4, the mean difference for sets (1) and (2) is 0.0025 which is 0.1279 minus 0.1255. The first set tested the equality of means for the right turns and left turns of sets (1) and (2). Therefore, it is testing whether the average lateral acceleration is different between right turns and left turns with the Corsa Box in a

Table 6.5: Test for Equality of Means between Right and Left Turn Curves of Control Test.

Curve ID	R (m)	Mean difference (g)	t-statistic	Degrees of freedom	p-value	Standard error of difference	Reject null hypothesis?
(a) Test for Equality of Means for Sets 1 and 2.							
3c4	273	0.0025	0.60	291	0.5520	0.0042	No
3c5	239	0.0324	11.70	211	<0.0001	0.0028	Yes
3c6	325	0.0250	8.14	99	<0.0001	0.0031	Yes
3c7	180	0.0472	7.99	69	<0.0001	0.0059	Yes
3c8	556	0.0250	4.93	81	<0.0001	0.0051	Yes
3c9	203	0.0201	4.37	129	<0.0001	0.0046	Yes
3c10	267	0.0362	13.27	225	<0.0001	0.0027	Yes
3c11	256	0.0160	4.66	346	<0.0001	0.0034	Yes
(b) Test for Equality of Means for Sets 3 and 4.							
3c4	273	0.0037	6.49	251	<0.0001	0.0057	Yes
3c5	239	0.0485	10.97	203	<0.0001	0.0044	Yes
3c6	325	0.0165	3.81	111	<0.0001	0.0043	Yes
3c7	180	0.0570	16.75	84	<0.0001	0.0034	Yes
3c8	556	0.0363	6.85	95	<0.0001	0.0053	Yes
3c9	203	0.0449	12.06	107	<0.0001	0.0037	Yes
3c10	267	0.0617	16.63	256	<0.0001	0.0037	Yes
3c11	256	0.0515	14.55	278	<0.0001	0.0035	Yes
(c) Test for Equality of Means for Sets 1 and 4.							
3c4	273	0.0037	0.69	241	0.4840	0.0034	No
3c5	239	0.0369	17.60	227	<0.0001	0.0021	Yes
3c6	325	0.0217	5.90	82	<0.0001	0.0036	Yes
3c7	180	0.0171	4.66	69	<0.0001	0.0037	Yes
3c8	556	0.0270	4.81	92	<0.0001	0.0056	Yes
3c9	203	0.0243	5.62	125	<0.0001	0.0043	Yes
3c10	267	0.0385	11.48	240	<0.0001	0.0033	Yes
3c11	256	0.0170	4.52	311	<0.0001	0.0038	Yes
(d) Test for Equality of Means for Sets 1 and 3.							
3c4	273	0.0337	6.93	284	<0.0001	0.0049	Yes
3c5	239	0.0116	2.76	202	0.0060	0.0042	Yes
3c6	325	0.0047	0.97	98	0.3330	0.0048	No
3c7	180	0.0403	27.69	95	<0.0001	0.0015	Yes
3c8	556	0.0093	2.01	96	0.0480	0.0046	Yes
3c9	203	0.0206	4.60	113	<0.0001	0.0045	Yes
3c10	267	0.0232	7.05	270	<0.0001	0.0033	Yes
3c11	256	0.0345	9.37	314	<0.0001	0.0037	Yes
(e) Test for Equality of Means for Sets 2 and 4.							
3c4	273	0.0062	1.23	264	0.2200	0.0050	No
3c5	239	0.0045	1.48	212	0.1410	0.0031	No
3c6	325	0.0039	1.3	112	0.1930	0.0029	No
3c7	180	0.0300	3.74	58	<0.0001	0.0080	Yes
3c8	556	0.0204	0.34	80	0.7330	0.0060	No
3c9	203	0.0042	1.00	124	0.3170	0.0042	No
3c10	267	0.0023	0.68	221	0.4960	0.0033	No

3c11	256	0.0009	0.29	310	0.7720	0.0010	No
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Null hypothesis (H_0): the mean lateral acceleration for the right turn of each curve is equal to that of the left turn.

certain direction. The results in Table 6.5 show that the difference was statistically significant for seven out of the eight curves on the route. The tests were repeated to test the equality of means for the right turns and left turns of sets (3) and (4), and the difference was statically significant on all eight curves. The third set of tests was between sets (1) and (4). The objective of this test was to eliminate the effect of body roll, if significant, from the comparison. This is because the effect of body roll in each run is in the opposite direction, and with the same value. Therefore, the resulting difference in lateral acceleration eliminates the effect of body roll in both runs. Again, the difference was statistically significant on seven curves. Finally, two more sets of tests were made, first between sets (1) and (3) then another between and (2) and (4). These tests compare the difference between the curves in the same direction. For example, in sets (1) and (3) curve 3c4 is left turn, and therefore the mean difference in lateral acceleration should not be significant. For the test of equality between sets (1) and (3) only one curve out of eight did not have a significant difference. On the other hand, for the test of equality between sets (2) and (4), seven out of eight curves did not have a significant difference in lateral acceleration mean, as expected. The unexpected result between sets (1) and (3) is minor given that four out of the five control tests of means of equality conform to expectations. This unexpected result could be attributed to not placing the Corsa Box at the exact same position when rotating it 180°.

In each of these sets, the right turns experienced greater lateral acceleration values than the left turns. Therefore, it was concluded that when negotiating right turns, drivers

experience higher values of lateral acceleration, making it the critical direction. This result follows the previous studies, however it is recommended to investigate in future research the reason behind this difference. Further regression analysis was consequently performed using right turns only.

6.5 Model Development

6.5.1. Modelling Approach

Similar to modelling the desirable spiral length, the variation model approach was used to model the mean lateral acceleration and its dispersion. The general model form can be written as:

$$a_{L,ip} = a_{L,AVG} + Z_p \cdot a_{L,STDEV} + \epsilon = \sum_j b_j \cdot X_{ij} + \sum_k c_k \cdot (Z_p \cdot X_{ik}) + \epsilon \quad (6.2)$$

In this form, $a_{L,ip}$ represents the lateral acceleration at any percentile (p), and is calculated by adding the average lateral acceleration ($a_{L,AVG}$) to the product of the standardized normal variable (Z_p) to the standard deviation of the lateral acceleration ($a_{L,STDEV}$). The percentile lateral accelerations from the 5th to the 95th percentiles in increments of 5 were determined for each site. Then, each of the explanatory variables was multiplied by the Z_p value corresponding to this particular percentile. The b_j coefficient is the effect of the X_j parameter on the average lateral acceleration and the c_k coefficient is the effect of X_k parameter on the standard deviation of the lateral acceleration (Medina and Tarko, 2005).

The descriptive summary of the data used for the regression analysis was shown earlier in Table 6.3. The available database for modelling consisted of data pertaining to rural and urban roads and ramps. From these curves there were 14 right turns and 9 left

turns. However, from the preliminary analysis, it was proven that the side friction demands for right turns were significantly higher than those of left turns. Thus, the right turn direction was the critical direction for which the analysis would continue. In addition, due to the generally higher traffic volumes on urban roads, the free flow observations on these curves were too low to be considered a representative sample. Therefore, for the purpose of this study, regression analysis was performed for rural roads and ramps only, with a filter for ramps.

6.5.2. Model Development

Based on the discussions in the earlier sections, a model was developed to describe lateral acceleration on horizontal curves for rural roads and ramps. Eight curves were used for the analysis, five rural and three ramps. The regression analysis was performed using the software SPSS v15, and produced:

$$a_{L,p} = 0.6853 - 0.0688 I_R - 0.0058 V_{A_{85}} + 0.00576 \Delta V_{85} + 0.0754 Z_p - 0.00046 Z_p V_{A_{85}} + 0.00038 Z_p \Delta V_{85} \quad (6.3)$$

Where $a_{L,p}$ = the lateral acceleration at any percentile (p) in units of g ;

I_R = dummy variable (=1 for a ramp, =0 for rural roads);

$V_{A_{85}}$ = 85th percentile approach speed (km/h); and

ΔV_{85} = 85th percentile speed reduction = $V_{A_{85}} - V_{C_{85}}$ (km/h).

Table 6.6 shows the results of the regression analysis. The model diagnostics show a high coefficient of determination of 99.7% and a low root-mean-square error for a_L of 1.47×10^{-5} g . In addition, each coefficient has a high t -statistic with p -values less than 0.0001 indicating the significant contribution of each variable to the model. Therefore, it

is concluded that this model provides a good estimate of driver comfort thresholds. The coefficient of the ramp indicator (I_R) is significant and equal to -0.0688 showing that when driving on rural roads and ramps, the drivers showed similar behaviour, with drivers on ramps having lower side friction demand for the same speeds. This could be the result of expecting sharp curves and speed reductions on ramps, as opposed to curves on rural roads.

Figure 6.6 shows the predicted mean lateral acceleration from Equation 6.3 compared to the observed mean lateral acceleration from the collected data. The figure shows the strength of the model in modelling the lateral acceleration tolerated by drivers on horizontal curves. In addition, a correlation analysis was performed to examine multicollinearity issues between the model descriptive variables, ΔV_{85} and $V_{A_{85}}$. The correlation analysis found correlation between the two variables ($r = 0.6$); however, there was no evidence of multicollinearity concerns (Variation Inflation Factor, $VIF = 4.2$).

Table 6.6: Model for Driver Comfort on Horizontal Curves.

Variables	Coefficients	SEE	<i>t</i> -statistic	<i>p</i> -value
Constant	0.6853	0.003	215.36	< 0.0001
Ramp Indicator, I_R	-0.0688	0.001	-69.904	< 0.0001
$V_{A_{85}}$	-0.0058	0.000	-166.74	< 0.0001
ΔV_{85}	0.00576	0.000	111.84	< 0.0001
Z_p	0.0754	0.003	26.94	< 0.0001
$Z_p \cdot V_{A_{85}}$	-0.00046	0.000	14.559	< 0.0001
$Z_p \cdot \Delta V_{85}$	0.00038	0.000	7.890	
Model diagnostics: Adjusted $R^2=0.996$, Degrees of freedom=151				
Root mean square of residual= 0.0044

Figure 6.6: Relation of the Observed Mean Lateral Acceleration and that Predicted by Model.

6.6 Model Application and Curve Design

6.6.1. Comparison of Side Friction Demand

Among the strengths of this model is the ability to predict any chosen percentile for lateral acceleration, including the 85th percentile which could be used in the design of horizontal curves. This section presents a comparison between the 85th percentile lateral acceleration ($a_{L_{85}}$), or side friction demand ($f_{s,d}$), values as recommended in the Green Book (AASHTO 2004), Bonneson's study (1999), and in this research. The point-mass formula (Equation 2.1) is the basic relationship used in this section to relate the curve speed ($V_{C_{85}}$) to $a_{L_{85}}$ or $f_{s,d}$. While this is a direct relationship for the design values in the

Green Book, the calculation for the proposed model (Equation 6.4) requires setting up two equations by substituting for ΔV_{85} as $(V_{A_{85}} - V_{C_{85}})$ and solving to determine $a_{L_{85}}$ for a specific curve radius, superelevation rate, and approach speed. It should be noted that the side friction values in TAC (1999) are almost similar to those in the Green Book, and therefore the comparison is valid for both guides.

Therefore, the two equations used to determine $a_{L_{85}}$ were Equation 6.4 and the point mass formula.

$$a_{L_{85}} = 0.76339 - 0.0688 I_R - 0.00628 V_{A_{85}} + 0.00615 \Delta V_{85} \quad (6.4)$$

Table 6.7 shows the calculations used to find $f_{s,d}$ for both the guides and the developed model for a range of radii. First, a range of suitable radii values for two-lane rural roads was chosen between 150 m and 550 m. The superelevation was assumed constant at 6%. Then from the Green Book, the values of $f_{s,d}$ corresponding to the design speed and radius was found. For the developed model, the same range of radii was used and the operating speed and side friction demand were determined using both Equations 6.4 and the point mass formula simultaneously. The approach speed was assumed as 103 km/h which previous research in Ottawa found to be the 85th percentile approach speed on tangents for two-lane highways (Misaghi and Hassan 2005). It should be noted that although the relationship between the operating speed and the radius is nonlinear, the relationship between the curve speed and $a_{L_{85}}$ is linear as shown in Equation 6.4.

Figure 6.7 shows the results of the comparison between $a_{L_{85}}$ developed in this study, based on actual driving conditions, and $f_{s,d}$ used in the design guides for high speed rural roads. It is obvious that there is a gap between the values adopted by the drivers and those used in the design, with the gap increasing as the speed decreases, a condition that

usually corresponds to sharper curves. Even though the objective of using lower $f_{s,d}$ values in the guides is to increase the safety margin between the designed values and the skidding values, two concerns arise from this gap. First, since $f_{s,d}$ is substantially less than the driver comfort threshold, the actual operating speed of the drivers inferred by the comfort threshold model will be greater than the design speed, especially for sharp curves. With the significant improvements in vehicle technology, drivers may not feel great discomfort when negotiating horizontal curves with higher speeds than what they were designed for. This is a major concern, as it would lead to inconsistency in the design of horizontal curves. The second concern is that the lower margin of safety between the comfort threshold and the sliding threshold may lead to a safety hazard of sliding on horizontal curves.

Table 6.7: Values of Radii and Design Speeds Used for Comparing $f_{s,d}$

R (m)	Guides		Developed Model	
	Design Speed (km/h)	$f_{s,d}$ (g)	V_C (km/h)	$f_{s,d}$ (g)
150	63	0.150	79	0.268
200	73	0.148	85	0.228
250	80	0.14	90	0.197
300	86	0.135	94	0.175
350	92	0.129	97	0.154
400	97	0.124	100	0.138
450	101	0.119	102	0.123
500	105	0.114	104	0.111

Figure 6.7: Comparison of Predicted Lateral Acceleration with Side Friction Factor of Design Guides and Developed Model.

6.6.2. Inferred Curve Speed Using Developed Model

A further step was made to find the implication of the difference between $f_{s,d}$ in the guides and that of the developed model. As a surrogate to the operating speed on horizontal curves, Figure 6.8 shows $V_{C_{85}}$ calculated using the point mass formula with superelevation of 6% and the three approaches for estimating $f_{s,d}$: the Green Book, the developed model, and Bonneson's model. First, the figure shows the insensitivity of the Green Book values to $V_{A_{85}}$. As for Bonneson's model, $V_{C_{85}}$ for a certain radius varies considerably for different $V_{A_{85}}$ values. For the developed model, $V_{A_{85}}$ was a significant parameter but $V_{C_{85}}$ values corresponding to different $V_{A_{85}}$ values were close to each other at the early stages corresponding to sharper radii. This implies that for different $V_{A_{85}}$ values, the drivers reduce their speeds differently to reach a certain curve speed.

This behaviour is closer to observed curve operating speed trends, where the spread in speed tends to be lower on sharper curves.

(a) Rural Roads.

(b) Ramps.

Figure 6.8: Comparison between the Designed Speed and the Inferred Speed using Measured Driver Comfort Thresholds.

The main concern found from the figure is that there is a significant difference between the curve speed inferred by the developed model and the design speed of the Green Book at the sharper range of curves (radii less than 350 m). For example, for a radius of 200 m, the Green Book would suggest a design speed of 74 km/h, while the inferred speed of the developed model is 80 km/h. For a curve of radius 150 m, the design speed would be 63 km/h, and the inferred speed would be 74 km/h. The difference increases as the radius decreases, and these differences between the inferred and design speeds would result in inconsistency in design and may also lead to safety issues. Therefore, it is recommended to revise the values of $f_{s,d}$ used in the design guides for speeds in the range of 80 to 90 km/h to reflect driving behaviour.

Finally, using the developed model for $a_{L_{85}}$ or $f_{s,d}$ in Equation 6.4 in combination with the point mass formula, the curve speed was calculated based on the approach speed, superelevation, and curve radius. Figure 6.9 shows the results of the analysis. A range of radii between 100 m and 600 m was studied for a range of approach speeds between 80 km/h and 110 km/h, and superelevation rates ranging from 4% to 12%. These ranges are within the limits of this research. The figure shows that drivers generally try to minimize speed reduction, and that they will reduce their speed only on sharp curves. Figure 6.9 (a) shows the application of the model for rural roads, and Figure 6.9 (b) shows the application on ramps. The comparison between the two figures illustrates that the speed reductions on ramps are higher than on two-lane rural roads, indicating that drivers tend to accept larger speed reductions on ramps as opposed to curves on a rural road.

(a) Rural Roads.

(b) Ramps.

Figure 6.9: Inferred Curve Speeds for Different Approach Speeds and Superelevations.

To illustrate the benefit of this figure, suppose a curve with a radius of 250 m and superelevation 12%. In case of a curve on a rural two-lane road, if the driver approaches the curve with a speed of 80 km/h, then, as Figure 6.9 (a) points out, there will be no speed reduction. However, if the driver approaches the curve with a higher speed of 100 km/h, then the expected curve speed as shown from the figure would be 95 km/h. Similarly, for a curve on a ramp with a radius of 250 m and superelevation 12%, then Figure 6.9 (b) points out that there will be no speed reduction in case of 80 km/h approach speed. However, for an approach speed of 100 km/h, the speed reduction would be 11 km/h, and the inferred curve speed would be 89 km/h.

On the other hand, the developed model combined with the point mass formula is sensitive to superelevation rate. For example, for a curve radius of 400 m on a rural two-lane highway, and a superelevation rate of 12%, if the approach speed were 100 km/h, there would be no speed reduction as shown from the figure. However, for the same curve, with superelevation rate of 4%, the inferred curve speed would be 90 km/h.

6.7 Conclusions

This chapter showed the results of the analysis of driver comfort thresholds on horizontal curves. The analysis proved that drivers experience higher values of lateral acceleration when negotiating right turns. In addition, this chapter presented a model describing driver comfort thresholds on horizontal curves for rural roads and ramps. The uniqueness of this model is that it is based on lateral accelerations measured under actual driving conditions. In addition, it models the distribution of lateral acceleration and can be used to determine any chosen percentile of side friction demanded. Based on the results of this study it was found that curve operating speeds as inferred from the

developed model would be higher than the design speed for curves of radius 350 m or less. This difference would result in two potentially unfavourable conditions. The first is inconsistency of design and the second is reduction in the margin of safety between side friction demanded and coefficient of friction at impending skidding.

The developed model can be used, in combination with the point mass formula, in determining the minimum horizontal curve radius for a specific speed environment, represented by the approach speed in the model. To achieve this, the point mass formula is used, substituting the side friction demand with calculated values using the developed model.

In addition, the model can be used in evaluating design consistency of horizontal curves. For a specific radius and superelevation rate, an inferred curve operating speed could be predicted and compared to the design speed. In addition, the inferred speeds of a series of horizontal curves could be predicted using this model and hence a check could be made for uniformity of speeds to ensure design consistency.

It should be noted here that even though the study used a relatively small number of sites, it has the unique advantage of utilizing measured lateral acceleration values under actual driving conditions. The reliability of the data and developed model is evident from their conformity with expected trends and proximity to the findings of Bonneson (1999), which were based on a much larger database but no measured lateral acceleration values.

CHAPTER 7. DECELERATION AND ACCELERATION

BEHAVIOUR FOR TWO-LANE RURAL ROADS

The objective of this chapter is to study the deceleration and acceleration rates before and after the horizontal curves on two-lane rural highways. These rates could help in evaluating drivers' behaviour at entrances and exits of curves and their relationship with expected safety performance. For example, a high deceleration rate at a curve entrance would indicate an unexpected curve or inability to judge the curve sharpness. Either condition would be expected to have negative safety impacts. In addition, average deceleration and acceleration rates are required in order to trace the speed profile for drivers on horizontal curves. Many studies have shown that one of the main factors affecting collision rates on horizontal curves is the speed reduction from an approach tangent to a curve (Lamm et al. 1986, Anderson et al. 1999, Ng and Sayed 2004). Therefore, the need to understand driver behaviour and a conforming geometric design is significant, especially on rural two-lane horizontal curves where collisions are more frequent. Evaluation of design consistency of an alignment is one of the significant tools used for assessing safety on horizontal curves.

Operating speed profiles have been used since the 1970s in the Swedish design guides to address inconsistencies in speeds and therefore in the design of roads (Swiss 1981). They are used to trace the change of speed between tangents and curves. Along with the operating speeds on tangents and curves, the length of speed reduction or increase is calculated using the deceleration and acceleration rates entering and exiting the curve respectively. Several models have been developed to predict the speeds on

horizontal curves and tangents for two-lane rural roads. On the other hand, there are few research studies that have been performed to predict acceleration and deceleration rates. However, driver behaviour in this research showed that using a constant rate on different curve radii, which is the assumption adopted in most speed profile models, is far from the reality. A thorough review of the approaches adopted for determining the deceleration and acceleration rates on two-lane rural highway was given previously in Section 2.8.1.d of Chapter 2.

In this chapter, the curves used in the analysis are first presented. Then, the methodology used for calculating deceleration and acceleration rates follows. Finally, analysis of these rates along with the development of models relating both the average and maximum acceleration and deceleration rates to the geometric characteristics of the curve are presented.

7.1 Preparation of Data Set

7.1.1. Curves Used in Analysis

Table 7.1 shows the characteristics of the curves used for analysis. For deceleration rate analysis, a total of thirteen curves were studied, ten curves on rural roads and three curves on ramps. For the acceleration rate analysis, fourteen curves were studied, the same ten curves on rural roads, and four curves on ramps. Ramps were included in the study as the preliminary analysis, which will follow shortly, showed that drivers adopt similar behaviour on the curves of ramps as that on curves of two-lane rural roads. One of the curves was not studied for the deceleration analysis on ramps (curve ID 6-416c1) since the vehicles were approaching this curve with very low speed as they were recovering from a yield sign and a right turn from Highway 6 to the ramp. Therefore, all

the vehicles were accelerating on the curve entrance. This behaviour is shown in Figure 7.1

Table 7.1: Geometric Characteristics of Curves Used in Analysis.

Curve ID	R (m)	L_c (m)
(a) Rural Roads.		
3c4	273.32	381.54
3c5	239.17	224.95
3c6	325.05	169.97
3c7	180.17	40.00
3c8	555.89	160.00
3c9	203.35	71
3c10	267.25	429.90
3c11	255.94	369.94
6c1	463.05	159.97
7c1	1167.95	1073.56
(b) Ramps.		
6-416c1	71.62	119.87
416-12c1	151.42	94.97
417-7c1	447.83	105
417-7c2	548.22	384.09

Note: R = radius; L_c = curve length.

Figure 7.1: Speed Profiles for Free-Flow Drivers on Curve 6-416c1.

One of the ramp curves from the test route was not included in the analysis (curve ID 416-12c2). The speed profile for curve 417-12c2 is shown in Figure 7.2. Due to the presence of a stop sign at the end of the curve, the acceleration and deceleration behaviours were not a result of the geometric features of the curve. Therefore, the curve could not be used in the analysis.

Figure 7.2: Speed Profile for Free Flow Drivers on Curve 416-12c2.

7.1.2. Calculation of Deceleration and Acceleration Rates Used in Analysis

Figure 7.3 shows an example of a deceleration profile for one of the drivers on the entrance of a horizontal curve with a radius of 273 m (Driver No. 18 on Curve 3c4). The deceleration profile shows that the deceleration rate increases gradually starting 50 m before the beginning of the curve (PC), reaches a maximum value of around -1.66 m/s^2 at 15 m from PC and continues with this value for 45 m, and then gradually the deceleration rate decreases again.

Figure 7.3: Horizontal Alignment, Deceleration Rate, Throttle, and Speed Profiles for Driver 18 on Curve 3c4.

From the speed profile it can be concluded that the speed reduction is from 50 m before *PC* to 105 m after *PC*. The throttle profile shows that the driver's foot was released from the throttle pedal also 50 m before *PC* and applied pressure once again 105 m after *PC*. The brake profile shows that maximum brake pressure is consistent with maximum deceleration values. Using these four profiles, along with the horizontal alignment of the curve, the following could be concluded for this driver on this curve:

1. The beginning and end points of deceleration could be detected as explained earlier. Therefore the deceleration length (L_d) is equal to 155m. The beginning and end of deceleration were decided based on multiple criteria including the throttle pedal position, which would be minimum during deceleration, the start and end of speed reduction, and the deceleration profile. When tracing the braking pedal position, it was found that drivers do not always brake in order to reduce their speed. Therefore, it was not a significant criterion when determining the deceleration length.
2. The brake profile, on the other hand, shows points where maximum deceleration occurs. Therefore, maximum deceleration values could be a good representative of hard or aggressive braking.
3. The average deceleration rate (d) is equal to -0.995 m/s^2 for the profile shown in Figure 7.3. It was calculated by taking the average of all deceleration rate readings in the range of the deceleration length of 155 m. The maximum deceleration rate (d_{max}) is equal to -1.66 m/s^2 . It is important to note here that both the average and maximum deceleration rates are used to describe driving behaviour. The average value is used to describe the speed reduction profile, and the maximum

deceleration rate is used to detect drivers' expectations to curves, especially sharp ones.

4. The percentage of deceleration on the tangent ($\%T_d$) is 32%.

Profiles for all curves are shown in Appendix E. The approach of measuring the average deceleration rate (d) in this research was compared to a traditional approach adopted by most studies to calculate the average deceleration rate using Equation 7.1, and which will be referred to as second approach.

$$d_{calculated} = \frac{V_1^2 - V_2^2}{2L_d} \quad (7.1)$$

The approach adopted in this study is more useful as it is readily extracted from the data acquisition instrument, rather than calculated from Equation 7.1. Therefore, it represents driving behaviour. Nonetheless, as a means of comparison, the average deceleration rate found in this study (d) was compared to the calculated deceleration rate ($d_{calculated}$) using Equation 7.1, for all drivers on all curves. As shown in Figure 7.4, the two variables were almost equal with d higher than $d_{calculated}$ on average by 0.05 m/s^2 as shown in Equation 7.2.

$$d = 1.0002d_{calculated} - 0.0508 \quad R_{adj.}^2 = 0.953 \quad (7.2)$$

A third approach for measuring the 85th percentile deceleration rate was using the difference of the 85th percentile speeds at the beginning and end of the deceleration length as shown in Equation 7.3.

$$d'_{85} = \frac{V_{A-85}^2 - V_{C-85}^2}{2L_d} \quad (7.3)$$

Figure 7.4: Relation between Calculated Deceleration Rate and Measured Deceleration Rate for Each Driver on Each Curve.

This third approach is used in most of the studies mentioned previously (Fitzpatrick et al. 2000, Crisman et al. 2007) as it corresponds to the 85th percentile speeds used in the speed profile. To find if there was a significant difference between the two approaches, a paired *t*-test was performed between the two values for the thirteen curves. The result showed that there was no significant difference between the two values at 5% level of significance (p -value=0.797). In this study, the first approach explained was used, as it represents the actual deceleration rate experience by the individual drivers, which is the scope of this study.

Using the first approach for measuring the average deceleration rate, for each curve a data set was created where for each driver the average deceleration rate (d) and

maximum deceleration rate (d_{max}) were determined. Using these datasets, the following were calculated for each curve:

1. The average, 85th percentile, and standard deviation for average deceleration rate (d_{Avg} , d_{85} , d_{SD});
2. The average, 85th percentile, and standard deviation for maximum deceleration rate (d_{max_Avg} , d_{max_85} , d_{max_SD});
3. The average and 85th percentile portion of deceleration length occurring on the tangent preceding the curve ($\%T_{d_AVG}$, $\%T_{d_85}$).

The resultant data are shown in Table 7.2. In addition, the descriptive statistics of these data are shown in Table 7.3. Table 7.3 shows that there was a significant spread in the deceleration rates depending on the curve parameters.

The same parameters were also extracted for the acceleration rate when exiting the horizontal curves following the same procedure. The average, 85th percentile, and standard deviation for the average acceleration rate (a_{Avg} , a_{85} , a_{SD}) were calculated. In addition, the average, 85th percentile, and standard deviation for the maximum acceleration rate (a_{max_Avg} , a_{max_85} , a_{max_SD}) were calculated. Finally, the average and 85th percentile portion of acceleration length occurring on the tangent following the curve ($\%T_{a_AVG}$, $\%T_{a_85}$) were determined. Table 7.2 and Table 7.3 show the values of as well as the descriptive statistics.

7.2 Data Analysis

Four single sample *t*-tests were performed before developing a model for the acceleration and deceleration behaviour on horizontal curves. The main objective of these tests was to validate the values used in previous studies. The following tests were made:

Table 7.2: Data Used for Analysis of Deceleration and Acceleration Rates.

(a) Deceleration						
Curve ID	$d_{max\ 85}$ (m/s ²)	$d_{max\ Avg}$ (m/s ²)	d_{85} (m/s ²)	d_{Avg} (m/s ²)	% $T_{d\ 85}$ (m)	% $T_{d\ AVG}$ (m)
3c4	1.724	0.882	1.108	0.597	86.28	43.83
3c5	0.692	0.559	0.728	0.410	100.00	90.67
3c6	1.058	0.541	0.582	0.392	100.00	93.03
3c7	0.812	0.441	0.642	0.316	100.00	89.94
3c8	0.594	0.377	0.479	0.270	100.00	62.78
3c9	1.121	0.614	0.690	0.423	100.00	60.10
3c10	1.460	0.861	0.916	0.564	68.07	43.92
3c11	0.920	0.431	0.509	0.297	100.00	63.65
6c1	1.687	0.641	0.970	0.383	100.00	96.60
7c1	0.342	0.226	0.064	0.061	100.00	38.36
416-12c1	2.218	1.639	1.535	1.185	100.00	90.63
417-7c1	0.984	0.611	0.548	0.374	100.00	90.37
417-7c2	1.020	0.629	0.559	0.380	100.00	89.77
(b) Acceleration						
Curve ID	$a_{max\ 85}$ (m/s ²)	$a_{max\ Avg}$ (m/s ²)	a_{85} (m/s ²)	a_{Avg} (m/s ²)	% $T_{a\ 85}$ (m)	% $T_{a\ AVG}$ (m)
3c4	0.547	0.353	0.468	0.242	27.69	9.46
3c5	0.656	0.545	0.556	0.409	87.6	63.53
3c6	0.491	0.340	0.321	0.232	0	1.39
3c7	0.782	0.622	0.602	0.468	100	90.02
3c8	0.449	0.287	0.361	0.201	100	86.72
3c9	0.529	0.383	0.422	0.273	100	80.22
3c10	0.699	0.529	0.610	0.454	61.63	30.08
3c11	0.789	0.537	0.656	0.414	100	79.50
6c1	0.744	0.413	0.632	0.278	92.65	57.10
7c1	0.333	0.207	0.182	0.080	100.00	100
416-12c1	0.647	0.391	0.536	0.319	100	70.77
417-7c1	0.542	0.453	0.391	0.287	94.51	53.29
417-7c2	0.500	0.383	0.395	0.283	100	96.88
6-416c1	1.370	1.202	0.941	0.787	100	87.20

Table 7.3: Descriptive Statistics of the Characteristics of Study Curves.

Curve Characteristics	Minimum	Maximum	Mean	Standard Deviation	Range
(a) Deceleration Statistics					
<i>R</i> (m)	151.42	1167.95	390.66	270.16	1016.53
<i>L_c</i> (m)	39.99	1073.56	281.92	272.56	1033.56
<i>d_{avg}</i> (m/s ²)	0.06	1.18	0.43	0.26	1.12
<i>d₈₅</i> (m/s ²)	0.06	1.53	0.72	0.36	1.47
<i>d_{max_avg}</i> (m/s ²)	0.23	1.64	0.65	0.35	1.41
<i>d_{max_85}</i> (m/s ²)	0.34	2.22	1.13	0.52	1.88
% <i>T_{d_avg}</i> (m)	38.36	96.60	73.36	21.80	58.24
% <i>T_{d_85}</i> (m)	100	96.40	93.42	31.93	100
(b) Acceleration Statistics					
<i>R</i> (m)	71.62	1167.95	293.27	152.07	484.27
<i>L_c</i> (m)	39.99	1073.56	200.09	142.28	389.90
<i>a_{avg}</i> (m/s ²)	0.08	0.79	0.34	0.17	0.71
<i>a₈₅</i> (m/s ²)	0.18	0.94	0.51	0.19	0.76
<i>a_{max_avg}</i> (m/s ²)	0.21	1.20	0.47	0.24	1.00
<i>a_{max_85}</i> (m/s ²)	0.33	1.37	0.65	0.25	1.04
% <i>T_{a_avg}</i> (m)	1.39	100	64.73	31.45	98.61
% <i>T_{a_85}</i> (m)	0	100	83.15	31.58	100

1. First Null Hypothesis: 85th percentile deceleration rate at curve entrance (*d₈₅*) = 0.85 m/s²

As shown previously in Table 7.3, the average 85th percentile deceleration rate of the studied sample was 0.718 m/s². A single sample *t*-test was performed to test if *d₈₅* was significantly different from the value proposed in previous studies (0.85 m/s²) at 5% level of significance. The result of the test showed that the average is not significantly different from 0.85 m/s² (*p*-value= 0.206). This is the same conclusion made by Collins and Krammes (1996). However, the wide range of *d₈₅* with the change in radius (0.06 m/s² – 1.53 m/s²) suggests that a single value should not represent all deceleration rates on all curves regardless of their radii. In addition, the kurtosis value is equal to 1.708, which is higher than a preferred value of 1 for normal distribution. This implies that the sample may be flatter than a normal distribution, resulting in a wide range of *d₈₅* values.

2. Second Null Hypothesis: 85th percentile acceleration rate at curve exit (a_{85}) = 0.85 m/s²

As shown previously in Table 7.3, the average 85th percentile acceleration rate of the studied sample was 0.51 m/s². Another single sample *t*-test was performed to test if a_{85} was significantly different than 0.85 m/s² at 5% level of significance. The result of the *t*-test showed that there is a significant difference between a_{85} and 0.85 m/s² (*p*-value <0.001). This conforms to results of Collins and Krammes (1996) and Fitzpatrick et al. (2000). Therefore, using this value in the speed-profile underestimates the acceleration length. In addition, the wide range of the acceleration rate (0.18 m/s² – 0.94 m/s²) also shows that the geometric characteristics of the curve affect the acceleration rate adopted by the drivers. Finally, the kurtosis value was found to be 6.7, which indicates that the sample has a very flat distribution, resulting in a wide range of a_{85} values. This in turn could be a factor in the result of this test.

3. Third Null Hypothesis: 85th percentile of percent of deceleration occurring on tangent section ($\%T_{d_{85}}$) = 100%.

It was mentioned earlier that most studies assume that the deceleration and acceleration occur on the tangent sections preceding and following the horizontal curve. Medina and Tarko (2007) developed a model using an iteration approach and found that 66% of the deceleration and 72% of the acceleration take place on the tangents preceding and following the horizontal curve respectively. To validate previous studies, a single sample *t*-test was performed to test if $\%T_{d_{85}}$ was significantly different than 100% at 5% level of significance. That implies that the full deceleration length was on the tangent section preceding the horizontal curve. As shown in Table 7.3, the average $\%T_{d_{85}}$ of the

studied sample was 96.5%. The result of the t -test showed that there is no significant difference between $\%T_{d_{85}}$ and 100% (p -value=0.200), which conforms to the assumption made in most studies.

4. Fourth Null Hypothesis: 85th percentile of percent of acceleration occurring on tangent section ($\%T_{a_{85}}$)=100%.

Similar to $\%T_{d_{85}}$, a single sample t -test was performed to test if $\%T_{a_{85}}$ was significantly different 100% at 5% level of significance. That means that the full acceleration length occurred on the tangent section following the horizontal curve. As shown previously in Table 7.3, the average $\%T_{a_{85}}$ of the studied sample was 83.15%. The result of the t -test showed that there is no significant difference between $\%T_{a_{85}}$ and 100% (p -value=0.076). The results of the latter two t -tests conform to the assumption made in most studies, and therefore will be adopted in this research.

Therefore, these tests have shown that there is a wide range of deceleration and acceleration rates observed for the curves analyzed. The tests indicate that there could be factors that contribute to this variation and affect the choice of deceleration and acceleration rate of drivers. In addition, the assumption made in most studies that the deceleration and acceleration at entrances and exits of curves occur on the tangents before and after the curves was found to be correct.

7.3 Model Development

7.3.1. Explanatory Variables

From the geometric characteristics of the curves, correlation analysis was performed for each of 85th percentile of average and maximum deceleration and

acceleration rates (d_{85} , a_{85} , d_{max_85} , and a_{max_85}) with the curve radius, curve length, and preceding tangent length.

7.3.1.a. Deceleration Rate

In case of d_{85} , there was a significant correlation at 5% level of significance with the inverse of the radius ($r=0.663$). The 85th percentile average deceleration rate increases with decrease of radius and increase of speed reduction. Figure 7.5 also shows the strong and significant relationship between d_{85} and R for the thirteen curves studied and that the deceleration rate adopted by the drivers decreases with increase in radius. There is a difference in the trend between curves preceded by independent tangents and curves preceded by nonindependent tangents, since the speed reduction in both cases is different. It should also be noted that including a term representing curves preceded by an independent tangent in the regression model implies that the greater the speed reduction the driver needs to make, the higher the deceleration rate adopted by the driver. Therefore, having a long independent tangent followed by a sharp curve would result in a higher deceleration rate than the same curve radius preceded by a nonindependent tangent where the speed reduction would be less.

It should also be noted that there was no significant correlation at 5% level of significance with the curve length ($r=-0.088$). In addition, the correlation between the deceleration rate and the preceding tangent length was also not significant at 5% level of significance ($r=0.357$). Therefore, no further analysis was performed with these two variables.

Figure 7.5: Relationship between Radius and 85th Percentile Deceleration Rate.

Similarly, Figure 7.6 shows the relationship between the radius and 85th percentile maximum deceleration rates. The figure shows that the maximum deceleration rate has a similar trend as the average deceleration rate. In addition, there was a significant correlation at 5% level of significance with the inverse of the radius ($r=0.497$).

Figure 7.6: Relationship between Radius and 85th Percentile Maximum Deceleration Rate.

7.3.1.b. Acceleration Rate

In case of the 85th percentile average acceleration rate, there was a significant correlation at 1% level of significance with $1/R$ ($r=0.796$). Figure 7.7 shows the relationship between a_{85} and the curve radius for the fourteen curves used in the analysis. Similar to deceleration rate, the figure shows that the acceleration rate decreases as the radius increases. In addition, the figure shows a clear similarity in the trend for both rural and ramp curves. Therefore, when modelling the acceleration rate, both curves will also be used in the analysis.

In case of acceleration analysis, the tangents were classified whether independent or nonindependent based on the departing tangent. That is, if a vehicle is exiting a curve, and the tangent connecting between this curve and the following curve is greater than 200 m then the tangent is considered independent, and if the tangent is less than 200 m then the tangent is nonindependent. The type of departure tangent was found significant when modelling the acceleration rate as will be shown in the following section. It was found that there was no significant correlation at 5% level of significance with the curve length ($r=-0.361$). Similarly, for the 85th percentile maximum acceleration, Figure 7.8 shows the relationship between the radius and 85th percentile maximum acceleration rates. The figure shows that the maximum acceleration rate has a similar trend as the average acceleration rate.

Figure 7.7: Relationship between Radius and 85th Percentile Average Acceleration Rate.

Figure 7.8: Relationship between Radius and 85th Percentile Maximum Acceleration Rate.

7.3.2. Modelling Approach

The variation model approach was used in modelling the driver's choice of deceleration and acceleration rates. The general model form in case of deceleration rate can be written as:

$$d_{ip} = d_{iAVG} + Z_p \cdot d_{iSTDEV} + \epsilon = \sum_j b_j \cdot X_{ij} + \sum_k c_k \cdot (Z_p \cdot X_{ik}) + \epsilon \quad (7.4)$$

In this form, d_{ip} represents the deceleration rate at any percentile (p), and was calculated by adding the average deceleration (d_{iAVG}) to the product of the standardized normal variable (Z_p) to the standard deviation of the deceleration rate (d_{iSTDEV}). The percentile deceleration rates from the 5th to the 95th percentiles in increments of 5 were

determined for each site. Then, each of the explanatory variables is multiplied by the Z_p value corresponding to this particular percentile. The b_j coefficient is the effect of the X_j parameter on the average deceleration rate and the c_k coefficient is the effect of X_k parameter on the standard deviation of the deceleration rate.

7.3.3. Model Development

The descriptive summary of the data used for the regression analysis was shown earlier in Table 7.3. The available database for modelling consisted of data pertaining to thirteen curves for the deceleration rate analysis and fourteen curves for the acceleration rate analysis. The regression analysis was performed using the software SPSS v15.

7.3.3.a. Deceleration Rate

1. Average Deceleration Rate

The regression analysis using the variation model developed the following model for the deceleration rate at any percentile:

$$d_p = 0.361 + \frac{198.271}{R} I_T - 0.497 I_T + 0.265 Z_p + \frac{37.936}{R} I_T Z_p - 0.101 I_T Z_p \quad (7.5)$$

Even though this model can be used to determine any percentile, speed profiles used for evaluating design consistency calculate the 85th percentile values. The main advantage of using this new approach was to model the factors that affect both the mean deceleration rate and the variation in the deceleration rate. In this case, Equation 7.5 shows that the mean deceleration rate and the variation in deceleration rate are affected by the inverse of the radius and whether the curve is preceded by an independent tangent or not.

Calculation of d_{85} , for use in speed profiles is done by substituting in Equation 7.5 with Z_p equal to 1.036, resulting in Equation 7.6.

$$d_{85} = 0.636 + \frac{237.573}{R} I_T - 0.602 I_T \quad (7.6)$$

Table 7.4 shows the results of the regression analysis. The model diagnostics show a high coefficient of determination of 95% and a low root-mean-square error of 0.077 m/s^2 . In addition, the values of the t -statistic for the coefficients indicate the significant contribution of each variable to the model. Therefore, it is concluded that this model provides a good estimate of the deceleration rate when entering a curve. Figure 7.9 shows the predicted 85th percentile deceleration rates from Equation 7.6 compared to the observed 85th percentile values from the collected data. The figure shows the strength of the model in predicting d_{85} on the curve entrance.

Table 7.4: Models for Average and Maximum Deceleration Rates on Horizontal Curves.

Variables	Coefficients	SEE	t -statistic	p -value
Constant	0.361	0.008	43.842	< 0.0001
$(1/R) \cdot (I_T)$	198.271	3.977	49.848	< 0.0001
I_T	-0.497	0.016	-31.247	< 0.0001
Z_p	0.265	0.01	27.712	< 0.0001
$Z_p \cdot (1/R) \cdot (I_T)$	37.936	4.625		< 0.0001
$Z_p \cdot (I_T)$	-0.101	0.019	-5.441	< 0.0001
Model diagnostics: Adjusted $R^2=0.95$, Degrees of freedom=246				
Root mean square of residual= 0.077 m/s^2				
(b) Maximum Deceleration Rate Model.				
Constant		0.014	38.367	< 0.0001
$(1/R) \cdot (I_T)$	239.525	6.94	34.512	< 0.0001
I_T	-0.551	0.028	-19.878	< 0.0001
	0.512	0.056	9.111	< 0.0001
$Z_p \cdot (1/R)$	-41.278	13.176	-3.133	0.002
$Z_p \cdot (1/R) \cdot (I_T)$	116.68	15.449	7.553	< 0.0001
$Z_p \cdot (I_T)$	-0.287	0.063	-4.582	< 0.0001
Model diagnostics: Adjusted $R^2=0.923$, Degrees of freedom=246				
Root mean square of residual= 0.141 m/s^2

Figure 7.9: Relation of the Observed and Model Predicted Values of d_{85} and d_{max_85} .

2. Maximum Deceleration Rate

The regression analysis using the variation model developed the following model for the maximum deceleration rate at any percentile:

$$d_{max_p} = 0.552 + \frac{239.525}{R} I_T - 0.551 I_T + 0.512 Z_p + \frac{116.680 I_T - 41.278}{R} Z_p - 0.287 I_T Z_p \quad (7.7)$$

This model is useful in finding the factors that affect the aggressive application in brakes while entering a sharp curve in terms of maximum deceleration rate indicating an unexpected curve or inability to judge the curve sharpness. The model shows that the

maximum deceleration rate and the variation in deceleration rate are affected by both the inverse of the radius and whether the curve is preceded by an independent tangent or not.

Calculation of d_{max_85} is done by substituting in Equation 7.7 with Z_p equal to 1.036, resulting in Equation 7.8.

$$d_{max_85} = 1.082 + \frac{(360.406I_T - 42.764)}{R} - 0.848I_T \quad (7.8)$$

Table 7.4 shows the results of the regression analysis. The model diagnostics show a high coefficient of determination of 92.3% and a low root-mean-square error of 2.17 m/s². In addition, the values of the *t*-statistic for the coefficients indicate the significant contribution of each variable to the model. Therefore, it is concluded that this model provides a good estimate of the maximum deceleration rate when entering a curve. In addition, Figure 7.9 shows the predicted 85th percentile maximum deceleration rates from Equation 7.8 compared to the observed 85th percentile values from the collected data. The figure shows the strength of the model in predicting d_{85} on the curve entrance.

7.3.3.b. Acceleration Rate

1. Average Acceleration Rate

The regression analysis developed the following model for the acceleration rate at any percentile:

$$a_p = 0.121 + \frac{44.563}{R} + 0.149Z_p + 0.046I_{TF} \quad (7.9)$$

Where I_{TF} = dummy variable (=1 for independent tangent following the curve, =0 for nonindependent tangent).

The mean acceleration rate is affected by the inverse of the radius for both curves preceded by independent and nonindependent tangents, where both curves followed by independent and non-independent curves have the same trend.

The model for a_{85} by substituting with $Z_p=1.036$ will render:

$$a_p = 0.275 + \frac{44.563}{R} + 0.046I_{TF} \quad (7.10)$$

The 85th percentile acceleration rate model diagnostics in Table 7.5 show that the model has a high coefficient of determination of 88.6% and a low root-mean-square error of 0.084 m/s². Each coefficient has a high t -statistic with p -values less than 0.0001 for all the coefficients indicating the significant contribution of each variable to the model. Therefore, this model also gives a good estimate of the 85th percentile average acceleration rate departing from a horizontal curve. In addition, Figure 7.10 also shows the predicted 85th percentile acceleration rates from Equation 7.8 compared to the observed 85th percentile values from the collected data. The figure shows the strength of the model in predicting a_{85} on the curve exit.

2. Maximum Acceleration Rate

The regression analysis using the variation model developed the following model for the maximum acceleration rate at any percentile:

$$a_{max-p} = 0.136 + \frac{64.552}{R} + 0.106I_{TF} + 0.140Z_p + \frac{8.499}{R}Z_p \quad (7.11)$$

The model shows that the maximum acceleration rate is affected by the inverse of the radius, where both curves followed by independent and non-independent curves have

Table 7.5: Models for Average and Maximum Acceleration Rates on Horizontal Curves.

Variables	Coefficients	SEE	<i>t</i> -statistic	<i>p</i> -value
(a) Average Acceleration Rate Model.				
Constant	0.121	0.011	11.106	< 0.0001
(1/ <i>R</i>)	44.563	1.7	26.211	< 0.0001
<i>I</i> _{TF}	0.046	0.011	4.14	< 0.0001
<i>Z</i> _{<i>p</i>}	0.149	0.006	24.468	< 0.0001
Model diagnostics: Adjusted <i>R</i> ² =0.834, Degrees of freedom=265				
Root mean square of residual= 0.084 m/s ²				
(b) Maximum Acceleration Rate Model.				
Constant	0.136	0.012	11.41	< 0.0001
1/ <i>R</i>	64.552	1.86	34.714	< 0.0001
<i>I</i> _{TF}	0.106	0.012	8.747	< 0.0001
<i>Z</i> _{<i>p</i>}	0.14	0.011	12.52	< 0.0001
<i>Z</i> _{<i>p</i>} ·(1/ <i>R</i>)	8.499	2.144	3.965	< 0.0001
Model diagnostics: Adjusted <i>R</i> ² =0.886, Degrees of freedom=265

Figure 7.10: Relation of the Observed and Model Predicted Values of a_{85} and a_{max_85} .

the same trend. It was also found that the inverse of the radius affects the variation in the acceleration rate, therefore, the coefficient of the term $[(1/R) \cdot Z_p]$ equals to 8.499. The model for a_{max_85} by substituting with Z_p equal to 1.036 will render:

$$a_{max_85} = 0.281 + \frac{73.357}{R} + 0.106I_{TF} \quad (7.12)$$

Table 7.5 shows the results of the regression analysis. The model diagnostics show a high coefficient of determination of 88.6% and a low root-mean-square error of 0.095 m/s². In addition, the values of the *t*-statistic for the coefficients indicate the significant contribution of each variable to the model. Therefore, it is concluded that this model provides a good estimate of the maximum acceleration rate when exiting a curve. In addition, Figure 7.10 shows the predicted 85th percentile maximum acceleration rates from Equation 7.8 compared to the observed 85th percentile values from the collected data.

7.4 Conclusions

This chapter showed the results of the analysis of deceleration and acceleration rates preceding and following horizontal curves based on driver behaviour. The analysis proved that drivers adopt different deceleration and acceleration rates based on the speed characteristics on the horizontal curve, which are affected by the geometric characteristics of the curve. This is different from what was assumed in most studies. In addition, the chapter presented a model describing the deceleration and acceleration rates for rural roads and ramps. The uniqueness of this model is that it is based on deceleration and acceleration rates measured under actual driving conditions. In addition it models the distribution of the accelerations and can be used to determine any chosen percentile.

In case of the average values of these rates, their main objective is tracing speed profiles for use in evaluation of design consistency. Therefore, the 85th percentile models were presented. Likewise, the objective of the maximum values of these rates is detecting the aggressive aspect of driving behaviour where high deceleration or acceleration rates are adopted in order to conform between desired driver's speed and the geometric characteristics of the curve.

Based on the results of this study it was found that the deceleration rate is significantly dependent on the inverse of the curve radius, and whether the tangent preceding the curve is independent or not. In addition, the acceleration rate depends on the inverse of the radius of the curve and whether the tangent following the curve is independent or not.

The developed models for average values of acceleration and deceleration rates can be used, in combination with the operating speed models, in drawing speed profiles for use in the evaluation of design consistency on two-lane rural roads. This is done given the 85th percentile speed on tangents and on curves. Then, using the deceleration and acceleration rates, the deceleration and acceleration lengths between the tangent and curve can be drawn, assuming that the speed change would occur on the tangents preceding and following the curve.

In addition, the developed models for maximum values of deceleration and acceleration rates can be used in the safety evaluation of horizontal curves. This will be presented in the following chapter. It should be noted here that even though the study used a relatively small number of sites, it has the unique advantage of utilizing measured deceleration and acceleration rates under actual driving conditions.

CHAPTER 8. RELATING DRIVER BEHAVIOUR TO SAFETY

PERFORMANCE ON HIGHWAY HORIZONTAL CURVES

In the preceding chapters of this thesis, several driver behaviour models were developed that are of potential value for the geometric design of horizontal curves in addition to assessing the quality and consistency of the horizontal alignments. These driver behaviour models proved to be statistically significant. However, it would be of great value to demonstrate that these parameters are in fact related to safety. Therefore, the objective of this chapter is to relate the developed driver behaviour models in previous chapters of this research to safety explicitly.

The chapter is organized into five sections. First, the database used in the analysis and the analysis approach are described. Second, an overview of the modelling technique used is given with a brief discussion of the different goodness of fit measures. Third, each driver behaviour parameter is analyzed and related to safety explicitly. Fourth, a sensitivity analysis is presented to demonstrate the validity of the safety prediction models developed in this chapter. Finally, a qualitative comparison between the developed safety models and two available safety models is performed to validate and evaluate the ability of the developed models in identifying collision prone sections (Said et al. 2008).

8.1 Modelling Database

The evaluation of the interrelationship between the modelled driver behaviour parameters and safety was performed using 70 horizontal curves on rural two-lane highways available in previous research at Carleton University (Awatta 2003, Awatta et

al. 2006) and not belonging to the test route of the experiment. These curves were used for two main reasons. First, the number of rural two-lane horizontal curves available in the test route is not large enough to develop a statistical safety model. Second, to avoid bias between the developed models since the curves belonging to the test route were used to develop the driver behaviour models.

Therefore, data for eight two-lane rural highway sections were provided by a previous research at Carleton University (Awatta 2003, Awatta et al. 2006). Details of the highways used in the analysis are shown in Table 8.1. This research was concerned with the quantitative evaluation of highway safety performance based on design consistency. The models developed from this study were demonstrated previously in Chapter 2, Table 2.7.

Table 8.1: Summary of Selected Highways.

Highway	Highway classification	Design classification	Segment length (km)	Number of selected curves
43	Secondary Highway (Former King's Hwy)	Unavailable	17.2	7
41	King's Highway	RC80	26.5	16
34	King's Highway	RC100	9.2	0
31	Secondary Highway (Former King's Hwy)	Not provided	33.5	10
17	King's Highway	RAU100	54.5	8
15	King's Highway	RAU100	16.0	10
12	County Road	Not Provided	31.4	9
7	King's Highway	Not Provided	13.5	10
Total curves				70

8.1.1. Geometric Alignment Data

The curve data obtained were categorized into geometric alignment data, traffic volume data, and collision history. The geometric alignment data included the radius and curve length, which were provided by the Eastern Region of the Ontario Ministry of Transportation for the King's Highways and by the Roads Department of the United Counties of Stormont, Dundas, and Glengarry for Secondary Highways and County Roads.

The traffic volume data were supplied by MTO's Traffic Volume Information System for King's Highway and Secondary Highways and the Municipal Road Appraisal Sheets for County Road 12. The descriptive statistics of the characteristics of the seventy curves chosen for analysis are shown in Table 8.2. All curve data are shown in Appendix F. The curves were chosen such that they satisfy certain criteria including low traffic volumes with $AADT \leq 10,000$ vehicles/day, curve radii less than 1,200 m and curve lengths greater than 50 m, no controlled intersections within 800 m of the curves, no features creating abnormal hazards, and marked and paved roadways with constant lane widths. As shown in Table 8.2, the curves are on roads that encompass a variety of highway classifications, design speeds, terrain, traffic volume, and collision history. The table also shows that the sample covers a wide range of curve radii from 317 m to 1117 m.

Table 8.2: Descriptive Statistics of the Characteristics of the Study Curves.

Curve Characteristics	Minimum	Maximum	Range	Mean	Standard Deviation
R (m)	317.52	1166.50	848.98	664.16	235.98
L_c (m)	53.02	1001.89	948.87	314.09	222.98
V_D (km/h)	80.00	100.00	20.00	88.00	9.87
V_{85} (km/h)	95.12	103.61	8.49	98.58	2.36
ΔV_{85} (km/h)	8.46	16.69	8.22	11.19	2.06
$AADT$ (veh/day)	1420.00	10670.00	9250.00	5162.43	2899.96
Collision Frequency	0	5	5	0.55	0.96

8.1.2. Collision Data

The collision data were obtained for the years 1997 to 1999 from the Accident Database for the King's Highways provided by the Highway Keypoints for Accident Location Identification and from the police collision reports for the Secondary Highways and County Road 12. The collision data available included data on the collision severity including property damage only (PDO), Injury (I), and Fatal (F). From the different combinations of severity levels, Awatta et al. (2006) found the most statistically significant models were based on all collision severity types (PDO+I+F).

In addition, the data were divided into three groups depending on the collision types. The first group included all collision types; the second group included all non-intersection related collisions; and the third group excluded collisions related to driver asleep, animal on the roadway, parked, turning, or passing vehicle, mechanical defect, and bicycle, motorcycle, or truck vehicles. From the three possible types of collisions, the group that yielded the most statistically significant models was based on non-intersection related collisions (Awatta 2003, Awatta et al. 2006). Therefore, this research uses the same collision group.

8.2 Modelling Technique

8.2.1. Overview of Available Modelling Techniques

Figure 8.1 shows the frequency distribution of collisions on the studied curves. The distribution clearly shows that the collisions are positively skewed with most of the sections experiencing zero collisions (around 60%). These two attributes are two of the main characteristics of collision events, which are discrete, rare, non-negative, and sporadic events.

Figure 8.1: Collision Frequency Distribution.

Collision prediction models have initially been developed using the conventional linear approach as demonstrated previously in Chapter 2 (Lamm et al. 1999). However, it was shown in many research works that assuming a linear relationship between the collision frequency and the explanatory variables is incorrect for the following reasons (Jovanis and Chang 1986, Hauer et al. 1988, Miao and Lum 1993):

1. Using linear regression assumes a continuous normal distribution. Collisions, on the other hand, are discrete, sporadic events.
2. Linear regression models assume a constant variance for the residuals, which is not the case for collision data. Collision data may have unequal variances at different predictor levels.
3. Linear regression models may result in negative collision frequency which is not the case for collisions which are positive events.
4. In addition, collisions are positively skewed with a greater proportion of the events occurring at the lower frequency side. This is because collisions are rare and many sections may experience none or very few collisions.

Therefore, such types of data are best modelled using regression models devoted for count data such as Poisson and negative binomial (NB) regression models. The Poisson regression model allows for describing a Poisson distributed response variable using a number of explanatory variables. For example, in case of this research, the number of collisions predicted (Y_i) at a certain curve (i) would be assumed to be a random variable with a mean (μ_i), where μ_i is a function of a number of explanatory variables (x_i) such that:

$$\mu_i = \exp(\beta_0 + \beta_1 x_1 + \beta_2 x_2 + \dots + \beta_n x_n) \quad (8.1)$$

Where β s are the coefficient of the explanatory variables that are determined through regression analysis. By definition, the number of collisions at a certain curve has the probability function:

$$P(Y_i = y_i) = \frac{\mu_i^{y_i} e^{-\mu_i}}{y_i!} \quad (8.2)$$

Where $P(Y_i = y_i)$ is the probability that y_i collision would occur at a certain curve i .

In Poisson models, an assumption is made for simplicity that the mean and variance equal to the parameter μ_i . Therefore,

$$Var(Y_i) = E(Y_i) = \mu_i \quad (8.3)$$

The nature of collisions, on the other hand, has been found to have variances exceeding the mean, a case called overdispersion. Therefore, a better descriptive model would be the negative binomial (NB) distribution, which is an extension of the Poisson where the variance is a quadratic function of the mean. Therefore,

$$E(Y_i) = \mu_i, \quad Var(Y_i) = \mu_i + k\mu_i^2 \quad (8.4)$$

Where k is the dispersion parameter. When k equals to zero the NB distribution yields the Poisson distribution again.

Miau and Lum (1993) explained that the reasons behind the overdispersion nature of collisions include the following:

1. The difficulty to include all the possible factors affecting collision occurrences. There are many factors affecting collision events among which are human factors, which are variable and difficult to quantify. Therefore, excluded or missing variables would result in overdispersion.
2. Exposure data and traffic variables such as the AADT or percentage of trucks are collected through a highway sampling system and therefore are subjected to sampling and nonsampling errors. Therefore uncertainties could arise from the collected data. An example of such errors includes under reporting of collisions as minor collision of property damage less than a certain value may

not be reported. Another example is the location of the collision which is determined by the police officials and may not be accurate. For short sections, such as curves, the collision location may be even more susceptible to error and collision may occur at one section and end at another.

3. The roadway environment, including the weather and lighting conditions, is non-homogeneous. Therefore, the traffic conditions during a collision are not homogeneous on each road section.

Therefore, the NB regression is used in this study. There are many forms for a safety performance model, the most suitable for collision experience being the exponential form shown in Equation 8.5:

$$Y_i = \exp(\beta_0 + \beta_1 x_1 + \beta_2 x_2 + \dots + \beta_n x_n) = \exp(\beta_0) \exp(x_1)^{\beta_1} \exp(x_2)^{\beta_2} \dots \exp(x_n)^{\beta_n} \quad (8.5)$$

Where Y_i is the expected collisions on a certain curve, x_1, x_2, x_n are the descriptive parameters, and $\beta_0, \beta_1, \beta_2, \beta_n$ are the intercept and model coefficients.

The exponential term is what makes this model suitable for collisions as it results in positive expected collisions. The coefficients β_i of the NB model are obtained by maximizing the log likelihood function $L(\beta, k)$.

$$L(\beta, k) = \sum \left[\sum_{j=0}^{y_i} (\log(1 + k_j)) - \log(1 + k y_i) + y_i \log \mu_i - \left(y_i + \frac{1}{k} \right) \log(1 + k \mu_i) - \log(y_i!) \right] \quad (8.6)$$

The maximum log likelihood estimates can be obtained using the generalized linear model (GLM) approach available in several statistical computer packages. For this study,

the SAS software is used, and the maximum log likelihood estimates are calculated through the PROC GENMOD. This procedure calculates the regression model coefficients, standard errors, p -values, scaled deviance, and Pearson chi-square statistics.

8.2.2. Exposure Term

For any experiment, the number of achieved successes greatly depends on the number of performed trials or opportunities given (Pasupathy et al. 2000). In case of collision analysis, the opportunity given or available is expressed in terms of exposure to collision, or the traffic volume and travel length.

In previous research, the relationship between collision experience and exposure levels was assumed linear. However, it has been recently proven that the increase in the probability of collisions with increase in exposure level is not proportional (Pasupathy et al. 2000, Qin et al. 2004). Therefore, including the exposure level as a variable allows the model to determine the best relationship between the exposure and the collision frequency.

The form of the expressing exposure in the model can be by including the traffic volume and section length separately, where each variable has a different coefficient. The regression model would then determine the relationship of these variables to the collision frequency. Alternatively, the two terms could be combined together in one exposure term ($EXPO$), such that:

$$EXPO = N \times AADT \times 365 \times L \times 10^{-6} \quad (8.7)$$

Where

$EXPO$ is a single exposure term (million vehicle-km)

N = number of years;

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

L = section length (km), in case of this research it is the curve length (km).

Another form of expressing exposure would be taking the natural logarithm of $AADT$ and L . Successful relations have been found using this method by Anderson et al. (1999), Swalha and Sayed (2001), and Ng and Sayed (2004). An example of this approach is shown in Ng and Sayed's model (2001) in Equation 8.8.

$$\text{Collisions/ 5 years} = \exp(-2.338) L^{1.092} AADT^{0.4629} \exp[IC(0.022\Delta V_{85} - 1.189\Delta f_r)] \quad (8.8)$$

Where:

IC = dummy variable (0 for tangents and 1 for horizontal curves); and

L = section length in km.

Another approach is using the single exposure term ($EXPO$) without transformation. The single exposure term could also be used as a scale factor, where the coefficient of $EXPO$ is set to one, and the coefficients of the remaining variables are determined. This approach was used successfully by Harwood et al. (2000). A final approach is taking the natural logarithm of $EXPO$, which was successfully used by Pasupathy et al. (2001).

In case of this research, the different modelling methods were examined. However, the only approach giving significant results at 5% level of significance was using the untransformed single exposure term.

8.2.3. Assessing Goodness of Fit

There are several criteria used to assess the appropriateness or goodness of fit for NB models:

1. First, the estimated regression coefficient for each variable should be statistically significant. That is, the null hypothesis that the coefficient equals to zero is rejected at a certain level of significance, for example 5%. This is determined by the p -value for each coefficient which should be less than 0.05 in case of 5% level of significance.
2. Another important measure is that the model should be logical in terms of the signs and the values of the coefficients.
3. The scaled deviance (D^m) and Pearson chi-square (χ^2) statistics are common measures for testing overdispersion and goodness of fit. The D^m is an approximately Chi-square random variable with degrees of freedom (DF) equal to the number of observations (n) minus the number of predictor variables (p).

$$D^m = 2(L^f - L^m) \quad (8.9)$$

Where L^f is the log likelihood when the model gives a perfect fit and L^m is the log likelihood of the developed model. Pearson Chi-square statistic (χ^2) is an approximately Chi-square random variable.

$$\chi^2 = \frac{\sum_{i=1}^n (y_i - Y_i)^2}{Y_i} \quad (8.10)$$

The maximum allowable values of D^m and χ^2 are determined from tabulated values of the Chi-square distribution corresponding to the model's DF and are compared with the calculated values.

Furthermore, the mean value of the random Chi-square is equal to DF . Therefore, if the model fits the data well, the ratio of the deviance to DF should be close to one.

Usually a value less than 1.2 is accepted. Values greater than 1.2 indicate that the model does not fit well with overdispersion. Similarly, cases with ratios less than 0.8 would indicate underdispersion (Long 1997).

4. A final criteria considered for assessing goodness of fit is one suggested by Miaou (1996) based on the dispersion parameter (k):

$$R_k^2 = 1 - \frac{k}{k_{max}} \quad (8.11)$$

Where k_{max} is the dispersion parameter with a model containing only the constant term. This measure is simple in its calculation and has a value ranging from 0 up to 1 when $k=0$. The parameter, R_k^2 , measures how much of the variability in the true means can be explained by the model.

8.3 Application of Driver Behaviour Models to Studied Curves

As explained previously the driver behaviour models developed in the previous chapters were used to develop relationships with safety on horizontal curves and were applied to seventy curves of rural two-lane highways. Four parameters were applicable, as they were related to geometric characteristics and speed behaviour on horizontal curves:

1. The rate of change of steering angle at the entrance of horizontal curves (RSA),

$$RSA = 7.129 + 2947.282 / R \quad (5.7)$$

2. The difference in curvature between the vehicle path and the alignment path ($\Delta Curve$),

$$\Delta Curve = -0.00057 \ln(R) + 0.0047 \quad (5.5)$$

3. The 85th percentile of lateral acceleration on horizontal curves which describes the driver comfort threshold (a_{L85}),

$$a_{L85} = 0.7634 - 0.0688 I_R - 0.00628 V_{A_85} + 0.00615 \Delta V_{85} \quad (6.4)$$

4. The 85th percentile of maximum deceleration rate at curve entrance (d_{max_85}),

$$d_{max_85} = 1.082 + \frac{(360.406 I_T - 42.764)}{R} - 0.848 I_T \quad (7.8)$$

RSA and $\Delta Curve$ are related to the curve radius, a_{L85} is related to the operating speed on the preceding tangent and the speed reduction, and d_{max_85} is related to both the curve radius whether the curve is preceded by an independent tangent or not.

Each of the previously explained parameters evaluates a different aspect of design for horizontal curves. That is, the rate of change of steering angle evaluates the workload imposed on drivers at the entrance of horizontal curves, the difference in curvature evaluates the curve cutting behaviour in the horizontal curve itself, the 85th percentile of lateral acceleration evaluates the driver comfort threshold within the horizontal curve, and the 85th percentile maximum deceleration rate describes the horizontal curve induced deceleration rate at the curve entrance and would indicate an unexpected curve or inability to judge the curve sharpness. However, all the parameters are related to the geometric features of the horizontal curves either in terms of R or operating speeds. Therefore, it is expected that since the parameters are interrelated that it would not be possible to incorporate them in a single model. Consequently, the relationship between each driver behaviour parameter and safety was quantified in a separate model.

8.3.1. Analysis of Rate of Change in Steering Angle

As explained in Chapter 6, drivers receive information while driving on the road through their perception of the geometric characteristics ahead, and control their vehicle accordingly. Misconception of the alignment ahead or violating driver's expectations may lead to extra workload on the driver. One of the parameters that was found to describe driver workload well is the rate of change of steering angle (*RSA*) at the entrance of the curve. It was found in Chapter 6 that as the curve radius decreases, the *RSA* increases (Equation 5.7). It was also found that greater workloads are imposed at the turning point of the relationship which was found to be at radii less than 200 m.

The results of the NB regression analysis are summarized in Table 8.7. The relationship between the two variables is logical and in the direction expected. As the *RSA* increases, the probability of collision occurrence increases. *EXPO* and *RSA* have *p*-values of less than 0.0001 and 0.0006 respectively, and therefore are statistically significant at 5% level of significance. The parameters, D'''/DF and χ^2/DF , are equal to 0.91 and 1.11, which are greater than 0.8 and less than 1.2, indicating an acceptable level of dispersion. The scaled deviance and χ^2 values are equal to 61.0677 and 74.6271 respectively, and are less than the maximum allowable value of 87.11. Finally, the value of R^2_k is equal to 0.76. Therefore, the model shows that the *RSA* has a significant impact on safety. The previous analysis is performed similarly to the remaining driver behaviour parameters in the following subsections.

8.3.2. Analysis of Difference in Curvature

As explained earlier, the difference in curvature describes the difference between the curvature adopted by the vehicle and the actual curvature of the alignment. Increased

differences would imply higher discomfort to the driver due to higher driver comfort thresholds. Therefore, it is expected that as $\Delta Curve$ increases, the safety on the curve decreases, and therefore the collision frequency increases.

Table 8.3: Models Relating Driver Behaviour Parameters to Collision Experience.

Parameter	Estimate	p-value	D^m χ^2 (Test)	D^m/DF χ^2/DF R^2_k	k
(a) Rate of Change of Steering Angle (RSA): $Y = \exp(\beta_0 + \beta_1(EXPO) + \beta_2(RSA))$					
β_0	-4.8805	0.0006	61.0677	0.9115	
β_1	0.368	<0.0001	74.6271	1.1138	
β_2	0.2719	0.0074	87.11	0.760	0.2940
(b) Difference in Curvature ($\Delta Curve$): $Y = \exp(\beta_0 + \beta_1(EXPO) + \beta_2(\Delta Curve))$					
β_0	-4.4301	0.0003	62.4344	0.9319	
β_1	0.3922	<0.0001	75.7361	1.1304	
β_2	2725.517	0.0052	87.11	0.823	0.2164
(c) Driver Comfort Threshold (a_{L85}): $Y = \exp(\beta_0 + \beta_1(EXPO) + \beta_2(a_{L85}))$					
β_0	-3.9188	0.0023	60.43	0.9020	
β_1	0.3289	0.0003	76.49	1.1416	
β_2	16.4984	0.0291	87.11	0.668	0.4066
(d) Deceleration Rate for Curves (d_{85}): $Y = \exp(\beta_0 + \beta_1(EXPO) + \beta_2(d_{max\ 85}))$					
β_0	-3.6760	0.0003	60.71	0.9061	
β_1	0.3819	<0.0001	72.65	1.0844	
β_2	2.5963	0.0103	87.11	0.741	0.3168

The results of the NB regression analysis relating the three-year collision frequency to $\Delta Curve$ are summarized in Table 8.3. Similar to *RSA*, the results show that $\Delta Curve$ has a significant impact on safety. The relationship is also logical and is in the direction expected. Both *EXPO* and $\Delta Curve$ have *p*-values of less than 0.0001 and 0.0052, and therefore are statistically significant at 5% level of significance. The model dispersion

parameters indicate an acceptable level of dispersion. The scaled deviance and χ^2 values are less than the maximum allowable value of 87.11.

8.3.3. Analysis of 85th Percentile Lateral Acceleration

It was previously proven in Chapter 7 through the developed model between the 85th percentile lateral acceleration (a_{L85}) and the approach tangent speed and speed reduction that the values of side friction coefficients in the design guides are substantially lower than the driver comfort threshold. Therefore, the relationship between a_{L85} and safety is investigated in this section.

For the curves used in the analysis, the speed reduction was taken as the difference between the 85th percentile tangent speed and the 85th percentile curve speed. To determine the 85th percentile curve speed, the model developed by Nie and Hassan (2007) was used. This model was based on the same experiment where the research in hand is based on, and the same curves used in the analysis of the research in hand. The model for speed on the middle of the curve is shown in Equation 8.13.

$$V_{85_MC} = 108.357 - 0.097CCR \quad df=8 \quad R^2=0.860 \quad (8.12)$$

Where

V_{85_MC} = 85th percentile speed at the midpoint of the circular curve (km/h);

$$CCR = \text{Curvature change rate} = \frac{63,700(L_{s1} / 2R + L_c / R + L_{s2} / 2R)}{L_{s1} + L_c + L_{s2}};$$

L_c = Length of curve (m); and

L_{s1} and L_{s2} = Length of entry and exit spirals

Although the study by Nie and Hassan (2007) resulted in a model for speed on tangents, the model requires the length of tangent preceding the curve. However, the available database from Awatta et al. (2006) used in the safety analysis of this section does not include data on tangent lengths. Therefore, to determine the speed on tangent, Equation 8.12 is used and the value of CCR is substituted by zero. This gives a tangent speed of 108.357 km/h.

The three-year collision frequency was modelled as a function of a_{L85} and $EXPO$, resulting in the model shown in Table 8.3. The results show that the direction of the relationship is logical, where the probability of collision increases with increase in a_{L85} . The relationship is also significant at 5% level of significance. The model indicates an acceptable level of dispersion and goodness of fit.

8.3.4. Analysis of 85th Percentile Maximum Deceleration Rate

The final driver behaviour parameter analyzed is the 85th percentile of maximum deceleration rate (d_{max_85}). The three-year collision experience was modelled as a function of d_{max_85} and $EXPO$. The results in Table 8.3 show that there is a logical and significant relationship between the collision experience and d_{max_85} . The coefficients of the descriptive variables are also significant at 5% level of significance. The model parameters indicate an acceptable level of dispersion and goodness of fit.

8.3.5. Overall Safety Model based on Different Driver Behaviour Parameters

Constructing an overall safety model containing the contribution of each driver behaviour parameter to the safety of the horizontal curves would have been of great benefit. However, since most of the driver behaviour parameters are related to the radius of the horizontal curve, multicollinearity was a major contributor in not achieving an

overall safety model. The correlation matrix, shown in Table 8.4, shows that the correlation coefficients for all driver behaviour parameters were greater than 0.79, indicating a significant degree of correlation. The high correlation between the driver behaviour parameters resulted in a statistically insignificant regression model as shown in Table 8.5.

Table 8.4: Pearson's Correlation Coefficient Matrix.

Variable	Collision Frequency	a_{L85}	$\Delta Curve$	$d_{max\ 85}$	RSA
Collision Frequency	1.000	0.139	0.123	0.107	0.138
a_{L85}	0.139	1.000	0.924	0.782	0.952
$\Delta Curve$	0.123	0.924	1.000	0.852	0.988
$d_{max\ 85}$	0.107	0.782	0.852	1.000	0.848
RSA	0.138	0.952	0.988	0.848	1.000

Table 8.5: Overall Safety Model Relating Driver Behaviour Parameters to Collision Experience.

Parameter	Estimate	p-value	D^m	D^m/DF	k
			χ^2	χ^2/DF	
			(Test)	R^2_k	
$Y = \exp(\beta_0 + \beta_1(EXPO) + \beta_2(RSA) + \beta_3(\Delta Curve) + \beta_4(a_{L85}) + \beta_5(d_{85}))$					
β_0	-4.1854	0.0481	62.4604	0.9759	
β_1	0.4503	<0.0001	71.4766	1.1168	
β_2	0.2720	0.98411	83.68	0.873	0.1551
β_3	4384.1120	0.5252			
β_4	-31.5315	0.2790			
β_5	0.6450	0.7596			

The table shows that the model is not logical in terms of the values of coefficients of $\Delta Curve$, a_{L85} , and d_{85} , where the values are much different than the coefficients resulting from the separate models and from the expected values. In addition, the sign of the coefficient of a_{L85} is also not logical, where it is negative. Finally, the coefficients of

all variables except for the coefficient of *EXPO* are not statistically significant at 5% level of significance.

8.4 Sensitivity Analysis

Collision frequency models have been used in many previous studies to investigate the individual contribution of each descriptive variable to safety using a sensitivity analysis. However, Swalha and Sayed (2001) argued that using sensitivity analysis to estimate the safety effect of the descriptive variables is a pitfall. The safety impact of one variable cannot be estimated by changing the value of this variable while keeping the others constant as correlation among the variables may exist. They also argued that even if there was no correlation between the variables, there might be correlations between the variables and other factors not included in the prediction model.

For the research in hand, it is not possible or reasonable to separate the effect of each of the driver behaviour parameters from the others as they are interrelated. For example, increasing the curve radius by a certain value would affect all driving behaviour parameters simultaneously. Therefore, each parameter would have a contribution to safety to a certain extent, and the contribution of each parameter individually would be very difficult to recognize. Sensitivity analysis however could help compare the sensitivity of the collision frequency to each driver behaviour parameter, which is the main variable in the collision prediction models at different exposure levels.

The collision reduction factor (*CRF*) is also another means of investigating the driver behaviour parameters that best describe their relationship with collision experience on horizontal curves. The collision reduction factor in this research corresponds to the percentage reduction in collisions due to 10 metres increase in radius. The radius is

chosen as a reference since most of the driver behaviour parameters derived in this research are related to radius except for the lateral acceleration. Therefore, for a unit of 10 metres increase in radius, the change in each of the driver behaviour parameters (δ) will be calculated, followed by the *CRF*. The collision reduction factor is calculated as follows:

$$CRF_j = \frac{\Delta Y}{Y} \times 100 = 100 \left[\frac{\exp[a_i(x_i + \delta_j)]}{\exp(a_i x_i)} - 1 \right] = 100 [\exp(a_i \delta_j) - 1] \quad (8.13)$$

This equation implies that for each driver behaviour parameter (j), when changing the curve radius by 10 metres, the driver behaviour parameter decreases by (δ), and the collision frequency (Y) changes by CRF_j . The value of the factor (δ) for each driver behaviour parameter was calculated through finding a linear relationship between each parameter and the radius. Therefore, the values of factor (δ) are the slope of the linear relationships. Hence, the values of CRF_j were calculated and shown in Table 8.6.

Table 8.6: Values of δ and *CRF* Factors for Each Driver Behaviour Parameter.

Driver Behaviour Parameter	δ	<i>CRF</i> (%)
<i>RSA</i>	-0.07535	-2.03
$\Delta Curve$	-8.727×10^{-7}	-2.35
a_{L85}	-8.958×10^{-5}	-1.47
d_{85}	-7.168×10^{-4}	-1.85

The table shows that the differences in the values of CRF_j for the different driver behaviour parameters are not significantly large. Results of the sensitivity analyses are shown in Figure 8.2 through Figure 8.5. All sensitivity analyses show that the change in collision frequency with each corresponding driver behaviour parameter is gradual and

realistic. There are no sudden increases or decreases in the collision prediction or unexpected values. Therefore, it can be concluded that all models provide logical and realistic relations between each driver behaviour parameter and collision frequency.

From the values of CRF_j shown in Table 8.6, the variation in collision frequency calculated by the values of CRF_j produced similar results to the figures. All driver behaviour parameters produced similar results. Therefore, based on these results, it could be concluded that all driver behaviour parameters could be used for safety evaluation and could be used to assess the quality and consistency of the design of horizontal curves.

Figure 8.2: Sensitivity of Safety to RSA.

Figure 8.3: Sensitivity of Safety to $\Delta Curve$.

Figure 8.4: Sensitivity of Safety to a_{L85} .

Figure 8.5: Sensitivity of Safety to d_{85} .

8.5 Validation of Developed Safety Models

8.5.1. Data Used

The objective of this section is first to validate the four developed safety models by comparing the results with two other available safety models. The second objective is to compare between the effectiveness of the models developed in this chapter and other available models to identify collision prone curves or inconsistencies on horizontal curves based on driver behaviour. The four safety models are compared with the Interactive Highway Safety Design Model's (IHSDM) safety model (Harwood et al. 2000) as well as the collision prediction model by Ng and Sayed (2004). These two models are specifically selected since the IHSDM consistency model predicts collisions based on the geometric characteristics of the curve, in terms of R , the length of curve (L_c), and the presence of spiral curves. Ng and Sayed's model predicts collisions based

explicitly on design consistency measures, namely the speed reduction on curves (ΔV_{85}) and the difference in side friction assumed and side friction demand (Δf_r).

To enable the comparison, a fictitious alignment similar to that developed by Hassan et al. (2001) that is designed with deliberate inconsistencies is used for the evaluation. The alignment's geometric characteristics are shown in Table 8.7. Some of the characteristics are changed from the original research by Hassan et al. (2001) in order to remain within the range of radii used for developing the models, or to test certain aspects in the alignment. In addition to the characteristics shown in Table 8.7, the alignment is assumed to have a constant lane width, maximum superelevation rate and is free of intersections. The *AADT* is assumed to be 10,000 veh/d. The alignment has a design speed of 100 km/h except for the final curve which has a design speed of 90 km/h. The approach speed to the horizontal curve is taken as 108.357 km/h as explained previously. The alignment has the following characteristics:

- All curves are preceded by independent tangents except for C2 and C3.
- C2 and C3 are reverse curves with equal radii separated by a short tangent to test the effect of successive curves.
- C4 is a compound curve composed of curves C4-1 and C4-2.
- C5 is longer than C1 with the same radius.
- C6 is long with a radius much greater than the minimum required value.
- C8 has the sharpest radius, and a design speed of 90 km/h.

Table 8.7: Geometric Characteristics of Fictitious Alignment Used (Hassan et al. 2001).

Element	L (m)	R (m)	L _s (m)
T1	464.1	-	-
C1	391.2	600	60
T2	177.4	-	-
C2	394.1	440	80
T3	75	-	-
C3	394.1	440	80
T4	292.7	-	-
C4-1	300	440	-
C4-2	500	660	-
T5	1000.2	-	-
C5	550	600	60
T6	350.3	-	-
C6	600	1000	-
T7	374	-	-
C7	234	440	80
T8	253.9	-	-
C8	128.2	330	60

As mentioned previously, the four safety models developed in this chapter are compared with the IHSDM safety model and Ng and Sayed's collision prediction model. The IHSDM model has the following form when applied on the fictitious alignment:

$$N_{rs} = N_{br} \times AMF_h \quad (8.14)$$

$$AMF_h = \frac{(1.55L_c + 80.2 / R - 0.012S)}{1.55L_c} \quad (8.15)$$

$$N_{br} = AADT \times 365 \times 10^{-6} \times L \times \exp(-0.4865) \quad (8.16)$$

Where:

N_{rs} = predicted number of collisions per year on a section;

N_{br} = predicted number of collisions for the base case;

AMF_h = the accident modification factor for horizontal curves;

L_c = the length of the horizontal curve in miles;

$S=1$ if a spiral curve is present and 0 if a spiral curve is not present; and

L = section length in miles.

Ng and Sayed's model was shown previously in Equation 8.8.

8.5.2. Evaluation

It is noteworthy to mention that the objective is to compare the predictive ability of safety for the developed models between successive sections rather than the value of collisions at a certain section in order to detect inconsistencies or collision prone sections. Therefore, a qualitative evaluation will be performed rather than using quantitative values of collisions at each section. Results of the comparison are shown in Figure 8.6 (a) for the four safety models developed in this research and in Figure 8.6 (b) for IHSDM and Ng and Sayed's safety models.

It is found from the figure that the predictive ability of the four safety models is almost similar and that all models show increase and decrease in collision for the same sections. This verifies that all four models could be used for detecting collision prone sections.

Next a comparison between the four developed models and IHSDM and Ng and Sayed's models is made. The two figures show that the trend in collision frequency is almost similar, although there are some differences. All models show an increase in the collision frequency between C1 and C2. This is because C2 has a sharper radius than C1. However, the developed models are more sensitive to the reduction in radius than the IHSDM model. All models predict a constant collision experience for C2 and C3 except for the safety model based on a_{L85} and Ng and Sayed's model. This is because C2 and C3

have equal radii, however they are separated by a short tangent. Therefore, the speed reduction on C2 is greater than C3, causing C3 to have a lower collision experience. The reduction in collision experience shows that the alignment follows the driver's expectancy as C3 has a similar curvature to C2.

(a) Based on Developed Models

(b) Based on IHSDM and Ng and Sayed (2004)

Figure 8.6: Evaluation of Safety Performance based on (a) Developed Models and (b) IHSDM and Ng and Sayed (2004).

There is a difference between the prediction models from C3 to C4-1. The two curves have equal radii, however C4-1 is shorter in length and has no spiral curves. In

addition, the speed reduction on C4-1 is greater as it is preceded by an independent tangent. Therefore, it is expected that the safety model based on a_{L85} and Ng and Sayed's model would predict an increase in collision experience. The IHSDM model also predicted a minor increase due to the absence of spiral curves for C4-1. The remaining driver behaviour safety models predicted a slight decrease in collisions due to the shorter length of C4-1, causing the exposure term to be less. Therefore, there could be a debate on which model is more reliable. However, the differences in the models are very minor compared to the other curves, concluding that C4-1 does not pose as a curve prone to collisions.

All models agree as expected on the reduction of collision experience between C4-1 and C4-2 except for the safety model based on d_{85} , due to the fact that C4-2 is a curve with a larger radius than C4-1, and the speed reduction on C4-2 is less than C4-1. Therefore, the curve does not violate the expectations of drivers. The safety model based on d_{85} predicts an increase due to the longer curve length of C4-2 than C4-1, and due to the sensitivity of the model to the exposure term. From C4-2 to C5 all models predict an increase in collision experience except for the IHSDM model. C5 has a moderate curvature and follows a long tangent, which can cause a relative violation in driver's expectancy. This is reflected in the speed reduction, causing the collision potential to increase. In addition the length of C5 is greater than C4-2, and the radius of C5 is less than C4-2, increasing the probability of collision experience. Therefore, it could be concluded that the IHSDM model is not as sensitive to speed reduction or to the exposure levels.

Moreover, although all models show a reduction in collision potential between C5 and C6, which has a radius much higher than the minimum required, the sensitivity of this reduction is least for the IHSDM model. In addition, all models show an expected rise in the collision potential from C6 to C7 except for the safety model based on a_{L85} since C7 has a sharper curve than C6, however C7 is shorter causing much less *EXPO* term. Finally, all models show an increase in collision potential from C7 to C8 since C8 has the sharpest curve on the alignment ($R=330$ m), and a different design speed. Therefore, this curve represents a violation to the driver's expectancy, and a great increase in the collision experience is expected. It is worth noting that although all models show this increase, Ng and Sayed's model shows the least increase from C7 to C8 compared to the other models. This confirms that this model is less sensitive to the geometric characteristics of the curves in terms of the radius, and more sensitive to speed reduction.

8.5.3. Results of Evaluation

The evaluation of the fictitious alignment using the six safety prediction models resulted in similar trends of detecting curves that are inconsistent and need evaluation. Therefore, the evaluation confirmed the validity of the four developed safety models in detecting collision prone sections. It also proved that a_{L85} was successful in detecting inconsistencies due to speed reductions on curves. Therefore, it is recommended that all four driver behaviour parameters be used simultaneously when evaluating road safety.

In addition, the safety evaluation proved that the developed models are more sensitive to the geometric characteristics of curves in terms of R and L_c than Ng and Sayed's model. This was conveyed in the transition from C1 to C2. The evaluation also

showed the over sensitivity of Ng and Sayed's model to ΔV_{85} on the expense of considering the geometric characteristics of the curve as portrayed at C2, C6 and C8. The *IHSDM* model failed to detect the inconsistency at C5 due to lack of sensitivity to speed reduction which is a drawback in this model. This agrees with results of a similar evaluation performed by Ng and Sayed (2004).

Therefore, from the evaluation it could be concluded that although all models showed similar trends, the developed safety models were successful in detecting inconsistencies and collision prone section relying on both geometric alignment characteristics and design consistency in terms of speed reduction. This is opposed to using *IHSDM*'s single model that relies only on geometric characteristics and Ng and Sayed's model which relies on design consistency without considering geometric characteristics.

8.5.4. Conclusions

Four NB regression models were developed in this chapter. The advantage of these models is to illustrate the nature of the relationship of each of the driver behaviour parameters to safety. All models were found to be statistically significant and sensitive to safety. This is a major finding as it implies that this research did in fact find potential measures of driver behaviour that should be considered when designing a safe and consistent horizontal alignment, which is the main objective of this research.

Moreover, to validate and evaluate the ability of the developed models in identifying collision prone sections a qualitative comparison was performed with two available safety models. Although the comparison was made based on a fictitious alignment it showed that the developed models when used together could identify

collision prone sections based on both geometric characteristics and design consistency. From the evaluation it could be concluded that although all models showed similar trends, the developed safety models were successful in detecting inconsistencies and collision prone section relying on both geometric alignment characteristics and design consistency in terms of speed reduction. This is opposed to using IHSDM's single model that relies only on geometric characteristics and Ng and Sayed's model which relies on design consistency without considering geometric characteristics.

Finally, it should be stressed that the accuracy of the developed models is greatly dependent on the range of the independent variables. Therefore, it is recommended to expand the database used to develop these models using a wider range of radii.

CHAPTER 9. CONCLUSIONS AND RECOMMENDATIONS

9.1 Summary

This research focused on updating the geometric design of horizontal curves with the road user in mind. In doing so, the current geometric design of horizontal curves was reviewed and recommendations were proposed in order to provide a design that accommodates driver behaviour in a quantitative manner. The following were covered in the research:

- The research started by a comprehensive literature review of driving behaviour including a detailed description of the relationship between geometric design and human factors as reported in previous research. The review covered the driver characteristics, including the physiological and mental characteristics. In addition, the North American geometric design guidelines current practices in designing horizontal curves were reviewed. The elements of concern were the design of spiral curves and the choice of driver comfort thresholds for determining the side friction demand. In addition, the different approaches used in evaluating geometric design consistency that relate to driving behaviour, and the models describing the relation between different design consistency measures and collision experience were reviewed and presented.
- A detailed description of the driver behaviour data collection experiment was then presented. The description covered the different pieces of equipment used to collect driver behaviour data, the test route traversed, the volunteers' recruitment, and the methodology of synchronization of the measurements collected from the different

equipment. Following the data collection, the actual geometric characteristics of the curves were identified to facilitate the analysis of the interrelationship between driver behaviour and the geometric characteristics of horizontal curves.

- The research presented an analysis of vehicle path on horizontal curves in terms of the curvature adopted by the drivers compared to the actual alignment curvature. In addition, the rate of change of steering position on horizontal curves was analyzed for different road classifications. Using the steering behaviour on horizontal curves, it was possible to model the desirable spiral length at entrance and exit of horizontal curves, which better conforms to driver behaviour. The models were developed for both two-lane highways and freeways. Desirable spiral lengths were found to correlate well with the radius of curvature for two-lane highways with high coefficient of determination. In addition, the desirable spiral lengths were compared with the different controls of spiral length found in the North American design guides. It was found that the minimum criteria found in North American guides need revision to better describe actual driver behaviour. Furthermore, recommended values for rate of change of lateral acceleration and spiral parameters were given.
- Furthermore, an investigation of the relationship between the driver comfort thresholds used in the design of horizontal curves and both the speed characteristics adopted by the driver and the horizontal curve elements was performed. A model was developed relating the side friction demanded by the driver with the radius of the curve, the 85th percentile approach speed, and the speed difference between the approach tangent and the curve. It was found that the minimum criteria found in the North American design guides need revision to better describe actual driver

behaviour. Furthermore, recommended values for design speeds and side friction demand were given.

- Moreover, four models were developed to relate the deceleration and acceleration rates before and after the horizontal curves on two-lane rural highways to the geometric characteristics of the curves. The models were developed using the variation modelling approach. For each of acceleration and deceleration rate, a model was developed for average values and another for maximum values. The models showed that there is a significant relationship between the deceleration rate when entering the curve and the inverse of the curve radius. In addition, there is a significant relation between the acceleration rate when exiting the curve and the inverse of the curve radius.
- Finally, the driver behaviour parameters developed in this research were related to safety explicitly. In doing so, a negative binomial regression analysis was performed to find the relationship of each of RSA , $\Delta Curve$, a_{L85} , and d_8 , to safety. Then, a sensitivity analysis was performed to demonstrate the validity of the safety prediction models. Finally, a qualitative comparison between the developed safety models and two available safety models, namely the IHSDM consistency model and Ng and Sayed's model, was performed to validate and evaluate the ability of the developed models in identifying collision prone sections. It was found that the developed safety models were successful in detecting inconsistencies and collision prone sections relying on both geometric alignment characteristics and design consistency in terms of speed reduction.

9.2 Contributions

The following are the main contributions of this research:

- First, a procedure for studying driver behaviour in terms of vehicle path and steering path was presented. Through a thorough investigation, it was found that there is a difference between the actual alignment, and the alignment that drivers adopt. A relationship was developed between the difference in curvature between the vehicle path and actual alignment and the inverse in curve radius. This relationship was further used to develop a safety model.
- In addition, the analysis of steering behaviour showed that it is recommended that the steering behaviour of drivers be used to find desirable spiral lengths that conform to driving behaviour. In doing so, a newly applied variation model was used for finding desirable spiral lengths on horizontal curves. Based on the results of this study, the spiral lengths follow a cubic relationship with the same trend as the comfort control in the current North American design guides. It was found that when comparing the desirable spiral lengths with the current control values in the design guides, the desirable lengths were greater than minimum values, which raised safety issues. In addition, the current practice in design guides is to take a constant value for the rate of change of lateral acceleration (c). However, through this study, it was found that c depends on the radius of the curve, with maximum values found at smaller radii. The maximum value of c was 0.75 m/sec³. Recommendations for corrections to the comfort control values and spiral parameters were made to the design guides in order to follow the 85th percentile desirable spiral length.

- Third, analysis of driver comfort thresholds on horizontal curves proved that drivers experience higher values of lateral acceleration when negotiating right turns than left turns. Two control tests were also presented that proved the validity of these results. This is an important result as it is reported in very few previous studies, however it has not been sufficiently proven or justified. In addition, the developed model describing driver comfort thresholds on horizontal curves for rural roads and ramps modelled the distribution of lateral acceleration and can be used to determine any chosen percentile of side friction demanded. Based on the results of this study it was found that curve operating speeds as inferred from the developed model were higher than the design speed for curves of radius 350 m or less. It was pointed out that this difference would result in two potentially unfavourable conditions. First, since $f_{s,d}$ is substantially less than the driver comfort threshold, the actual operating speed of the drivers inferred by the comfort threshold model would be greater than the design speed, especially for sharp curves. With the significant improvements in vehicle technology, drivers may not feel great discomfort when negotiating horizontal curves with higher speeds than what they were designed for. This is a major concern, as it would lead to inconsistency in the design of horizontal curves. The second concern is that the lower margin of safety between the comfort threshold and the sliding threshold may lead to a safety hazard of sliding on horizontal curves.
- The developed model could be used, in combination with the point-mass formula, in determining the minimum horizontal curve radius for a specific speed environment, which would replace the approach speed in the model formulation. In addition, the model could be used in evaluating design consistency of horizontal curves. For a

specific radius and superelevation rate, an inferred curve operating speed could be predicted and compared to the design speed. In addition, the inferred speeds of a series of horizontal curves could be predicted using this model and hence a check could be made for uniformity of speeds to ensure design consistency.

- Furthermore, Chapter 7 proved that drivers adopt different deceleration and acceleration rates based on the geometric characteristics of the curve. This is different from what was assumed in most studies. In addition, the model developed describing the deceleration and acceleration rates for rural roads and ramps modelled the distribution of the accelerations and can be used to determine any chosen percentile. The developed models could be used, in combination with the operating speed models, in drawing speed profiles for use in the evaluation of design consistency on two-lane rural roads. Moreover, models describing maximum deceleration and acceleration behaviour were developed, that would model the aggressive aspect of driving behaviour where high deceleration or acceleration rates are adopted in order to conform between desired driver's speed and the geometric characteristics of the curve
- Finally, Chapter 8 presented four models relating the driver behaviour parameters developed in this research to safety. All models were found to be statistically significant and sensitive to safety. This is a major finding as it implies that this research did in fact find potential measures of driver behaviour that should be considered when designing a safe and consistent horizontal alignment, which is the main objective of this research. All driver behaviour parameters were considered well correlated to safety and could be considered when designing horizontal curves. From

the evaluation it was concluded that the developed safety models were successful in detecting inconsistencies and collision prone section relying on both geometric alignment characteristics and design consistency in terms of speed reduction.

Finally, it is important to note that the importance of this research as outlined in the introduction of this thesis lies in its economic and social impacts. The economic impacts of updating the geometric design guides to design with the driver in mind are invaluable. The research in hand proved the strong relationship between the driver behaviour parameters described in this thesis and safety. Reducing collisions on roads has significant economic and social impacts. Economic impacts can be realized in terms of reduced property loss, large savings in terms of medical expenditure and insurance claims, and significant reduction in lost working days due to injury or loss of earning power due to death. On the other hand, the social benefits of improving highway safety are in terms of reduced fatalities and injuries and are even more significant.

9.3 Recommendations for Future Research

Based on the findings of this research, the following can be recommended:

- This research was successful in its methodology for collecting driver behaviour data. Therefore, it is recommended that this methodology be used with a larger driver sample along with a larger sample of horizontal curves. In doing so, the models developed in this research could be refined with a wider range of radii, and a larger sample of drivers.
- This study was based on a minivan passenger car as a test vehicle. Further analysis with different types of passenger cars and heavy trucks would be valuable in covering a wider range of vehicles on the road.

- This research accounted for only the horizontal alignment and its impact on driver behaviour, mainly since the test route traversed level terrain. Further research studying the combined effect of horizontal and vertical alignments would be valuable.
- The methodology used for collecting driver behaviour data could be applied to analyze driver behaviour on both compound and reverse curves. In doing so, a large sample would be required.
- The methodology could be used to study driver behaviour under different traffic conditions, such as peak-hour versus off-peak hour conditions, day-time versus night-time conditions. It could also be used to study other driver behaviour parameters such as those related to stopping and passing sight distances. It could also be used to study driver behaviour at different types of traffic controls at intersections.
- Finally, quantitative knowledge and understanding of driver behaviour using a computer-based interactive driving simulator could be achieved using the results of this study. Using a driving simulator to continue this research will lead to a more complete understanding of driver behaviour and subsequently updating Canadian geometric design guides. No doubt, research pertaining to driver behaviour is considered of greater validity when performed under actual driving conditions using instrumented vehicles. On the other hand, using simulators allows researchers to design experiments with high-risk scenarios such as a series of sharp curves, which are hard to apply on real roads. In addition, almost any kind of road environment could be simulated, including 3D alignments.

REFERENCES

- Alexander A. and H. Lunenfeld. 1986. *Driver Expectancy in Highway Design and Traffic Operations*. U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.
- American Association of State Highway and Transportation Officials. 2004. *A Policy on Geometric Design of Highways and Streets*. Washington, D.C.
- Anderson, I., M. Bauer, D. Harwood, K. Fitzpatrick. 1999. Relationship to Safety of Geometric Design Consistency Measures for Rural Two-Lane Highways. *Transportation Research Record 1658*, TRB, National Research Council, Washington, D.C., pp. 43-51.
- Awatta, M. 2003. Highway Design Consistency and Safety: Individual and Overall Evaluation Criteria. *M.A.Sc. Thesis, Carleton University, Ottawa*.
- Awatta, M., Hassan, Y., and Sayed, T. 2006. Quantitative Evaluation of Highway Safety Performance Based on Design Consistency. *International Journal of Advances in Transportation Studies*. Volume IX: pp. 29-44.
- Barnett, J. 1936. Safe Side Friction Factors and Superelevation Design. *Highway Research Board*, Vol. 16, pp. 69-80.
- Bellis, M. 2004. How the Wheels Got Turning, A Historical Perspective on American Roads. <http://inventors.about.com/library/inventors/blcar3.htm> (Accessed March 5, 2006)
- Bonneson, J. 1999. Side Friction and Speed as Controls for Horizontal Curve Design. *Journal of Transportation Engineering*, ASCE, Vol. 125 (3), pp. 473-480.

- Cafiso, S., A. Di Graziano, G. La Cava, R., Heger, R., Lamm. 2004. In Field Data Collection for Driving Behaviour Analysis using the DIVAS Instrumented Car. *XIV International Congress SIIV*, Italy.
- Cafiso, S., A. Di Graziano, and G. La Cava. Actual Driving Data Analysis for Design Consistency Evaluation. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1912, TRB, National Research Council, Washington, D.C., 2005, pp. 19–30.
- Cooper, Casey 2002. History of the US Highway System, From Dirt Paths to Super Highways. <http://gbcnet.com> (Accessed March 2, 2006)
- Dimitropoulos, I. and G. Kanellaidis. 1995. Highway Geometric Design: The Issue of Driving Behaviour Variability. *International Symposium on Highway Geometric Design Practices*.
- Drader, M. 1996. The Evolution of Transportation in the Northwest. *Fort St. John, The Energetic City*. [http:// collections.ic.gc.ca/north_peace/transport/index.html](http://collections.ic.gc.ca/north_peace/transport/index.html) (Accessed March 5, 2006)
- Easa, S. 2003. Improved Speed- Profile Model for Two-Lane Rural Highways. *Canadian Journal of Civil Engineering*, Vol. 30 (6), pp. 1055-1065.
- Easa, S. and E. Dabbour. 2003. Design Radius Requirements for Simple Horizontal Curves on Three-Dimensional Alignments. *Canadian Journal of Civil Engineering*. Vol. 30 (6), pp. 1022-1033.
- Easa, S. and W., He. 2006. Modeling Driver Visual Demand on Three-Dimensional Highway Alignments. *Journal of Transportation Engineering*, ASCE, Vol. 132 (5), pp. 357-365.

- Farber E. and V. Gallagher. 1972. Attentional Demand as a Measure of the Influence of Visibility Conditions on Driving Task Difficulty. *Highway Research Record 414*, pp. 1-5.
- Flipe, E. and Navin, F. 1998. Automobiles on Horizontal Curves, Experiments and Observations. *Transportation Research Record 1628*: 50-56.
- Fitzpatrick, K., and J.M. Collins. 2000. Speed-Profile Model for Two-Lane Rural Highways. *Transportation Research Record 1737*, Transportation Research Board, Washington D.C., pp. 42-49.
- Fitzpatrick K., I. Anderson, K. Bauer, J. Collins, L. Elefteriadou, P. Green, D. Harwood, N. Irizarry, R. Koppa, R. Krammes, J. McFadden, K. Parma, K. Passetti, B. Poggioli, O. Tsimhoni, M. Wooldridge, 2000a. *Evaluation of Design Consistency Methods for Two-lane Rural Highways, Executive Summary*. FHWA-RD-99-173.
- Fitzpatrick K., I. Anderson, K. Bauer, J. Collins, L. Elefteriadou, P. Green, D. Harwood, N. Irizarry, R. Koppa, R. Krammes, J. McFadden, K. Parma, K. Passetti, B. Poggioli, O. Tsimhoni, M. Wooldridge. 2000b. *Alternative Design Consistency Rating Methods for Two-lane Rural Highways*. FHWA-RD-99-172.
- Fitzpatrick, K. and Wooldridge, M. 2001. Recent Geometric Design Research for Improved Safety and Operations. *The National Academy Press*, Washington D.C., NCHRP Synthesis 299. (Cited in Easa 2003).
- Gartner, W. and Murphy, M. 1976. Pilot Workload and Fatigue: A Critical Survey of Concepts and Assessment Techniques. Washington, D.C.: NASA TN-D-8365. (cited in Hicks and Wierwille 1979)

- Glennon J. and D. Harwood. 1978. Highway Design Consistency and Systematic Design Related to Highway Safety. *Transportation Research Record 681*, TRB, National Research Council, Washington, D.C., pp. 77-88.
- Glennon, J.G. and G.D. Weaver. *The relationship of vehicle paths to highway curve design*. (TTI Report 134-5). 1971. Texas A&M University, Texas Transportation Institute, College Station, Texas. (cited in Tom 1995).
- Gopher, D. and R. Braune. 1984. On the Psychophysics of Workload: Why Bother with Subjective Measures? *Human Factors*, Vol. 26 (5), pp. 519- 532.
- Hassan, Y. 2004. Highway Design Consistency - Refining the State of Knowledge and Practice. *Transportation Research Record 1881*, TRB, National Research Council, Washington, D.C., pp. 63-71.
- Heger, F. 1995. Driving Behaviour and Driver Mental Workload as Criteria of Highway Geometric Design Quality. *First International Symposium on Highway Geometric Design*, Germany, Mainz, 1995.
- Hicks T. and Wierwille W. 1979. Comparison of Five Mental Workload Assessment Procedures in a Moving-Base Driving Simulator. *Human Factors*, Vol. 21(2), pp. 129-143.
- Imran, M. 2005. Development of GPS-Based Procedure for Tracking Vehicle Path. *M.A.Sc. Thesis, Carleton University, Ottawa*.
- Imran , M., Hassan, Y., Patterson, D. 2006. GPS-GIS-Based Procedure for Tracking Vehicle Path on Horizontal Alignments. *Computer-Aided Civil and Infrastructure Engineering*, Vol. 21, pp. 383-394.

- Johnston, I.R. 1983. The effects of roadway delineation on curve negotiation by both sober and drinking drivers. *Australian Road Research Board*, ARR No 128. Victoria. Australia. (cited in Tom 1995).
- Kanellaidis, G. 1996. Human Factors in Highway Geometric Design. *Journal of Transportation Engineering*, ASCE, Vol. 122 (1), pp. 59-66.
- Knowles, W. 1963. Operator Loading Tasks. *Human Factors*, April 1963, pp. 155-161.
- Krammes, R. and S. Glascock. 1992. Geometric Inconsistencies and Accident Experience on Two-Lane Rural Highways. *Transportation Research Record* 1356, TRB, National Research Council, Washington D.C., pp. 1-10.
- Krammes, R., R. Brackett, M. Shafer, J. Ottesen, I. Anderson, K. Fink, K. Collins, O. Pendleton, and C. Messer. 1995. *Horizontal Alignment Design Consistency for Rural Two-Lane Highways*. FHWA-RD-94-034. Federal Highway Administration, Washington, D.C.
- Lamm, R., A. Beck, S. Cafiso, G. La Cava. 2003. A New Procedure for Evaluating Traffic Safety on Two-Lane Rural Roads. *Proceedings of the XXIIInd PIARC World Road Congress*. Durban, South Africa.
- Lamm, R., B. Psarianos, and T. Mailaender. 1999. *Highway Design and Traffic Safety Engineering Handbook*. McGraw-Hill, New York, NY.
- Lamm, R., Choueiri, E., Hayward, J., and Paluri, A. 1988. Possible Design Procedures to Promote Design Consistency in Highway Geometric Design of Two-Lane Rural Roads. *Transportation Research Record* 1195, Transportation Research Board, Washington D.C., pp. 111-122.

- Lawless, J. 1987. Negative Binomial and Mixed Poisson Regression. *Canadian Journal of Statistics*. Vol. 15(3).
- Leica Geosystems 2002. GPS System 500, Technical Reference Manual, Version 4.0. *Leica Geosystems Ag*. Heerbrugg, Switzerland.
- Leisch J. E. and J.P. Leisch. 1977. New Concepts in Design-Speed Application. *Transportation Research Record 631*, pp. 5-14.
- Long, J. S. 1997. *Regression Models for Categorical and Limited Dependent Variables*. Thousand Oaks, CA: Sage Publications.
- McFadden, J., Elefteriadou, L. 2000. Evaluating Horizontal Alignment Design Consistency of Two-Lane Rural Highways: Development of New Procedure. *Transportation Research Record 1737*, TRB, National Research Council, Washington D.C., pp. 9-17.
- Medina, A., and A. Tarko. 2005. Speed Factors on Two-Lane Rural Highways in Free-Flow Conditions. *Transportation Research Record: Journal of the Transportation Research Board*, No. 1912, TRB, National Research Council, Washington, D.C., pp. 39-46.
- Messer, C.J. 1980. Methodology for Evaluating Geometric Design Consistency. *Transportation Research Record 757*, Transportation Research Board, Washington, DC, pp. 7-14.
- Misaghi, P. 2003. Modelling Operating Speed and Speed Differential for Design Consistency Evaluation, *M.A.Sc. Thesis, Carleton University, Ottawa*.

- Misaghi, P. and Y. Hassan. 2005. Modeling Operating Speeds and Speed Differential on Two-Lane Rural Roads. *Journal of Transportation Engineering*, Vol. 131(6), pp. 408-418.
- Morall, J. and Talarico, R. 1994. Side Friction Demanded and Margins of Safety on Horizontal Curves. Transportation Research Record 1435, Transportation Research Board, Washington, D.C., pp. 145-152.
- Morgenstern, B., D. Mahoney, and B. Warady. 2002. Estimating Total Body Weight in Children on the Basis of Height and Weight: A Reevaluation of the Formulas of Mellits and Cheek. *Journal of the American Society of Nephrology*, Vol. 13, pp. 1884-1888. (cited in Medina and Tarko (2005))
- Morrell, C., J. Pearson, L. Brant, and S. Gordon-Salant. 1997. Construction of Hearing Percentiles in Women with Non-Constant Variance from the Linear Mixed-Effects Model. *Statistics in Medicine*, Vol. 16, pp. 2475-2488. (cited in Medina and Tarko (2005))
- Ng, J. and T. Sayed. 2003. Quantifying the Relationship Between Geometric Design Consistency and Road Safety. Transportation Research Board Annual Meeting, January 2003, Paper No. 03-2363.
- Otteson J. and Krammes R. 2000. Speed-Profile Model for a Design Consistency Evaluation Procedure in the United States. *Transportation Research Record 1701*, Transportation Research Board, Washington D.C., pp. 76-85.
- Perco, P. 2006. Desirable length of Spiral Curves for Two-Lane Rural Highways. *Transportation Research Record*, 1961, Transportation Research Board, Washington, D.C., 1-8,

- Perco, P. 2005. Comparison between Vehicle Paths Along Transition Sections with and without Spiral Curves, *3rd International Symposium on Highway Geometric Design*, June 29-July 1, 2005.
- Polus, A., Fitzpatrick, K., and Fambro, D. 2000. Predicting Operating Speeds on Tangent Sections of Two-Lane Rural Highways. *Transportation Research Record 1737*, Transportation Research Board, Washington D.C., pp. 50-57.
- Road Safety in Canada –2003. 2006. Road Safety and Motor Vehicle Regulation Directorate. Accessed on February 5, 2008. <http://www.tc.gc.ca/roadsafety/tp/tp13951/2003/pdf/tp13951%20EN-S.pdf>
- Said, D., Y. Hassan, and A.O. Abd El Halim. 2006. Methodology for Analysing Vehicle Trajectory and Relation to Geometric Design of Highways. *Advances in Transportation Studies – An International Journal*, Issue X.
- Said, D., Hassan, Y., and Abd El Halim, A. 2007. Quantification and Utilization of Driver Path in Improving the Design of Highway Horizontal Curves. 86th Annual Meeting, Transportation Research Board, January 21, 2007, Paper No. 07-2037.
- Said, D., Hassan, Y., and Abd El Halim, A. 2008. Driver Comfort Thresholds for Horizontal Curve Design. Submitted to the *Canadian Journal of Civil Engineering*, Paper No. 08140.
- Said, D., Abd El Halim, A., and Hassan, Y., 2008. Relating Driver Behaviour to Safety Performance on Highway Horizontal Curves. Paper presented at the *7th Transportation Specialty Conference*, CSCE Annual Conference, June 10 – 18, Paper No. TR-656.

- Said, D., Abd El Halim, A. and Hassan, Y. 2009. Desirable Spiral Length Based on Driver Steering Behavior. Paper submitted for publication in *Transportation Research Record: Journal of the Transportation Research Board* and presentation in the 88th Annual Meeting, TRB, National Research Council, Washington, D.C., January 2009.
- Senders et al. 1967. The Attentional Demand of Automobile Driving. *Highway Research Record*, Vol. 195, pp. 15-33, 1967.
- Shinar, D., E.D. McDowell, E.D., and J.H. Rockwell. 1974. "Improving driver performance on curves in rural highways through perceptual changes." (Report EES 423). The Ohio State University, Department of Industrial Engineering, Columbia, Ohio. (cited in Tom (1995).
- Stewart, D. and C. Chudworth. 1990. A remedy for accidents at bends. *Traffic Engineering and Control*. Vol. 31, No. 2, pp 88 –93.
- Syed, L. and Y. Hassan. 2005. "Role of Uneven Vehicle Weigh Distribution in Curve Driving." *2005 Canadian Society of Civil Engineers Conference Proceedings*. TR-162.
- The Math Forum Drexel University. 2000. "Finding the Center of a Circle Given 3 Points." <<http://mathforum.org/library/drmath/view/55239.html>> (January 18, 2005)
- Tom, G. Accidents on Spiral Transition Curves. *ITE Journal*. pp. 49-53. September 1995.
- Transport Canada. 2004. "Transportation in Canada Annual Report."
<http://mathforum.org/library/drmath/view/55239.html> <http://Tc.gc.ca>. (Accessed March 2, 2006)

- Transportation Association of Canada. 1999. *Geometric Design Guide for Canadian Roads*. Ottawa, Ontario.
- Wierwille W. and F.T. Eggemeier. 1993. Recommendations for Mental Workload Measurement in a Test and Evaluation Environment. *Human Factors*, Vol. 35 (2), pp. 263-281.
- Wierwille, W. and J. Gutmann. 1978. Comparison of Primary and Secondary Task Measures as a Function of Simulated Vehicle Dynamics and Driving Conditions. *Human Factors*, Vol. 20, pp. 233- 244, 1978.
- Wikipedia contributors. 2006. Automobile. *Wikipedia, The Free Encyclopedia*.
<http://en.wikipedia.org/w/index.php?title=Automobile&oldid=42058068>.
(Accessed on March 3, 2006)
- Wildervanck, C., G. Mulder, and J. Michon. 1978. Mapping Mental Load in Car Driving. *Ergonomics*. Vol. 21, No. 3, pp. 225-229.
- Wooldridge, M.D. 1994. Design Consistency and Driver Error. *Transportation Research Record 1445*, Transportation Research Board, Washington, DC, pp. 148-155.
- Yerkes, R. and J. Dodson. 1908. The Relation of Strength of Stimulus to Rapidity of Habit-Formation. *Journal of Comparative Neurology of Psychology*, Vol. 18, pp. 450-482 (cited in Wooldridge 1994).
- Zeitlin L. 1995. Estimates of Driver Mental Workload: A Long-Term Field Trial of Two Subsidiary Tasks. *Human Factors* Vol. 37 (3), pp. 611-621.

Appendix A

Driver Consent Form

Informed Consent Form

I _____, agree to participate in the research study program titled "Evaluation of Driver Behavior and its Effect on Highway Design and Safety." I clearly declare that I have accepted the invitation to participate in this particular research study and according to the following:

Supervisor of the research:

Dr. Yasser Hassan
Phone: 520 – 2600 ext. 8625
e-mail: yasser_hassan@carleton.ca

Prof. A. O. Abd El Halim, Chair
Phone: 520 –2600 ext. 5789
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Student of the research: Dalia Said

Phone: 520-2600 ext 7471
Email: dsaid@connect.carleton.ca

Ethics Committee Chair: Prof. Antonio Gualtieri

Carleton University Research Ethics Committee
Phone: 520-2517
E-Mail: ethics@carleton.ca

This research study is not a commercial project or for the sake of profit. It is a research study part of a Ph.D. program the Department of Civil and Environmental Engineering. The aim of this research study is to improve highway safety by reducing or preventing crashes on highways by studying human behavior while driving and the workload imposed on him.

I will be involved in the following:

- Filling out a questionnaire to mark the answer that represents every participant. Names of (or personal information that can uniquely identify) the participants are not required. Questions to gain the information of the followings: education, profession, gender, age, usage of corrective eye, driving experience, driving education, driving frequency on rural highways will be in the questionnaire. Participants may decline answering any question.
- Driving the instrumented vehicle as I would drive my own on a highway. There are no known and/or reasonably anticipated risks, discomforts, and/or inconveniences (physical, psychological, emotional, economic, and social) from participating in this research other than normally experienced risks of daily driving.
- I will not be asked to perform specific maneuver and I will perform the normal tasks and maneuvers of daily driving I usually perform.
- The data collected are the longitudinal and lateral values of the driving speed and acceleration while driving on the road. The data will be collected automatically and recorded on the data acquisition card.
- I will neither be taped (audio or video) nor photographed (conventional or digital).
- The amount of the compensation is honorarium and covers only the cost of parking at Carleton University if the volunteer has to pay during the experiment time. The experiment time is estimated to be 3 hours.

I understand that the data from the research study will be stored in a locked filing cabinet at the office of the supervisor of research at Carleton University. Only the student and the supervisor of research will have access to the data. At the end of the research, this data will be part of the thesis of the researcher's Ph.D. degree and will be kept at the Carleton University library.

I understand that my participation is voluntary. I have the right to withdraw at any time or to refuse to answer any questions. In case of withdraw I will / will not give (circle one) the researcher permission to use the data I have provided to that point.

I have the right to contact the researcher and the supervisor of this project at any time with questions that I may have before starting, during, or after the research study.

I may contact the Ethics Committee with concerns or complaints.

Finally, I understand that the nature of the collected data does not allow the researcher to prove that I am committing an illegal act, as law enforcement and speed controls are not used during the experiment. Therefore, I am not subjected to any risk affecting my security or confidentiality because of participating in the experiment. However, I will not purposely break the basic rules of the road as I should obey rules and limits on the road as usual driving.

Participant:

Date:

Signature:

Supervisor: Prof. A. O. Abd El Halim

Date:

Signature:

Supervisor: Dr. Yasser Hassan

Date:

Signature:

Student: Dalia Said

Date:

Signature:

Appendix B

85th Percentile Curvature Profiles on Study Curves

Figure B- 1: Curve 3c4 and Curve 3c5

Figure B- 2: Curve 3c6 and Curve 3c7

Figure B- 3: Curve 3c8 and Curve 3c9 **Figure B- 4: Curve 3c10**

Figure B- 5: Curve 3c11

Figure B- 6: Curve 6c1

Figure B- 7: Curve 12c1

Figure B- 8: Curve 12c2 and Curve 12c3

Figure B- 9: Curve 12c4 and Curve 12c5

Figure B- 10: Curve 7c1

Figure B- 11: Curve 73c1

Figure B- 12: Curve 73c2

Figure B- 13: Curve 73c3

Figure B- 14: Curve 73c4

Figure B- 15: Curve 73c5 and Curve 73c6

Figure B- 16: Curve 73c7 and Curve 73c8

Figure B- 17: Curve 73c9

Figure B- 18: Curve 416c1

Figure B- 19: Curve 416c2

Figure B- 20: Curve 416c3

Figure B- 21: Curve 416c4

Figure B- 22: Curve 417c1

Figure B- 23: Curve 417c2

Figure B- 24: Curve 417c3

Figure B- 25: Curve 417c4

Figure B- 26: Curve 417c5, Curve 417c6, Curve 417c7

Figure B- 27: Curve 417c8

Figure B- 28: Curve 417c9

Figure B- 29: Curve 417c10

Figure B- 30: Curve 417c11

Appendix C

Steering Profiles for All Drivers on Study Curves

Figure C- 1: Curve 3c4 and Curve 3c5

Figure C- 2: Curve 3c6 and Curve 3c7

Figure C- 3: Curve 3c8 and Curve 3c9

Figure C- 4: Curve 3c10

Figure C- 5: Curve 3c11

Figure C- 6: Curve 6c1

Figure C- 7: Curve 12c1

Figure C- 8: Curve 12c2 and Curve 12c3

Figure C- 9: Curve 12c4 and Curve 12c5

Figure C- 10: Curve 73c1

Figure C- 11: Curve 73c2

Figure C- 12: Curve 73c3

Figure C- 13: Curve 73c4

Figure C- 14: Curve 73c5 and Curve 73c6

Figure C- 15: Curve 73c7 and Curve 73c8

Figure C- 16: Curve 73c9

Figure C- 17: Curve 416c1

Figure C- 18: Curve 416c2

Figure C-19: Curve 416c3

Figure C-20: Curve 416c4

Figure C- 21: Curve 417c1

Figure C- 22: Curve 417c2

Figure C- 23: Curve 417c3

Figure C- 24: Curve 417c4

Figure C- 25: Curve 417c5, Curve 417c6, Curve 417c7

Figure C- 26: Curve 417c8

Figure C- 27: Curve 417c9

Figure C- 28: Curve 417c10

Figure C- 29: Curve 417c11

Figure C- 30: Curve 6-416 c1

Figure C- 31: Curve 416-12c1

Appendix D

Matlab Function for Finding Lateral Acceleration Values on Horizontal Curves

```

z_score = 1.96; AllMat = xlsread('trialavgmatlab');

Distance = AllMat(:,1); % First column is the distance on the curve

LatAcc = AllMat(:, 2:n); % Rest of the matrix is lateral acceleration

% Each column corresponds to one driver

n = n-1; % Reduce number of columns for lateral acceleration matrix

AvgW = nanmean(LatAcc); % Average for each driver with the outliers

StdW = nanstd(LatAcc); % Standard Deviation for each driver with the outliers

MeanMat = repmat(AvgW,m,1); % Matrix with all column elements having same value (average)

StdMat = repmat(StdW,m,1); % Matrix with all column elements having same value (standard
deviation)

outliers = ones(size(m,n));

while outliers ~= zeros(m,n)

outliers = abs(LatAcc - MeanMat) > z_score * StdMat;

LatAcc(outliers == 1) = NaN;

AvgN = nanmean(LatAcc); % Average for each driver without the outliers

StdN = nanstd(LatAcc); % Standard Deviation for each driver without the outliers

end

```

Appendix E

Longitudinal Acceleration, Throttle, and Brake Profiles for All Drivers on Rural Two-Lane Highways

Figure E- 1: Curve 3c4 and Curve 3c5

Figure E- 2: Curve 3c6 and Curve 3c7

Figure E- 3: Curve 3c8 and Curve 3c9

Figure E- 4: Curve 3c10

Figure E- 5: Curve 3c11

Figure E- 6: Curve 6c1

Figure E- 7: Curve 7c1

Figure E- 8: Curve 416-12c1 and Curve 416-12c2

Figure E- 9: Curve 417-7c1 and 417-7c2

Figure E- 10: Curve 6-416c1

Appendix F

Curve Data Used for Safety Analysis (Awatta 2003,
Awatta et al. 2006)

Curve#	Station# (PI)	Radius (m)	Length (m)	Spiral (m)	Average Traffic Volume (vpd)	CollFreq
1	126+90.93	388.00	159.79	60.96	1550	1
2	136+93.09	582.12	110.91	60.96	1550	2
3	205+35.50	873.19	430.66	45.72	1550	0
4	256+68.03	873.19	208.79	45.72	1550	0
5	336+30.65	806.00	541.13	45.72	1550	1
6	356+86.57	582.12	71.12	45.72	1550	0
7	382+00.02	436.59	100.14	45.72	1550	0
8	397+06.63	436.59	204.66	45.72	1550	0
9	11+915.302	388.08	314.227	60.96	1420	2
10	15+238.372	873.19	358.183	60.96	1420	0
11	16+570.919	388.08	194.959	60.96	1420	0
12	17+031.136	1164.25	226.905	60.96	1420	0
13	18+188.717	388.08	87.996	60.96	1420	0
14	20+886.703	436.59	53.023	60.96	3470	0
15	26+043.167	537.35	836.897	60.96	3470	1
16	27+621.685	635.05	1001.89	60.96	3470	1
17	27+042.694	1166.50	76.2	10670	10670	1
18	20+347.506	993.00	76.2	10670	10670	1
19	22+700.738	1000.00	76.2	7950	7950	1
20	19+396.383	876.00	76.2	7950	7950	3
21	15+700.826	880.00	76.20	7950	7950	1
22	13+200.664	1164.00	76.20	6450	6450	0
23	11+683.094	1160.00	76.2	6450	6450	1
24	10+312.542	866.00	76.2	6450	6450	1
25	11+011.695	582.13	165.713	60.96	5970	0
26	16+364.029	873.19	83.337	60.96	5970	0
27	20+705.088	873.19	58.095	60.96	5970	0
28	11+050.986	873.19	281.365	60.96	5970	0
29	17+324.484	600.00	365.696	60.96	5170	0
30	17+882.942	600.00	227.085	60.96	5170	1
31	19+619.382	900.00	234.313	56	7600	0
32	20+565.516	900.00	365.024	56	7600	0
33	21+383.625	600.00	205.202	59	7600	0
34	22+501.418	600.00	155.946	60.96	7600	0
35	12+782.386	873.19	726.236	57.977	7230	1
36	10+484.786	873.19	501.097	60.96	7230	0
37	16+501.665	873.19	462.568	60.96	10010	1
38	15+805.846	698.55	340.603	60.96	10010	1
39	15+003.084	410.91	455.368	60.96	10010	5
40	14+304.34	873.19	116.459	60.96	10010	1
41	13+963.29	582.13	66.45	60.96	10010	0
42	13+063.321	582.13	75.447	60.96	10010	0

Curve#	Station# (PI)	Radius (m)	Length (m)	Spiral (m)	Average Traffic Volume (vpd)	CollFreq
43	11+687.846	582.13	199.221	60.96	10010	0
44	10+114.883	436.59	105.666	60.96	10010	3
45	115+82.16	873.19	873.19	604.65	5690	0
46	220+46.43	388.00	388.00	203.99	5690	0
47	237+45.38	388.00	388	240	5690	0
48	254+49.29	582.12	582.12	101.60	5690	0
49	262+51.60	582.12	582.12	110.91	5690	0
50	297+20.01	436.59	436.59	259.08	5690	0
51	313+91.55	436.59	436.59	263.65	5690	0
52	516+50.57	873.20	873.2	77.6	4820	0
53	537+48.61	873.20	873.2	76.6	4820	0
54	583+91.94	873.20	873.2	464.2	4820	0
55	621+53.91	873.20	873.2	474	4820	0
56	121+04.34	436.60	436.6	162.4	4820	0
57	12+572.902	580.00	580	170	4820	2
58	16+551.184	583.00	583	377.329	4200	0
59	17+978.435	583.00	583	783.641	4200	0
60	26+621.125	349.00	349	472.377	5500	2
61	27+731.038	388.00	388	527	5580	3
62	110+87.82	367.66	367.659	579.976	2170	0
63	232+09.23	388.08	388.083	124.855	2170	0
64	239+82.24	388.08	388.083	128.074	2170	0
65	38+51.72	698.55	698.55	352.96	2170	0
66	79+33.66	436.59	436.59	233.04	2170	0
67	239+30.10	698.55	698.55	121.92	2170	0
68	246+37.77	698.55	698.55	113.486	2170	0
69	286+90.81	317.52	317.52	443.34	2170	0
70	305+58.89	317.52	317.52	445.47	2170	1