FREEWAY MERGING BEHAVIOUR AND SAFETY OF ACCELERATION LANES: FIELD STUDY

by

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ABSTRACT

Appropriate geometry of the entrance terminal allows the vehicles entering into a freeway to merge into the mainline safely and comfortably, without interfering with the through traffic movement. Several aspects of traffic behaviour on speed-change lanes (SCL) and freeway right lane were examined using data from 23 merging sites on Highway 417 in Ottawa (Canada). Analysis of traffic behaviour has shown that the merging speed of entering vehicles is affected by geometrics of the entrance ramp on both upstream and downstream the gore. Lower merging speeds were shown to be associated with higher collisions on the SCL. Right lane traffic volumes and merging speed of entering vehicle were shown to affect the right lane speed along the SCL. Six statistically significant models were developed for the prediction of merging speed, acceleration on the SCL, effective length used for acceleration, freeway right lane speed, and safety performance of the SCL.

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LIST OF SYMBOLS

a_{85Max} = 85th percentile passenger car maximum acceleration rate, m/s^2

a_{85Mean} = 85th percentile passenger car mean acceleration rate, m/s^2

a_{85Over} = 85th percentile passenger car overall acceleration rate, m/s^2

AADT = Annual average daily traffic, veh/day

AASHTO = American Association of State Highway and Transportation Officials

$D_{85Merge}$ = 85th percentile passenger car merging distance, m

EnAADT = Entering *AADT*, veh/day

Expo = Total traffic exposure on section, million veh-km

ExpoAcc = Traffic exposure on the acceleration lane, million veh-km

HCM = Highway Capacity Manual

LACC = SCL length, km

L_{Extd} = Length of extended *SCL*, m

L_{Lim} = Length of limited length *SCL*, m

Q_{RL} = Right lane equivalent hourly traffic volume, pc/h/lane

Q_{SCL} = Equivalent hourly traffic volume on the SCL, pc/h/lane

R = Radius of controlling ramp curve, m

R^2 = Coefficient of determination

RampGr = Ramp Grade code

RampTyp = Ramp Type code

RL = Right lane

SCL = Speed-change lane

TAC = Transportation Association of Canada

$TCol$ = Total Collisions on the section along the SCL

$TColAcc$ = Total Collisions on the Acceleration lane

Typ = Code for the SCL Type

V_{85Gore} = 85th percentile passenger car speed at the gore (beginning of the SCL), km/h

$V_{85Merge}$= 85th percentile passenger car merging speed, km/h

V_{85RL} = 85th percentile passenger car right lane speed (downstream the gore), km/h

θ = Angle of convergence at physical nose, deg.

ΔV_{85} = Speed disparities between the V_{85RL} and $V_{85Merge}$, km/h

1.0 INTRODUCTION

1.1 BACKGROUND

The prime benefit of a roadway is the mobility provided by that road to its users. However, the benefit of mobility is subdued due to the cost associated with road traffic crashes, which impose an enormous public health burden globally. Over a million people die on World's roadways each year with over 3,000 each day, and several millions more are injured. In the US, 42,815 deaths and almost three million non-fatal injuries were recorded during 2002, attributable to road traffic crashes. This makes road traffic crashes the leading cause of injury-related death (Sleet et al. 2004). Traffic crashes claim more human year loses than any other incident or disease. The US crash costs for the year 2000 were estimated to be well above $200 billion. Driver behaviour, including choice of speed, roadway design, weather and other factors all play role in the occurrence of highway crashes (Kockelman et al. 2004).

Freeways are fully access controlled facilities that carry large volume of traffic at high speeds with high levels of safety and efficiency, and therefore, provide the highest level of mobility. The concept of controlled access facilities for movement of vehicular traffic at high-speed first came into being during the 1920s, with isolated grade separations. There has since been continuing evolution in geometric design of freeways and interchanges. However, as noted by Koepke (1993), many of the criteria for interchange design are based on data collected as far back as the 1940's and 1950's. In fact, those criteria are still reflected in the latest editions of design guides by AASHTO (2001,

2004) and TAC (1999). Because of improved vehicle performance and design, over the past several decades, the applicability of those data and research findings to today's drivers or vehicles and in turn to road design may be questioned. To maintain the concept of freeway, access to and exit from freeways are provided at selected intersecting roads through interchanges. According to the Transportation Association of Canada (TAC 1999), the fundamental principle of an interchange is the movement of vehicles through the interchanges in the safest, most efficient manner possible. Interchanges consist of a system of loops and ramps, with safe and efficient operation of freeway merge and diverge areas sensitive to the geometry of these elements.

Drivers' entrance to the freeways is facilitated through entrance ramps, for travelling to their destinations in the fastest way possible. Adequate lengths of the entrance terminals, adjoining the through roadway, should enable the entering drivers to accelerate to desired speed, find acceptable gaps in the mainstream, and merge into the freeway in a safe and comfortable manner. Accordingly, TAC (1999) stated that safety could be increased on acceleration lane with increased length.

Several research studies identified interchanges as the most crash prone area in terms of number and severity of collisions. For example, based on data from the US Fatality Analysis Reporting System (FARS) and General Estimates System (GES), McCartt et al. (2004) mentioned that an estimated 82,609 police reported crashes occurred at interchanges on interstate freeways in 2001, of which 24,996 crashes resulted in injuries and 544 were fatal. Overall, 18% of all interstate freeway crashes, 17% of injury crashes, and 11% of fatal crashes occurred at interchanges, although such locations constitute

less than an estimated 5% of total freeway mileage. The entrance or exit ramps experience the maximum proportion of collisions that occurs at interchange. Janson et al. (1998) mentioned that nationally (in US) 20 to 30% of freeway truck crashes occur on or near ramps, excluding an additional 10 to 15% that occur at intersection of ramps and surface streets.

Despite these observations, researchers are paying relatively little attention to safety performance of interchange ramps. An attempt is made in this research to evaluate the driver behaviour at freeway merging areas. Freeway to freeway entering or exiting, which is usually facilitated through high speed ramps, is different from that of freeway to urban arterials. Entrance to freeway from the urban arterials is usually associated with substantial change in speed by design definition of these two facilities. Speed disparities would still exist between the freeway and entering vehicles although ramps and speed-change lanes are provided to allow speed transitions. Therefore, in this study only the speed-change lanes that allow arterial to freeway transition were taken into consideration.

1.2 RESEARCH GOALS

This research represents a second phase of a comprehensive study into driver behaviour at freeway merge and diverge areas, and safety of these areas, following Sarhan (2004) who studied the safety performance using the existing data. The specific goals of this research were to examine the effect of length of acceleration lane on merging behaviour of entering vehicles. Availability of suitable equipment enabled the capturing of speed and traffic data covering both acceleration and freeway right lane. The research has

therefore been intended to include several aspects of traffic behaviours on speed-change lane as well as freeway right lane. With the availability of relevant data and based on pertinent literature review, the objectives of this thesis were then set as follows:

- ◊ To analyze the effect of length of acceleration lane on driver behaviour including merging speed of entering vehicles, acceleration behaviour on the speed-change lanes and the effective length used for merging into the through lane.
- ◊ To analyze the effect of entering vehicles on the operation of the freeway right-most through lane.
- ◊ To analyze the effect of presence of heavy vehicles on merging speed, acceleration behaviour, merging distance, and freeway right lane speed.
- ◊ To develop statistical models for the prediction of passenger car behaviour on the acceleration as well as freeway right-most lane, and collision frequency on freeway merge areas.

The findings of this research are expected to assist the highway professionals in selecting appropriate geometry of the entrance terminal at interchanges. This would in turn, assist in safe and efficient operation of the freeway merge areas. The findings and recommendations would also pave the way to other researchers as well as designers for further study on the merging and diverging areas of freeways.

1.3 SCOPE OF RESEARCH

With the limitations of time and cost as well as availability of geometric data, the scope of this research was limited to a portion of Highway 417 located within the City of

Ottawa, Ontario (Canada). The study area extends from Interchange (IC) 110 (intersection with Walkley Road) in the east to IC 145 (intersection with Highway 7) in the west. Practical considerations and emphasis on quality of data that are free from possible interference in driver behaviour, limited the scope of research to 23 entrance terminal sites at fifteen interchanges out of 66 entrance terminals available within the study area at 27 interchanges. These 23 entrance terminal sites vary with respect to geometry and/ or traffic they serve as well as land use patterns, and hence are expected to provide sufficient variability in explaining the variation in traffic behaviour on the speed-change lanes as well as through lane adjoining the speed-change lanes.

1.4 THESIS ORGANIZATION

An introduction to the research study is already presented in this chapter (Chapter 1.0). Chapter 2.0 provides the review of design guide criteria and available published literatures pertinent to the research goals of this study. The data collection and processing are presented in Chapter 3.0. Chapter 4.0 provides the analysis of traffic behaviour on acceleration and freeway right-most lanes. Regression modelling attempts are then presented in Chapter 5.0. Chapter 6.0 provides some application examples of the developed models. Finally, Chapter 7.0 provides a summary of the findings of this study and recommendations for future research.

2.0 LITERATURE REVIEW

This chapter provides review of design guides and available literature associated with the objectives of this research. Section 2.1 presents a review of design guides, which include Transportation Association of Canada (TAC 1999), the American Association of State Highway and Transportation Officials (AASHTO 2004), and the Highway Capacity Manual (HCM 2000). Section 2.2 presents a review of the available literature of research work pertaining to freeway speed-change lanes.

2.1 REVIEW OF DESIGN GUIDES

The main design guides used in North America were reviewed in regard to interchange design features and safety, with emphasis on the speed-change lanes. The following sections provide in brief the design features outlined in those geometric design guides.

2.1.1 Freeway Concept

As mentioned in TAC (1999), the road network, providing the movement of people and goods, is composed of different types of roads. Each type of road performs a particular service in providing vehicular movement between the points of origin and destination, and access to property. Freeways, expressways, and arterials provide mainly through traffic movements while local roads and public lanes are used almost exclusively for access purposes. Collectors provide both services. A comparison of the service functions of traffic movement and property access is shown graphically in Figure 2.1.

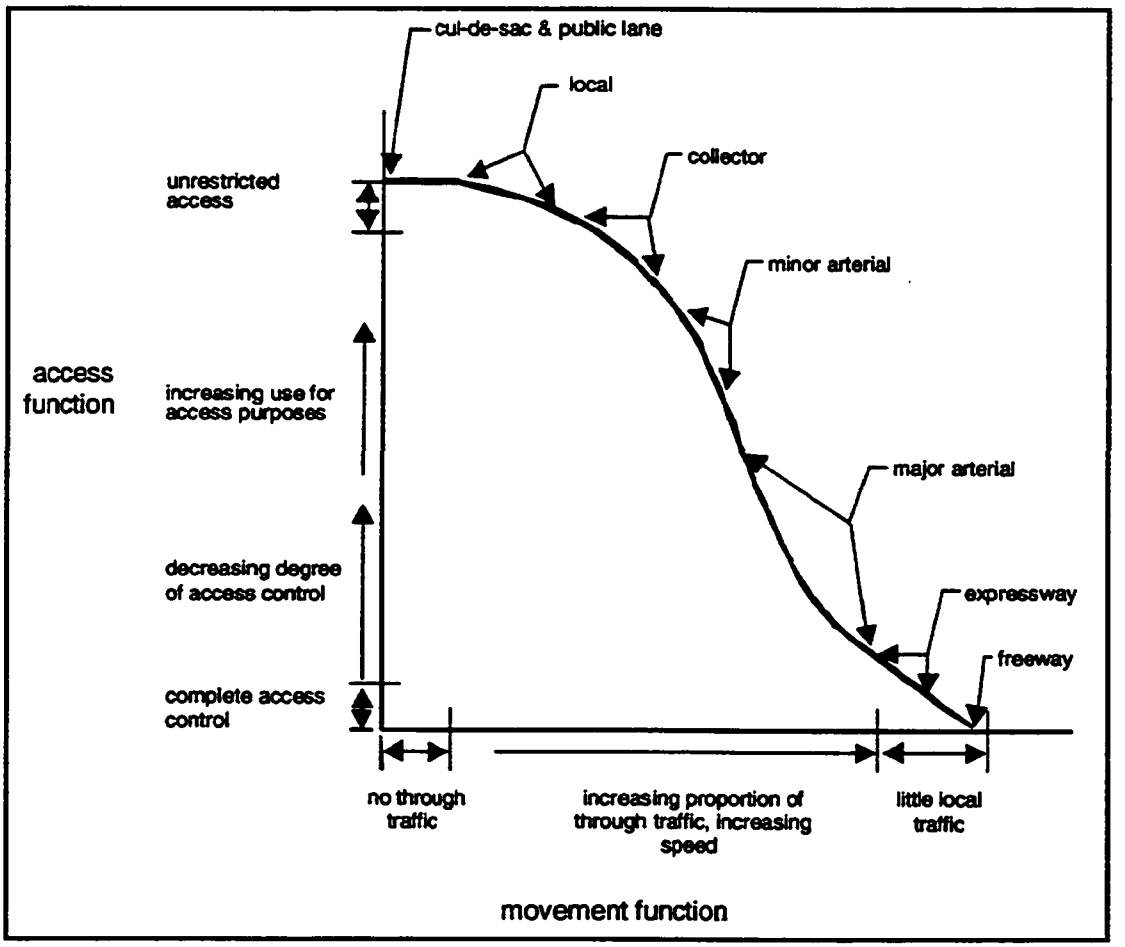


Figure 2.1: Service Function of Different Types of Road (TAC 1999)

As defined in AASHTO (2004), freeways are highways with full control of access. They are intended to provide movement of large volume of traffic at high speeds with high level of safety and efficiency. Urban freeways usually carry higher traffic volumes with four to sixteen through-traffic lanes in the two directions. However, their design is sometimes constrained due to the limited right-of-way. The basic concepts of rural freeways are similar to the urban ground-level freeways, but usually associated with fewer through lanes due to lower traffic volumes. In addition, design of alignment and cross section elements of rural freeways are more generous due to availability of

sufficient right-of-way at lower cost, and usually associated with higher design speeds. As the freeways are fully access controlled facilities, preference is given to through traffic by providing access connections with selected intersecting roads only.

2.1.2 Interchange Configurations and Design Features

TAC (1999) defined an interchange as a system of interconnecting roadways in conjunction with one or more grade separations that provide movement of traffic between roads on different levels. Interchanges usually allow all movements between intersecting highways. Interchanges that allow a limited number of movements between the intersecting roads are known as partial interchanges.

As mentioned in AASHTO (2004), the interchange configurations vary from a single ramp connecting local streets to some complex and comprehensive layouts involving two or more highways. The basic interchange configurations as provided in AASHTO (2004) are shown in Figure 2.2. Any one of the basic configurations shown in the figure can vary extensively in shape and scope, and there are numerous possible combinations of interchange configurations.

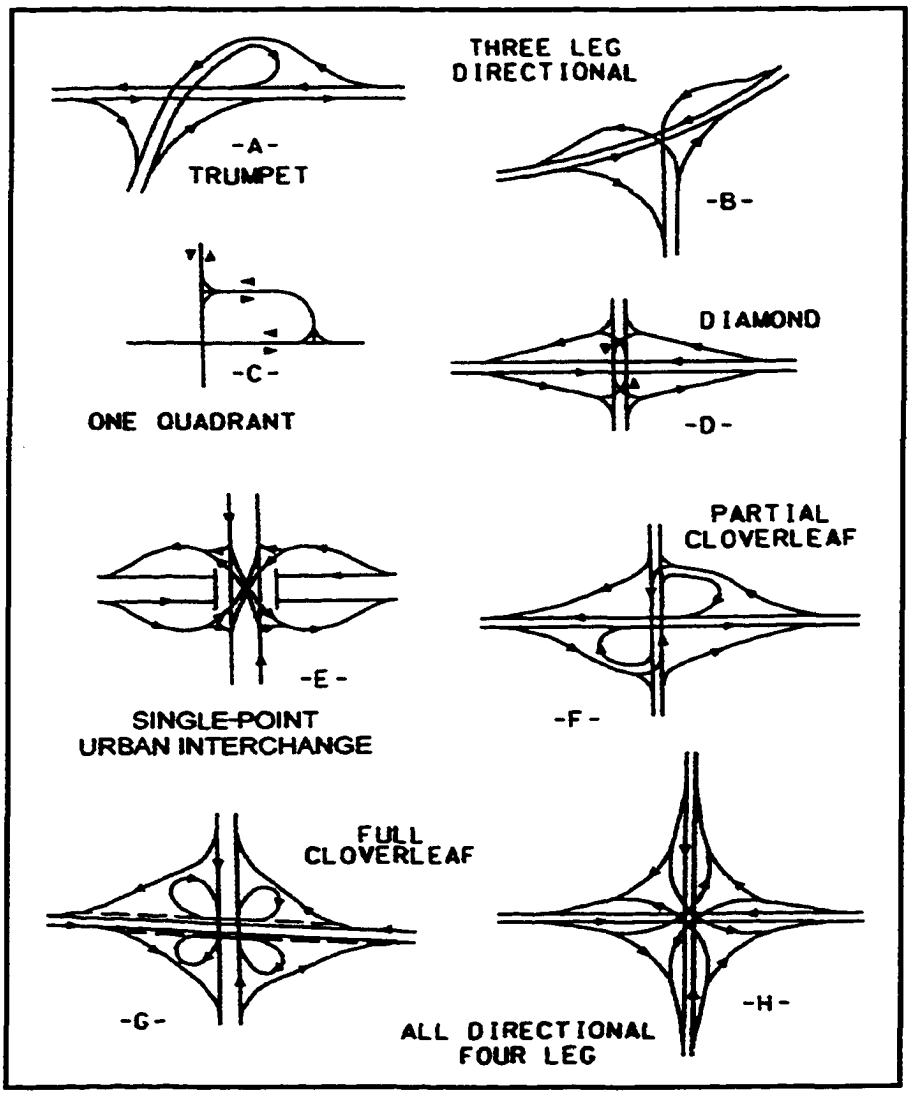


Figure 2.2: Basic Interchange Configurations (AASHTO 2004, Exhibit 10.1)

The selection of the appropriate or most suitable interchange configuration for any particular application and its detailed design depend on a number of controls and other considerations. Among them, as outlined in TAC (1999), are: safety, functional and design classification of intersecting roads, adjacent land use patterns, design speed of intersecting roads, traffic volume and traffic mix, number of intersecting legs, traffic control devices to be provided, topography of the area, right-of-way requirements and

availability, service requirement for adjacent communities, overall systems considerations and design consistency, environmental factors, and economics. TAC (1999) also emphasized the uniformity of interchange patterns and design consistency to best promote the diver understanding and facilitating safe manoeuvring through an interchange such as selecting a suitable speed, accelerating or decelerating, choosing appropriate lanes, and then diverging or merging successfully.

AASHTO (2004) mentioned that while the interchanges are custom designed to fit specific site conditions, it is desirable to provide uniformity in exit and entrance patterns to satisfy the need for high capacity, appropriate level of service, and maximum safety in conjunction with freeway operations. Therefore, the overall pattern of exits along the freeway should have some degree of uniformity. It is also desirable that all interchanges have one point of exit located in advance of the crossroad, whenever practical, to favour the driver expectancy.

TAC (1999) stated that left exits and entrances have experienced much higher collision rates than right exits and entrances due to weaving and turbulence movements in the vicinity of interchanges. Therefore, it is desirable that all exits from and entrances to freeways should be on the right, unless an unavoidable situation exists to place them on the left. Advance and special signing is recommended to enable the drivers to merge or diverge at left safely and without causing undue hazard to other road users. The importance of maintaining basic freeway lanes, lane balance and the proper design of weaving sections in achieving the smooth operation, flexibility and the desired level of service on the freeway was also emphasized. TAC (1999) recommended that exit ramps

should be on upgrades to assist in deceleration and entrance ramps should be on downgrades to assist in acceleration of vehicles. Therefore, it is desirable that freeways should pass under the cross-road, unless the opposite is unavoidable.

As a general guide, TAC (1999) recommended that in rural areas interchanges may normally be spaced at 3 to 8 km. Urban freeways are usually associated with higher traffic volumes and higher access demand. The traffic conditions and driver behaviour and expectations on urban freeways are different from those of rural freeways, and this influences interchange spacing on urban areas. As mentioned in TAC (1999), interchanges spaced at more than 3 km cannot normally provide adequate service to urban development, and closer spacing is called for. Minimum interchange spacing in urban context is determined by the distance required for weaving, length of speed-change lanes, and distance required for the placement of directional signs at appropriate locations. As a general rule, 2 to 3 km spacing between interchanges in urban areas was recommended. TAC (1999) also stated that urban freeway collision rates tend to increase as the spacing between interchanges decreases.

AASHTO (2004) mentioned that interchange spacing has a pronounced effect on freeway operations. In general, the guide recommended a minimum interchange spacing of 1.5 km (1 mile) in urban areas and 3.0 km (2 miles) in rural areas. HCM (2000) mentioned *"the ideal average interchange spacing over a reasonably long section (8 to 10 km) of freeway is 3 km or greater. The minimum average interchange spacing considered possible over a substantial length of freeway is 1 km (p 10-6)"*.

2.1.3 Interchange Ramps

2.1.3.a Ramp Design Features

TAC (1999) defined an interchange ramp as a connecting roadway carrying traffic between two grade-separated through roads. The interchange configuration, selected to suit a particular access site is in fact made up of different ramp patterns. Figure 2.3 illustrates several types of ramps and their typical shapes as provided in AASHTO (2004).

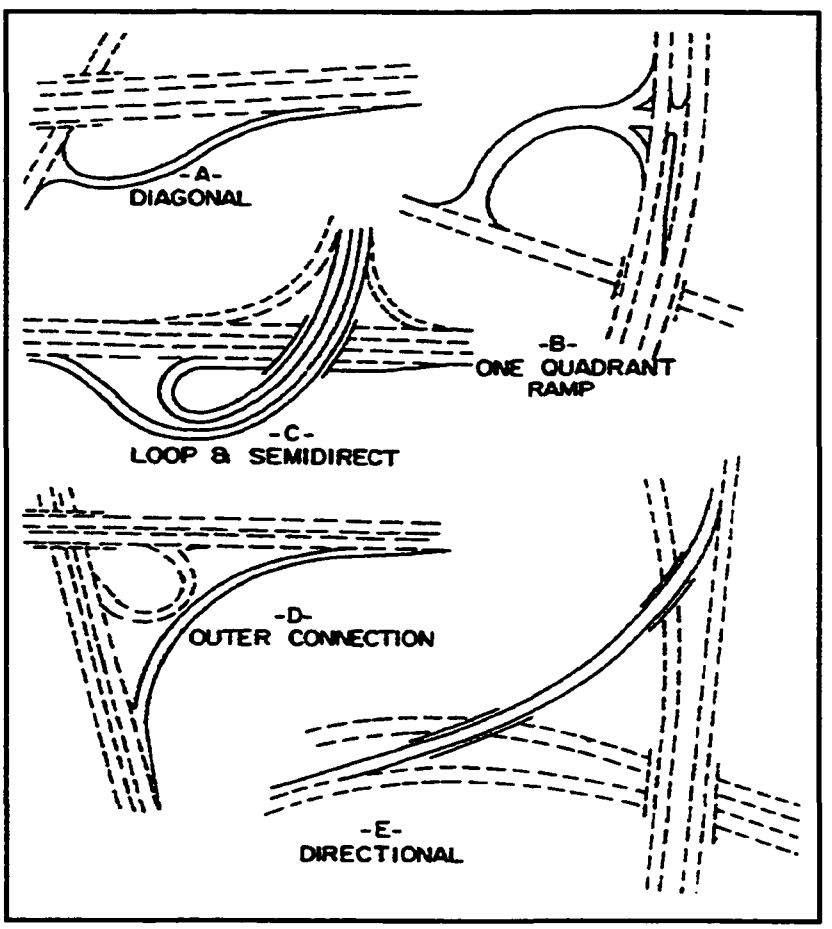


Figure 2.3: General Types of Ramps (AASHTO 2004, Exhibit 10-55)

As mentioned in TAC (1999), an interchange ramp consists of the exit terminal, the ramp proper or the connecting roadway between the terminals, and the entrance terminal. The geometric features of ramp are influenced by a number of factors such as traffic volume and traffic mix, geometry of the connecting roads, operational characteristics of the through roads, site terrain, traffic control devices and driver expectation. The design domain guide values for ramp design speed in terms of roadway design speed are shown in Table 2.1. These design values apply to the controlling ramp curve on the ramp proper. For directional ramps and outer loops, higher values within the design domain are recommended. For inner loops, higher design speed, above 50 km/h, would require a large area which is usually not available, particularly in urban areas. The guide recommended that the minimum inner loop design speed, in concentrated urban areas, should be 40 km/h.

Table 2.1: Ramp Design Speed (TAC 1999, 2.4.6.1)

Roadway Design Speed (km/h)	Ramp Design Speed, Design Domain* (km/h)
60	50 – 40
70	60 – 40
80	70 – 40
90	80 – 50
100	90 – 50
110	100 – 60
120	110 – 60
130	110 – 70

AASHTO (2004) recommended compound or spiral curve transitions following the controlling ramp curve to obtain the desired alignment of the ramps, assist in smooth

transition between the design speeds of the through and turning roadways, and fit natural path of the vehicles.

2.1.3.b Gores

In AASHTO (2004), the "gore" is defined as an area downstream from the freeway and ramp shoulder intersection point as shown in Figure 2.4. The physical nose is defined as a point upstream from the gore, having some dimensional width (1.2 to 2.4 m) that separates the roadways (mainline and ramp), while the painted nose is a point, having no dimensional width, occurring at the separation of the traveled ways of freeway and ramp terminal. The triangular area between the painted and the gore nose in referred as the neutral area and it incorporates the physical nose.

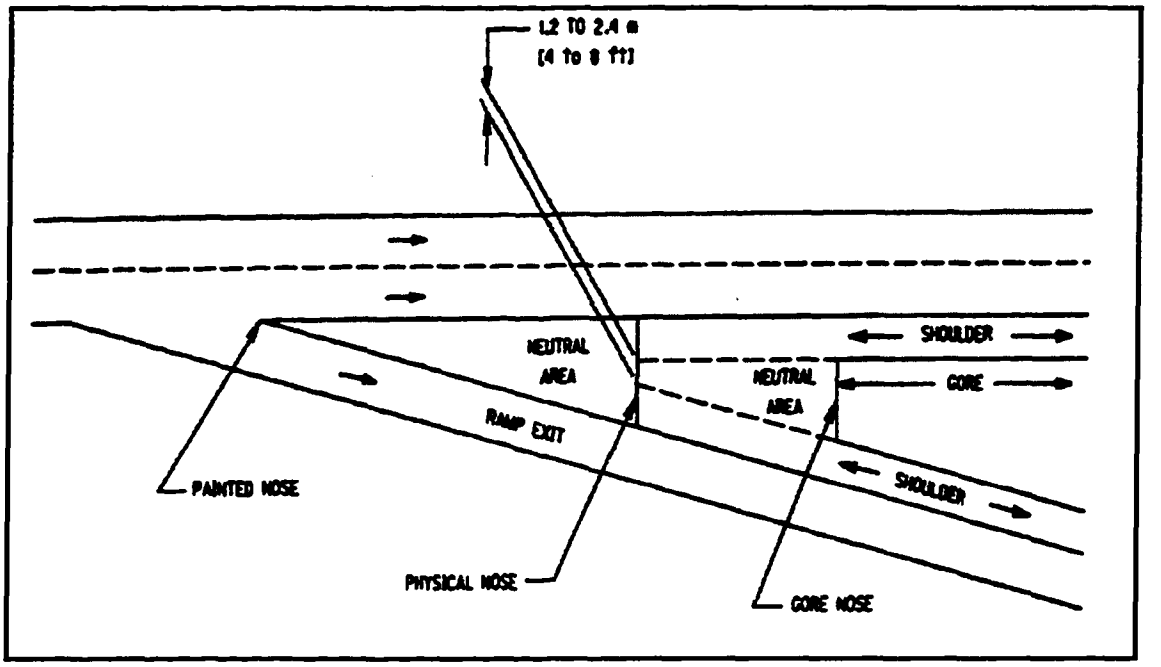


Figure 2.4: Typical Gore Area Characteristics (AASHTO 2001, Exhibit 10-59)

At an entrance terminal, the term "gore" refers to the similar area between a through roadway and a converging entrance ramp. The shape, layout, and extent of the triangular manoeuvre area at an entrance terminal are similar to that at an exit terminal. However, entrance gore converges in a direction opposite to the exit gore (points downstream). The point of convergence, at an entrance terminal, is the beginning of all paved area and is referred as the "merging end".

The geometry of the nose is important for the driver to make a merging manoeuvre safely. As mentioned in TAC (1999), collision rates in the vicinity of gore areas are higher than that at other locations, and therefore the gore area is usually kept as free of obstructions as possible. The drivers on a ramp begin accelerating in the vicinity of the spiral curves and start looking for a gap in the traffic stream on freeway through lane. The geometry of the gore area, such as ramp grade and curvature, and angle of convergence, should provide the driver a clear view of an object of 1.0 meter height at the centre of the right lane at an angle of 120° with respect to the direction of travel as shown in Figure 2.5. In addition, to allow the driver to make the merging manoeuvre safely, after accelerating to the desired speed, the driver should have a clear view of the entire speed-change lane from the nose.

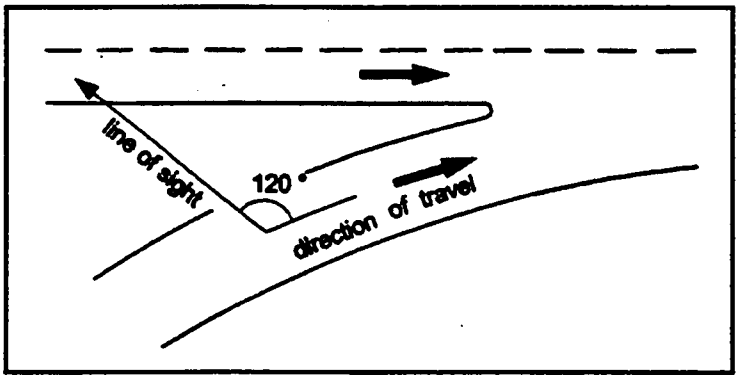


Figure 2.5: Line of Sight at Entrance Terminals (TAC 1999, 2.4.6.4)

2.1.3.c Ramp Terminal

As mentioned in AASHTO (2004), the ramp terminal includes speed-change lanes, tapers, and islands adjacent to the through traveled ways. At the cross road, the ramp terminals are usually at-grade type. At the high speed road, the ramp terminal is free-flow type, where ramp traffic merges with or diverges from the high speed through traffic at flat angles. Ramp terminals can also be classified as either single lane or multilane terminals based on the number of travelled way lanes on ramp at the terminal location, and either a taper or parallel type based on the configuration of the speed-change lane downstream (for entrances) or upstream (for exits) the nose area.

Two or more ramp terminals are sometimes located in close succession, particularly on urban freeways. However, to provide sufficient weaving distance and adequate space for signing, the spacing between successive ramp terminals should have a reasonable minimum depending on the classification of the interchanges involved, the function of the ramp pairs (entrance or exit), and weaving potentials. In case of a ramp-pair where an entrance ramp is followed by an exit ramp, the absolute minimum distance between the

noses at terminals is selected based on weaving requirements. AASHTO (2004) recommended that when the spacing between successive noses, where an entrance ramp is followed by an exit ramp (En-Ex pair), is less than 450 m (1500 ft), the speed-change lanes should be connected to provide an auxiliary lane. This auxiliary lane is believed to improve traffic operation over relatively short sections of the freeway route and is not considered an addition to the basic number of lanes. The auxiliary lane between an entrance and an exit ramp, referred to as extended speed-change lane in the research study at hand, serves both as acceleration and deceleration lanes for entering and exiting traffic, respectively.

2.1.4 Speed-Change Lane

2.1.4.a Design Features

Drivers leaving a highway at an interchange are required to reduce their speed as they exit to a ramp. On the other hand, drivers entering a high speed road facility from an entrance ramp are required to accelerate until the desired merging speed is reached. This desired merging speed should be close to the running speed of the through traffic. Since the necessary change in speed between a ramp and a freeway is usually substantial, provision should be made for deceleration and acceleration to be accomplished on auxiliary lanes. Such an auxiliary lane, including tapered lengths, is called a speed-change lane. The term speed-change lane, deceleration lane, or acceleration lane refers to the added lane adjoining the travelled way of the roadway and does not necessarily imply a definite lane of uniform width. It is a part of the ramp terminal area at merge and

diverge areas.

The two forms of acceleration lanes, as provided in TAC (1999), are shown in Figure 2.6. As mentioned in TAC (1999), both of the parallel and the direct taper design, when properly designed for the appropriate site and traffic conditions, will operate satisfactorily. However, the guide indicated that for the "direct taper" design, some design agencies are concerned about the forced merge from the tapered entrance terminal (acceleration lane), especially when the speed on the through traffic lane is high.

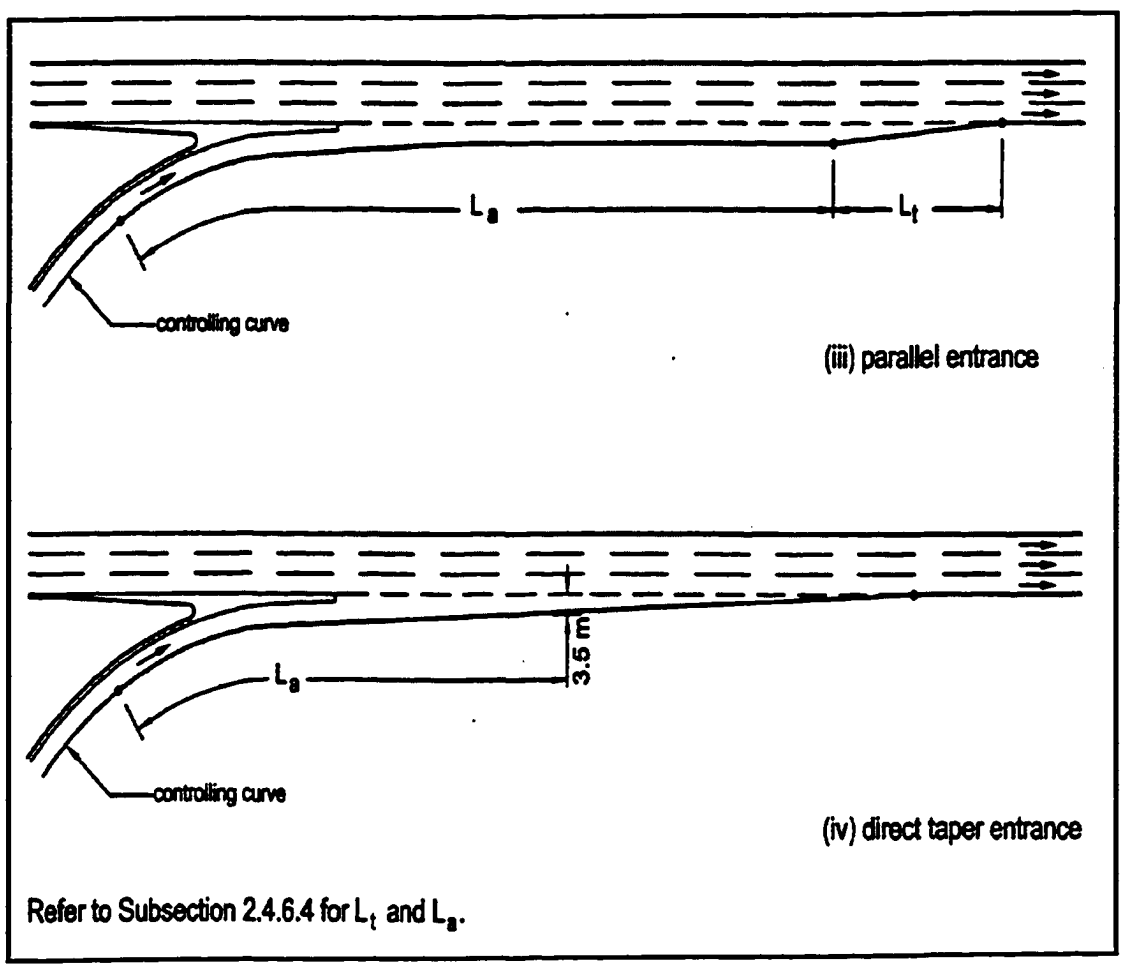


Figure 2.6: Acceleration Speed-Change Lanes (TAC 1999, 2.4.6.1)

In parallel-type entrance terminal design, an auxiliary lane of uniform width is added to the right of the outermost through traffic lane and is ended some distance downstream by means of a taper. As mentioned in TAC (1999), the process of merging from the parallel type acceleration lane into the through roadway lane is similar to a lane change to the left, after accelerating to a speed close to the through traffic speed. The geometric design of parallel type speed-change lane usually include a long acceleration lane which provides more time and flexibility to the merging vehicles to find a suitable gap in the through-traffic stream. The parallel type allows the drivers of entering vehicles to freely observe traffic conditions on the main line. Thereby, the parallel design can provide operational and safety benefits which are particularly important when the headways are short and traffic gaps are infrequent.

On the other hand, the direct taper type entrance follows a direct alignment from the entrance nose to the edge of the through lane some distance downstream the nose and works on the principle of direct entry. As mentioned in TAC (1999), the taper design provides a more natural and direct path but on a comparatively short merging length, which is a concern under heavy-volume conditions. The guide mentions also that where the freeway mainline is on a significant right hand curve, and visibility is limited, the direct taper could be less effective as the angle of convergence tends to be too great.

2.1.4.b Length of Acceleration Lane

According to TAC (1999), a speed-change lane should have adequate length to enable a driver to make the necessary change of speed between the operating speeds of the

through roadway and ramp. The guide also mentions that in case of acceleration speed-change lane, an additional length should be provided so that the driver of the entering vehicle can manoeuvre into a gap in the through traffic stream before reaching the end of the acceleration lane.

According to TAC (1999), for parallel type entrance, the length of the acceleration lane, L_a, is measured from the end of ramp curve to the beginning of taper. For the direct taper type entrance, L_a is measured from the end of ramp curve to the point at which the auxiliary lane is 3.5 meters wide. The measurement of L_a for both of parallel and tapered type design, and the tapered portion for parallel type design (L_t) is shown in Figure 2.6. As mentioned in TAC (1999), the length of an acceleration speed-change lane is based on three factors in combination, which are: the running speed on the through lanes, the control speed on the ramp proper, and the manner of acceleration.

Table 2.2 shows the TAC (1999) design domains of acceleration lane lengths for acceleration lanes situated on grades not steeper than 3%. These design domains were developed using the acceleration-distance curves presented in the AASHTO (1994) geometric design guide that were developed based on acceleration of passenger car tested in the US during 1940s or 1950s. As shown in the table, in recommending the length of acceleration lanes, the guide assumes mainline operating speeds that are 5 to 25 km/h lower than the design speed of 60 to 130 km/h, though research has shown that operating speeds of vehicles are well above the design speeds of the road facilities (Seneviratene et al. 1992). Therefore, the recommended acceleration lane lengths may result in large speed disparities between mainline and entering vehicles merging speeds. This in turn

may result in higher collisions at freeway merge areas, as speed inequalities cause accidents (Baker 1980).

Table 2.2: Design Length of Acceleration Lanes (TAC 1999, 2.4.6.5)

Speed of Roadway (km/h)		Length of Taper (m)	Length of Acceleration Lane Excluding Taper (m) L_a							
			Design Speed of Turning Roadway Curve (km/h)							
Design	Assumed Operating	L_t	Stop Condition	20	30	40	50	60	70	80
60	55-60	55	85-115	70-100	60-80	45-60	20-35			
70	63-70	65	120-160	115-150	100-135	80-115	50-85	15-40		
80	70-80	70	160-225	150-215	130-200	115-185	85-160	40-100		
90	77-90	80	215-325	200-310	180-300	160-285	140-250	50-200	40-145	
100	85-100	85	275-450	250-440	240-420	225-405	200-375	140-325	100-285	40-230
110	91-110	90	330-650	320-645	305-630	290-600	260-575	210-525	150-475	100-410
120	98-120	95	410-730	400-725	375-710	370-690	340-660	285-590	250-515	195-430
130	105-130	100	550-885	540-880	510-870	500-850	470-820	400-745	340-655	300-550

Notes: 1. The selection of ramp design speed as discussed in Subsection 2.4.6.2 should be referred.
2. The acceleration distance curves in 1994 AASHTO are used in developing the design domain.

TAC (1999) also mentioned that the recommended lengths of acceleration lanes, shown in Table 2.2, should be adjusted by the appropriate grade factors if the acceleration lanes are situated on grades steeper than 3%. The length of an acceleration lane to be provided at a specific site also depends on the volumes of through roadway and entering traffic. Longer acceleration lanes, i.e. higher values of the design domain, are recommended to enable the entering traffic to merge with the through traffic safely and conveniently on higher volume roads. The guide also mentioned that trucks and buses require longer acceleration lanes than passenger cars, and therefore longer acceleration lanes are recommended where a substantial number of large vehicles are expected to enter the road.

Figure 2.7 shows typical single-lane entrance ramp terminals for both taper type and parallel type entrance as provided in AASHTO (2004). The taper-type acceleration lane is merged with the outer-most through lane by means of a long, uniform taper of about 50:1 to 70:1. As mentioned in AASHTO (2004), the geometry of the ramp proper should permit motorists to attain a speed that is within 10 km/h of the operating speed of the freeway by the time they reach the point where right edges of the through lane and ramp travelled way is 3.6 m apart. The distance (L_a) to be provided in advance of this point of convergence is governed by the speed differential between the operating speed on the controlling curve of entrance ramp and the operating speed of the highway. For parallel-type, the added lane should enable the vehicle to accelerate to near-freeway speed prior to merging. A taper length approximately 90 m, suitable for design speed up to 110 km/h, is recommended at the end of the added parallel lane.

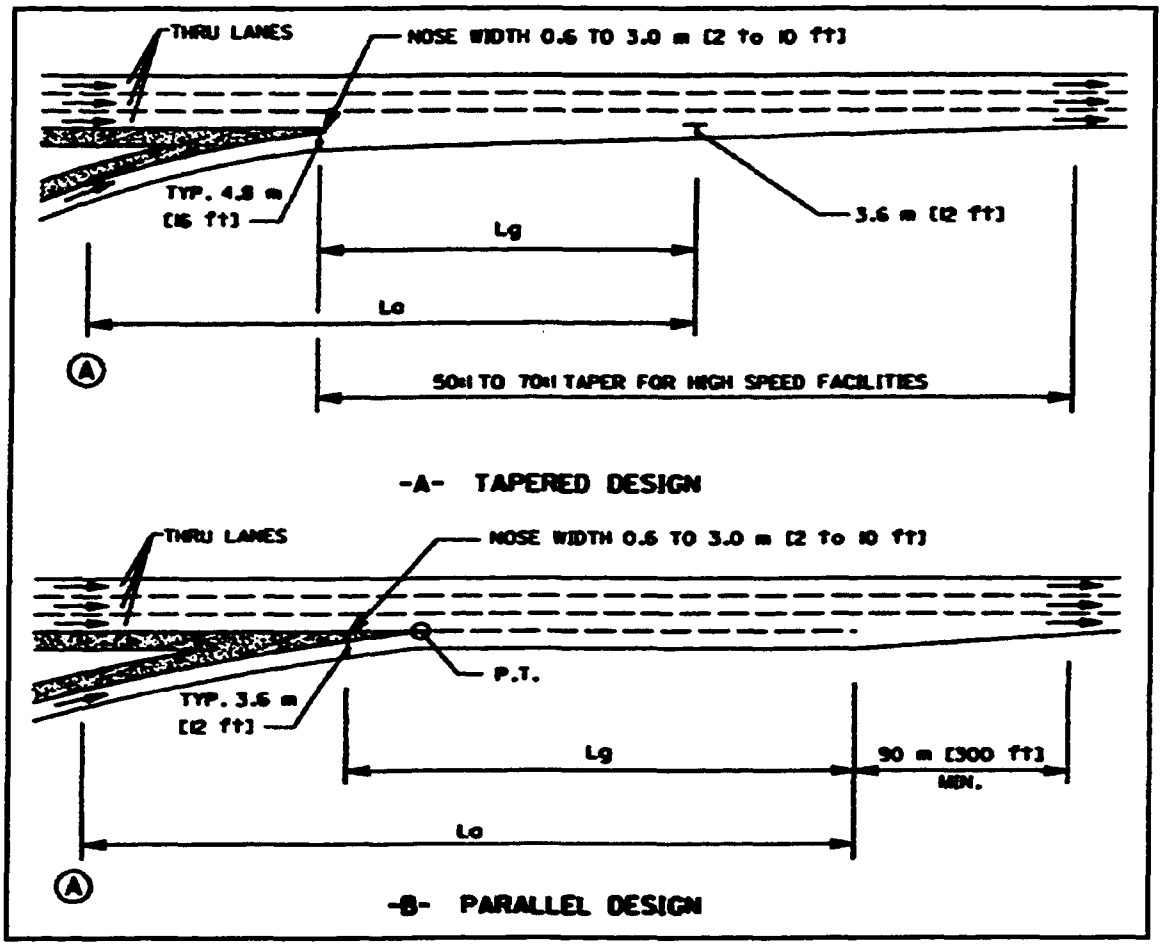


Figure 2.7: Typical Single Lane Entrance Terminal (AASHTO 2001, Exhibit 10-69)

The minimum length required for gap acceptance (L_g) as well as for acceleration (L_a) is specifically shown in Figure 2.7, as provided in AASHTO (2004). L_a is measured from the downstream end of ramp curve, unless the radius is greater than 300 m, and L_g is measured from the point where the right edge of through lane and left edge of ramp travelled way is 0.6 to 3.0 m apart. The length L_g should be a minimum of 90 to 150 meters depending on the nose width. The value of L_g or L_a, which produces the greatest distance downstream the point where nose width is 0.6 m, is suggested for use in the design of the entrance terminal.

Table 2.3 shows the AASHTO (2004) recommended minimum lengths of acceleration distance for entrance terminals based on the design speeds of highway and controlling ramp curve. The guide also recommends an acceleration lane length of at least 360 m, plus the taper, for parallel type design, wherever it is anticipated that the ramp and freeway will frequently carry traffic volumes approximately equal to the design capacity of the merging area. As shown in the table, the estimated speed (V_a) reached by the entering vehicles at the end of recommended acceleration lane length is 13 to 32 km/h lower than highway design speed in the range of 50 to 120 km/h. It should be noted however that the guide mentioned that the geometrics of the ramp proper should be such that motorists may attain a speed that is within 10 km/h of the operating speed of the freeway by the time they reach the point where right edges of the through lane and ramp travelled way are 3.6 m apart. This further shows inconsistencies in the guide's recommendations. Therefore, the recommended length of acceleration lanes may lead to large speed disparities between the operating speed of freeways and merging speed of entering vehicles. Such speed disparities may contribute to higher collisions at the merge areas.

Table 2.3: Minimum Acceleration Lane Lengths for Entrance Terminals with Flat Grades of 2 Percent or Less (AASHTO 2004, Exhibit 10-70)

Metric

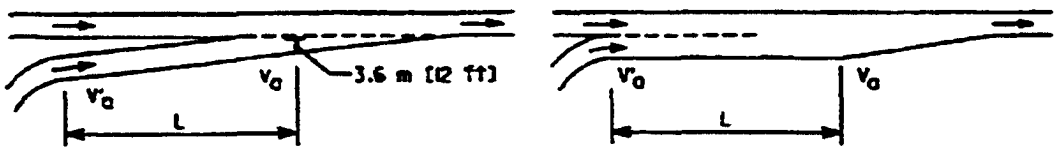
Highway		Acceleration length, L (m) for entrance curve design speed (km/h)							
		Stop condition	20	30	40	50	60	70	80
Design speed, V (km/h)	Speed reached, V_a (km/h)	and initial speed, V_a' (km/h)							
		0	20	28	35	42	51	63	70
50	37	60	50	30	—	—	—	—	—
60	45	95	80	65	45	—	—	—	—
70	53	150	130	110	90	65	—	—	—
80	60	200	180	165	145	115	65	—	—
90	67	260	245	225	205	175	125	35	—
100	74	345	325	305	285	255	205	110	40
110	81	430	410	390	370	340	290	200	125
120	88	545	530	515	490	460	410	325	245

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 400 m.

US Customary

Highway		Acceleration length, L (ft) for entrance curve design speed (mph)								
		Stop condition	15	20	25	30	35	40	45	50
Design speed, V (mph)	Speed reached, V_a (mph)	and initial speed, V_a' (mph)								
		0	14	18	22	26	30	36	40	44
30	23	180	140	—	—	—	—	—	—	—
35	27	280	220	160	—	—	—	—	—	—
40	31	360	300	270	210	120	—	—	—	—
45	35	560	490	440	380	280	160	—	—	—
50	39	720	660	610	550	450	350	130	—	—
55	43	960	900	810	780	670	550	320	150	—
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.



TAPER TYPE PARALLEL TYPE

Finally, in regard to extended type acceleration lanes, the TAC (1999) guide recommended a length of at least 550 m to 700 m for weaving section between two arterial interchanges. The criteria for measuring the length of extended speed-change lane as provided in HCM (2000) is shown Figure 2.8. As shown in the figure, the length is measured from a point at the entrance gore where the right edge of freeway through lane and left edge of the merging lane is 0.6 meter apart to a point at the diverge gore where the two edges are 3.7 m apart. Similar measurement is recommended in TAC (1999) with the exception of the start point at the entrance gore, which is taken as 0.5 m separation, instead of 0.6 m, of the two pavement edges.

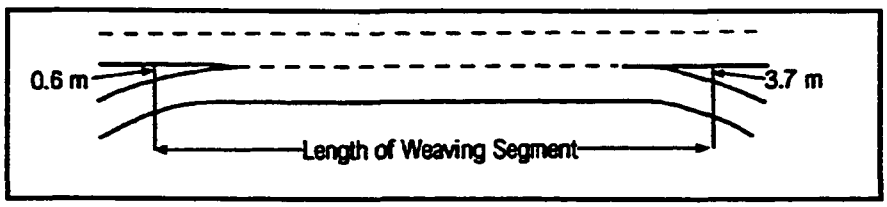


Figure 2.8: Measuring the Length of a Weaving Section (HCM 2000, Exhibit 13-11)

2.1.4.c Safety

The AASHTO design guide has not introduced an explicit safety evaluation in the design recommendations. Rather, the guide provides an implicit safety criterion that long acceleration lanes provide operational and safety benefits. TAC (1999), on the other hand, has introduced explicit evaluation of safety in the design domain guide. However, for interchange design the guide state, "*in general, reliable, specific and explicit quantitative correlations between interchange parameters and safety are not available (p 2.4.1.1)*". The guide then mentions that collisions on ramps and connecting roads

generally increase with the increase in traffic volume and with decrease in curve radius. Upgrade exit ramps experience lower collisions rates and thus, preferable from a safety view point. Tight radius curves on ramps and short speed-change lanes cause problems with heavy trucks and higher incidents involving trucks.

2.2 REVIEW OF RELEVANT RESEARCH

This section presents a summary of the available literature relevant to the objectives of this research.

2.2.1 Driver Behaviour on Freeways Merge Areas

Wattleworth et al. (1967) analyzed the effect of entrance ramp geometrics on freeway merging using data from 24 entrance ramps on freeways in eight US cities, with speeds greater than 40 mph (64 km/h). The length of the acceleration lanes (physical gore to end of taper) ranged from 240 to 1500 ft (73 to 457 m) and angle of convergence ranged from 1 to 14°. Reference marks were placed at 200 ft intervals starting from 400 ft upstream the gore and vehicle position were obtained by aerial photography during free flow conditions. No well defined relationship between the merging speed, acceleration lane length and angle of convergence at physical nose was found. A model was suggested for average merging distance from the nose to the merging point of entering vehicles during each three-minute period. The model produced an excellent R^2 value of 0.90. The statistically significant independent variables were acceleration lane length, ramp length, relative grade between nose and 400 ft upstream the nose, angle of convergence, ramp

offset at nose, and the product of the three-minute flow rate and average speed of the freeway vehicles approaching the nose. However, the justification of this product was not clarified. The merge volume, the percent trucks in the merge volume, and the percent ramp vehicles in the merge volume was found not to be statistically significant.

Blumenfeld et al. (1971) presented a model for the expected delay on an acceleration lane of finite (limited) length. The model assumed that a car in the acceleration lane travels at constant speed v until the end of acceleration lane while the traffic on main road travels at constant speed $V>v$. The model assumes that no delay occurs if the merge occurs before the vehicle reaches the end of acceleration lane, which in turn assumes $V = v$ for no delay on the acceleration lane. It was found that the expected delay does not depend strongly on the length of the acceleration lane, and that most of the merging occurs near the beginning of the lane. It was concluded that increasing the length of the acceleration lane beyond 600 ft (183 m) results in relatively little further benefit in merging. However, this conclusion was based on a curve plotted for the expected delay against acceleration lane length, assuming a large speed difference between the mainline and vehicle on acceleration lane (V = 60 mph and v = 20 mph). The justification of such assumption or conclusion was not clarified.

Olsen et al. (1976) studied merging behaviour on three on-ramps and indicated that the merge manoeuvre is likely to be strongly affected by the overall length of acceleration lane. On a long parallel acceleration lane of 264 m (physical gore to end of taper), the last 140m was not utilized by 86.6% of vehicles, and the tapered section (67 m) was not utilized by 97.3% of the drivers. Less of the acceleration lane was utilized when traffic

volume increases. On a short taper acceleration lane of length 61 m, 27% of the entering vehicles came to a complete stop on the acceleration lane, while 56% percent of all vehicles either stopped or slowed drastically, and 3.3% of all vehicles run-off the road (lane) which prevented proper and full-speed merging. At another site, with provision of an additional 35 m, only 0.5% of all vehicles run-off the road. It was recommended that during design, part of the taper should be kept for evasive action.

Huberman (1982) collected speed data using radar emitter/receiver system, located at selected distance intervals on five entrance ramps in Edmonton, Alberta (Canada), to examine the vehicle performance on ramp. All ramps were diamond type situated on downgrades less than 4%, with high operating speeds (speed limit 60 to 70 km/h), and radar emitters were placed upstream the gore on the ramp proper. It was found that 85% or more of the entering vehicles travelled at speeds fairly close to the design speeds of the ramp curves. 85% or more of the highway vehicles, in the vicinity of ramp terminal, travelled at the average running speed of the highway. The 85th percentile passenger car speed ranged from 102 to 112 km/h on highways with design speed of 110 to 120 km/h. Higher acceleration rates than that presented in AASHO (1965) design guide were derived from vehicle survey over 20 to 70 km/h speed range, and considered to be attributed to the improved operating characteristics of the current vehicle population. Exponential relationship between speed and acceleration rate was assumed for speed ranging from 30 km/h to 70 km/h and a model was developed. However, the developed model did not consider the effect of ramp geometry such as ramp curvature, angle of convergence, grade, length of the acceleration lanes, etc. As the acceleration of the vehicle was computed from the speeds, model correlating these two again without

considering the effect of other geometric variable (s) is practically meaningless. The model was given as:

$$y = 159.91e^{-0.44x} \qquad R^2 = 0.91 \qquad (2.1)$$

Where, y = Passenger car speed in km/h, and x = Passenger car acceleration rate in km/h/s.

Fazio (1988) developed a user-based model for prediction of vehicle speed at freeway weaving sections using a total of 28 observations, collected by time-lapse aerial photography, from three sites in California and Georgia (US). All sections contained four lanes (three through lanes and one auxiliary lane) forming a simple weaving sections. Multiple regression analysis was performed using average running speed by lane within the weaving section as dependent variable. However, the model used the orderly number of different lane positions as an independent variable together with other predictors. Use of ordinal number is not correct from a statistical point of view, as ordinal numbers should be transformed to categorical variables for use in statistical model. The resulting model was given as:

$$S_i = 29.3 - 0.0066[V/(PHF * f_{HV})] + 0.0392 * Length + (Lane_i) \qquad (2.2)$$

Where, S_i = Average running speed of the vehicles in lane i (mph), i = Lane position number (1 for the auxiliary lane, 2 for the freeway lane adjacent to auxiliary lane, 3 for the next freeway lane, and 4 for the outermost lane), V = Sum of flow rates in lane 1 through 4 (vehicles/h), PHF = Peak-hour factor based on

five-minute intervals, f_{HV} = Heavy vehicle adjustment factor, and *Length* = Length of weaving section (ft).

Michaels et al. (1989) suggested a model for angular velocity of approaching vehicles relative to vehicle on the speed-change lane. The model was tested during peak-hour traffic on 102 vehicles at two ramps, one loop and one diamond connector. The threshold of angular velocity, for ramp vehicle to steer onto the freeway, was taken as 0.004 rad/sec as median value of the distribution of angular velocity that ranged from -0.043 to +0.053. It was mentioned that if the observed gap between vehicles on mainline (lead and lag) and vehicle on acceleration lane generate a less than the threshold angular velocity, the entering driver initiates the steering manoeuvre required to merge. It was concluded that a speed-change lane length of 800 ft (244 m) is sufficient to ensure an acceptable gap 85 percent of the time for all freeway volumes over 1,200 passenger cars/h/lane and ramp design speeds over 30 mph (48 km/h), assuming an acceleration capability of the ramp vehicle of more than 1.5 ft/sec^2. However, the central tendency of the distribution of angular velocity was appeared to be approaching to zero or may not be statistically different from zero. The significance of the central tendency was not indicated, and the applicability of such a model may be questioned.

Kou et al. (1997) presented a methodology to model ramp vehicles acceleration–deceleration behaviour at freeway entrance ramp terminal based on video data collected at four entrance ramps in Houston and Austin, Texas (US). The entrance ramps included short and long acceleration lanes of both taper and parallel configurations. A complex model was presented for vehicle acceleration-deceleration, using car following

technique, as a function of ramp vehicle position and speed, position and speed of freeway lead and lag vehicles, an assumed distance lag, etc. However, it was concluded that the findings in their study cannot be directly utilized in practice, but useful in updating the existing microscopic freeway simulation models. Modelling attempts for merging position showed that none of the individual traffic parameters such as traffic flow levels, ramp vehicle approach speeds, time lag between entering and freeway lag as well as lead vehicles, and relative speeds between entering and freeway lag vehicles has a statistically significant effect on the merge position of entering vehicles. However, collecting more freeway merge data from different sites in developing such models was recommended.

Liang et al. (1998) found, from a study on 25 km stretch of interstate highway with a posted speed of 88.5 km/h in Idaho and Utah (US), that passenger car speeds are consistently higher than those of trucks by about 6.4 km/h. Dixon et al. (1999) studied the effect of change in posted speed limit from 89 to 105 km/h (55 to 65 mph) on rural freeways in Georgia (US). Cars and heavy vehicles were found to have similar free-flow speed during daylight at 89 km/h posted speed condition, but at higher posted speed condition of 105 km/h, heavy vehicles drove approximately 0.8 km/h slower than cars. During night-time, heavy vehicles drove faster than cars at both posted speed conditions, and passenger car daytime speed was 1.0 km/h higher than the night-time speed. Limited visibility likely contributed to the lower night-time speeds.

Hunter et al. (2001) studied ramp and freeway speed-distance relationship based on data collected by video taping at six non-loop ramps, with no substantial horizontal

curvature, in three Texas cities. Acceleration lane lengths were measured from the painted ramp gore or nose until full-width lane was no longer available, which varied from 55 to 241 m. Selected acceleration lanes included both of the substandard design that not meeting AASHTO and standard design meeting the current AASHTO design practices in terms of length, grade, visibility, presence of barrier, presence or absence of shoulder, etc. Vehicle speed was calculated from the distance and travel time between a pair of marks, located on the shoulder at 18.3 to 36.6 m intervals. It was concluded that regardless of the ramp design speed, drivers in the 85th percentile region attempt to drive at speeds within 70 to 80 % of the typical 112.7 km/h freeway design speed and thus, a 50th percentile design speed, i.e. selected ramp design speed is about 50% of the highway design speed, might have negative safety implications. It was also found that good geometrics typically produce a small positive acceleration rate with a mean of 0 to 3.2 km/h/s, whereas bad geometrics produce larger values of positive and negative accelerations with a mean of -6.4 to 6.4 km/h/s. Furthermore, freeway right-lane speeds were not found to be largely affected by ramp vehicles if the ramp has good geometrics, whereas bad geometrics tend to cause significant reductions in right lane speeds, particularly under high freeway and ramp traffic volumes.

Liapis et al. (2001) developed two models for prediction of operating speed (V_{85}) on curved sections of the ramp proper based on data collected at twenty on and off ramps in Greece. The variables used were rate of change of curvature (*CCR*), superelevation rate (e), pavement width, paved shoulder width, sight distance, and grade. Only *CCR* and superelevation were statistically significant at 0.05 significance levels. The models for

off and on ramp curves were given as:

Off-ramps: $V_{85} = 75.161 - 0.009886CCR - 3.6835e$ $R^2 = 0.75$ (2.3)

On-ramps: $V_{85} = 85.186 - 0.01295CCR - 3.795879e$ $R^2 = 0.73$ (2.4)

Where, V_{85} = Operating speed in km/h, CCR = Rate of change of curvature of ramp curve including transition curves (gon/km), e = superelevation rate (%).

Using Equation 2.4, the predicted speed on entrance ramps were 6 to 8.6 km/h higher than the ramp design speed for curve radii between 50 to 250 m. It was also concluded that for same the CCR and e, the speed difference between an on ramp and an off ramp is approximately 10 km/h.

Voigt et al. (2003) studied the comfort levels for drivers of various types of vehicles on freeway-to-freeway connector ramps in the Houston urban area. Each of the vehicles was driven through seven freeway-to-freeway ramps at speeds ranging from 30 to 55 mph (48.3 to 88.5 km/h). Drivers of passenger cars, light trucks, and sports-utility vehicles generally exceeded the posted advisory speed limit by more than 10 mph (16 km/h) whereas the drivers of trucks generally exceeded the posted advisory speed limit by about 5 mph (8 km/h). On most connector ramps, maximum comfort levels, determined by ball-bank indicator, for drivers of passenger cars and sports-utility vehicles appeared to be 5 to 10 mph (8 to 16 km/h) higher than that for longer vehicles. The 85th percentile speed on a particular curve typically corresponded very well to the maximum "comfortable" speed of the test vehicles.

A large number of other models have been developed and or suggested by researchers worldwide for prediction of operating speed on horizontal curves of two-lane rural highways and design consistency evaluations.

2.2.2 Safety Performance

Cirillo (1970) studied the effect of geometry of acceleration and deceleration lanes, and weaving areas on the accident rate per hundred million vehicles using accident data of 2,288 sections, collected during 1961-1965 in twenty US states. It was concluded that additional length of speed-change lanes would be beneficial in terms of reduced accident rate if the percentage of merging or diverging traffic is above 6% percent of the mainline traffic. However, the increased length of acceleration lanes appeared to be more beneficial than the increased length of deceleration lanes. It was also mentioned that deceleration lanes are, in general, safer than acceleration lanes regardless of the length of the lane and the percentage of merging or diverging traffic.

Harwood et al. (1993) stated that AASHTO policies for freeway ramp design are adequate as long as the drivers adjust their speed to levels that are less than or equal to the design speed. Safety problems on ramp curves are most likely for vehicles, especially trucks, travelling faster than design speed. Most critical conditions occur for horizontal curves with lower design speed of 20 to 30 mph (32 to 48 km/h). A truck on a curve with a 20 mph (32 km/h) design speed can rollover when travelling at 25 mph (40 km/h) and may skid off the road under critical wet-pavement conditions when travelling at about 27 mph (43 km/h). A truck on a curve with 30 mph (48 km/h) design speed can rollover at a

travelling speed of about 38 mph (61 km/h). It was recommended that design policy for horizontal curves should ensure an adequate margin of safety against both rollover and skidding at the travel speeds actually used by vehicles, and accordingly, the projected ramp operating speed was recommended for use in design when the operating speed exceeds the pre-selected design speed.

Persaud et al. (1993) developed macroscopic and microscopic models for predicting of accident rates using 1988 and 1989 accident data. The macroscopic model used yearly accident rates per kilometre and average daily traffic volumes data from approximately 500 segments of freeways in Ontario (Canada). In the macro level, it was found that, for the same total traffic volume, four-lane freeways have lower collision risk than those with more than four lanes, where the higher collision risk was considered to be associated with rush hour congestion in urban areas. However, no comparison was made between segments with four-lanes and more than four-lanes within the urban context. In fact, the evaluation of relative safety of geometric elements was based on collision rates, which has been subject to wide criticism among researchers, since it is based on false assumption of linear relationship between the collisions and traffic volumes.

Twomey et al. (1993) reviewed past research on the safety of interchange design features in terms of accident rates per 100 million vehicles. It was indicated that the relative safety of entrance and exit terminals is enhanced with geometric designs that provide 800 ft (244 m) or longer acceleration or auxiliary lane and 900 ft (274 m) or longer deceleration lanes. It was concluded that interchange rehabilitation projects, such as lengthening acceleration and deceleration lanes, adding ramp lanes, and optimizing existing or

installing new traffic signals, adding collector-distributor roads, lengthening weave areas, etc. are effective in reducing accident experience.

Bauer et al. (1997) developed several statistical models for interchange ramps and speed-change lanes defining the relationship of traffic accidents to highway geometric design elements and traffic volumes. Three-year accidents (1993-1995) on 551 ramps of the highways in Washington State were used in the modeling attempts using negative binomial regression. Most of the variability was explained by ramp AADT, area type (urban or rural), ramp type (on or off), ramp configuration, and combined length of ramp and speed-change lane (up to the end of taper). The models were statistically significant at 10 to 20% significance level with R^2 value of 0.10 to 0.42.

Khorashadi (1998) evaluated the effect of ramp configuration, type, and geometry on accidents based on three-year accident data on 13,325 ramps on the California State Highways. No statistically significant difference was found in accidents between different ramp types. In general, scissor ramps, rest area ramps, and slip ramps configurations were the most common among the ramp types that experienced high accident rates per million vehicles. Rural ramps accounted for 2.82% of the accidents occurring on rural highways, whereas the urban ramps accounted for 18.41% of the accidents occurring on the urban highways.

Khan et al. (1999) presented accident models, for urban and rural segments separately, using two sets of predictor variables (a) segment length (*SL*) and average daily traffic (*ADT*), and (b) vehicle miles traveled (*VMT*). Models were developed by aggregating as well as disaggregating ten-year accident data on 97 segments over 160 miles (258 km)

of I-15, an interstate highway between Denver and Pueblo, Colorado. The best models were selected based on Pearson's χ^2 statistics. For injury, property damage, and total accidents, the Poisson regression model with log-transformed predictors performed significantly better than other regression models. For fatal accidents, the log normal generalized regression performed better than other regression models examined. Models based on accident data disaggregated for urban and rural segments by median *SL*, *ADT*, and *VMT* performed better than aggregated models. *SL* and *ADT* were found to be more significant for predicting injury and property damage accidents whereas for fatal accidents *VMT* was found to be more significant.

Golob et al. (2004) examined the accidents that occurred on 55 weaving sections covering all three types of weaving (Type A – C) sections in five Orange Country freeways of Southern California. No difference among these three types in terms of overall accident rates over one year (1998) was found. However, there were significant differences in terms of the types of accidents and its severity within these weaving types. It was found that the least severe accidents occur on Type A, followed by Type C. More severe accidents, involving vehicle lane changing to left or right, were found to occur on type B weaving sections, and the root cause was believed to be the disparity in speeds between lane changing movement and through movement.

Kockelman et al. (2004) studied average speed and speed variation patterns to identify any connection between such patterns and crash occurrence. The data was derived from single loop detectors upstream and within 2,000 ft (610 m) of the crash locations on six orange country freeways in California. It was concluded that no evidence had emerged

to support a hypothesis that speed is influencing crash occurrences.

McCartt et al. (2004) examined a sample of 1,150 crashes that occurred on 176 heavily travelled urban interstate ramps in Virginia (US). About half of all crashes occurred when at-fault drivers were exiting and 36% occurred when drivers were entering the interstate freeways. Six percent of crashes occurred on mid-point of access roads or on ramps connecting two interstate freeways. The crash type most frequently associated with exiting was run-off-road and the crashes most common with entering drivers were rear-end or sideswipe/cut-off types. An increase in the ramp design speed, extension of the acceleration lanes, and enforcement were suggested as counter-measures for run-off-road, sideswipe/cut-off, and rear-end crashes, respectively.

Sarhan (2004) successfully developed several models for the prediction of five-year total (fatal + injury + property damage only) collision frequencies using data from 94 segments at 26 interchanges of Highway 417 located in the urban corridor of the City of Ottawa, Ontario. Negative binomial regression was found to fit better the collision data, covering period from 1998 to 2002, over the Poisson regression in terms of over-dispersion parameters. Modelling attempts for "equivalent property damage only (*EPDO*)" collisions were unsuccessful, as no predictor was found to be statistically significant at 95% confidence level. The explanatory variables, for prediction of total collisions, those were found to be statistically significant at 95% confidence level include traffic exposure, type and length of speed-change lane, number of lanes, the weaving types, and advisory speed on exit ramps. The specific significant model for the prediction of number of collisions at merging section was given as:

$$T_{Col} = e^{2.5389+0.0168(Expo)-0.0020(LAcc)+1.6269(AccCo)} \quad (2.5)$$

Where, T_{Col} = Total five-year collisions on a segment, *Expo* = Traffic exposure in million-vehicle-km of travel, *LAcc* = Length of acceleration lane (m), *AccCo* = Code of acceleration lane (0 for limited length type, 1 extended type acceleration lane).

Sarhan (2004) found also that the collisions are expected to increase with increase in traffic volume or exposure, and decrease with increase in length of acceleration lanes. It was also concluded that carrying the full width of the speed-change lane to the gore of the following ramp might increase the number of collisions on the segment between two ramps. Accordingly, in situations that need an increase or decrease in the basic number of lanes, extending the speed-change lanes to next ramp was not recommended based on safety performance. As an alternative, it was recommended that changing the number of lanes should be implemented within the basic section and away from the influence of speed change lanes to improve safety.

Maze et al. (2005) performed a study to characterize traffic safety performance of the expressway segments, in the vicinity of at-grade intersections, based on three-year crash data (1999-2001 for Minnesota and 1998-2000 for Iowa). Two separate models were presented for Minnesota and Iowa State Expressways, respectively. Similar trends were observed between the traffic volumes (*ADT*) and crash density in terms of number of crashes per mile (averaged over three years), crash severity, and proportion of intersection-involved crashes. It was concluded that safety performance of expressway

facilities deteriorates with increasing traffic volumes, suggesting that appreciable safety benefits exist for converting high-volume expressway segments to full access control (grade separation).

2.3 SUMMARY

As evident from the review of design guides and available literature, outlined above, some design criteria are inconsistent and may not reflect the behaviour of current vehicle/ driver population. A limited number of research studies were conducted on the freeway merging behaviour. However, the methodology used and findings in most research were either inappropriate or of limited use for application to geometric design of freeway merge area. A few additional studies were conducted involving safety of the interchange ramps and speed-change lanes. Many of them were based on the inappropriate assumption of linear relationship between traffic volumes and collisions in terms of collision rates. Conclusions or design recommendations resulting from those studies may lead to a practically undesirable or hazardous feature. No research has been found in the available literature attempting to model the operating speed and acceleration behaviours on the SCL. In this research, an attempt has been made to examine the effect of freeway merge area geometry on driver behaviour and to develop predictive models. The developed models are expected to be helpful in selecting appropriate geometry of the entrance terminal for safe and efficient operation of freeway merge areas.

3.0 DATA COLLECTION AND PROCESSING

The data collection process required a number of steps that involved several organizations associated with highway design, operation, management, and law enforcement. Selection and acquisition of appropriate equipment was also a major issue. This chapter presents the data collection and preparation process as well as statistics of collected data. Section 3.1 covers the site selection process including the rationale for this selection and an overview of the sites. Section 3.2 describes the equipment used for speed data collection. Section 3.3 describes the speed data collection while Section 3.4 provides the preparation of the collected speed data. Section 3.5 presents the traffic data. Description of the maps used and the geometric data are provided in Section 3.6. Section 3.7 presents the collision data.

3.1 SITE SELECTION

As mentioned earlier, this study covered several aspects of traffic behaviour on freeway acceleration lanes and the adjacent right lane. Accordingly, data were collected from Highway 417, known as the Queensway, one of the major freeways in Ontario and part of the Trans-Canada highway. Highway 417 is a high speed freeway facility with posted speed limit of 100 km/h. It is aligned in the east-west direction and passes through the City of Ottawa, the national capital and fourth largest city in Canada. It continues in the east through the province of Quebec as Highway 40, and continues in the west as Highway 17. It also intersects with Highways 416 and 174 within the City of Ottawa

area. Highway 417 is the main freeway passing through the City of Ottawa and serving huge commuter traffic. The number of basic freeway lanes varies from two to four in each direction within the city area (total four to eight lanes in the two directions).

The length of the speed-change lanes vary widely among different sections. The eastern and western parts along Highway 417 and within the City of Ottawa can be characterized as urban or sub-urban commercial and industrial zones, while the central part can be characterized as a highly urbanized central business district (CBD). Interchanges outside the urban area serve less traffic compared to those within the urban core, and are generally more widely spaced. On the other hand, within the urban area, the geometry of the interchanges and speed-change lanes is constrained by the limited right-of-way. The acceleration lanes within the city area thus differ in terms of geometry, traffic volume, and/or land use pattern, and are therefore expected to provide sufficient variability in driver behaviour for the study purposes. This led to the selection of the portion of Highway 417 that lies within the City of Ottawa as the study area, as shown in Figure 3.1.

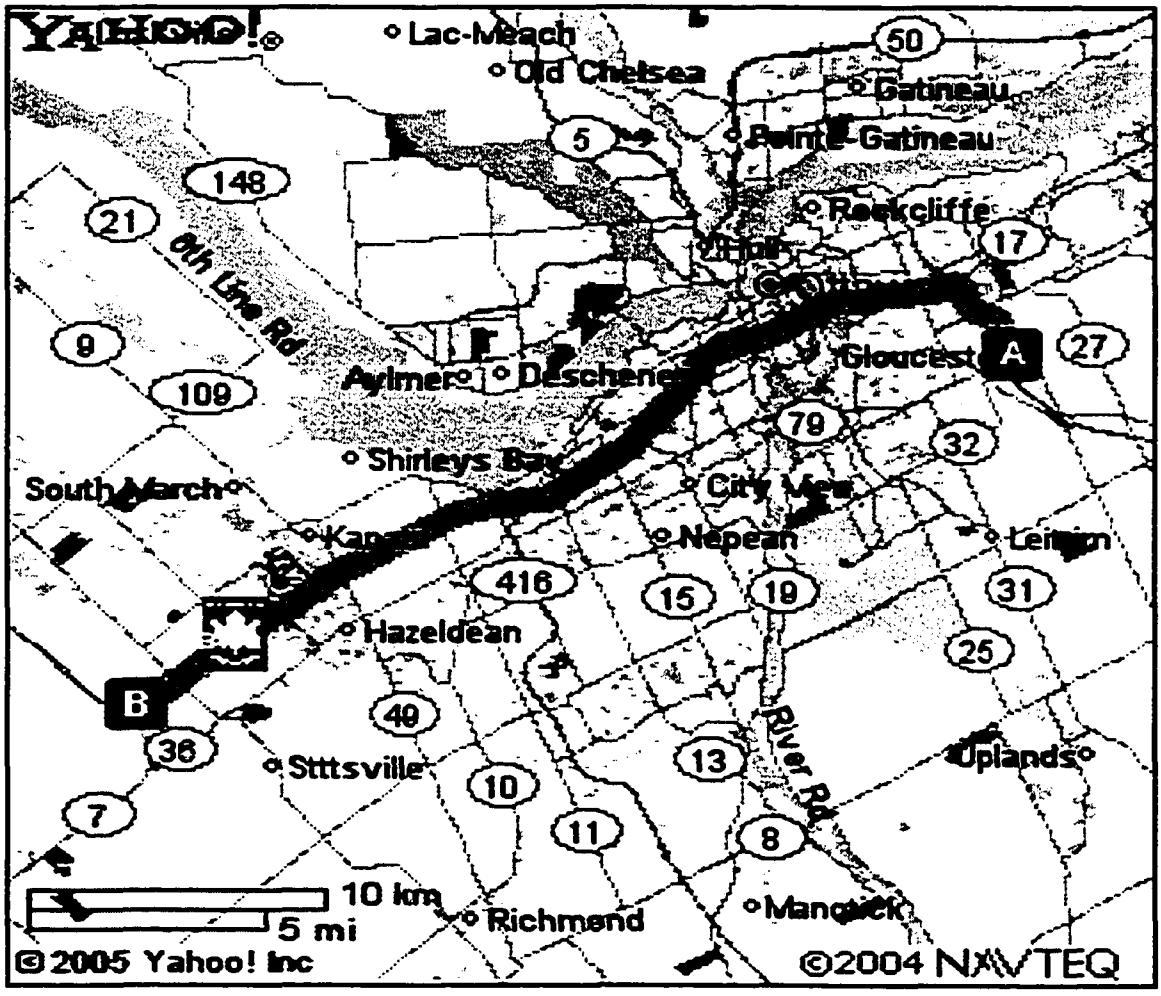


Figure 3.1: Map of the Study Area

There are 27 interchanges with 66 entrances and 27 exits in the two directions within the study area. Excluding the freeway to freeway ramps, there were sixty entrance ramps within the study area. After a preliminary survey, forty acceleration speed-change lanes were selected as candidate sites based on availability of suitable position for data collection, accessibility to the proposed position, and visibility of full speed-change lanes from the proposed position. The proposed equipment positions included overpasses, adjacent building roofs or upper floors with window facing the freeway, mall parking along the freeway, and some spots within freeway right-of-way but far from the shoulder.

Several meetings, correspondence or conversations with MTO, Ontario Provincial Police (OPP), building managements, and secondary surveys were conducted for selecting the appropriate data collection positions. The governing factors in this selection process were:

1. Refusal from MTO: SCL sites at which access to the proposed data collection positions were refused by MTO, due to their concern about safety, were excluded.
2. Driver's Distraction: SCL sites at which equipment positions were readily visible to the driver and may cause interference in driver behaviour were excluded.
3. Permission from building management: Not all building managements permitted access to their facilities, and therefore SCL sites at those locations could not be included.
4. Availability of sidewalk on the overpass: Overpasses without sidewalks were not suitable for equipment/ operator position, and hence SCL sites at those locations could not be included.

Not all acceleration lanes at a particular interchange with sidewalk on overpass could be included because of restricted visibility of the merging areas from the overpasses. This was mainly due to the distant position of the SCL and/ or freeway curvature and/ or the merging area located under the overpass. All these factors limited the selection to 27 acceleration lane sites for inclusion in this study. However, four of the selected sites were abandoned due to limitation of the equipment or traffic condition. These sites are Carling/ Kirkwood NS-W and O'Conner-W, where the laser beam did not pass through the glass

wall of buildings, and Walkley E-E and Palladium N-E, where entering volumes were very low (<five veh/h). Finally, the data collection was ended with 23 acceleration lanes at fifteen interchanges, all of which were observed from overpasses. Noting that the central part of the freeway within the study area passes mostly over the urban arterials, only four acceleration lanes out of the available fourteen sites within the central part were included in the study.

Table 3.1 shows a list of the selected SCL for which speed and traffic data were collected. Out of the 23 SCL, nine were situated in the west-bound and the remaining fourteen were in the east-bound. The selected entrance terminals include both extended and limited length type acceleration lanes with outer connection or loop ramps. The length of the SCL varies from 188 meters (limited length type) to 1366 meters (extended type). The limited length type includes both taper and parallel configurations. The outer connection ramps have flat curvature or are fairly straight type and/ or the ramp curves followed by relatively long flat transition until the beginning point of SCL, while the loop ramps with sharp curvature are followed by relatively short transition segments. Most of the selected SCL are preceded by downgrade ramps (where the freeway passes under the arterials), while only two SCL are preceded by upgrade ramps (freeway passes over the arterials). The mainline AADT (Average Annual Daily Traffic) in one direction, along the selected SCL, ranges from 11,705 to 66,600 veh/day. The ramp AADT ranges from 1,263 to 14,811 veh/day for the selected SCL.

Table 3.1: List of Selected Study Sites

SL.	IC #	Dir.	Ramp and SCL	Data Collection Position	Data Category
1	110	WB	Walkley Rd. W – W	Walkley Rd. Overpass	Merge Speed Only
2	112	EB	Innes Rd. E – E	Innes Rd. Overpass	Merge Speed Only
3	115	WB	St. Laurent Blvd. N – W	Belfast Rd. Overpass	Full Profile
4	117	WB	Vanier Pwy N – W	Vanier Pwy Overpass	Full Profile
5	117	EB	Vanier Pwy N – E	Vanier Pwy Overpass	Merge Speed Only
6	118	EB	Nicholas N – E, Lees – E	Lees Ave. Overpass	Merge Speed Only
7	122	WB	Parkdale NS – W	Parkdale Ped. Overpass	Full Profile
8	126	EB	Maitland Ave. NS – E	Maitland Ave. Overpass	Full Profile
9	127	WB	Woodroffe Ave. NS – W	Woodroffe Ave. Overpass	Full Profile
10	127	EB	Woodroffe Ave. N – E	Woodroffe Ave. Overpass	Merge Speed Only
11	127	EB	Woodroffe Ave. S – E	Woodroffe Ave. Overpass	Merge Speed Only
12	129	EB	Greenbank Rd. S – E	Greenbank Overpass	Merge Speed Only
13	130	EB	Richmond Rd. S – E	Richmond Rd. Overpass	Full Profile
14	134	WB	Moodie Drive N – W	Moodie Dr. Overpass	Full Profile
15	138	WB	March Rd. N – W	Castlefrank Ped. Overpass	Merge Speed Only
16	138	EB	Eagleson Rd. S – E	March/Eagleson Overpass	Full Profile
17	139	EB	Castlefrank Rd. NS – E	Castlefrank Rd. Overpass	Full Profile
18	140	WB	Terry Fox Drive N – W	Terry Fox Dr. Overpass	Full Profile
19	140	WB	Terry Fox Drive S – W	Terry Fox Dr. Overpass	Merge Speed Only
20	140	EB	Terry Fox Drive N – E	Terry Fox Dr. Overpass	Merge Speed Only
21	140	EB	Terry Fox Drive S – E	Castlefrank Overpass	Full Profile
22	144	EB	Carp Rd. N – E	Carp Rd. Overpass	Merge Speed Only
23	144	EB	Carp Rd. S – E	Carp Rd. Overpass	Full Profile

For fifteen acceleration lanes, vehicle speeds were measured over the full length of SCL, from the vicinity of the physical gore to the end of taper; such sites are designated here as "Full Profile". For the remaining eight acceleration lanes, vehicles could be seen from a

point some distance downstream the start of SCL but upstream the painted nose, and only merge speeds could be measured; such sites are designated here as "Merge Speed Only". Figure 3.2 and Figure 3.3 show sample pictures of SCL for "Full Profile" and "Merge Speed Only" data, respectively. The beginning of SCL in this study refers to the point where the pavement edges of freeway mainline and ramp terminal are 1.25 m apart.

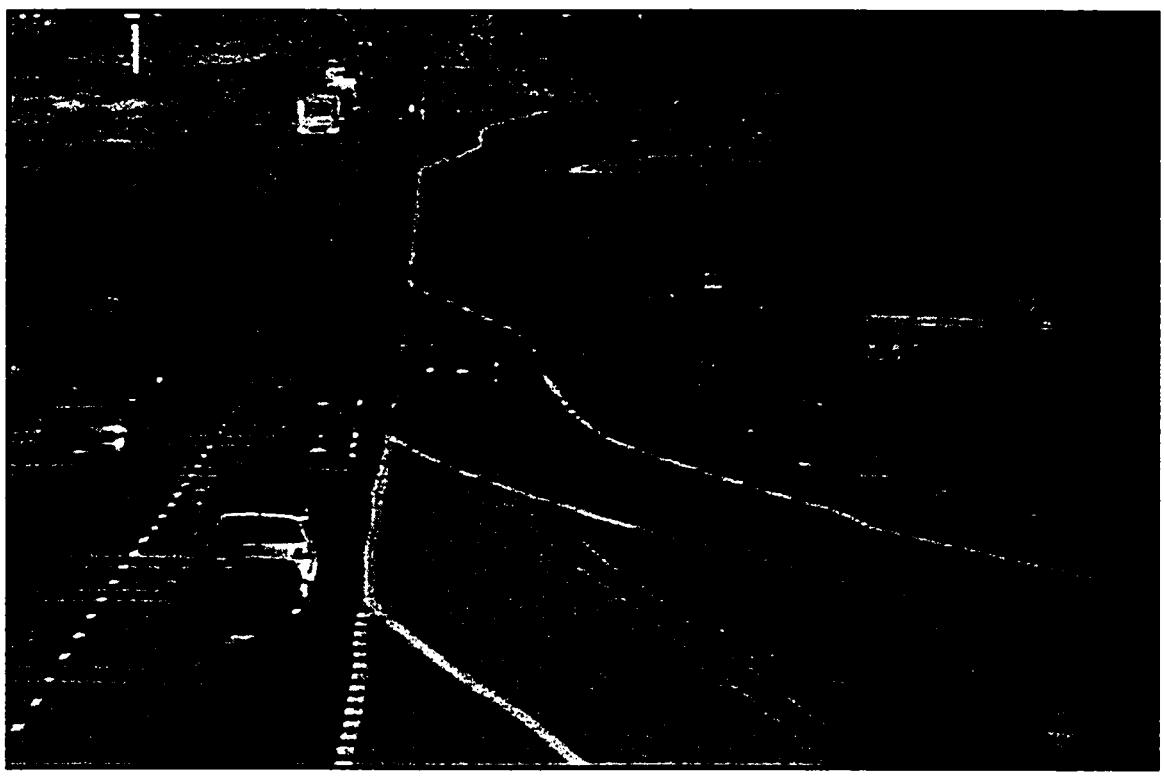


Figure 3.2: Typical SCL with "Full Profile" Speed Data

Figure 3.3: Typical SCL with "Merging Speed Only" Data

3.2 EQUIPMENT FOR SPEED DATA COLLECTION

This research study involved speed and traffic data collection in the field. Finding and selecting the appropriate equipment is an important step before any field work. An equipment that is capable of measuring as well as storing individual vehicle speed and distance in dense freeway traffic condition, without having to be on road or its right-of-way, was the main basis for the selection. Due to their several advantages, laser speed guns, manufactured by Laser Atlanta (US), were selected, acquired, and used for speed data collection in this research. Figure 3.4 shows a picture of the Laser Atlanta laser speed gun used in this study.

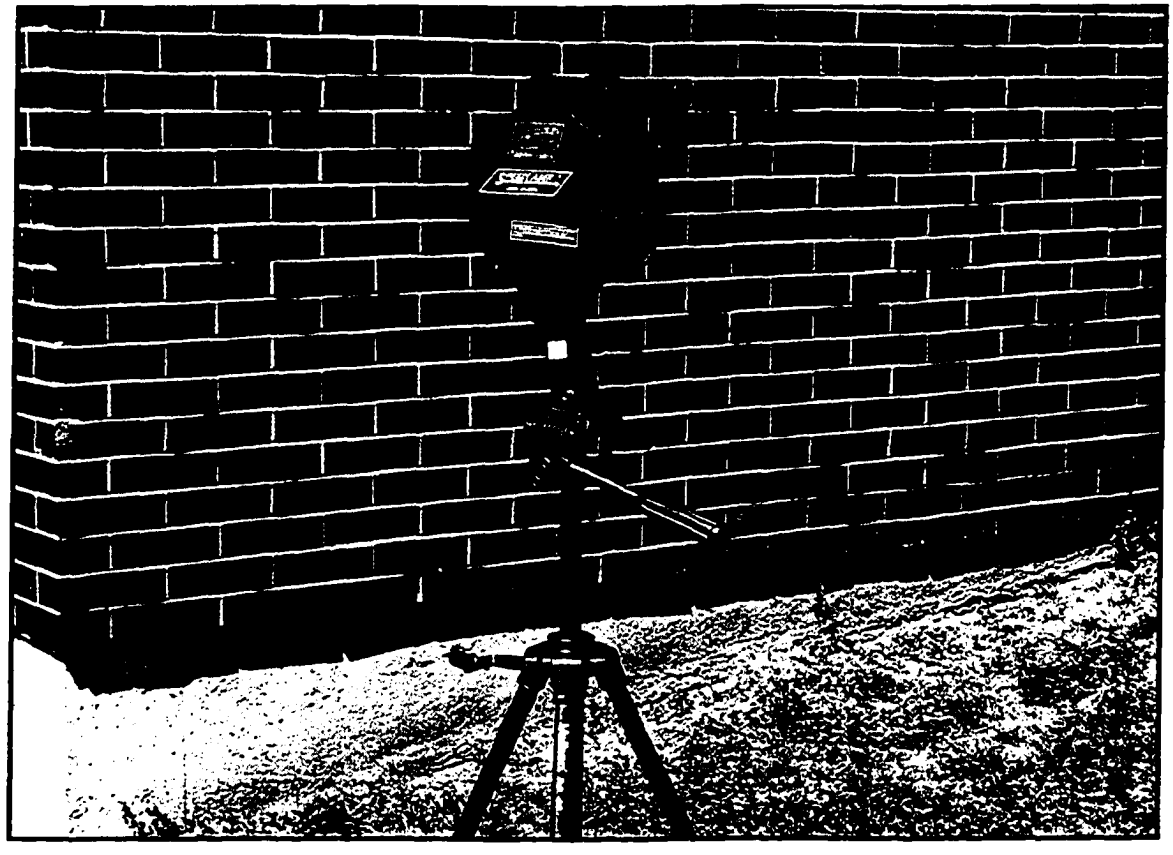


Figure 3.4: Laser Speed Gun Manufactured by Laser Atlanta (US)

This equipment is capable of measuring the distance to a vehicle position and speed at each position as it traverses along its path, and records each speed and distance data point with time on a PCMCIA Type II SRAM card. The laser gun has speed accuracy of ± 1.6 km/h (1.0 mph), maximum range of 1,220 m (4,000 ft) for speed measurement and speed range of 16 to 320 km/h (10 to 200 mph). It can measure distance within a range of 1.5 to 9,146 m (5 to 30,000 ft) with an accuracy of ± 0.3 m (1.0 ft). The precision of speed and distance measurements are 0.1 km/h, 0.01 m, respectively. The time is recorded to two decimal places of a second (0.01 second). When the laser is triggered, the speed, distance, and time profiles of the tracked vehicle are automatically stored in an individual null file, created and stored in the PCM card, in a sequential order. The data or profiles stored in

the PCM card could then be transferred to a laptop computer with SRAM drive, for further processing.

3.3 SPEED DATA COLLECTION

After acquiring the selected equipment, a schedule was prepared for field data collection in coordination with authorities/personnel responsible for the selected study sites and/or data collection positions. Two researchers collected the field data side-by-side for two concurrent research studies.

3.3.1 Access

Overpasses and adjacent buildings were selected as the data collectors' position for the speed and traffic data collection for two concurrent research studies. The selected freeway is owned by the Ministry of Transportation (MTO), Ontario, while the overpasses are City of Ottawa's property. Therefore, consents were taken from both MTO and the City of Ottawa. The adjacent buildings are private properties, and therefore each property owner/ management was contacted and permission was obtained for access to their facilities. Part of the study area, from Eagleson Road to Terry Fox Drive interchanges was designated as construction zone and Morrison Hersfield (Consulting Engineers) was responsible for this section. Their consent was also taken for data collection within this section.

For being a provincial property, Ontario Provincial Police (OPP) is responsible monitoring the freeway, while the Ottawa Police is responsible for monitoring the

municipal roads and highways including overpasses on freeways. Therefore, both OPP (Ottawa region) and Ottawa Police were informed about the study and data collection activities. Weekly schedules and changes in schedule for data collection were provided to all concerned parties. In addition, Ottawa Police was informed on a daily basis just before going out for data collection.

3.3.2 Data Collection Time

Data were collected during the months of October - November (2004), during off-peak hours, (between 10:00 am to 3:00 pm), and on dry pavement condition. Data collection activity schedules were updated based on weather forecast. No data were collected during rain, snow or fog. No data were collected when the wind speed was high enough to shake the equipment and losing the target, or to likely affect vehicle speeds.

3.3.3 Data Collection Process

The field data were collected jointly by two researchers engaged in two concurrent studies, with mutual cooperation, as the field data collection for a particular SCL required two persons to act together with two laser guns. Over two hundred null (zero-link) files were created and saved in laptop for transferring to the PCMCIA SRAM card after each use. On each scheduled day, with favourable weather conditions, Ottawa Police was informed and the data collection activity was set. The equipment was mounted on tripod and set on the sidewalk of the overpass. The equipment positions were selected to maximize the visibility and distance coverage along the vehicle path. Attempts were also made to minimize the horizontal offsets with respect to vehicle trajectory. It should be

noted that the vehicles were targeted either at the back or front surface depending on equipment position with respect to the direction of vehicle movement.

Once the final equipment positions were set, the horizontal offsets of the equipment positions, with respect the centre-line of respective lanes (SCL or RL), were measured using a measuring tape. The vertical offsets from the pavement surface to the top of the sidewalls were measured using the laser gun, except for two pedestrian overpasses. At the Parkdale pedestrian overpass, the vertical clearance between top of bridge slab and target surface was calculated using GPS equipment. The vertical clearance between pavement surface and top of bridge deck at Castlefrank pedestrian overpass was provided by Morrison Hersfield. The portions of the vertical offsets between the top of wall or slab and centre of two lenses of speed laser were obtained using a measuring tape.

A data collection worksheet was prepared in advance and used to record the data collection date; SCL site information such as highway direction, interchange (IC) and SCL description; vertical and horizontal offsets; data categories such as "full profile" or "merge speed only", vehicle number and its movement such as entering vehicle on extended SCL did "not merge", right lane (RL) vehicles changed lane to the left or diverged onto the extended SCL; vehicle type (HV marked); RL vehicle "impeded" by entering vehicle; and other special cases such as cancel (out), obstructed, etc., and the computer file number used to save the recorded profiles at each SCL site. Table 3.2 shows a sample worksheet used to record the speed data collection.

Table 3.2: Sample Worksheet for Recording Speed Data Collection

HWY 417 SPEED DATA COLLECTION: MERGING & ACCELERATION BEHAVIOURS

DATE: *Oct. 14/04* HWY DIR.: *WB* IC NO. : *115* IC NAME: *St. Laurent Blvd.*

LANE DESCRIPTION: *St. Laurent Blvd. N-W, Extended* DATA TYPE: *Full Profile*

OPTR.'S POSITION: *Belfast Road Overpass* VERTICAL OFFSET (m): *SCL = 8.25, RL = 8.25*

HORIZONTAL OFFSET (m): *SCL = 0, RL = 0* COMPUTER FILE: *St. Laurent (N-W)*

SPEED-CHANGE LANE				*RIGHT LANE*					
Veh. No.	H. V.	Not Merged	Others	Veh. No.	H. V.	Impeded	Merge Left	Diverged	Others
001			Start	001					Start
↓				↓					
004		x		004		x			P-1 Out
↓				↓					
012	x			007	x	x			
↓				↓					
066			Out	015					Out
↓				↓					
105		x	End	018		x	x		
				047				x	
				↓					
				103					End

On the SCL, each vehicle speed profile was captured from the proximity of physical gore, where possible, and kept targeting until it merged onto the freeway right lane. The merging point was taken as the position where ramp vehicles occupy the right lane, which corresponds to the vehicle position halfway between the SCL and right lane. For "merge speed only" cases of SCL, vehicles could not be targeted from the gore, and merging speeds were mainly measured. For freeway right lane, each vehicle was tracked from the proximity of physical gore to the end of SCL. However, for those few cases of right lane vehicles, diverging into the extended SCL or changing lane to the inner (left) lane after travelling on the right lane for substantial distance downstream the gore, speeds were measured until the lane change manoeuvre started. In addition, right lane speeds were also collected from a section upstream the gore area, where possible, to compare with the right lane speeds downstream gore (along the SCL). The typical speed profile for vehicle on freeway right lane (along the SCL) and speed-change lane is shown in Figure 3.5 and Figure 3.6, respectively. The distances shown in the figures refer to the distance of vehicle positions from the equipment position.

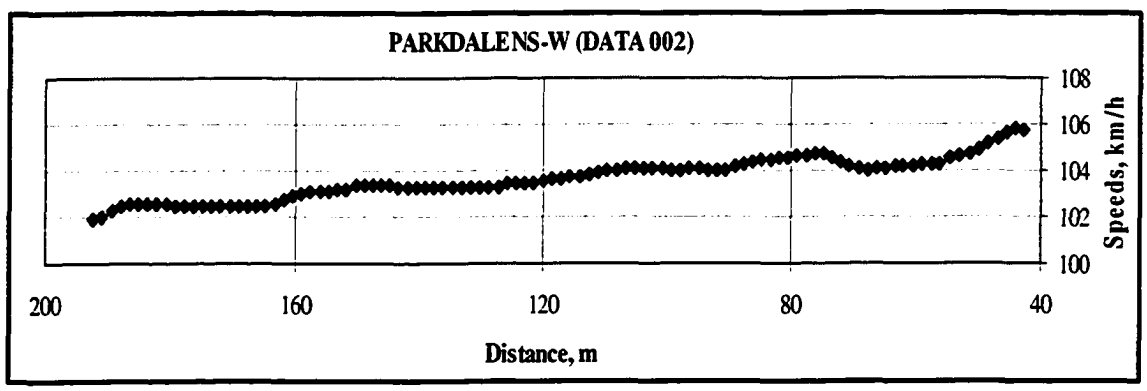


Figure 3.5: Typical Speed Profile of Right Lane Vehicles

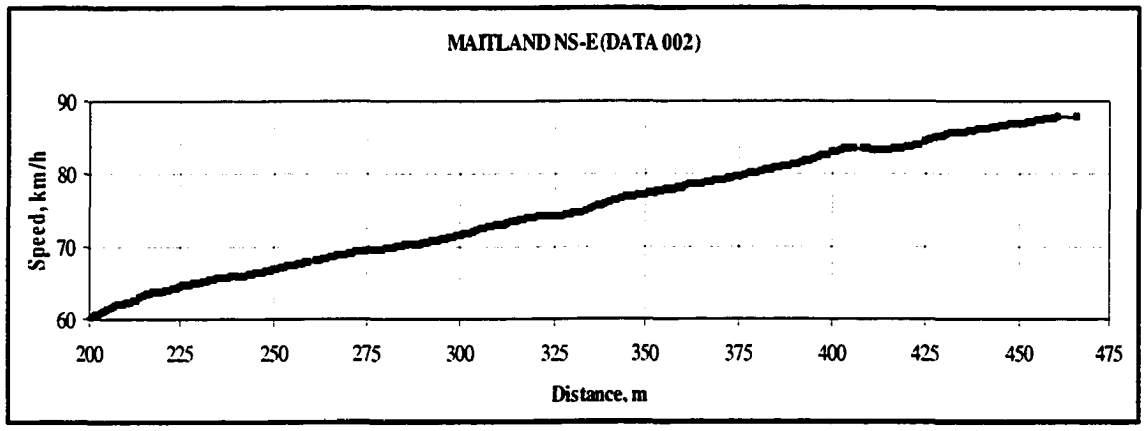


Figure 3.6: Typical Speed Profile of Vehicle on SCL

At each site, 75 to 150 vehicle speed profiles were measured on each of the SCL and right lane during a period of 75 to 150 minutes, spent only in targeting the vehicles (excluding the travel and equipment setting/ preparation time), depending on the traffic volume at the respective sites. Both passenger car and heavy vehicle (bus/truck) speeds were measured randomly at each site. The data noted as "out" or "cancel" were the vehicle profiles for which the merge speed could not be captured or was obstructed by another vehicle in front or back of the target vehicle. The impeded vehicles are the right lane vehicle cases when the ramp vehicle merged in front it with a relatively short gap, even though the impeded vehicles may or may not slow down. Such a gap was estimated visually as the vehicles may slow down without applying the brake.

In some cases, the target was lost due to obstruction by another vehicle or shaking of the equipment by a heavy vehicle movement on the overpass. Effort was made in tracking back the target vehicle; otherwise the respective profile was noted as "out" or "cancel", as mentioned earlier. Any error in targeting back such as targeting another vehicle instead of original target vehicle were examined during the data preparation phase and corrected

for error or full profile deleted as considered appropriate. When the data collection at a particular site was completed, the files with vehicle profiles were transferred to a laptop computer brought to the site. The vehicle profiles noted as "cancel" or "out" during data collection were deleted immediately after transferring to the laptop. New null files were entered in PCM cards for next use.

3.4 DATA PROCESSING

The speed profile data saved, after deleting those mentioned in Section 3.3, were further examined and prepared for use in the analysis and modelling process. In addition to speed, the laser speed gun used in data collection recorded other information such as distance and time. The following sections outline the data preparation process.

3.4.1 Correction for Offsets

The speed and distance readings at each point along a vehicle path were corrected for the vertical and horizontal offsets, as applicable, using the techniques of vector mechanics. These corrections are popularly known as cosine corrections. Figure 3.7 shows graphically the correction for vertical offset.

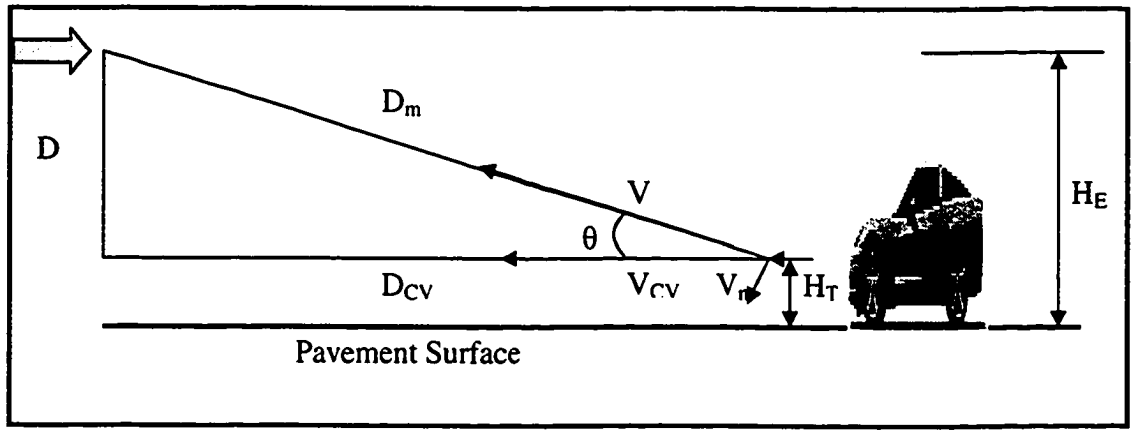


Figure 3.7: Correction for Vertical Offset

In Figure 3.7, H_E = height of the equipment from the pavement surface (m); H_T = height of the target from the pavement surface (m); D_V = vertical offset (m) = H_E - H_T; D_m = measured distance (m); θ= vertical angle, D_{CV} = vertical offset corrected distance (m); V_m = measured speed (km/h); V_{CV} = vertical offset corrected speed (km/h); and V_n = normal component of speed. Using vector mechanics:

$$\theta = Sin^{-1}(D_V / D_m) \tag{3.1}$$

$$D_{CV} = D_m Cos\theta \tag{3.2}$$

$$V_{CV} = V_m / Cos\theta \tag{3.3}$$

Alternatively,

$$D_{CV} = \sqrt{(D_m^2 - D_V^2)} \tag{3.4}$$

$$V_{CV} = V_m / Cos\theta = V_m / (D_{CV} / D_m) = (V_m * D_m) / D_{CV} \tag{3.5}$$

The target height refers to the average height of target surface, on the back or front of the vehicle, above the roadway surface. The heights of target surface for a typical sedan, passenger mini-van and pickup (light) truck were measured and average height, separate for back and front targeting, was calculated for use in correction for vertical offset. Target heights for heavy vehicles including dump trucks, buses, medium and large size delivery van were also measured and averaged separately for back and front targeting. For passenger cars, the average target heights were 0.5 and 1.0 m for front and back targeting, respectively. For heavy vehicles, the average target heights were 1.0 and 2.0 m for front and back targeting, respectively. Corrections were applied for passenger cars and heavy vehicles as well as front and back targeting separately, as applicable. The vertical offset corrected speeds and distances were further corrected for horizontal offsets, where applicable, as shown in Figure 3.8.

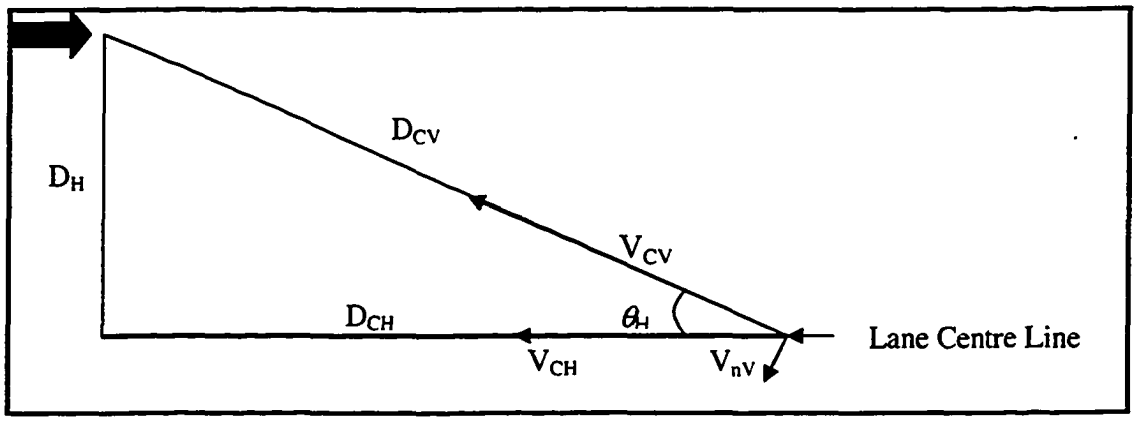


Figure 3.8: Correction for Horizontal Offset

In Figure 3.8, D_H = horizontal offset between the centre of the equipment position and the centre-line of the lane on which vehicles were targeted (m); θ_H = horizontal angle; D_{CV} = vertical offset corrected distance (m); V_{CV} = vertical offset corrected speed (km/h); D_{CH} =

horizontal offset corrected distance (m); V_{CH} = horizontal offset corrected speed (km/h); and V_{nV} = normal component of vertical offset corrected speed. Using the same technique:

$$D_{CH} = \sqrt{\left(D_{CV}^2 - D_H^2\right)} \tag{3.6}$$

$$V_{CH} = (V_{CV} * D_{CV}) / D_{CH} \tag{3.7}$$

It should be noted that the offsets (5.4 to 9.4 m) were very small compared to the distance of vehicle positions from the equipment. Applying the correction for the vertical as well as horizontal offsets, vehicle speeds were found to increase by 0 to less than 1 km/h.

3.4.2 Correction of Speed Profile and Acceleration Calculation

As mentioned earlier, the laser speed gun precision was 0.1 km/h for speed, 0.01 m for distance, and 0.01 second for time. With these precisions, the speed readings were sometimes recorded with no change in time or distance readings. Therefore, a criterion was required to set a gap between successive readings before using the speed profiles in the analysis of traffic behaviour.

One of the objectives of this study is the analysis of acceleration behaviour on the speed-change lanes. Since the laser speed gun, used for data collection, is capable of measuring the speed and distance, and records the time as well, it was decided to calculate the acceleration rates based on two formulas of kinematics and use the average of two accelerations in the analysis. The formulas are:

$$a = v_2^2 - v_1^2 / 2d \tag{3.8}$$

$$a = (v_2 - v_1) / t \tag{3.9}$$

Where, v_1 = speed at first position of two successive positions (m/s); v_2 = speed at second position of two successive positions (m/s); d = distance between two successive positions (m); t = travel time between two successive positions (s); a = acceleration between two successive positions (m/s^2)

The above acceleration formulas were transformed to calculate the acceleration in m/s^2 from speed (corrected for offsets) in km/h, distance (corrected for offsets) in meters and elapsed time in seconds as follows:

$$a = 12.96 * (V_2^2 - V_1^2) / 2d \tag{3.10}$$

$$a = 3.6 * (V_2 - V_1) / t \tag{3.11}$$

After calculating the accelerations using Equations 3.10 and 3.11, and looking at the values in two acceleration columns it was found that the differences in two acceleration values were sometimes very high, even infinite for successive readings with no change in distance or time values, which are impractical. An interval between two successive speed readings was then set so that acceleration values in two columns are close to each other, and not differing by more than single decimal, in general. Accordingly, the intermediate reading(s) were deleted taking care that the deletion should not increase the difference. The same procedure was applied to both of the speed-change and right lane speed profiles. The justification of choosing this interval is that the two formulas gave matching

(closer in values) accelerations implying that the interval is long enough to capture changes in vehicle manoeuvres.

The average of two acceleration values was taken as the acceleration profile of each vehicle. Figure 3.9 shows a sample of acceleration profile of a vehicle on SCL. The mean, maximum, and overall acceleration rates are indicated in the mentioned figure. The mean and maximum acceleration rates refer to the mean and maximum of all successive accelerations on SCL, respectively. Overall acceleration of a vehicle is the acceleration rate based on two boundary readings, at the beginning of SCL and the merging point.

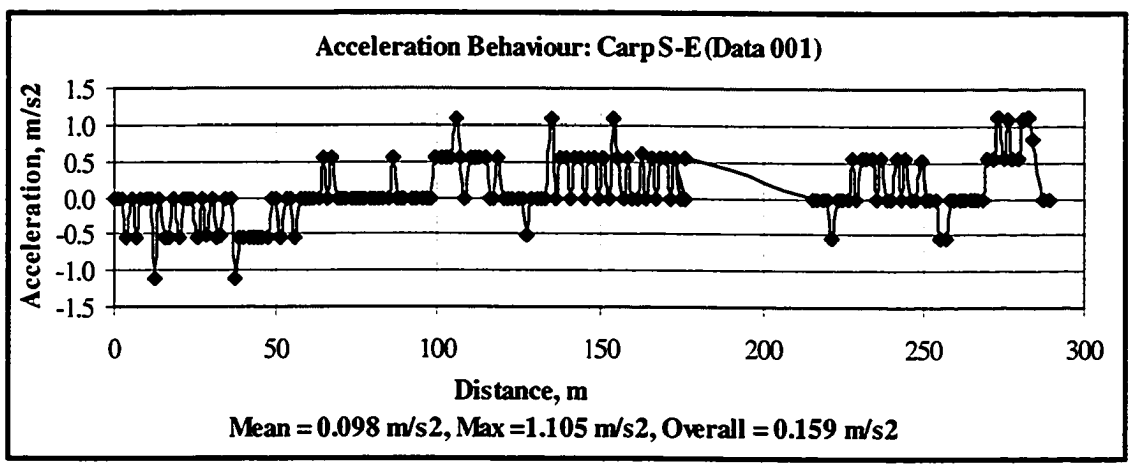


Figure 3.9: Sample Acceleration Profile on SCL

As shown in Figure 3.9, a typical vehicle's acceleration profile takes an oscillating shape, indicating high fluctuation in vehicle acceleration on the SCL. While no specific guidelines were available to set interval between successive readings, two more options were taken into consideration. These include establishing a time interval or a speed interval between successive readings. Figure 3.10 shows a sample acceleration profile with 0.3 second time interval between successive readings, selected based on the data

acquisition time of the laser equipment. With such an interval, the mean acceleration value as compared to the original interval established for matching acceleration values (Figure 3.9) changed from 0.098 to 0.120 m/s^2 in this example. The maximum acceleration value changed very significantly, from 1.105 to 0.733 m/s^2. It was seen that the longer the time gap the smaller the maximum acceleration value, while the changes in mean acceleration showed no particular trend. Deletion of readings to create such a gap would also result in neglecting some sections with zero and or constant accelerations.

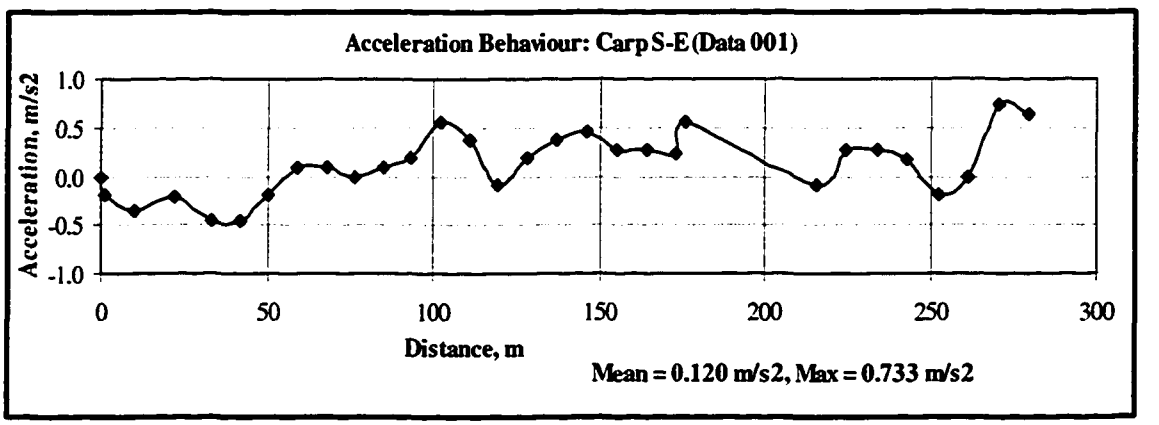


Figure 3.10: Acceleration Profile with Longer Time Gap

Another option was to establish a (higher) speed interval between successive readings, such as 2 km/h as the equipment has accuracy of 1.6 km/h. This option resulted in flatter oscillations, with deletion of over 90% of the readings of the speed profiles. In addition, deletion of readings to create such an interval would result in neglecting sections with zero and/ or constant accelerations, which affects the mean, while maximum acceleration tends to get closer to the overall acceleration. After comparison of the possible options, it was decided in this research to adopt the first option that gives matching accelerations (Figure 3.9).

Another limitation of mean and maximum accelerations, in addition to that associated with interval between successive readings, is that they did not cover section(s) where the target was lost for some distance, in few vehicle cases. Overall acceleration was considered to be the best indicator of vehicle acceleration on SCL, which takes into account only the two boundary readings and uses the whole distance and time in acceleration calculation. On the other hand, a vehicle may decelerate and/ or travel at constant speed between these two boundary points. These changes in driving manoeuvres are not accounted for in overall acceleration. Nevertheless, all of the three acceleration rates were used in the analysis and compared whether they represent the vehicle's manoeuvre on the SCL.

In addition to setting the intervals between the successive readings, each vehicle profile was carefully examined and evaluated for any discrepancy such as tracking more than one vehicle in a single profile, which would be evident from abnormal, abrupt changes between successive readings, changes in the sequence of distance and/ or time readings. Such discrepancies were corrected by examining the speed trend in each profile and deleting the reading(s) that appeared to be part of another vehicle profile. If continuity in speed profile of a vehicle was doubtful, the whole vehicle profile was deleted from the data set. It should be noted that such a problem had occur sometimes where target vehicle positions were very far from the equipment position, and target was lost due to obstruction by another vehicle or shaking of the equipment, as mentioned in Section 3.3.

As mentioned earlier, the lengths of the speed-change lanes were measured from the gore where the ramp and mainline pavement edges are 1.25 m apart, whereas the vehicles were targeted from the proximity of gore nose or physical nose for full profile data. Each

vehicle speed at the point where two pavement edges are 1.25 apart was obtained by interpolation between readings behind and in front this point. Then, the portion of the speed profiles upstream the beginning of SCL was deleted. Figure 3.11 shows an example of the profiles of all vehicles on the Maitland NS-E SCL where locations of the equipment, physical nose, beginning of SCL and end of SCL are marked. The figure shows that all vehicles had merged before the end of SCL. Only readings between the start and end of SCL were taken for further analysis. This was done for speed profiles on all SCL used in this study. Table 3.3 shows a sample of corrected profile (reduced to fit in a page) of a vehicle, with the headings of different columns are explained at the bottom of the table.

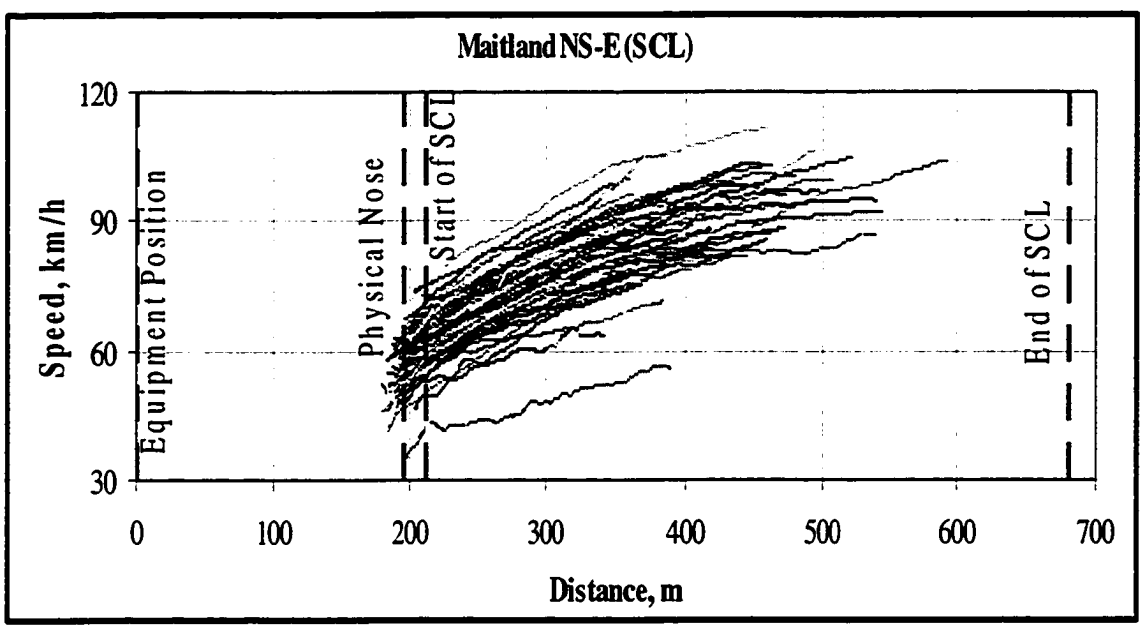


Figure 3.11: Treatment of Speed Profile

Table 3.3: Typical Profile of a Vehicle on SCL

R_Dist.	*R_Speed*	*R_Time*	*C_Dist.*	*Dist_Gore*	*C_Speed*	*E_Time*	*Acc_S_D*	*Acc._S_T*	*Avg. Acc.*
469.68	-105.1	27:11.0	469.62	-0.38	105.11	0.00			
			*470.00	0.00	105.11	0.01			
471.16	-105.1	27:11.0	471.10	1.10	105.11	0.05	0.000	0.000	0.000
472.62	-105.1	27:11.1	472.56	2.56	105.11	0.10	0.000	0.000	0.000
474.08	-105.1	27:11.1	474.02	4.02	105.11	0.15	0.000	0.000	0.000
475.53	-105.0	27:11.2	475.47	5.47	105.01	0.20	-0.560	-0.556	-0.558
477.00	-105.0	27:11.2	476.94	6.94	105.01	0.25	0.000	0.000	0.000
478.45	-104.9	27:11.3	478.39	8.39	104.91	0.30	-0.559	-0.556	-0.558
↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
752.80	-110.2	27:20.6	752.76	282.76	110.21	9.61	1.082	1.111	1.097
754.35	-110.4	27:20.6	754.31	284.31	110.41	9.66	1.098	1.111	1.105
757.46	-110.7	27:20.7	757.42	287.42	110.71	9.76	0.823	0.833	0.828
759.01	-110.7	27:20.8	758.97	288.97	110.71	9.81	0.000	0.000	0.000
760.57	-110.7	27:20.8	760.53	290.53	110.71	9.86	0.000	0.000	0.000
Mean					***105.61***		*0.096*	*0.100*	**0.098**
Maximum					***110.71***		*1.098*	*1.111*	**1.105**
Overall Average							0.160	0.158	**0.159**

R_Dist. = Recorded Distance (m), R_Speed = Recorded Speed (km/h), R_Time = Recorded Time, C_Dist. = Corrected Distance (m), Dist_Gore = Distance of vehicle from Gore (start of SCL) in meters, C_Speed = Offset Corrected Speed (km/h), E_Time = Elapsed Time (Sec), Acc_S_D = Acceleration based on Speed & Distance (Equation 3.9.8), Acc_S_T = Acceleration based on Speed & Time (Equation 3.9.9), and Avg. Acc. = Average Acceleration which is average of Acc_S_D and Acc_S_T (m/s^2)

** Start of SCL (distance from equipment position, m)*

3.4.3 Analysis Parameters of Vehicle Behaviour

After applying the correction for offsets and processing the speed profiles as outlined in the previous sections, the parameters listed in the following were calculated from each vehicle speed and acceleration profile for use in the analysis of traffic behaviour.

a) Average speed of each right lane vehicle.

b) Merging speed of each entering vehicle.

c) Speed at the start of SCL (gore speed) for full profile SCL vehicles.

d) Overall acceleration on the SCL.

e) Maximum acceleration on the SCL (maximum of all successive accelerations).

f) Mean acceleration on the SCL (mean of all successive accelerations).

g) Merging distance (distance to merging point from the beginning of SCL).

3.4.4 Frequency Distribution of Vehicle Data

Figure 3.12 shows the frequency distribution of the number of entering vehicles for which speed data were collected, excluding those deleted during data processing. The same figure also shows the frequency distribution of the number of right lane vehicles data entered in the analysis.

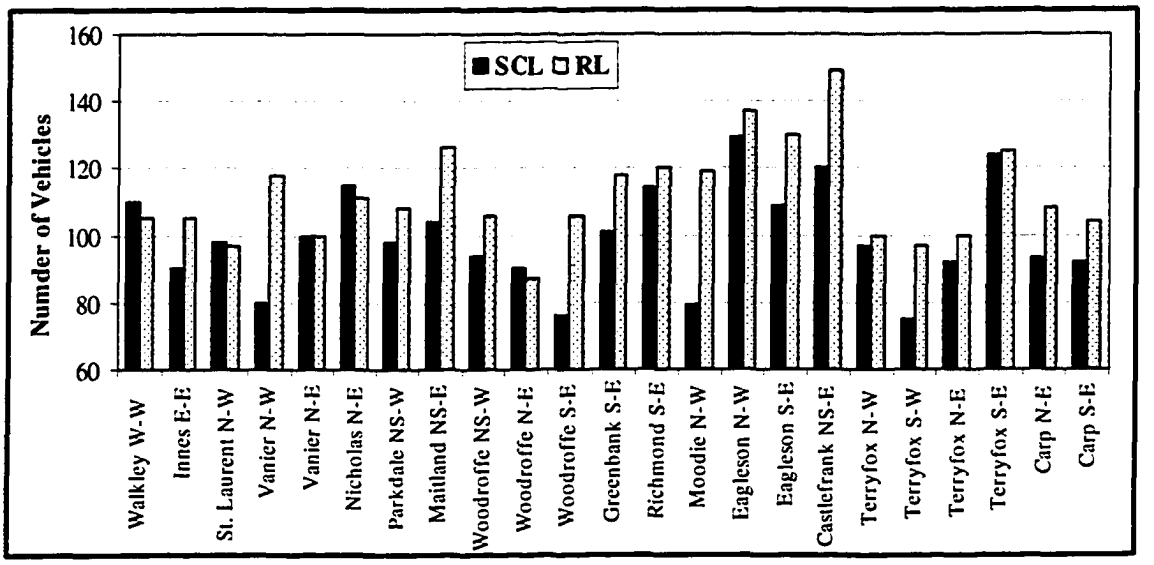


Figure 3.12: Frequency Distribution of Entering Vehicles Data

3.4.5 Distribution of Vehicle Movement

This section presents the distribution of vehicle movement on freeway right lanes in the vicinity of the SCL as well as on the SCL. Figure 3.13 graphically shows a sample of vehicle distribution and their movements at two entrance sites for which speed data were collected, and used in the analysis. Figures showing these distributions for all of the entrance sites, used in this study, are placed in Appendix A. The numbers along or followed by arrows indicate the number of vehicles, and the arrow indicate the direction of vehicle movements. The number in the parenthesis indicates the mean merging speed of all the entering vehicles from a particular SCL or mean speed of all right lane vehicles, where speed of each right lane vehicle represents the average speed along the SCL or upstream the gore, as applicable.

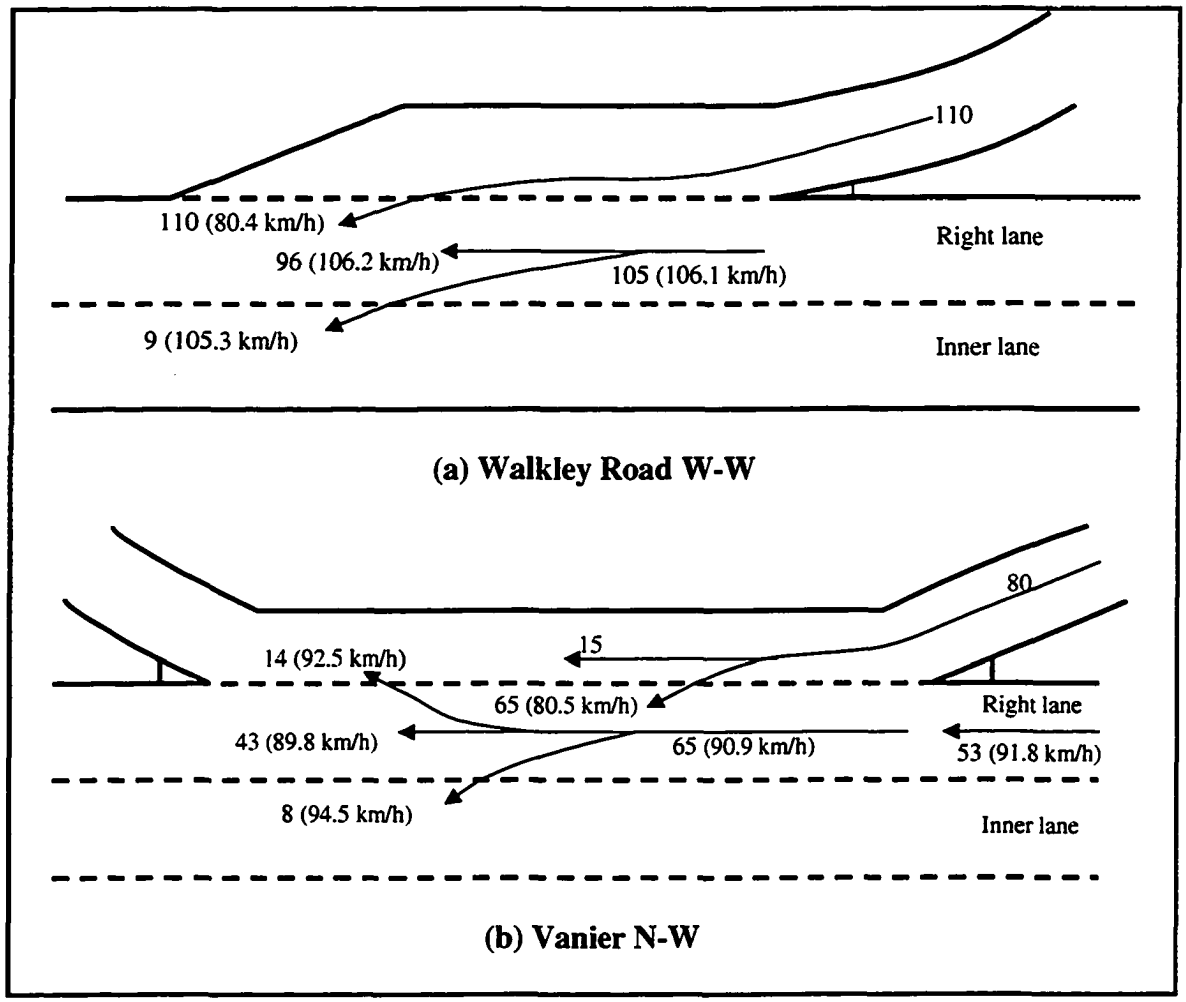


Figure 3.13: Sample Distribution of Vehicles and their Movements

For example, Figure 3.13(a) "Walkley W – W" shows the distribution for a limited length type SCL site. It shows that all 110 ramp vehicles merged into the freeway and the mean merging speed of these vehicles is 80.4 km/h. The mean speed of the 105 right lane vehicles, collected from downstream the gore, is 106.1 km/h. It also shows that 96 vehicles travelled straight and five vehicles merged left (changed lane) into the inner lane. The mean speeds of these two sets are 106.2 km/h and 105.3 km/h, respectively. Figure 3.13(b) "Vanier N – W" shows a sample distribution for an extended type SCL site. It shows that 15 out of the 80 ramp vehicles did not merge within the range of the

equipment, and no merging speed is indicated for them. Figure 3.13(b) also shows that 65 right lane vehicles speed data were collected from section downstream the gore (along the SCL), while 53 right lane vehicles speed data were collected from upstream (behind) the gore. The distribution of the 65 right lane vehicles downstream the gore is also shown indicating that 14 vehicles had diverged into the extended SCL. The ramp vehicles that did not merge within the range of equipment were excluded from the analysis.

The number of right lane vehicles that merged left and/ or diverged on extended SCL was quite small to make any statistical comparison with other vehicle groups. Therefore, comparison was made only between all movements and straight movement (excluding those turning left and/ or diverging). The frequency distributions of right lane vehicles mean speeds (along the SCL), for different movements, are shown in Figure 3.14. For all movements (all directions together), the right lane vehicles mean speeds along the SCL ranged from 89.2 to 109.7 km/h. For straight movement, the mean speeds ranged from 88.7 to 109.7 km/h.

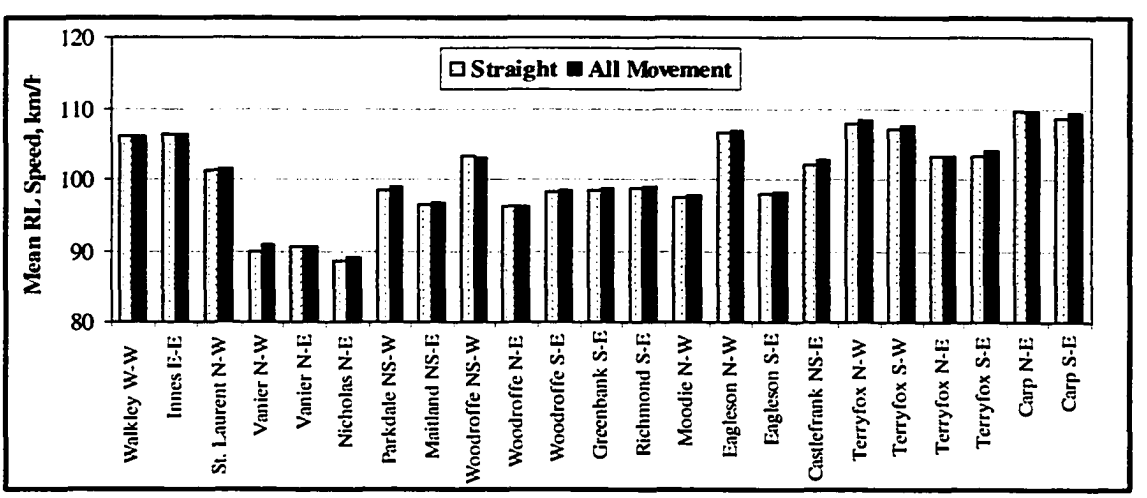


Figure 3.14: Distribution RL Vehicles Mean Speed for Different Movements

The comparison of mean speed of all right lane vehicles along the SCL (all movements together) with that of straight vehicles (excluding those turning left and/ or diverging) showed that the differences range 0 to 1.1 km/h. A *t*-test showed that these differences are not statistically significant even at a 0.01 level of significance. Furthermore, the vehicles turned left and/ or diverged on SCL are part of right lane vehicles until it turned left and/ or diverged, and travelled a significant distance downstream the gore before turning left and/ or diverging. Therefore, combined speeds of all right lane vehicles were taken for further analysis.

3.5 TRAFFIC DATA

3.5.1 Prevailing Traffic Data

Traffic volume data were recorded during the speed data collection to examine the relationship between speed/ acceleration behaviour and prevailing traffic conditions, in addition to the AADT. Accordingly, the prevailing traffic data were compiled from 30 to 45 minute video recordings using a video camera mounted on a tripod adjacent to the laser gun settings. The video camera was set to record the traffic movement on both the SCL and right lane (RL). The traffic video at each SCL site was then transferred to a DVD using a DVD writer for easy play-back later. The video writing process took 3 to 5 hours for each lane depending on the length of recording.

Traffic volumes on SCL as well as right lane were counted for the whole duration of each video recording (30 to 45 minute) at a slow speed. The numbers of passenger cars, buses and trucks, and recreation vehicles (RV) were determined. The equivalent hourly

volumes (pc/h/lane) were obtained from the whole count duration and converting bus, truck and RV volumes to passenger car equivalent. The heavy vehicle adjustment factors, in calculating the passenger car equivalent, were taken as 1.5 and 1.2 for bus/truck and RV, respectively, as recommended in HCM (2000) for level terrain. The freeway is assumed to satisfy this condition, as it is situated on fairly level terrain.

The frequency distribution of equivalent hourly traffic volumes, during the period of speed data collection, on SCL and right lanes are shown in Figure 3.15. Figures 3.16 and 3.17 show the distribution of vehicle classifications (percentages of passenger car and heavy vehicle) on SCL and right lanes, respectively. The equivalent hourly traffic volumes on SCL ranged from 95 to 1242 pc/h/lane with a mean volume of 436. The equivalent hourly volumes on right lanes ranged from 345 to 1604 pc/h/lane with a mean volume of 951. The percentages of heavy vehicles on the SCL ranged from 0 to 24.8 with a mean percentage of 5.8. The percentages of heavy vehicles on right lanes ranged from 1.8 to 20.2 with a mean percentage of 12.2.

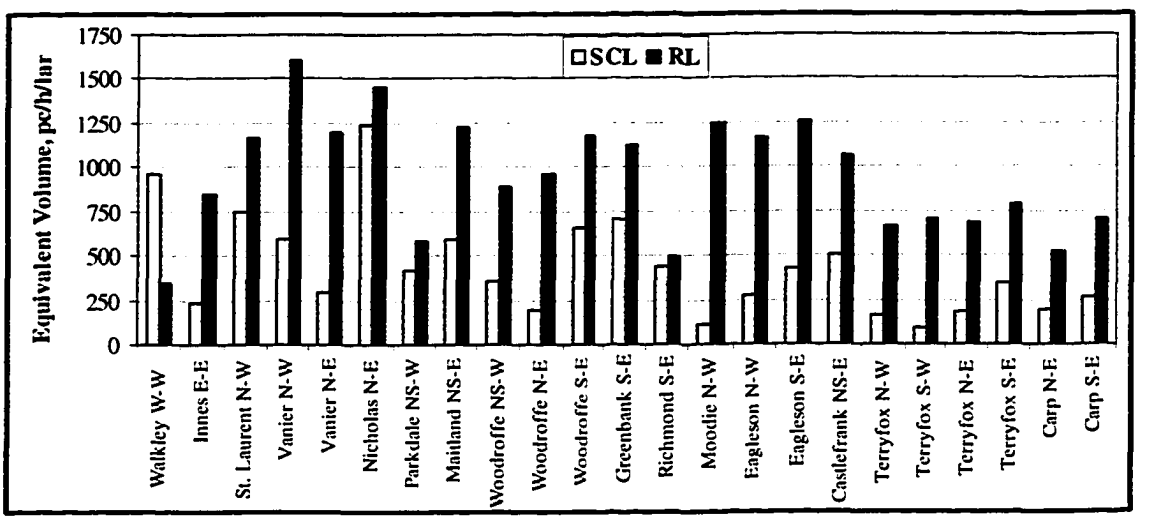


Figure 3.15: Equivalent Hourly Traffic Volume on SCL and RL

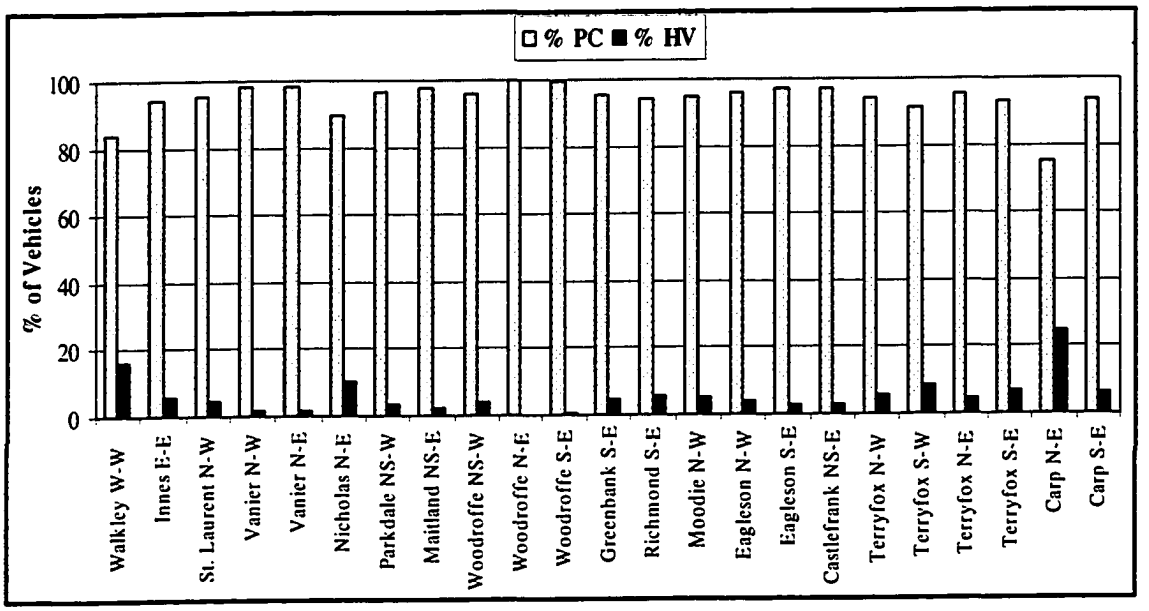


Figure 3.16: SCL Vehicle Classification

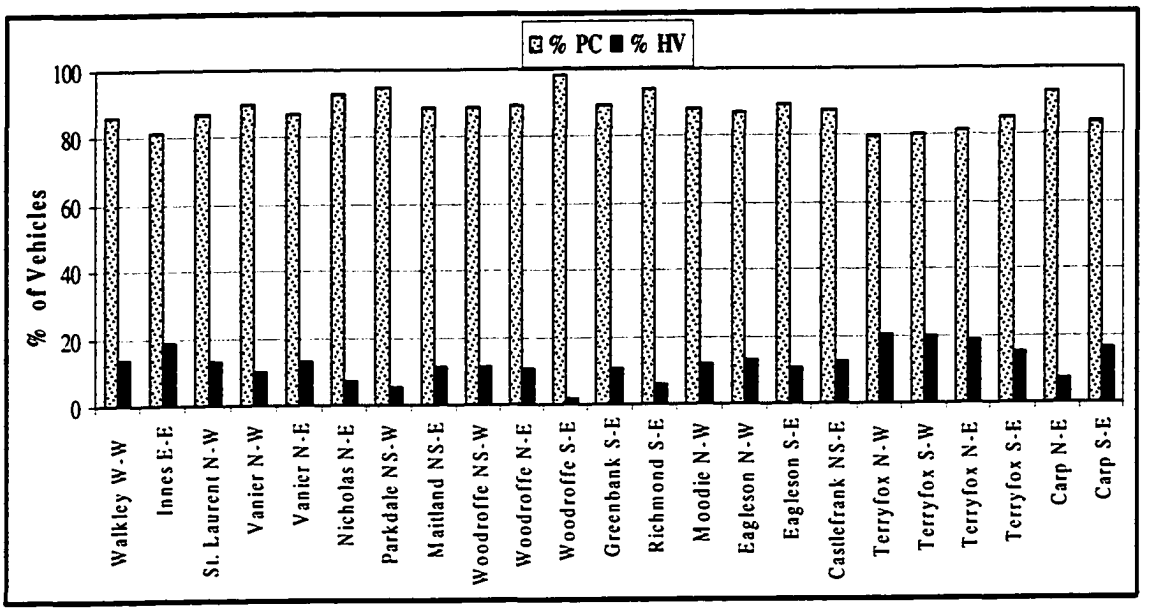


Figure 3.17: RL Vehicle Classification

3.5.2 Peak-Hour Traffic and AADT Data

Traffic volume data for year 2000 and 2002, for the freeway mainline and ramps within the study area, were collected from MTO traffic volume database. Both the AADT and peak-hour volumes were given for some ramps, while only the peak-hour volume was given for the rest. Sarhan (2004) converted the peak-hour volumes, for the set containing only the peak-hour data, to AADT using a conversion factor calculated from the other set of data containing both of the AADT and peak-hour volume. With the available data the peak-hour volume were found to be 9.5% of AADT, and was used as the conversion factor. The calculated conversion factor was close to the typical value of 9.3% specified in HCM (2000), indicating that this conversion is quite acceptable. These peak-hour traffic and AADT data were used in collision modelling attempts in this study.

The ramp peak hour volumes ranged from 120 to 1407 with a mean volume 579 pc/h/lane, while the average mainline peak hour volumes ranged from 556 to 1947 with a mean volume of 1206 pc/h/lane. Figures 3.18 and 3.19 show the comparison of equivalent hourly volumes (during data collection) with the peak volumes for SCL and mainlines, respectively. The peak hour volumes were collected by MTO during 2000 and 2002. Figure 3.19 shows that the current mainline off-peak volumes (during the data collection time) are above the average peak-hour volumes at two sites. Construction of a new interchange at Castlefrank Road, and the resulting redistribution of traffic movement, and/or significant traffic growth in that area probably contributed to this change. At Eagleson N-W, located upstream the Castlefrank interchange, the entering ramp traffic volume during the data collection was slightly higher than the peak-hour volume (Figure 3.18). This increase has probably resulted from the reason(s) mentioned

above. At Nicholas N-E, located within the downtown core, the ramp traffic volume during data collection was also higher than the peak-hour volume, probably further indicating a growth in vehicular traffic in some areas.

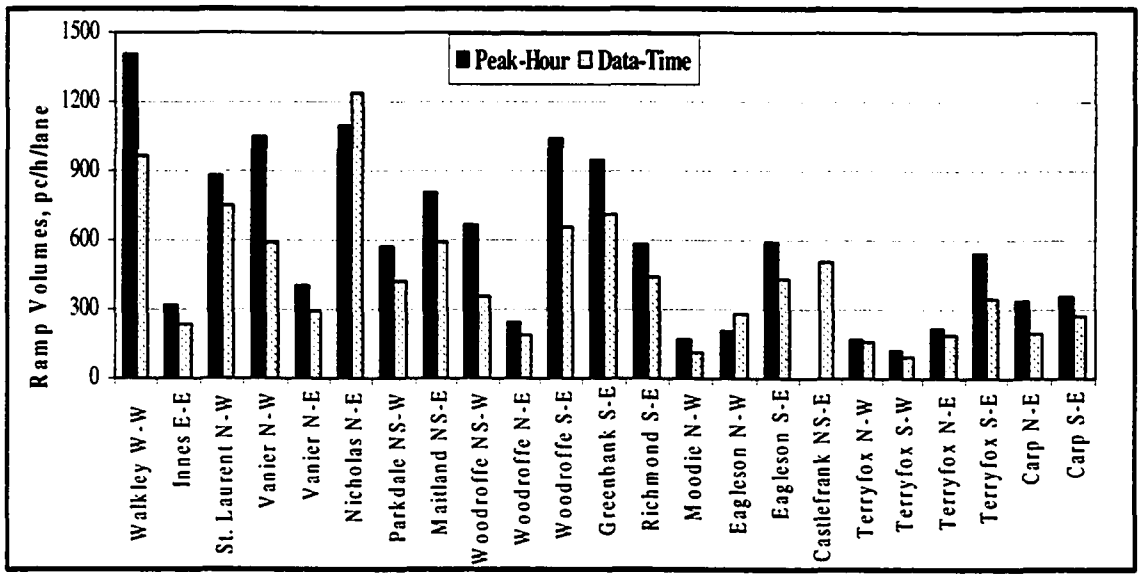


Figure 3.18: Comparison of Ramp Traffic Volumes (Peak Hour versus Data Time)

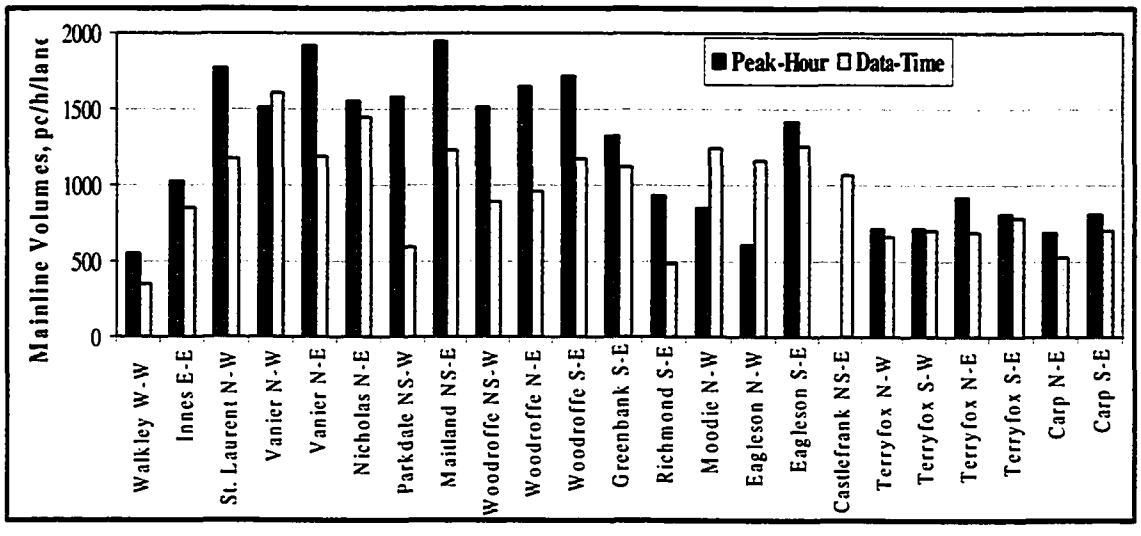


Figure 3.19: Comparison of Mainline Traffic (Peak Hour versus Data Time)

3.6 MAPS AND GEOMETRIC DATA

3.6.1 Description of Maps

A soft copy of maps was received from the MTO Eastern Region and printed for retrieving the geometric data. The soft copy consists of scanned (TIF) images of MTO Engineering and Title Records (ETR) maps for the projects carried out on Highway 417. These maps were carefully examined and compared with the existing conditions within the study area. During the site visits, it was noted that a new interchange has been constructed at Castlefrank Road, with two ramps, and already opened for traffic some time ago. Construction of this new interchange also resulted in a change in the length of Eagleson N-W acceleration lane. It should be noted that this site was used in analyzing the speed and acceleration behaviours, but excluded in modelling attempts for total collision on the acceleration lane. Maps for the section from Eagleson to Castlefrank interchange were collected from Morrison Hersfield. In addition, maps for two other sections, Castlefrank Road to Moodie Drive and Carp Road to Palladium drive interchanges, were missing in the electronic copy. Drawings for these two sections were collected from the MTO Ottawa Office. Figure 3.20 shows a typical MTO maps used in retrieving the geometric data.

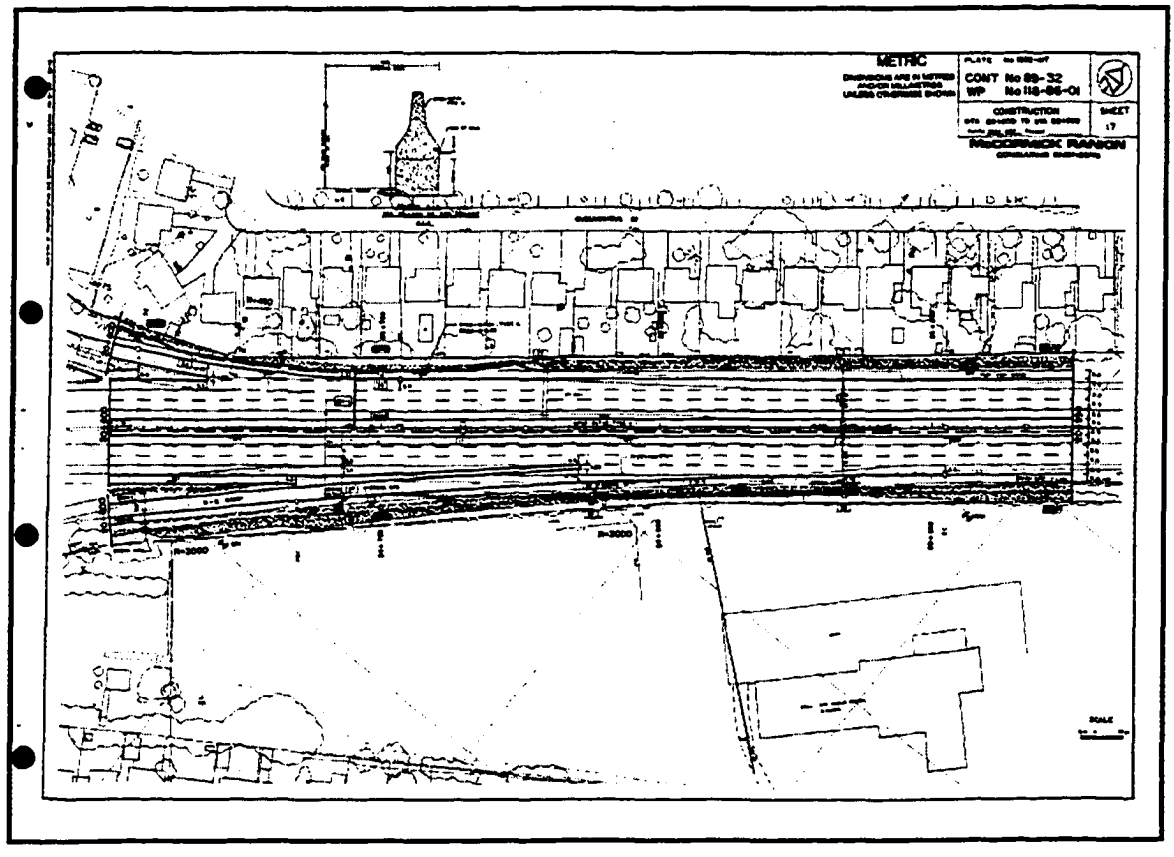


Figure 3.20: Typical MTO ETR Maps

3.6.2 Geometric Data

A total of 128 maps covering the whole study area were used to retrieve the geometric data of the ramps and speed-change lanes. The data retrieved from these maps include:

- ◊ Stations at physical nose and point where the ramp and mainline pavement edges are 1.25 m apart at entrance as well as exit gore, start of taper (parallel type SCL), point where the acceleration lane width is 3.5 m (tapered type SCL), and end of taper.
- ◊ Radius of controlling ramp curve, angle of convergence and pavement width at physical nose.

◊ Stations of equipment positions (sidewalk on overpass).

For this study, the length of speed-change lanes was measured from the point where the pavement edges of the ramp and mainline are 1.25 m apart at the entrance gore to the similar point at the exit gore for extended type SCL or to the end of taper for limited length type SCL. This 1.25 m separation point falls within the AASHTO (2004) design guide criteria of 0.6 to 3.0 m for the start of SCL, required for gap acceptance and merging. Similar start points were recommended in TAC (1999) and HCM (2000) for measuring the length of weaving sections. Another reason for selecting the 1.25 meters separation point as the start of SCL length, and also end for extended type, is that these points are specifically marked on some MTO maps, implying the start or end of SCL.

3.6.3 Descriptive Statistics of Geometry Features

The geometric features of the sites are summarized in Table 3.4. The quantitative geometric data include the length of SCL, length of transition, angle of convergence, ramp pavement width, and number of basic freeway lanes. The categorical geometric data include ramp type, ramp grade, SCL type, and SCL configuration for the limited length type.

Table 3.4: Geometric Features of the Sites

SCL Name	Ramp Type	SCL Config.	Length, m		Fwy Lane	Ramp Radius, m	Ramp Grade	Converge Angle, deg.	Ramp Width, m
			SCL	Transition					
Walkley W-W	Loop	Parallel	422	19	2	69	Down	5.0	4.9
Innes E-E	Loop	Parallel	354	32	2	43	Down	7.0	4.9
St. Laurent N-W	Outer	Extended	881	64	3	45	Up	3.0	4.8
Vanier N-W	Outer	Extended	800	43	3	82	Down	5.0	4.8
Vanier N-E	Loop	Parallel	363	32	3	60	Down	8.0	4.8
Nicholas NS-E	*Outer	Extended	710	78	3	N/A	Down	3.0	4.8
Parkdale NS-W	*Outer	Taper	188	36	4	N/A	Up	4.0	3.5
Maitland NS-E	+Loop	Parallel	468	17	3	74	Down	10.5	4.0
Woodroffe NS-W	+Loop	Parallel	346	56	3	113	Down	3.5	4.6
Woodroffe N-E	Loop	Parallel	248	32	3	50	Down	5.0	4.8
Woodroffe S-E	Outer	Extended	1222	104	3	90	Down	3.0	4.8
Greenbank S-E	Outer	Extended	1366	102	3	95	Down	5.0	4.8
Richmond S-E	Outer	Parallel	321	72	4	295	Down	5.0	4.8
Moodie N-W	Outer	Parallel	323	74	2	238	Down	3.0	4.8
Eagleson N-W	Outer	Extended	673	78	2	95	Down	3.0	5.0
Eagleson S-E	Outer	Parallel	327	78	2	350	Down	3.0	4.8
Castlefrank NS-E	*Outer	Extended	696	72	2	N/A	Down	2.0	4.8
Terryfox N-W	Outer	Parallel	419	83	2	155	Down	3.0	5.0
Terryfox S-W	Loop	Parallel	395	40	2	70	Down	5.0	5.0
Terryfox N-E	Loop	Parallel	418	46	2	70	Down	6.0	5.0
Terryfox S-E	Outer	Parallel	437	69	2	155	Down	3.0	5.5
Carp N-E	Loop	Parallel	425	7	2	70	Down	9.0	4.8
Carp S-E	Outer	Parallel	430	35	2	200	Down	3.0	4.8

** Fairly straight ramp, +Reverse loop ramp*

Length of Speed-Change Lanes

As mentioned earlier, data were collected for 23 entrance terminals of which sixteen were limited length type, and other seven were extended type. Only one SCL has length less

than 200 m and two SCL have lengths greater than 1,000 m. Table 3.5 shows a statistical summary of quantitative geometric data for the study sites. For the limited length SCL the lengths vary from 188 to 468 m with mean length of 368 m, while for the extended type they vary from 673 to 1366 m with mean length of 907 m.

Table 3.5: Summary of Quantitative Geometric Data

Description	Length of Limited Length SCL (m)	Length of Extended SCL (m)	Transition. m	Radius (m)	Number of Basic Freeway Lanes	Angle of Convergence (degree)	Ramp Pavement Width (m)
Minimum	188	673	7	43	2	2.0	3.5
Maximum	468	1366	104	113	4	10.5	5.5
Mean	368	907	55	68.8	2.57	4.65	4.75

Length of Transition Segment

A transition segment, in this study, refers to the section from the physical nose at the entrance gore to the beginning of SCL. The length of this section varies widely among the ramp terminals and is likely to have an effect on the merging speed and acceleration rate on the SCL. This length varies from 7 to 104 m with mean of 55 m.

Radius of Ramp Curvature

The radius of the controlling ramp curves varies from 43 to 350 m, excluding three fairly straight ramps (Table 3.4), with a mean of 69 m. The outer connection ramps were

relatively flat with large radius of curvature, followed by a long transition or spiral curve, or fairly straight type.

Number of Basic Freeway Lanes

The number of basic freeway lanes varies from two to four among the sites used in this study with mean of 2.57. Out of the 23 sections, twelve sections have two basic freeway lanes, nine have three basic lanes, and the remaining two sections have four basic lanes.

Angle of Convergence

After traversing the ramp curve at fairly constant speeds, the entering vehicles begin accelerating in the vicinity of the spiral curves and start looking for a gap in the mainline traffic stream. Therefore, the geometry of the nose is important for the driver to make a merging manoeuvre safely, and is therefore likely to have some effect on driver behaviour on the acceleration lane. The angle of convergence was measured at the physical nose where all paved area begins. As shown in Table 3.5, the angle of convergence varies from 2.0 to 10.5 degrees with a mean value of 4.65 degrees.

Ramp Pavement Width

The widths of the ramp travelled way at physical nose vary from 3.5 to 5.5 meters with a mean width of 4.75 meters.

Ramp Type

Table 3.6 provides the summary of categorical geometric data. As shown in the table, fourteen ramps are outer connection type with large radius of curvature, followed by long transition or spiral, or are fairly straight connection type. These ramps generally assist the vehicles to accelerate and gain substantial speed before reaching the gore area or entering the SCL. Nine ramps are loop or reverse curve on which drivers usually drive at or near the control speed with most acceleration and speed gain taking place in the vicinity of or downstream the physical gore.

Table 3.6: Summary of Categorical Geometric Data

Geometry	Group	Frequency	Total
Ramp Type	Outer	14	23
	Loop/Reverse Curve	9	
Ramp Grade	Upgrade	2	23
	Downgrade	21	
SCL Type	Limited Length	16	23
	Extended	7	
Configuration: Limited Length SCL	Parallel	15	16
	Taper	1	

Ramp Grade

As shown in Table 3.6, 21 ramps are situated on downgrade (arterials passing over the freeway), and two ramps on upgrade (freeway passing over the arterials). Downgrades assist in vehicle acceleration while the upgrades tend to retard the speed gains. Therefore, the ramp grade upstream the gore is likely to have some effect on the merging behaviour.

SCL Type

As shown in Table 3.6, out of the 23 SCL used in this study, sixteen are limited length type and other seven are extended type. On the extended type SCL, drivers have the option to take time and merge with a longer gap. However, unfamiliar drivers may treat it as a continuous lane and may abruptly merge to the left close to the exit gore when they discover the discontinuity of the lane. For the limited length type, drivers have to accelerate and merge within the available length, and the varying lengths are likely to affect the merging behaviour.

SCL Configurations

As shown in Table 3.6, out of the sixteen limited length type SCL, fifteen SCL are parallel type and only one is tapered type. The number of tapered SCL in this study is not sufficient to make a meaningful comparison between the two configurations.

3.7 COLLISION DATA

A summary of five years collision data, covering the period from 1998 to 2002, for the whole study area was received from Sarhan (2004). Those data were collected from the

City of Ottawa. The collision record from the City contained the location, date, time, day of the week, accident type such as fatal injury, non-fatal injury, or property damage only, etc. As mentioned in Sarhan (2004), a collision was classified as fatal if the collision resulted in the death of at least one person within 30 days of the collision. Injury collision refers to a collision resulting in a bodily injury of at least one person, but not resulting in death of anyone. The property damage only (PDO) collisions refer to those collisions in which no bodily injury occurs and only property damage was reported. The PDO collisions were reported if the property damage exceeded the collision-reporting threshold of $1,000, set for the study period from 1998 to 2002. Sarhan (2004) sorted and summarized the collision data by type and segment for use in the first phase of research covering the same study area. An Excel file covering the summary was collected for use in this second phase, and the number of collisions on the study sites included in this study was obtained.

Three types of collisions were included in this study that includes fatal, injury and PDO collisions. Sarhan (2004) grouped the collisions as "Total" and "Equivalent Property Damage Only (EPDO)" collisions for developing the safety performance models, where EPDO collisions are the sum of collisions with each type of collisions weighted by some factors. However, no model was successfully developed for the EPDO collisions. Therefore, only the total collisions are used in this study for developing the safety performance model, which is the simple arithmetic summation of all three types of collisions mentioned above (fatal + injury + PDO). The distributions of total collisions on segments including the SCL (*TCol*), and total collisions on the through plus acceleration lanes (*TColThAcc*) are shown in Figure 3.21. Figure 3.22 shows the distribution of total

collisions on the acceleration lanes (*TColAcc*). The total collisions on segments ranged from 0 to 126 with mean collisions of 45. Segment at Innes E-E experienced no collision during 1998 to 2002. As mentioned earlier, Castlefrank NS-E did not exist during that period, and the length of Eagleson N-W acceleration lane was changed due to construction of Castlefrank interchange. Therefore, these two sites were excluded from modelling attempts for total collision on the SCL.

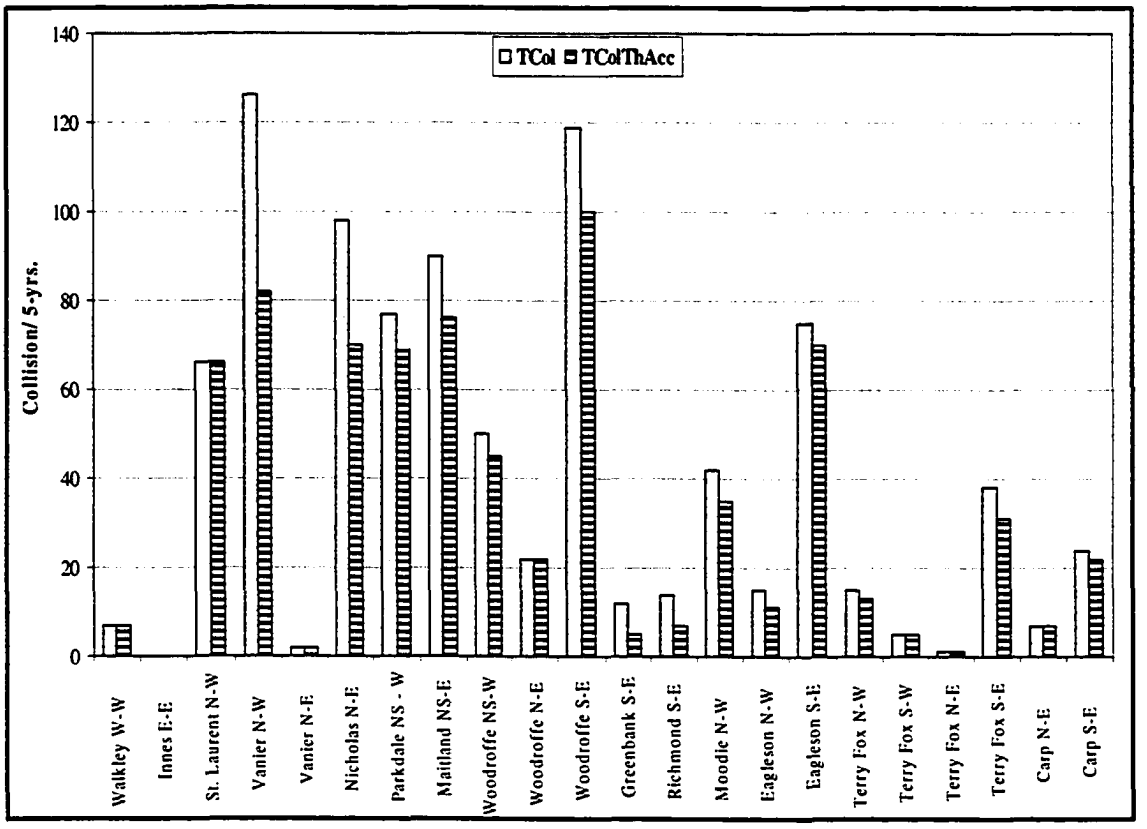


Figure 3.21: Distribution of Total Collisions on Segments

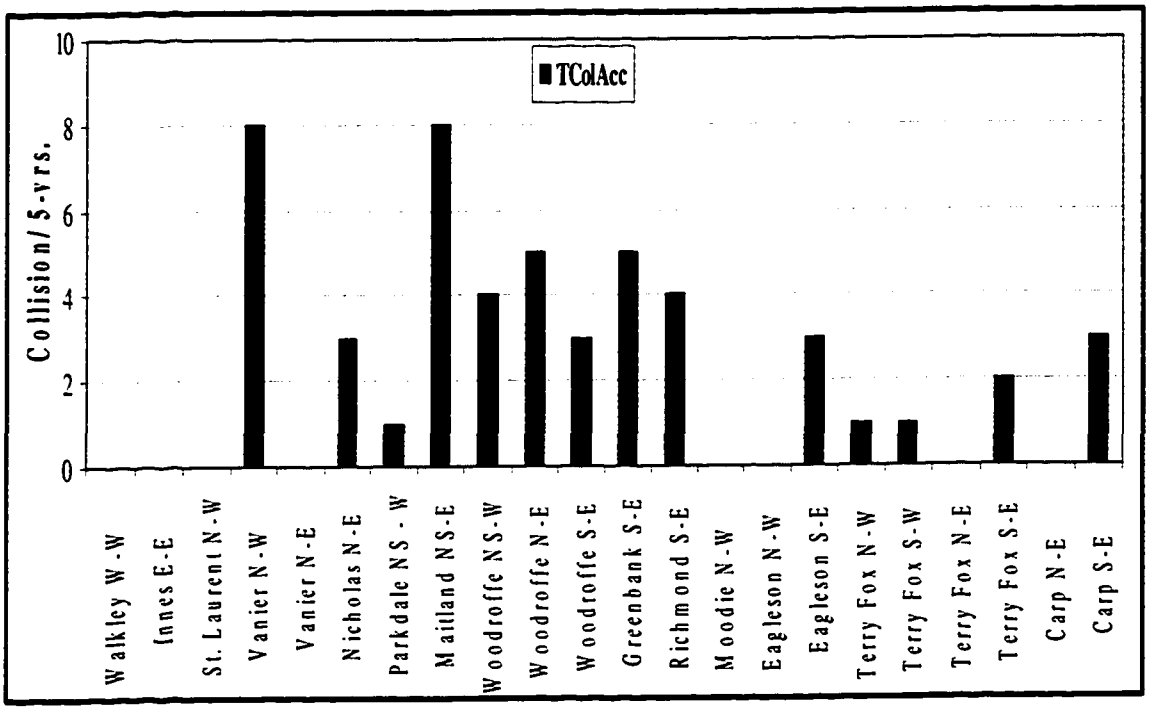


Figure 3.22: Distribution of Total Collisions on SCL

4.0 TRAFFIC BEHAVIOUR

This chapter provides an analysis of different features of driver behaviour on the freeway right-most lane and acceleration lane, including the effect of merge area geometry and traffic conditions. Section 4.1 presents a brief of statistical tests and cumulative probability distributions of speed data. Section 4.2 covers the distribution and analysis of freeway right lane speeds. Section 4.3 presents the distribution and analysis of entering vehicles speeds. Section 4.4 presents a comparison between the entering vehicles merging speed and right lane speed downstream the gore. Section 4.5 presents the distribution and analysis of overall accelerations on the SCL, while Section 4.6 presents the distribution and analysis of maximum accelerations. Section 4.7 presents the distribution of mean accelerations on SCL. Section 4.8 presents the distribution and analysis of effective acceleration distances (merging distances). Section 4.9 presents a summary of the traffic behaviour discussed in the chapter.

4.1 STATISTICAL TESTS AND CUMULATIVE DISTRIBUTIONS

4.1.1 Minimum Sample Size

The speed data consist of several groups and sub-groups. Histograms of speed data were plotted for various groups to examine whether the samples follow a normal distribution. It was seen that the samples are normally distributed with respect to the mean with sample size of thirty and over, except for very few groups. The central limit theorem, which states that the samples may be assumed to be normally distributed when the sample size is thirty or greater, was then adopted in this study. Accordingly, the

minimum sample size was set as thirty for studying any behaviour or in any comparison between groups.

4.1.2 Detection of Outliers

Outliers are inconsistent or errant data points in a sample. These errant points, which lie outside the normal trend or range or represent an unusual behaviour, are likely to influence the true behaviour. The most obvious and widely accepted method for determining the outliers in samples is the calculation of standardized (z) score for each sample point. Accordingly, z-scores were computed for each of the data point. The choice of a cut-off point for z-score also varies among statisticians and scientists. However, an absolute value of 3.0, as a rule of thump, is widely accepted (Mendenhall et al. 1995). Therefore, in this study, points with z-scores greater than 3.0 in absolute value were treated as outlier, and excluded from the analysis following an iterative process. For a sample size of thirty, this z-score value corresponds to a 99.7% confidence level.

4.1.3 Cumulative Probability Distributions

To determine the true 85th percentile values of speed, acceleration, and merging distance of the vehicle population, the cumulative probability distribution was plotted for each group of sample. Figure 4.1 shows a cumulative distribution example for the merging speed of entering vehicles from a SCL. From the cumulative distribution function, the 85th and 95th percentile speeds, accelerations, and merging distances were computed. The sample and population 85th percentile values were almost the same for all of the traffic behaviours analyzed in this study. Cumulative distributions were also plotted for

passenger cars and heavy vehicles combined (combined PC and HV) as well as separately for passenger cars. Finally, the minimum, maximum, mean, 85th, 95th percentile values, and standard deviations were summarized for each group in a tabular format, as shown in Appendix B, for further analysis.

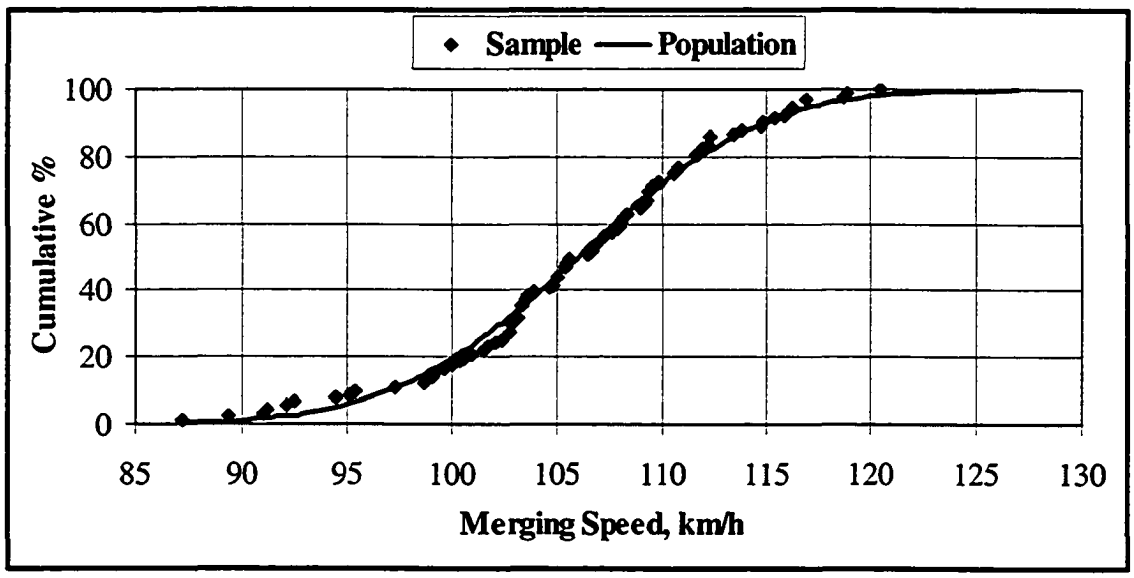


Figure 4.1: Cumulative Distribution Example of Merging Speeds (Carp Rd. S-E)

4.2 Distribution And Analysis Of Right Lane Speed

This section presents the distribution and analysis of speeds on freeway right-most lane together with comparison between different cases that include combined (PC and HV) versus passenger vehicles, impeded versus unimpeded cases, and downstream versus upstream the gore. It should be noted that the right lane speed of a vehicle refers to the average speed (average of all successive speed readings) over the whole section upstream or downstream the gore, as applicable.

4.2.1 RL Speed Downstream the Gore

Figure 4.2 graphically shows the distribution of passenger cars minimum, maximum, mean, and 85th percentile speeds. As shown in the figure, passenger vehicle speeds on the freeway right-most lane, downstream the gore, range from as low as 67.0 km/h to as high as 137.3 km/h within the study sections. The lowest right lane speed, 85th percentile passenger car speed (V_{85RL}) of 98.7 km/h, was observed at Nicholas N-E. A high right lane equivalent hourly traffic volume of 1449 pc/h/lane and the highest entering volume of 1242 pc/h/lane were observed at this site, with the highest total (right lane plus entering) equivalent hourly traffic volume of 2691 pc/h/lane among the sites. Similarly, the highest right lane speeds among the sites, V_{85RL} of 118.9 km/h was observed at Walkley W-W and at Carp Road N-E. The right lane equivalent hourly volume at Walkley W-W was as low as 345, the lowest among the sites used in this study. On the other hand, at Carp Road N-E, the total (right lane plus entering) equivalent hourly volume was 714, the lowest among the sites. These indicate that through, particularly the right lane, vehicle speed at the freeway merge area is affected by both right lane and entering traffic.

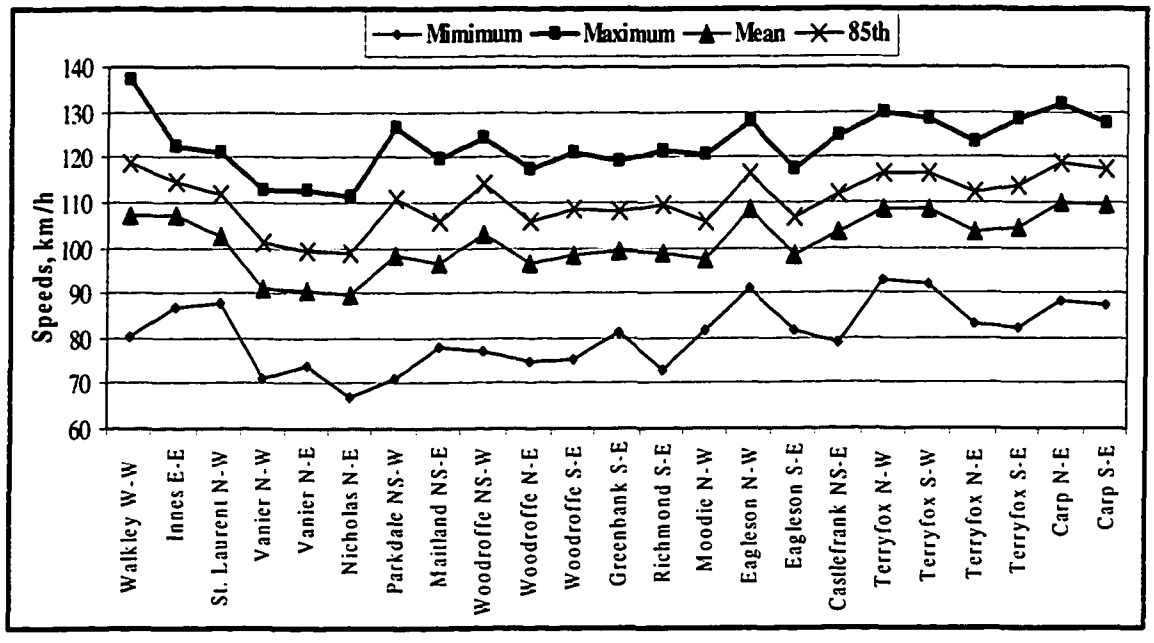


Figure 4.2: Variation of Passenger Car RL Speeds (Downstream the Gore)

4.2.2 Passenger Car versus Combined Vehicles RL Speeds

As mentioned in Chapter 3.0, vehicles were targeted randomly including the heavy vehicles (bus and truck). The number of heavy vehicles targeted was not sufficient to make any direct comparison between passenger cars and heavy vehicles. Therefore, comparisons were made between speeds of combined vehicles (HV and PC combined) and passenger cars. Figure 4.3 graphically shows the comparison of the mean and 85th percentile combined (Comb) vehicles and passenger cars (PC) right lane speeds along the SCL. The mean and 85th percentile passenger car speeds were observed to be -0.2 to 1.2 and 0 to 1.5 km/h, respectively, higher than that of the combined vehicles, where the percentages of heavy vehicles ranged from 1.8 to 20.2.

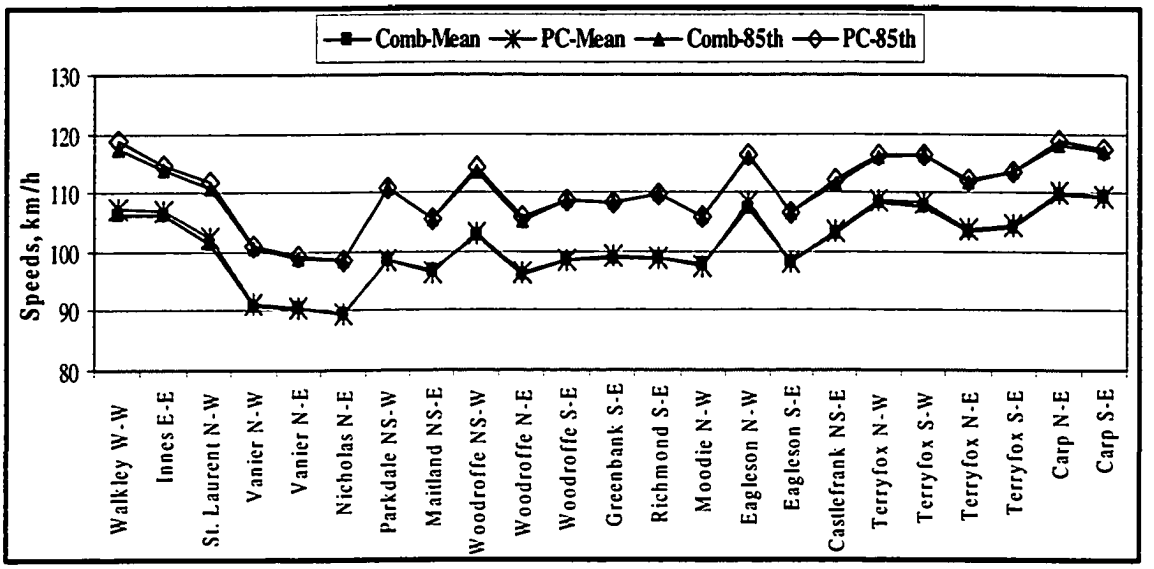


Figure 4.3: Comparison between Combined Vehicles and PC Right Lane Speeds

The highest difference in 85th percentile speeds of 1.5 km/h was observed at Walkley W-W with HV percentages of 13.6% and the lowest RL traffic volume among the sites. Furthermore, statistical testing showed that the differences in mean speeds between the combined and passenger vehicles are not statistically significant at a 5% level of significance (p-value = 0.087, for the highest speed difference). This is probably due to the small numbers of HV speed observations during the study. Since the differences in RL speeds between the combined and passenger vehicles were not remarkable (Figure 4.3) or statistically significant as well, the rest of the analysis uses PC speed only.

4.2.3 Speed of Impeded and Unimpeded RL Vehicles

In this study, an impeded vehicle refers to a right lane vehicle in front of which a ramp vehicle merged with a relatively short gap. Visual judgement was used to decide whether a RL vehicle was impeded or not, as vehicles may slow down without braking. The number of impeded passenger cars on each section ranged from zero to 36. Only at three sections, the recorded impeded cases were thirty or more. For these three sections, a comparison was made between the impeded and unimpeded speeds. However, for all sections additional comparison were made between all (impeded plus unimpeded) vehicles and unimpeded vehicles.

Table 4.1 shows the results of comparing the differences in mean speeds, on freeway right lane downstream the gore, between the unimpeded and impeded passenger cars at three sections. The speed differences ranged from 3.6 to 12.4 km/h at these sites. As shown in the table, higher differences are associated with shorter lengths of SCL. The highest difference of 12.4 km/h was observed at Parkdale NS-W with the shortest SCL among the sites used in this study. A *t*-test between unimpeded and impeded cases at each of these sites showed that the differences in mean speeds are statistically significant at a 5% level of significance (*p*-values ranged from 0.000 to 0.003). This implies that right lane speeds are affected by entering vehicles, and higher interruption occurs when the acceleration lane is shorter in length.

Table 4.1: Mean PC Unimpeded versus Impeded Speeds

Section Name	Recorded Impeded Cases	SCL Length, m	Speed Diff., km/h	*t*- value	*p*- value
Parkdale NS-W	34	188	12.4	7.99	0.000
Vanier N-E	36	363	7.4	6.05	0.000
Terryfox S-E	35	437	3.6	3.04	0.003

The comparison of mean passenger car speeds between all and unimpeded cases showed that the mean speeds of unimpeded cases are 0 to 4.2 km/h higher than that of all (impeded plus unimpeded) PC cases. Figure 4.4 shows that the impediment in mean passenger car speed, i.e. difference in speed between the unimpeded and impeded cases (slowing down of right lane vehicle due to entering vehicle), decreases as the length of limited length SCL (L_{Lim}) increase. This indicates that shorter SCL results in higher interruption in right lane speed at freeway merge areas.

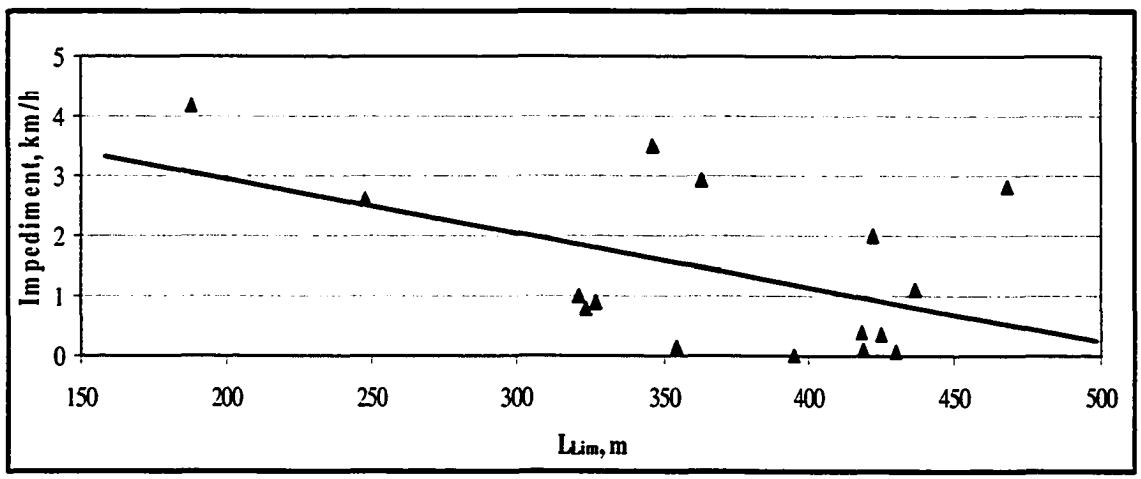


Figure 4.4: Impediment in Mean PC Speeds versus Limited Length SCL Lengths

It should be noted that not all the vehicles recorded as impeded had necessarily slown down. Furthermore, due to the distant positions of merge areas from the equipment, impeded cases could not be identified in some cases, indicating that higher impediment, than that presented here, may have resulted in the field.

4.2.4 RL Speed Upstream versus Downstream the Gore

At eight sites, right lane speed was measured from sections both upstream and downstream the gore for further studying the effect of entering vehicles on through vehicle speed. Figure 4.5 graphically shows the comparison of mean and 85th percentile passenger car speeds between these two sections over the respective sites. The mean passenger car right lane speeds ranged from 91.2 to 103.5 km/h and 92.3 to 107.4 km/h for sections downstream and upstream the gore, respectively.

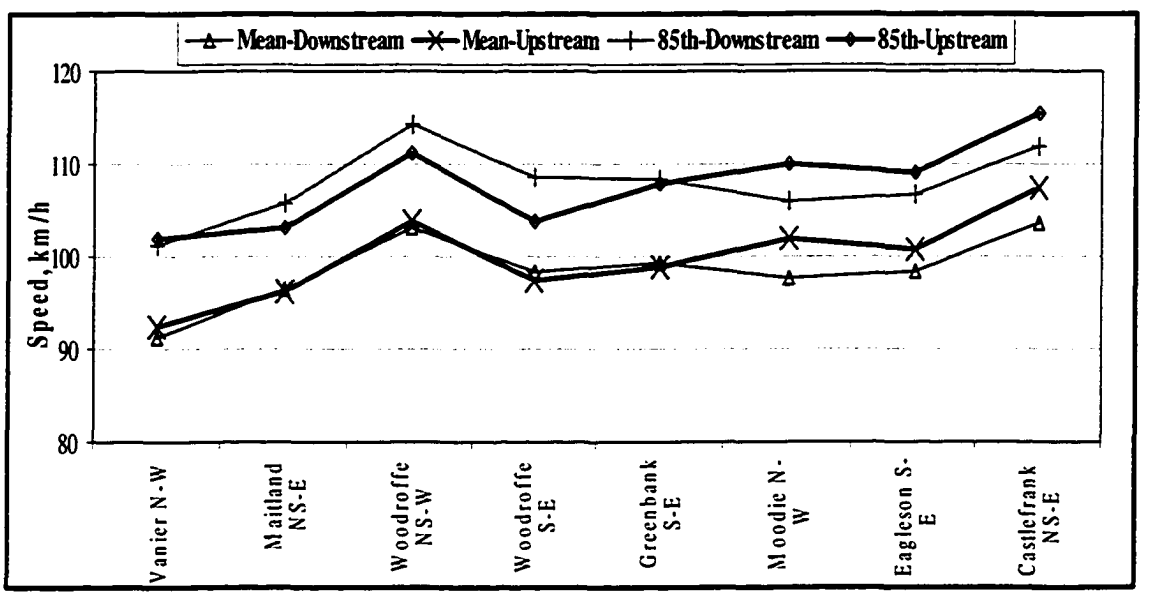


Figure 4.5: PC Speed Comparison- Upstream versus Downstream the Gore

As shown in the figure, the mean speeds upstream the gore were 0.7 to 4.3 km/h "higher" at four sites, and 0.3 to 1.0 km/h "lower" at other four sites than that downstream the gore. The largest difference corresponded to a mean upstream speed 4.3 km/h "higher" than that downstream at Moodie N-W, followed by Castlefrank NS-E where the mean upstream speed was 4.0 km/h "higher" than that of the downstream section. Another merging area does not exist closely, at upstream the SCL under consideration, at these two sites. On the other hand, the smaller differences, positive or negative, may be explained by the existence of another merging area close to the subject SCL. A *t*-test, for comparing the right lane PC speeds between sections upstream and downstream the gore, showed that the differences are statistically significant at a 5% level of significance (p-values ≤ 0.002) for the above mentioned two sites with speed differences of ≥ 4.0 km/h. This indicates that the through lane speed is higher at sections away from the influence of merge area.

4.3 Distribution And Analysis Of Speed On the SCL

4.3.1 Distribution of Gore Speeds

Gore speed is the speed of ramp vehicles attained at the beginning point of the SCL, where the ramp and mainline pavement edges are 1.25 m apart. The speed at this point captures the effect all of geometric variables preceding this point. Therefore, the speed at gore is likely to explain greatly the variability in merging behaviour. Figure 4.6 shows the variation in mean and 85th percentile passenger cars gore speeds among the fifteen sites with full profile speed data. The lowest 85th percentile passenger car gore speed (V_{85Gore}) of 67.9 km/h was observed on Maitland NS-E. The gore area at this site is preceded by a

short transition, 17 m from physical nose to the beginning of SCL, the largest convergence angle of 10.5° at physical nose, and a reverse loop ramp curve with a controlling radius of 74 m. The highest V_{85Gore} of 110.1 km/h was observed at Terryfox N-W, where the gore area is preceded by a long transition, 83 m from physical nose to the beginning of SCL, a smaller convergence angle of 3°, and an outer connection ramp with controlling radius of 155 m. These indicate that the gore speeds depend on the geometry of the entrance ramp upstream the beginning point of the SCL.

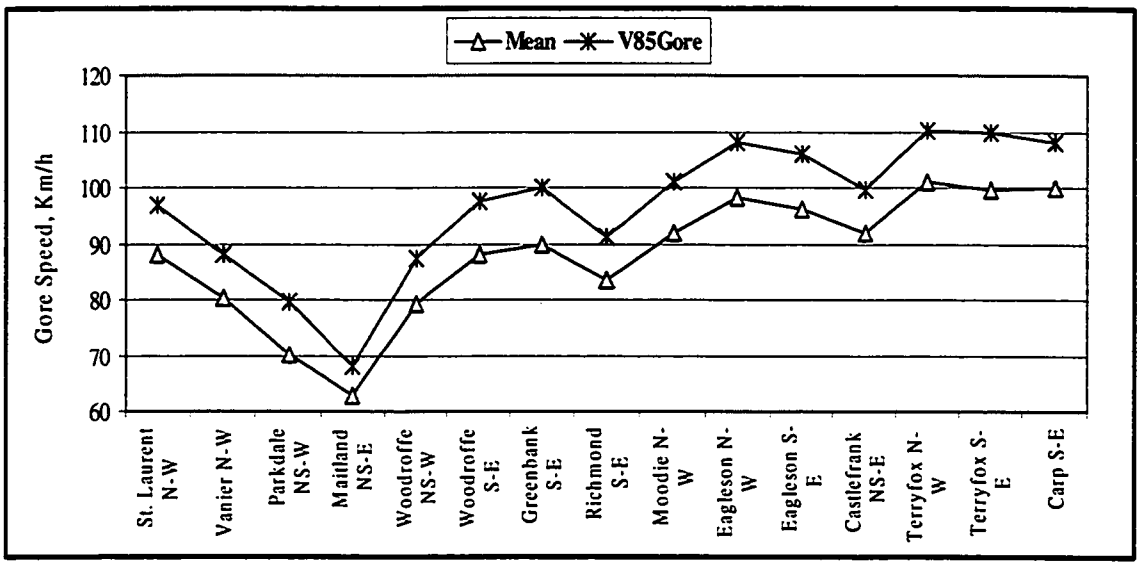


Figure 4.6: Distribution of Passenger Car Speeds at Gore

The mean gore speeds of PC were 0 to 1.6 km/h higher than that of combined vehicles for a HV volume of 0.4 to 6.7% on acceleration lanes with full profile data. These HV volumes are not adequate for a meaningful statistical test for the differences in gore speeds between the passenger and combined vehicles. However, the trend in Figure 4.7 shows that difference in the mean gore speed between the PC and combined vehicles increases as the HV% increases, indicating that geometry of the ramp upstream the gore

has some additional effect on heavy vehicles performance. The rest of the analysis uses only the V_{85Gore}.

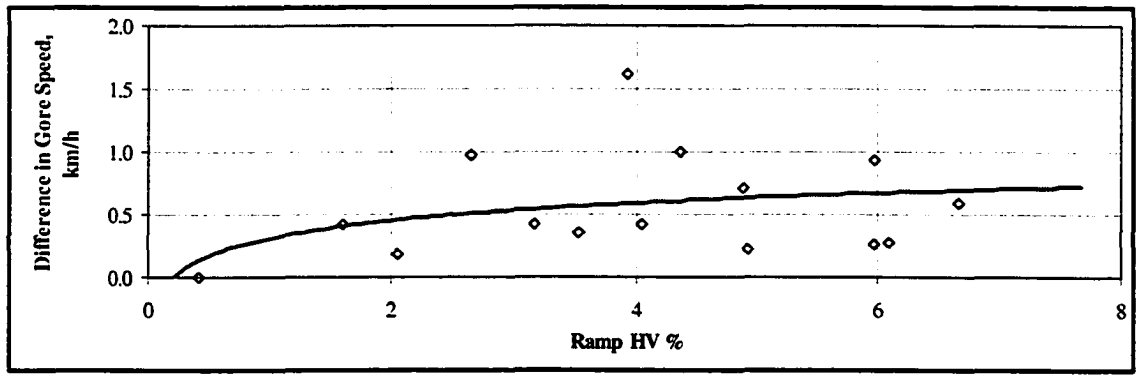


Figure 4.7: Effect of HV Volumes on Mean Speed at Gore

4.3.2 Distribution of Merging Speeds

This section presents the description of merging speed variations among the vehicles at each SCL and among the SCL. Figure 4.8 shows graphically the variations in minimum, maximum, mean, and 85th percentile passenger cars merging speeds. As shown in the figure, passenger vehicle merging speed on the study test sites ranged from as low as 54.4 km/h at Parkdale NS-W with the shortest acceleration lane of 188 m (including the taper) to as high as 129 km/h at Terryfox S-E with a long acceleration lane of length 437 m (including the taper).

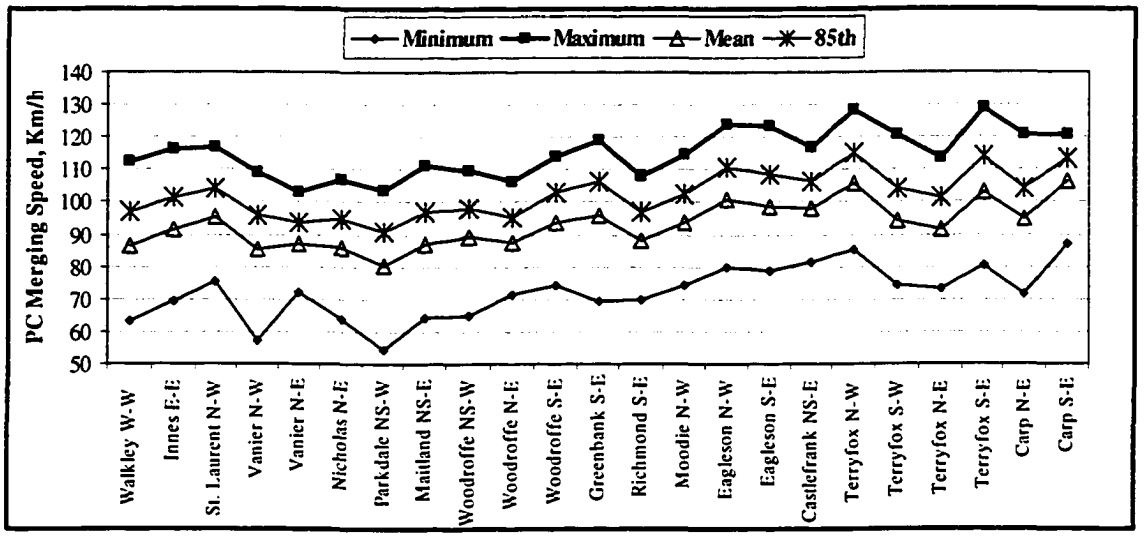


Figure 4.8: Distribution of Passenger Car Merging Speeds

At Parkdale NS-W, the 85th percentile PC merging speed ($V_{85Merge}$) was observed to be 90.6 km/h (the lowest among the sites) with an entering and total (entering plus right lane) equivalent traffic volume of 422 and 1010 pc/h/lane, respectively. At Terryfox N-W, the $V_{85Merge}$ was observed to be 115.1 km/h, the highest among the sites. The equivalent hourly entering volume at this site was observed to be 161, while the total equivalent volume was 822 pc/h/lane, both are lower than those at Parkdale NS-W. The merging area at Parkdale NS-W is preceded by a fairly straight but short ramp situated on an upgrade. The SCL at Terryfox N-W is 419 m in length, and is preceded by an outer connection flat ramp situated on a downgrade and a long transition from the end of controlling ramp curve to the beginning of the SCL. These indicate that shorter acceleration lanes, tight ramp geometrics, and higher traffic volumes are associated with lower merging speeds, while longer acceleration lanes and flat ramp geometrics enable the entering vehicles to merge at higher speed.

4.3.3 Passenger Car versus Combined Vehicles Merging Speeds

As mentioned in Section 4.2.2, the number of heavy vehicles targeted was not sufficient for any direct comparison between passenger cars and heavy vehicles, and therefore comparisons were made between speeds of passenger cars alone, and combined heavy vehicles and passenger cars. The comparisons of the mean and 85th percentile merging speeds between these two groups are shown in Figure 4.9. The figure shows very slight differences in the merging speed between the combined vehicles and passenger car.

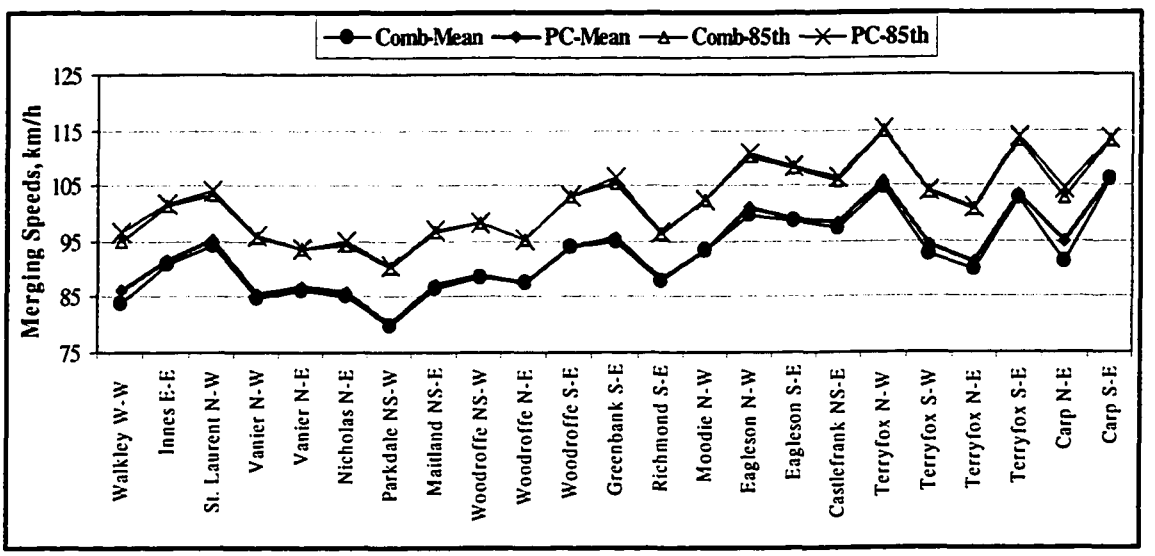


Figure 4.9: Comparison between Combined and Passenger Vehicles Merging Speeds

The differences in mean merging speeds between passenger cars and combined vehicles were 0 to 3.7 km/h. The largest difference, mean passenger car merging speed higher than that of the combined vehicles by 3.7 km/h, was observed at Carp Road N-E, with the highest HV volume of 24.8% among the SCL used in this study. It was followed by Walkley W-W (2.3 km/h) where the HV volume was 16.3%. The smallest differences

were noted at Woodroffe N-E (0 km/h) and Woodroffe S-E (0.1 km/h) where the HV volumes were less than 1%. Statistical t-test (Table 4.2) for the differences in the mean merging speeds between the combined vehicles and passenger cars showed that the differences are not statistically significant at a 5% level of significance, except Carp Road N-E where the truck volume was 24.8% with difference in mean speed of 3.7 km/h. However, the trend in mean speed differences indicates that truck volume has some effect on the merging speed. Figure 4.10 shows that the difference in mean merging speed between passenger and combined vehicles increases as the truck volume increases. The relationship shown in the figure has a good R^2 value of 0.75. This probably implies that longer SCL may be required where the percentages of truck traffic are higher for the trucks to attain acceptable merging speeds. However, the number of heavy vehicles, with relatively long acceleration lanes, at most sites was not adequate to study the effect of length of the SCL on the merging speed of heavy vehicles. Therefore, subsequent analysis uses PC speed only.

Table 4.2: Mean Combined versus PC Merging Speeds

Section Name	% HV	SCL Length, m	Speed Diff., km/h	*t*- value	*p*- value
Carp Road N-E	24.8	425	3.7	2.42	0.017
Walkley W-W	16.3	422	2.3	1.83	0.073

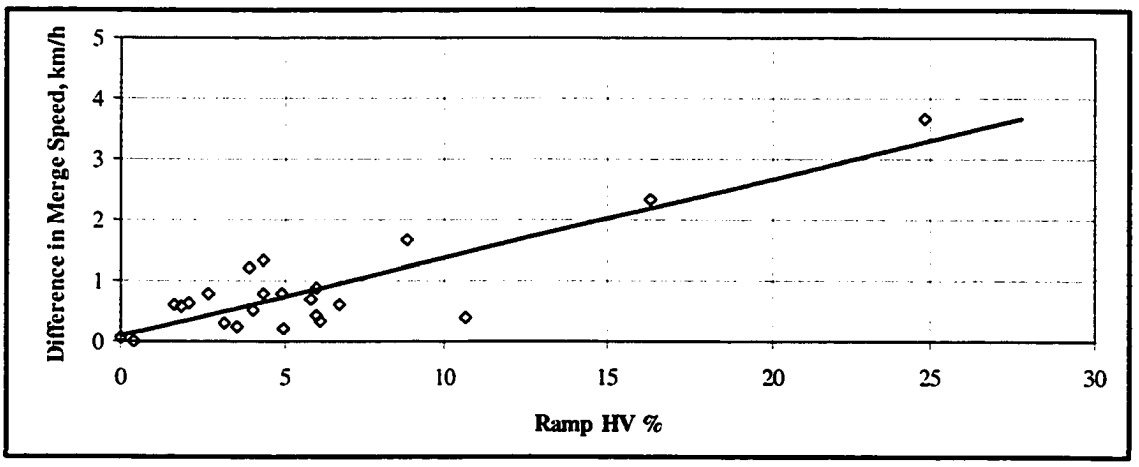


Figure 4.10: Effect of Heavy Vehicles on Differences in Mean Merging Speeds

4.3.4 Effect of Length of SCL on PC Merging Speed

Figure 4.11 shows graphically the relationship between the $V_{85Merge}$ and length of the limited length acceleration lanes (L_{Lim}). The slight scattered pattern, as shown in the figure, is probably due to the fact, indicated earlier, that merging speeds do not depend on the length of the SCL alone, rather on several factors in combination such as geometry of the ramp upstream the SCL, and traffic conditions on SCL as well as through lane. However, the trend in the figure indicates that the merging speed increases as the length of limited length SCL increases. The maximum merging speeds were observed for SCL length of about 425 m, at which the points are mainly clustered in Figure 4.11. The expected maximum $V_{85Merge}$, as shown on the trend line in the figure, is about 105 km/h corresponding to this length.

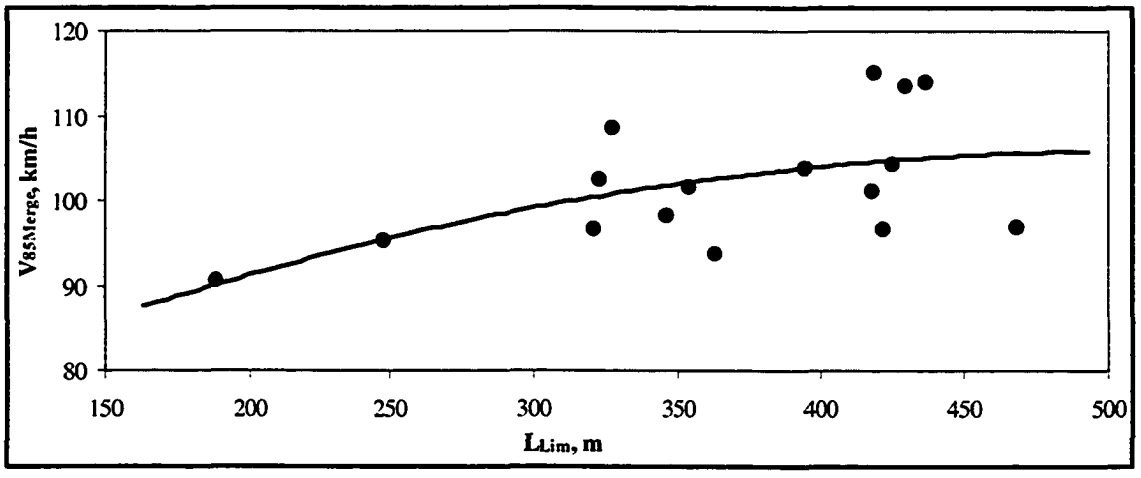


Figure 4.11: Effect of Limited Length SCL Lengths on PC Merging Speeds

Figure 4.12 shows the relationship of $V_{85Merge}$ with the length of the extended acceleration lanes (L_{Extd}). The trend in the figure shows no remarkable change in merging speed of entering vehicle with the change in the length of extended acceleration lane. In fact, the trend for the $V_{85Merge}$ is fairly constant within a range of 104 to 106 km/h (105 km/h on average) regardless of provided lengths. This is probably an indication that extended SCL allows the drivers mainly to carryout the waiving manoeuvre, and the provided length of SCL in excess of a certain maximum has little or no effect in improving the merging speed, regardless of provided ramp geometry upstream the gore. For example, the SCL at Maitland NS-E is provided with the longest acceleration lane of 468 m with tight ramp geometry upstream the gore. The ramp geometry at this site consists of a reverse loop curve with a controlling curve radius of 74 m, a convergence angle of 10.5° at the physical nose (the largest among the sites), and a short transition from end of ramp controlling curve. The V_{85Gore} at this site was observed to be 67.9 km/h, the lowest among the sites, with a $V_{85Merge}$ of 96.9 km/h, which is 5.5 km/h lower than the average $V_{85Merge}$ of all sites and 18.1 km/h lower than the highest $V_{85Merge}$ observed. Therefore, it may be

concluded that tighter ramp geometry upstream the gore may not be fully compensated with longer acceleration lane lengths downstream the gore. An acceleration lane of length in excess of about 425 m (including the taper) was shown not to be beneficial in increasing the merging speed, as observed for the limited length SCL.

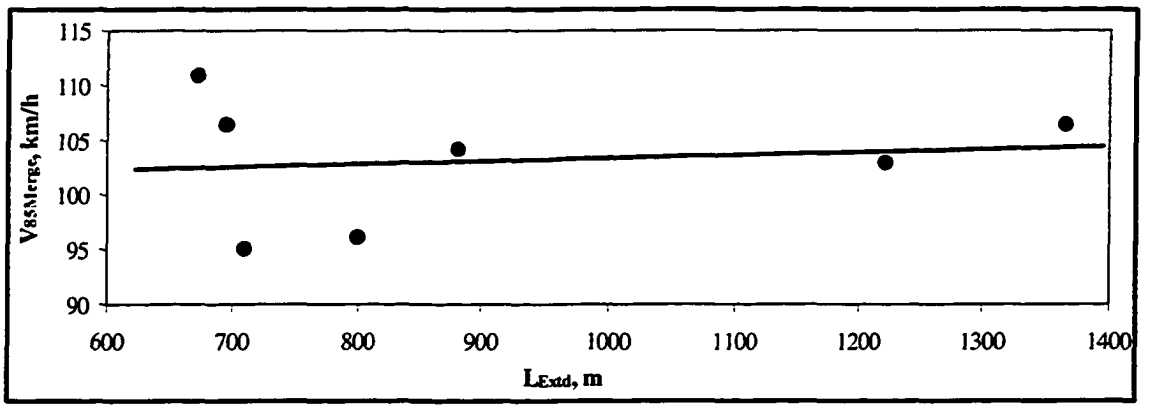


Figure 4.12: Effect of Extended SCL Lengths on PC Merging Speed

4.3.5 Effect of Gore Speed on Merging Speed

Figure 4.13 shows graphically the relationship between the gore speed and merging speed of entering vehicles. The trend in the figure shows that as the V_{85Gore} increases, the $V_{85Merge}$ also increases. The relationship has a good R^2 value of 0.78, indicating that these two speeds are highly correlated. Higher gore speed allows the driver to merge into the freeway at higher speed. As mentioned earlier, lower gore speed was observed where the geometry of the ramp upstream the gore is tighter. This in turn indicates that a flatter geometry of ramp upstream the gore is desired to allow the entering vehicle attains a higher speed at gore, and thereby a higher merging speed.

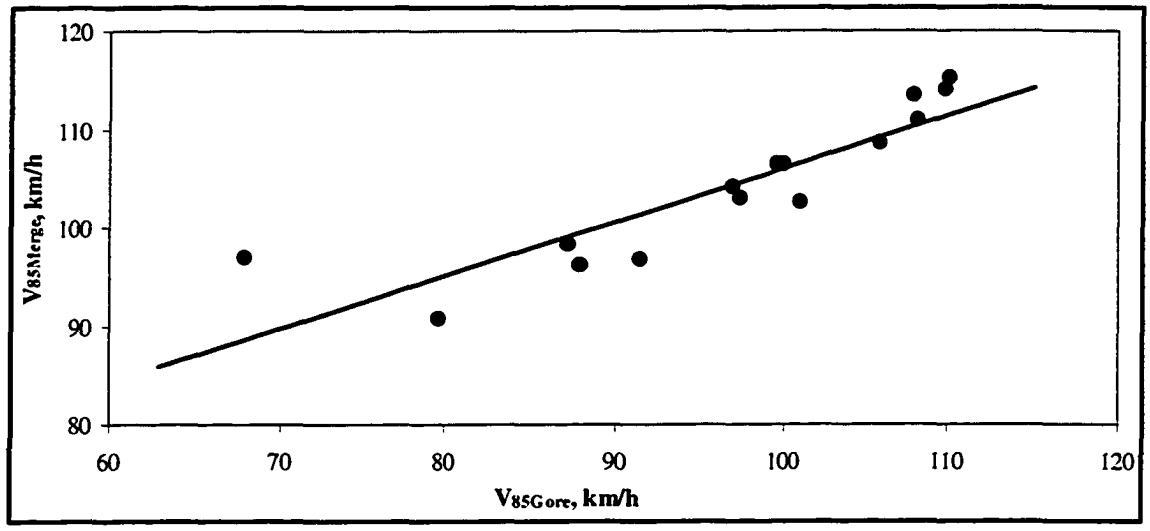


Figure 4.13: Effect of Gore Speed on Merging Speed

4.3.6 Effect of Traffic Volumes on Merging Speed

Figure 4.14 shows graphically the relationships of the $V_{85Merge}$ with the ramp equivalent hourly traffic (Q_{SCL}) and right lane equivalent hourly traffic (Q_{RL}) volumes. The trend in the figure indicates that the merging speed decreases as the entering volume increases. The trend for merging speed as related to right lane volume shows no remarkable change in merging speed with change in right lane volume. This indicates that right lane volumes have no significant effect (R^2 = 0.008) on merging speed during the off peak hours when the traffic merges freely into the freeway. The entering volume is shown to have better correlation with the merging speed with a correlation coefficient of 0.31 (R^2 = 0.097).

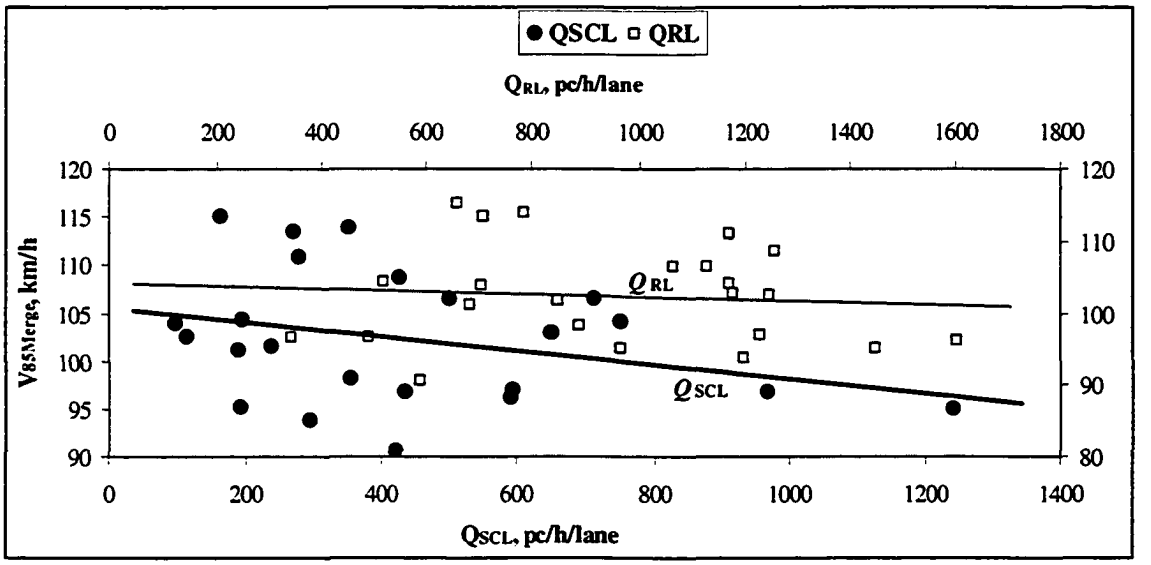


Figure 4.14: Effect of Traffic Volumes on Merging Speed

4.3.7 Effect of Ramp Curvatures on Merging Speed

Figure 4.15 shows the relationship of the angle of convergence at nose, where all paved area begins at upstream end of physical gore (θ), and radius of ramp controlling curve (R) with the $V_{85Merge}$. Only ramp radii of 300 m or less are used in this analysis, since for radius greater than 300 m the acceleration begins before the upstream end of ramp controlling curves. The trends in the figure show that as θ increases, the $V_{85Merge}$ decreases, and the $V_{85Merge}$ increases with increase in ramp radius. These further indicate that flatter ramp geometry allows the entering vehicles to merge at higher speeds.

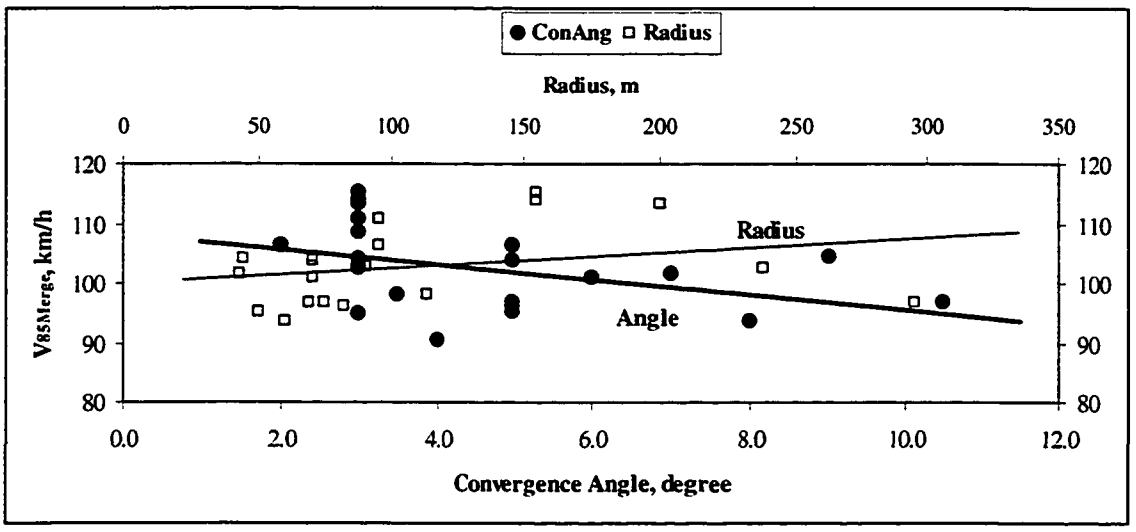


Figure 4.15: Effect of Ramp Curvatures on Merging Speed

4.4 SPEED DIFFERENTIALS BETWEEN RL AND SCL

A comparison of mean and 85th percentile PC merging speeds with those of freeway right lane speeds downstream the gore is shown in Figure 4.16. The histogram shows that these two speeds (merging and right lane speeds) vary widely among the sections used. Figure 4.17 shows the trend of speed disparities between the acceleration and right lane vehicles as related to the length of the acceleration lanes. As shown in the figure, speed disparities (ΔV_{85}) between the V_{85RL} and $V_{85Merge}$ decreases as the length of the limited length SCL increases.

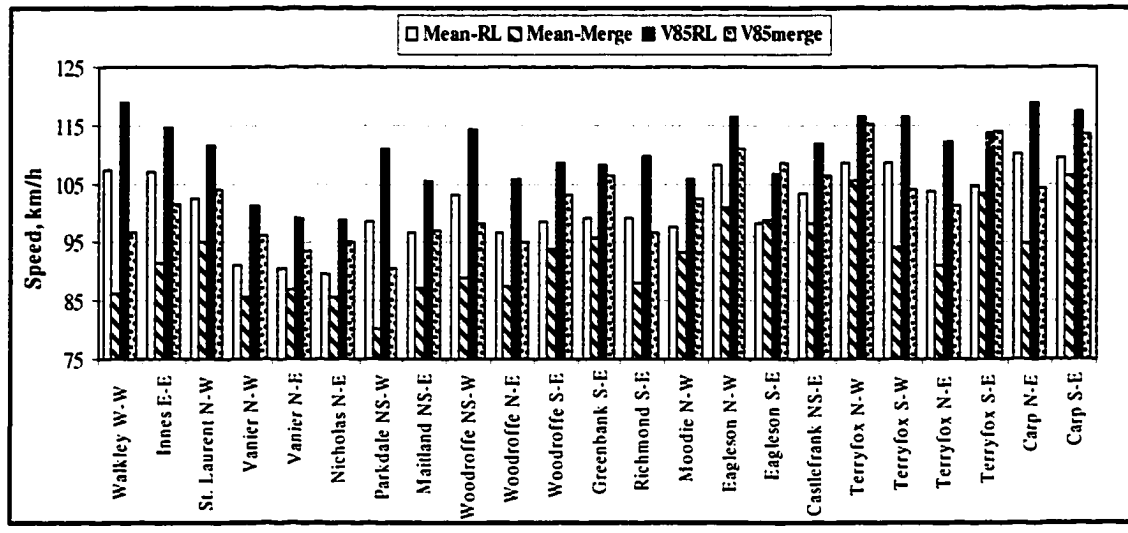


Figure 4.16: Comparison between Passenger Cars RL and Merging Speed

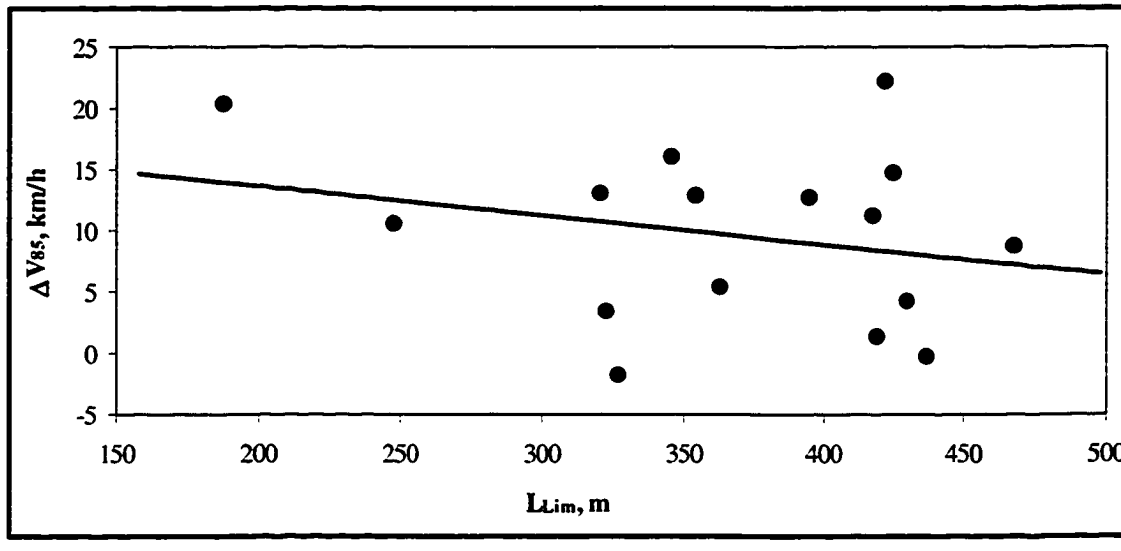


Figure 4.17: Effect of SCL Length on Speed Disparities between SCL and RL

Figure 4.16 shows that the differences in the mean and 85th percentile passenger cars right lanes and merging speeds ranged from -0.50 to 21.2 km/h and -1.90 to 22.2 km/h, respectively. At Eagleson S-E and Terryfox S-E, the $V_{85Merge}$ were 1.9 and 0.3 km/h higher than the V_{85RL}, respectively. The merging area at both of these sites is preceded by

outer connection ramps situated on downgrade and long transition from the end of controlling ramp curve, 69 to 78 m just from physical nose to the beginning of SCL. The radius of controlling ramp curve is over 300 m at Eagleson S-E and 155 m at Terryfox S-E. The lengths of the acceleration lane at these sites are 327 and 437 m, respectively. The relatively long SCL and/ or a long flat section upstream the gore justifies the smaller differences in speeds at these sites.

At all other sites, the mean and 85th percentile passenger car merging speeds were lower than those of the right lane speeds by 3.0 to 21.2 and 1.3 km/h to 22.2 km/h, respectively. The largest difference, $V_{85Merge}$ is 22.2 km/h lower than V_{85RL}, was observed at Walkley W-W. The speed difference at this site was followed by that at Parkdale NS-W, where the $V_{85Merge}$ was 20.3 km/h lower than the V_{85RL}. The acceleration lane at Walkley W-W is 422 m long, but it is preceded by a loop ramp of 69 m radius with a short transition from the end of controlling ramp curve, 19 m from physical nose, to the beginning of SCL. The equivalent hourly entering traffic (965 pc/h/lane) as well as entering and right lane HV (16.3 and 13.0 %, respectively) volumes at this site were high. All these factors probably justify the largest difference in speed at this site. On the other hand, the merging area of 188 m in length at Parkdale NS-W is the shortest one among the sites included in this study. The SCL at this site is preceded by a fairly straight but short ramp and a short transition, 36 m from physical nose to the beginning of SCL, and situated on upgrade. All these features explain the larger difference in speed at this site.

As noted from the above analysis, no single factor explains the differences in speeds between right lane vehicles and entering ramp vehicles. In general, higher differences may be attributed to a shorter length of acceleration lane, higher traffic volume, and/or

tighter ramp geometry upstream the gore, while lower differences may be attributed to longer length of SCL, lower traffic volume, and/or flatter geometry upstream the gore.

A *t*-test in SPSS showed that the differences in mean speeds between the right lane and merging vehicles are statistically significant at 5% significance level, except for two sites with difference in mean speed less than 3.0 km/h.

4.5 Overall Acceleration On The SCL

4.5.1 Distribution of Overall Acceleration Rates

The variations in passenger cars minimum, maximum, mean, and 85th percentile overall acceleration rates over the fifteen SCL with full profile data are shown in Figure 4.18. The highest of the 85th percentile passenger car overall acceleration rate (a_{85Over}) of 1.110 m/s^2 among the sites was observed on the SCL at Maitland NS-E, despite the longest (468 m) acceleration lane provided downstream the gore. The merging area at this site is preceded by a short transition, 17 m from physical nose to the beginning of SCL, largest (among the sites) angle of convergence at physical nose of 10.5°, and a long reverse curve loop ramp with controlling radius of 74 m. The speed attained by the 85th percentile passenger car at the beginning of SCL (V_{85Gore}) was 67.9 km/h which is the lowest among the sites, and the $V_{85Merge}$ was observed to be 96.9 km/h. Similarly, on the SCL at Woodroffe NS-W, the a_{85Over} was observed to be 0.931 m/s^2, the second highest among the sites. The SCL at this site is also preceded by a long reverse loop curve with a controlling radius of 113 m. The $V_{85Merge}$ was 98.2 km/h at this site, close to that at Maitland NS-E. An overall acceleration rate of 0.897 m/s^2 was observed at the SCL of

Parkdale NS-W. Both the ramp and SCL at this site are the shortest among the sites, and situated on an upgrade. All these resulted higher acceleration rate on the SCL at this site.

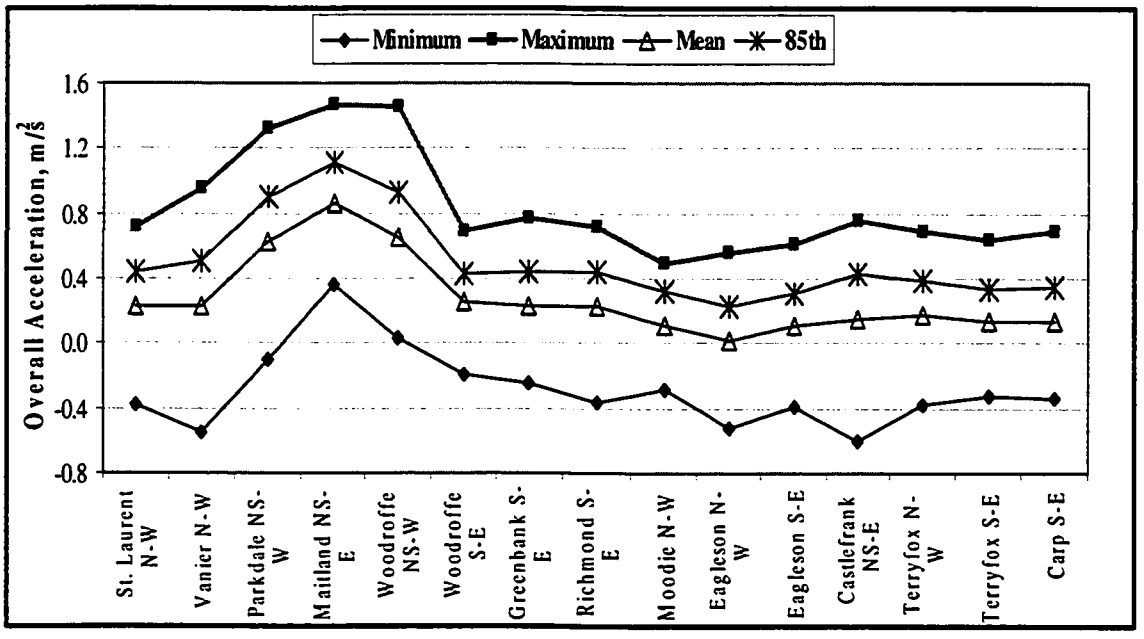


Figure 4.18: Distribution of Passenger Car Overall Acceleration Rates

On the other hand, the lowest (among the sites) a_{85Over} of 0.235 m/s^2 was noted at Eagleson N-W, an extended SCL of 673 m in length. The SCL at this site is preceded by a relatively long transition section, 78 m just from physical nose to the beginning of SCL, a flat angle of 3° at the physical nose, and a ramp with controlling curve radius of 95 m. The V_{85Gore} on this SCL was very high of 108.2 km/h as against a merging speed of 110. 9 km/h. Among the limited length SCL, the lowest a_{85Over} of 0.311 m/s^2 was observed at Eagleson S-E. The geometric features at this site include an acceleration lane of 327 m in length, preceded by a long flat transition section, also 78 m from the physical nose to the beginning of SCL, with convergence angle of 3° and a flat ramp curve of radius over 300 m. The V_{85Gore} and $V_{85Merge}$ on this SCL were 105.9 and 108.6 km/h, respectively.

The above analysis indicates that tighter ramp geometry upstream the gore and longer ramp curve restrain vehicle acceleration on section upstream the gore. At these situations, higher acceleration occurs on the SCL within a short length, even though a longer SCL is available downstream the gore. Therefore, it may further be concluded that tighter ramp geometry upstream the gore may not be fully compensated by longer acceleration lane lengths downstream the gore. All these together emphasize the importance of consistency in design. Since the different features of a roadway are parts of a system, the coordination among the elements are more important than just attempting to compensate one for the deficiency in other(s).

4.5.2 Effect of Length of SCL on PC Overall Acceleration Rates

As mentioned in previous section, the overall acceleration rates on the SCL at Maitland NS-E and Woodroffe NS-W are not dependent on the SCL lengths, rather they are controlled by the tight geometry of the ramp upstream the gore. The acceleration rates on these SCL fall outside the normal trend, and therefore these two sites are excluded from further analysis of the overall acceleration rates. Figure 4.19 shows graphically the relationship between the a_{85Over} and L_{Lim}. The trend in the figure shows that the overall acceleration rate decreases as the length of the SCL increases to 375 m. The overall acceleration rate does not decrease any more for SCL lengths of over 375 m (the lowest point on the trend).

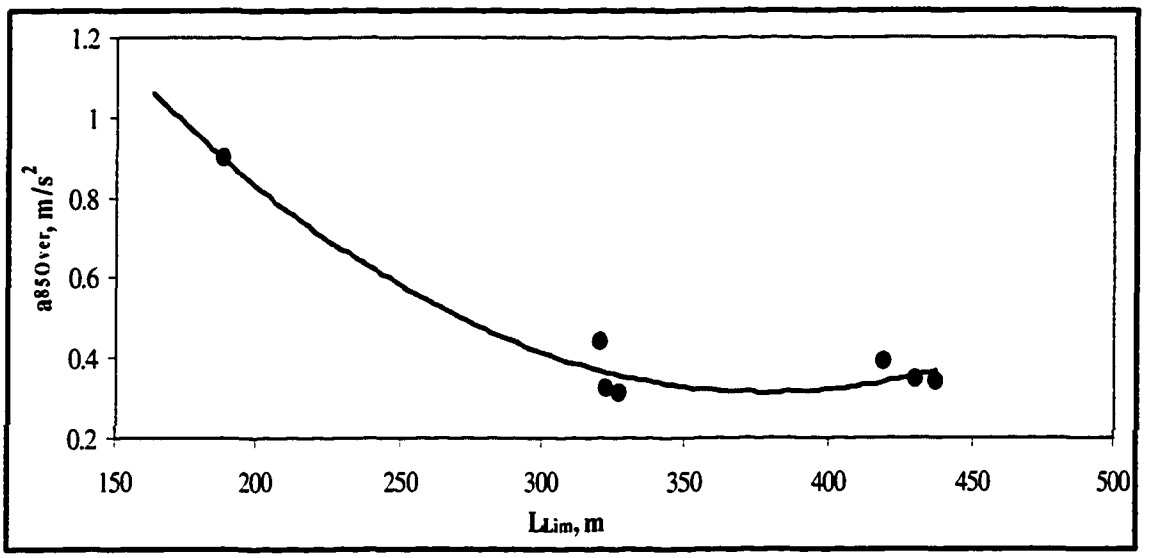


Figure 4.19: Effect of Limited Length SCL Lengths on Overall Acceleration

Among the extended SCL, the overall acceleration rate on Eagleson N-W was outside the trend of other sites. The V_{85Gore} and $V_{85Merge}$ were 108.2 and 110.9 km/h, respectively indicating that most acceleration occurs on the ramp. Therefore, this site is excluded from further analysis of overall acceleration on the extended SCL. The acceleration pattern on the extended SCL is shown in Figure 4.20, where a_{85Over} is shown to be fairly constant at about 0.4 m/s^2 (slight decrease with increase in length) for all SCL lengths. This indicates that the overall acceleration rate from the gore (beginning of the SCL) to the merging point is fairly constant for any length of the SCL of over 375 m. Therefore, it is probably an indication that the minimum length of SCL downstream the point, where the ramp and mainline pavement edges are 1.25 m apart, should be 375 m (including the taper) for completing the merging manoeuvres comfortably, given the availability of a suitable gap.

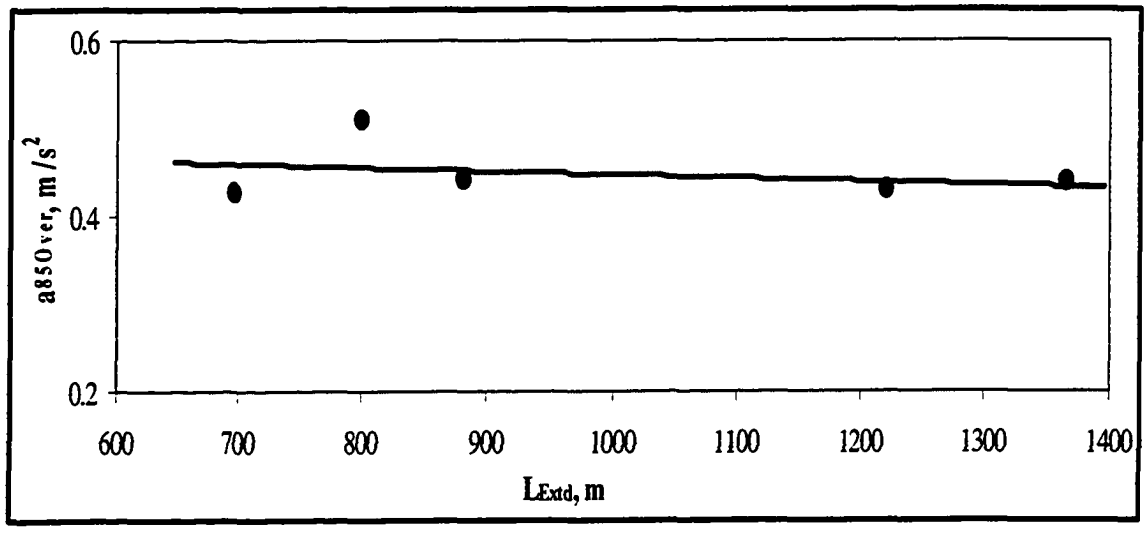


Figure 4.20: Effect of Extended SCL Lengths on Overall Acceleration

4.5.3 Effect of Gore Speed on Overall Acceleration Rates

Figure 4.21 shows the relationship between the V_{85Gore} and overall acceleration rates. The trend in the figure shows that as the V_{85Gore} increases the a_{85Over} decreases sharply. The relationship showed a very good correlation with R^2 of 0.83. This further indicates that the overall acceleration rate is highly dependent on the geometry of the entrance ramp upstream the beginning of SCL, since gore speed is dependent on the ramp geometry.

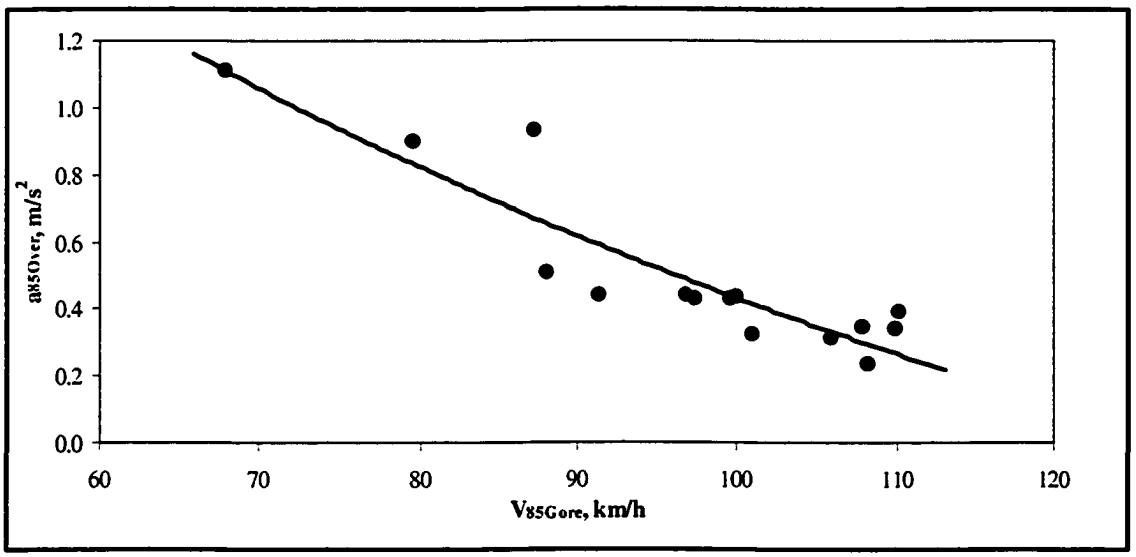


Figure 4.21: Effect of Gore Speeds on Overall Acceleration Rates

4.6 MAXIMUM ACCELERATION ON THE SCL

4.6.1 Distribution of Maximum Acceleration Rates

Figure 4.22 shows the variation in mean and 85th percentile passenger cars maximum acceleration rates on the SCL. These acceleration rates ranged from 1.169 to 2.300 m/s^2 and 1.551 to 2.971 m/s^2, respectively. The highest value of 85th percentile maximum acceleration rate of 2.971 m/s^2 was observed on Parkdale NS-W with the shortest SCL of 188 m in length and the shortest ramp situated on an upgrade. The lowest rate was observed at Moodie N-W with a SCL of 323 m in length and a long flat outer connection downgrade ramp with controlling curve radius of 234 m. The V_{85Gore} at Parkdale NS-W and Moodie N-W were 79.6 and 101.0 km/h, respectively. This probably indicates that the maximum acceleration rate of vehicles on the SCL depends on the geometry of the entrance terminal, both of downstream and upstream the gore that include SCL length,

ramp curvature and grade, length of transition from end of ramp curve to the beginning of SCL, etc.

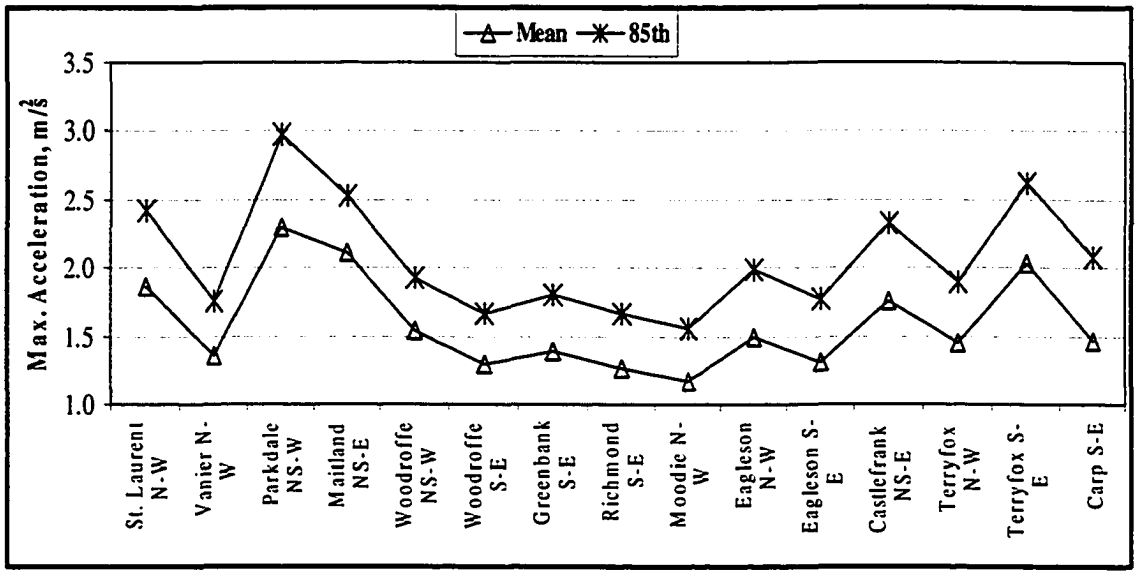


Figure 4.22: Variations in Passenger Car Maximum Accelerations

4.6.2 Effect of Geometry on Maximum Acceleration Rates

The maximum acceleration rates on both the limited length and extended SCL showed similar trends, and therefore they are combined in the analyzing the effect of SCL lengths on maximum acceleration rate. The maximum acceleration rate on Parkdale NS-W was outside the average trend of other sites because the entering vehicles have to accelerate hard to attain a higher speed within a short length available downstream the gore after traversing a short and upgrade ramp. The maximum acceleration rates on all other SCL with relatively longer lengths were shown to follow a constant trend. Figure 4.23 shows the relationship between the length of acceleration lane and the 85th percentile passenger car maximum acceleration rates (a_{85Max}), excluding Parkdale NS-W. The trend in the

figure shows that the a_{85Max} is fairly constant at 2.0 m/s^2 regardless the SCL lengths. Such a rate is most probably the maximum comfortable acceleration rate for the 85th percentile passenger vehicle.

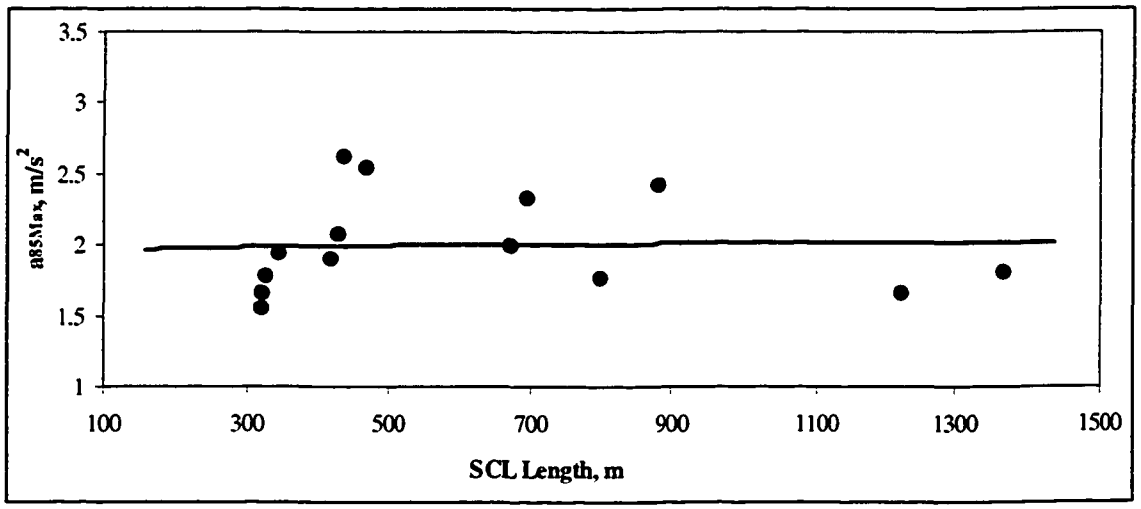


Figure 4.23: Effect of SCL Length on Maximum Acceleration

Figure 4.24 shows the relationship between the V_{85Gore} and a_{85Max}. The trend shows that the a_{85Max} decreases with the increase in the gore speed, although at a flatter rate than the overall acceleration. This probably indicates that maximum acceleration rate depends to a less degree on the geometry of the entrance terminal. Two errant points in the figure are Parkdale NS-W (short SCL and short upgrade ramp) and Maitland NS-E (long SCL but tight ramp geometry). The full geometric features of these sites were described earlier. Flat and smooth transition from the ramp to the SCL reduces stress on the vehicles, whereas tighter geometry of the ramp or acceleration lane places higher stress on vehicles. This further emphasizes the importance of coordination between the geometry at upstream and downstream the gore. It can then be concluded that the geometry of the

entrance ramp, both upstream and downstream the gore, should allow the entering vehicles not to exceed this maximum comfortable acceleration rate of 2.0 m/s^2.

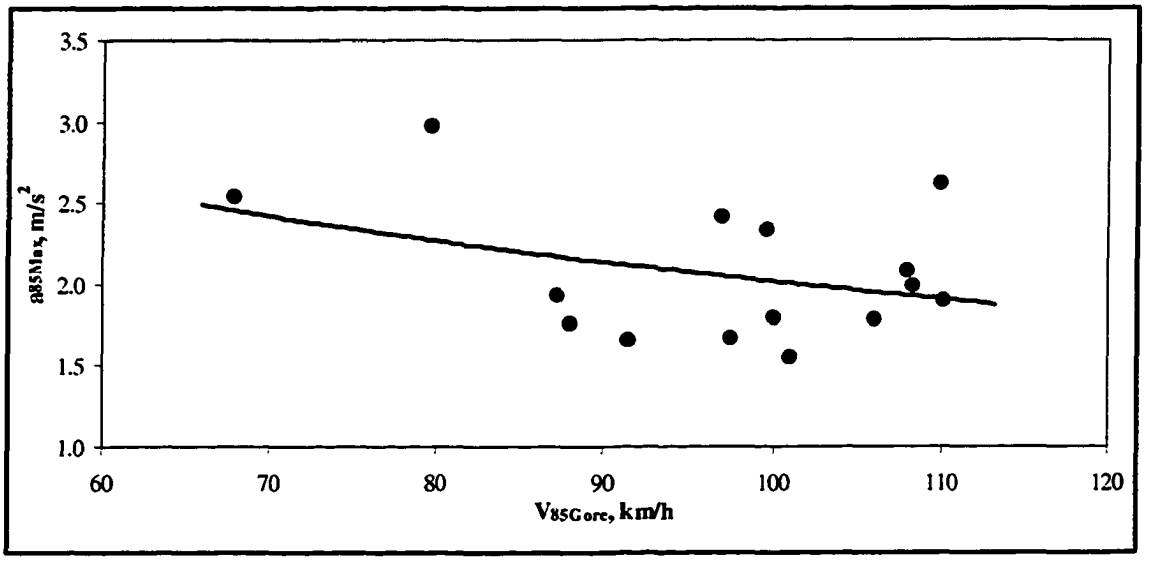


Figure 4.24: Effect of Gore Speeds on Acceleration Rates

4.7 MEAN ACCELERATION ON THE SCL

Figure 4.25 shows a graphical comparison between the 85th percentile passenger car mean and overall acceleration rates on the SCL. As shown in the figure, the variations in overall and mean acceleration rates are similar, and the differences between these two accelerations are not remarkable. The 85th percentile passenger cars mean acceleration rates (a_{85Mean}) ranged from 0.297 to 1.115 m/s^2 compared to the range of a_{85over} from 0.235 to 1.110 m/s^2 among the SCL. These indicate that the method used in calculating mean acceleration has successfully captured the variation in successive accelerations on the SCL, and chose interval between successive speed readings is the appropriate one.

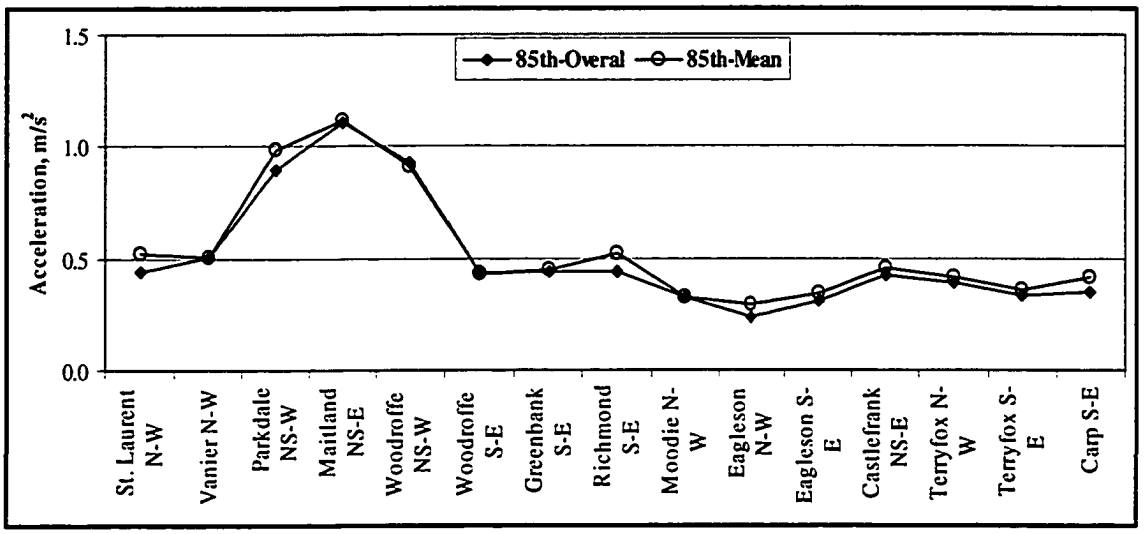


Figure 4.25: Passenger Car Mean and Overall Acceleration Rates

4.8 EFFECTIVE ACCELERATION (MERGING) DISTANCES

This section provides an analysis of the effective SCL lengths used for accelerating by entering vehicles. The effective length used is the distance from the beginning of SCL, where the ramp and mainline pavement edges are 1.25 m apart, to the point where the entering vehicles merge into the freeway.

4.8.1 Distribution of Merging Distances

The variations in passenger cars minimum, maximum, mean and 85th as well as 95th percentile merging distances are shown in Figure 4.26. The merging distances varied from a minimum of 3.4 m to a maximum of 450.6 m among the SCL. No vehicle merged upstream the beginning of SCL. This justifies the selection of the beginning point for calculating the SCL length. The 85th and 95th percentile passenger cars merging distances ranged from 112 to 293 m and 128 to 351 m, respectively. For the limited length SCL

alone, the 95th percentile passenger car merging distances ranged from 128 to 335 m. A comparison for the mean and 85th percentile merging distances between the combined and passenger vehicles are shown in Figure 4.27. The figure shows no noticeable difference in merging distances between these two groups for the SCL cases used in this study. In fact, the range of 85th percentile combined vehicle merging distances (112 to 292 m) was almost the same as that of passenger car (112 to 293 m). Subsequent analysis uses PC merging distances only.

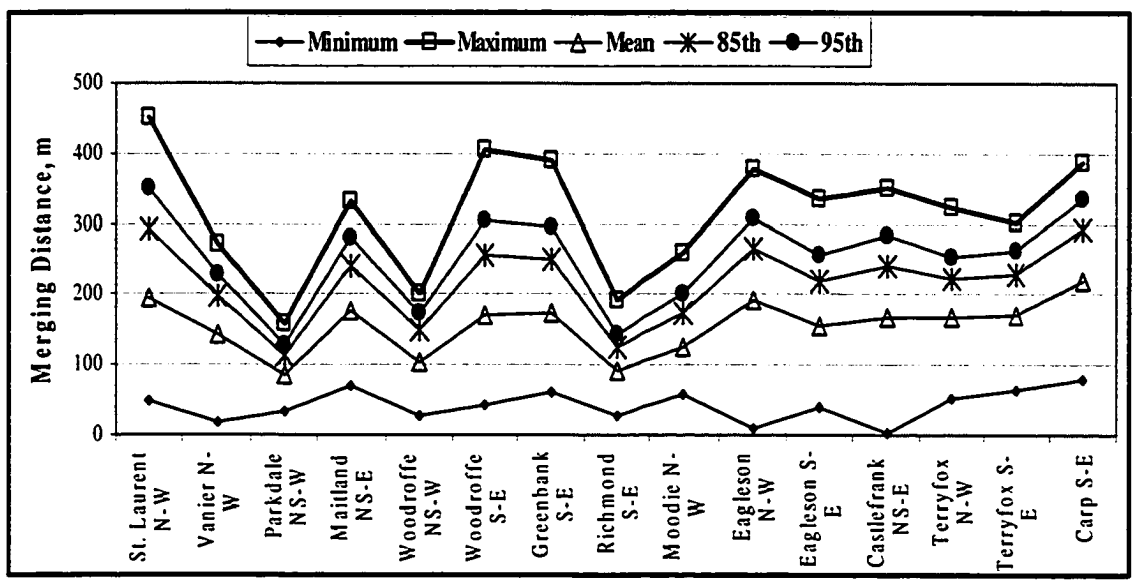


Figure 4.26: Variations in Passenger Car Merging Distances

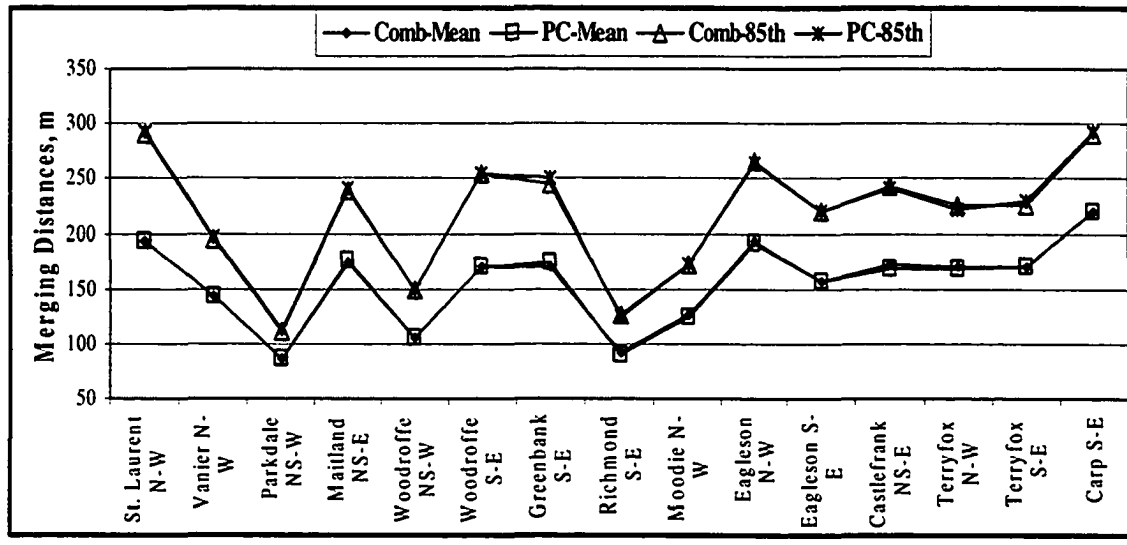


Figure 4.27: Combined versus Passenger Vehicles Merging Distances

4.8.2 Effect of SCL Lengths on Merging Distances

Figure 4.28 shows the relationship between the limited length acceleration lane and distance from the beginning of SCL to the 85th percentile passenger car merging points ($D_{85Merge}$). The figure shows that the $D_{85Merge}$ increases as the length of limited length SCL increases. However, only 39 to 68% of the provided L_{Lim} were used by 85% of vehicles. Figure 4.29 shows that the $D_{85Merge}$ is fairly constant regardless of the provided lengths of extended SCL. As shown in the figure, only parts (18 to 39%) of the extended SCL lengths were used in merging by 85% of entering vehicles. This further indicates that the SCL lengths in excess of certain minimum have no effect in improving the entering vehicles merging speed.

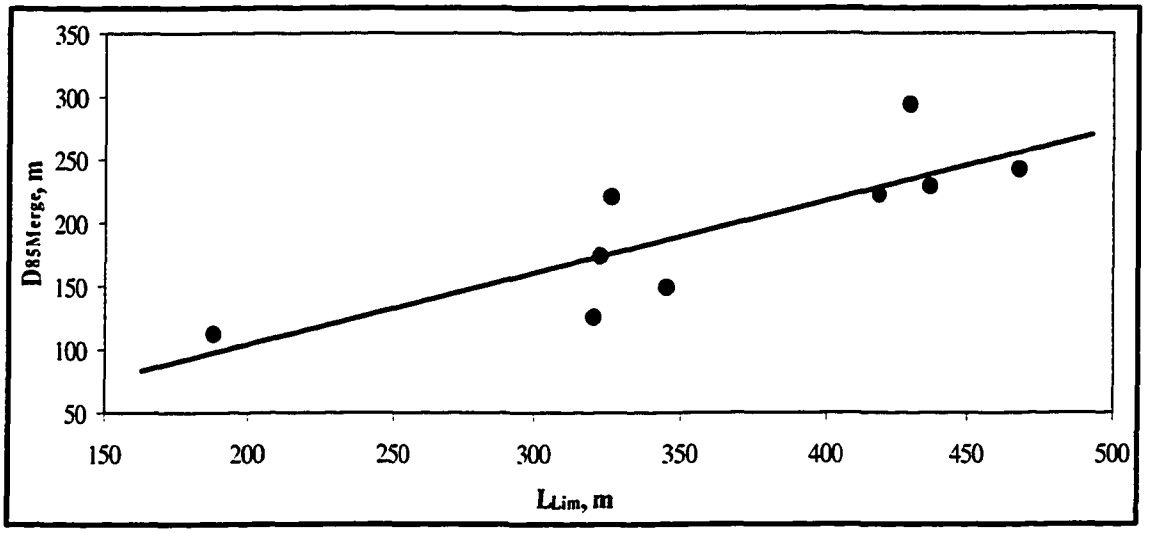


Figure 4.28: Effect of Limited Length SCL Length on Merging Distances

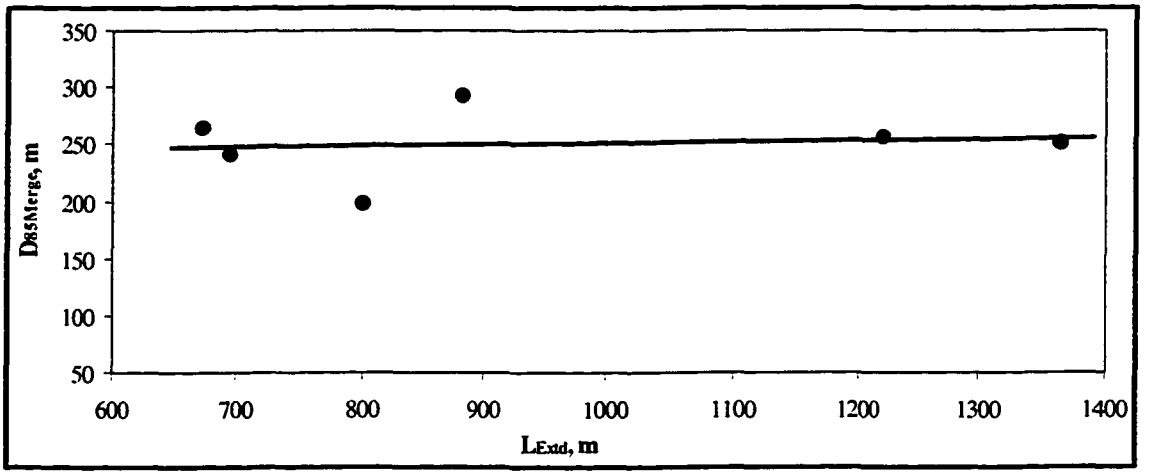


Figure 4.29: Effect of Extended SCL Lengths on Merging Distances

As mentioned in Section 4.8.1, the 85th percentile passenger car merging distance was found to be a maximum of 293 m. With a 90 m tapered section, to be provided for evasive action, as recommended in AASHTO (2004), the desired SCL length is 383 m to accommodate 85% of vehicles population. This desired length falls with the lengths required for acceleration and maximum merging speed, as found in Section 4.3 and 4.5.

Furthermore, the 95th percentile passenger car merging distance was observed to be a maximum of 335 m among the limited length SCL. Adding a 90 m taper, the desired SCL length is 425 m to accommodate 95% of vehicles population. This 95th percentile desired length matches with the length required for maximum merging speed, as found in Section 4.5. These indicate that a length of 425 m (including the taper), downstream the point where the ramp and mainline pavement edges are 1.25 m apart, is adequate for all ramp geometric and traffic conditions that allows the entering 85th percentile vehicle to attain a gore speed of at least 68 km/h. The minimum required length should be 375 m to complete the acceleration and merging manoeuvre comfortably. The length of limited length type SCL in excess of 425 m is not required in improving the merging behaviour, and therefore the additional cost may not be justified. Coordination between the geometry at upstream and downstream the gore is rather important in improving operation at merge areas.

4.8.3 Effect of Gore Speed on Merging Distances

Figure 4.30 shows the relationship between entering passenger vehicles speed at gore and merging distances from the gore. The trend in the figure shows that as the V_{85Gore} increases the $D_{85Merge}$ also increases. Such a trend seems to be counter intuitive, as higher gore speed should allow the drivers to merge within a short distance from the gore. The trend might have been disturbed by other factors such as length of acceleration lane and/or traffic volume.

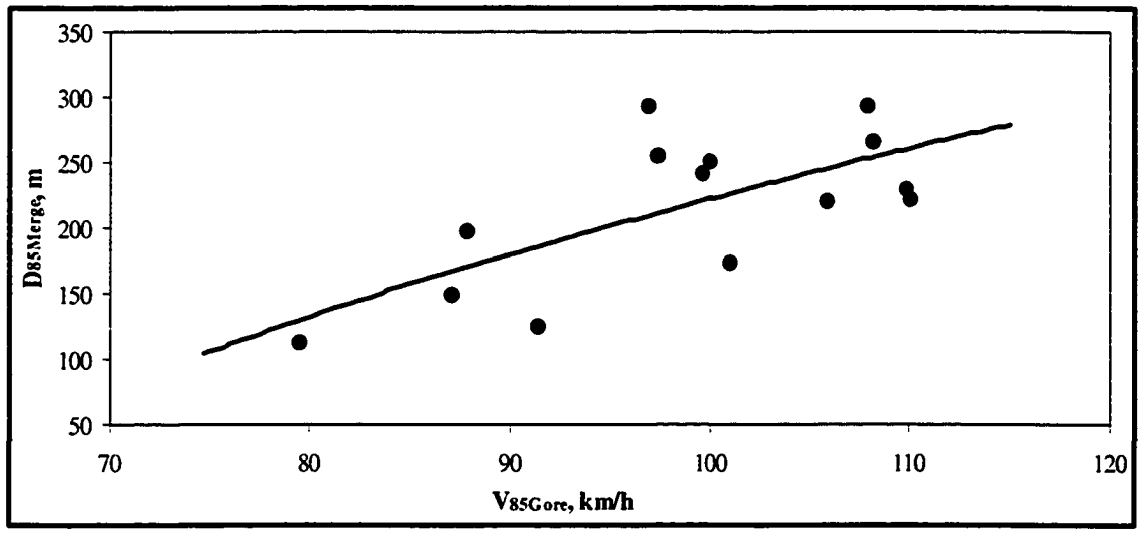


Figure 4.30: Effect of Gore Speeds on Merging Distances

4.8.4 Effect of Traffic Volumes on Merging Distances

This section examines the relationships of the merging distances with the right lane as well as the ramp traffic volumes. The trend for the $D_{85Merge}$ as related to the Q_{SCL} (Figure 4.31) indicates that the $D_{85Merge}$ is fairly constant (the trend showed poor correlation with R^2 of 0.046) for any entering traffic volume of ≤ 800 pc/h/lane. The merging distance increases with increase in right lane equivalent hourly volume to about 1200 pc/h/lane. For Q_{RL} >1200 pc/h/lane, the merging distance decreases with the increase in volume. The justification of such behaviours are that at lower right lane volumes, the through lane speeds are high as the vehicles can travel more freely, and entering vehicles have to wait for suitable gaps. On the other hand, at higher right lane volumes, the through lane speed is low. The entering vehicles attempt to occupy the right lane quickly, whereas the right lane vehicles would slow down to allow entering vehicles complete the merging manoeuvre.

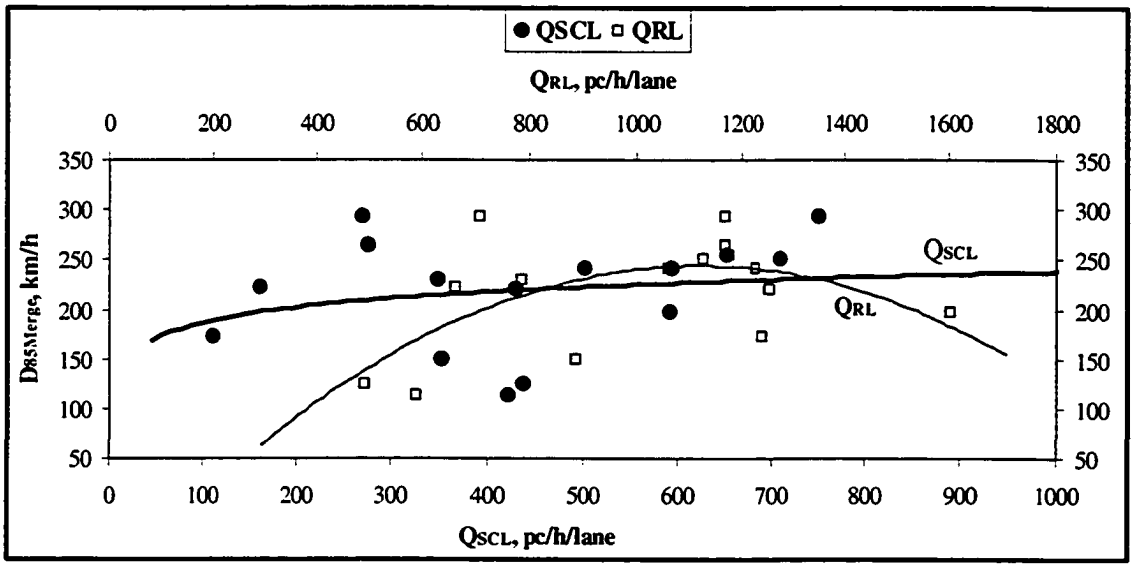


Figure 4.31: Effect of Traffic Volumes on Merging Distances

4.8.5 Effect of Ramp Curvature on Merging Distances

Figure 4.32 shows the relationship of the convergence angle (θ), and controlling ramp curve radius (R) with the $D_{85Merge}$. The trends in the figure show that the $D_{85Merge}$ decreases as both of the θ and R increases. Flatter ramp curve allows the drivers to merge within short distance from the beginning of the acceleration lane. This in turn demands a shorter acceleration lane to be provided at an entrance ramp. Smaller convergence angle at physical nose also demands a shorter SCL.

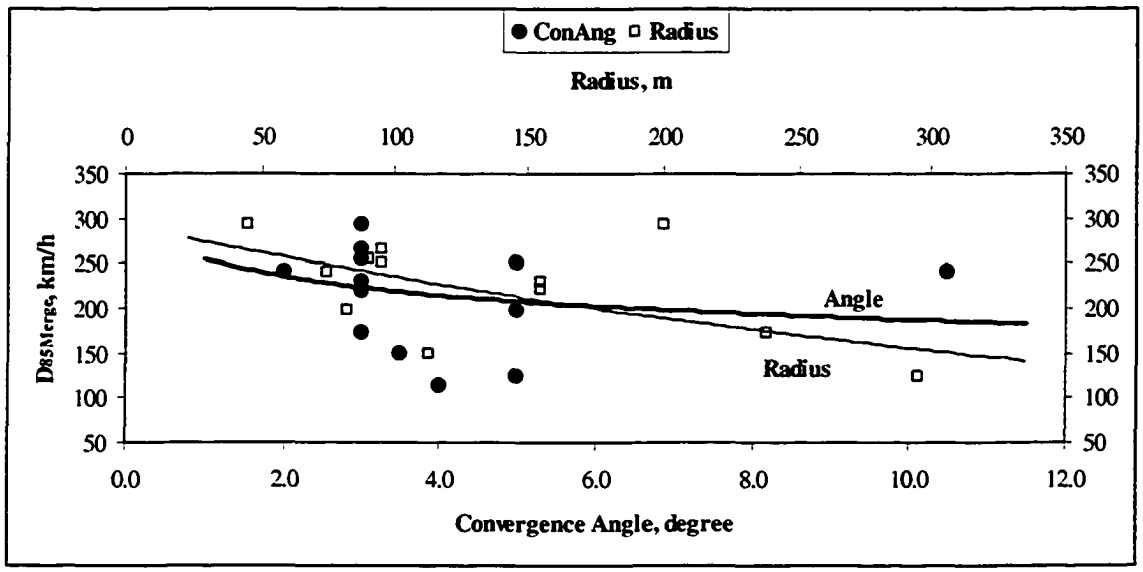


Figure 4.32: Effect of Ramp Geometry on Merging Distances

4.9 SUMMARY OF TRAFFIC BEHAVIOUR

The right lane speed along the SCL was shown to be affected by both right lane and entering traffic. The through lane speed was higher at sections away from the influence of merge area. Mean speeds of unimpeded right lane passenger vehicles were 3.6 to 12.4 km/h higher than those of impeded vehicles along the SCL, and higher speed differences were associated with shorter SCL. Both ramp and SCL geometrics were shown to affect the merging speed of entering vehicles. The $V_{85Merge}$ was shown to increase for SCL lengths up to about 425 m. The expected $V_{85Merge}$ corresponding to this length is about 105 km/h. An acceleration rate of 2.0 m/s^2 appeared to be the maximum comfortable acceleration rate for the 85th percentile passenger car. The merging distances were shown to increase with the increase of right lane equivalent volume up to 1200 pc/h/lane, a moderate traffic volume. For higher right lane volume over 1200 pc/h/lane, the merging distances were shown to decrease with increase in volume. An acceleration lane of 425 m

(including the taper) was shown to be adequate for all geometric and traffic conditions to allow comfortable merging of 95% of vehicle population. The additional cost for the limited length acceleration lane in excess of 425 m in length may not be justified.

Tighter ramp geometry and longer ramp curve were shown to restrain vehicle acceleration on the section upstream the gore. At this situation, higher acceleration was shown to occur downstream the gore and within a short length, despite the very long acceleration lane provided at downstream. This indicated that tighter ramp geometry may not be fully compensated for by longer acceleration lane, emphasizing the importance of consistency in design and coordination among the geometric features. Good coordination may be attained with the provision of a flat or downgrade ramp, long transition from the end of controlling ramp curve to the beginning of the SCL, and a reasonable length of the SCL for allowing the entering traffic to merge in a suitable gap. A downgrade ramp also has an advantage in terms of improved sight distance. It should be noted that the desired maximum length, mentioned above, could not be compared with design guide recommendations, since the lengths recommended in the guides begin at the end of controlling ramp curve.

5.0 REGRESSION MODELS

This chapter describes the modelling attempts covering all of the available geometric and traffic variables in multiple regression analysis. Stepwise multiple linear regression analysis method in SPSS 12.0 software was used in all modelling attempts, expect for the safety performance models for which SAS 8.2 was used. The description of the developed models as well as variables included in each model is also presented. The modelling attempts include prediction/ estimation of the 85th percentile passenger car merging speeds, merging distances, overall and maximum accelerations on the SCL, freeway right lane speeds downstream the gore, and safety performance of the acceleration lanes. Section 5.1 presents the modelling attempts for merging speeds, while Section 5.2 presents the merging distance modelling attempts. Section 5.3 presents the modelling attempts for prediction of acceleration on the SCL whereas Section 5.4 presents the modelling attempts for prediction of freeway right lane speed downstream the gore. Finally, Section 5.5 presents the modelling attempts for safety performance of acceleration lanes.

5.1 MERGING SPEED MODELS

This section presents the modelling attempts for estimation of the entering 85th percentile passenger car merging speeds ($V_{85Merge}$), and description of the developed models. The explanatory variables entered in the modelling attempt are:

i. SCL Length (Combined = L_{Comb}, Limited Length = L_{Lim}, Extended = L_{Extd}), m

ii. Radius of controlling ramp curve (R), m

iii. Angle of convergence at physical nose (θ), degrees

iv. Transition length (L_t), m

v. Pavement width (P_W), m

vi. SCL equivalent hourly volumes (Q_{SCL}), pc/h/lane

vii. RL equivalent hourly volumes (Q_{RL}), pc/h/lane

viii. PC Gore Speed (V_{85Gore}), km/h

ix. PC RL Speed (V_{85RL}), km/h

x. Ramp type code (*RampTyp*), outer connection = 1 and loop ramp = 0

xi. SCL type code (*Typ*), extended SCL = 1, Limited Length SCL = 0

xii. Ramp grade code (*RampGr*), downgrade = 1, upgrade = 0

xiii. Number of basic freeway lanes (*BL*)

xiv. RL heavy vehicle percentages (*RLHV*)

xv. SCL heavy vehicle percentages (*RampHV*)

Each predictor variable was transformed to several forms. The transformation of a predictor variable that showed a better trend in correlating with the dependent variable and improved the prediction power of the model, in terms of statistical significance (*p*-value), R^2 value and practical significance, was finally selected.

5.1.1 Merging Speed Models for Limited Length SCL

In the first modelling attempt for merging speeds for the limited length acceleration lanes, SPSS was run combining the gore speed with all other predictor variables listed above, except the upstream geometry. The model summary for the prediction of merging speeds is shown in Table 5.1. As shown in the table, only gore speed and length of SCL are

statistically significant at 5% level of significance. The model showed an excellent coefficient of determination ($R^2 = 0.99$). This means gore speed and length of the limited length SCL together explained 99% of variability in 85th percentile passenger car merging speeds. The gore speed alone explained 72.6% variability in merging speeds, as the entering vehicles attain this speed after traversing the section all the way from the ramp curve or intersection with arterials to the beginning of the SCL, and the effect of all geometric variables upstream the gore point (beginning of the SCL) is captured in the gore speed itself.

Table 5.1: Merging Speed Model Summary for Limited Length SCL (All Predictor Variables Included)

Model		Coefficients	t	p-value	R^2
1	(Constant)	59.934	4.761	0.005	0.726
	V_{85Gore}	0.470	3.639	0.015	
2	(Constant)	29.944	7.571	0.002	0.990
	V_{85Gore}	0.484	17.610	0.000	
	L_{Lim}	0.073	10.311	0.000	

The regression model for the estimation of passenger cars merging speed for the limited length SCL may be written as:

$$V_{Merge} = 29.944 + 0.484V_{85Gore} + 0.073L_{Lim} \qquad R^2 = 0.99 \qquad (5.1)$$

Where,

$V_{85Merge}$ = 85th percentile passenger car merging speed (km/h)

V_{85Gore} = 85th percentile passenger car gore speed (km/h)

L_{Lim} = Length of limited length SCL (m)

The positive signs assigned to the regression coefficients associated with both predictors, the gore speed and length of the SCL indicate that the merging speed increases as the gore speed and/or the length of SCL increases. Therefore, provision of ramp geometrics that allows the entering vehicles to attain a higher gore speed would increase the merging speeds, and thereby improve the traffic operations at freeway merge areas. Similarly, longer SCL would also assist in attaining higher speeds, and complete the merging manoeuvre safely and comfortably.

Attempts were made to develop a model excluding the speed at gore, as it may not always be practical to have an accurate estimate of the gore speed to estimate the merging speed. The resulting model is shown in Table 5.2. The inverse of the angle of convergence at physical nose, length of SCL and the equivalent hourly volumes of entering vehicles appeared to be the statistically significant predictors at 5% level of significance, in absence of gore speed, in explaining the variability in merging speeds for the limited length acceleration lanes. The model is statistically significant at 5% significance level (p-value = 0.00) with a very good coefficient of determination (R^2 = 0.852).

Table 5.2: Merging Speed Model Summary for Limited Length SCL (Based on Geometry and Traffic Volumes only)

Model		Coefficients	t	p-value	R^2
1	(Constant)	90.638	22.335	0.000	0.445
	$1/\theta$	53.682	3.105	0.009	
2	(Constant)	66.812	8.091	0.000	0.705
	$1/\theta$	55.869	4.236	0.001	
	L_{Lim}	0.061	3.112	0.010	
3	(Constant)	66.756	10.892	0.000	0.852
	$1/\theta$	49.640	4.971	0.001	
	L_{Lim}	0.075	4.941	0.001	
	Q_{SCL}	-0.013	-3.157	0.010	

An alternative model for the prediction of merging speed of entering vehicles for the limited length SCL, in absence of gore speed, may be given as:

$$V_{85Merge} = 66.756 + 49.640/\theta + 0.075L_{Lim} - 0.013Q_{SCL} \qquad R^2 = 0.852 \quad (5.2)$$

Where,

$V_{85Merge}$ = 85th percentile passenger car merging speed (km/h)

θ = Angle of convergence at physical nose (degrees)

L_{Lim} = Length of limited length SCL (m)

Q_{SCL} = Equivalent hourly volume of entering (ramp) vehicles (pc/h/lane)

The model in Equation 5.2 also shows that as the length of SCL increases, the merging speed increases, i.e. longer acceleration lanes improve traffic operation at freeway merge areas. As the volume of entering traffic increases, the merging speed decreases. Larger angles of convergence at nose would mean lower merging speeds compared to smaller angles, i.e. smaller angles improve merging behaviours and traffic operation at merge areas. Provision of a smaller angle at nose would mean flatter ramp curvatures and/or long flat transition upstream the physical nose at entrance ramps. However, smaller angle at nose should not be attained by abrupt change in curvatures. Rather, the section upstream the nose should provide a natural path that assists drivers in smooth transition from ramp curve to the acceleration lanes.

5.1.2 Merging Speed Models for Extended Type SCL

Attempts have been made with different combination of variables listed in Section 5.1 for developing merging speed model for the extended SCL. The summary of regression coefficients and their significance are shown in Table 5.3. For the extended type SCL the gore speed appeared to be the only significant predictor, at 5% significance level. The model parameters (regression coefficients) are statistically significant at less than 5% level of significance. The length of the extended SCL has no statistically significant effect on the merging speed.

Table 5.3: Merging Speed Model Summary for Extended (All Predictor Variables Included)

Model		Coefficients	t	p-value	R^2
1	(Constant)	31.163	4.601	0.010	0.967
	V_{85Gore}	0.745	10.843	0.000	

SPSS was run after excluding the gore speed and with different combination of variables. The Q_{RL} and Q_{SCL} were statistically significant at 5% level of significance, but the length was not statistically significant, as shown in Table 5.4. This further proves that traffic behaviour on the extended SCL is different from that on the limited length type.

Table 5.4: Merging Speed Model Summary for Extended (Based on Geometry and Traffic Volumes only)

Model		Coefficients	t	p-value	R^2
1	(Constant)	134.208	15.698	0.000	0.730
	Q_{RL}	-0.025	-3.676	0.014	
2	(Constant)	133.573	24.734	0.000	0.914
	Q_{RL}	-0.019	-4.179	0.014	
	Q_{SCL}	-0.009	-2.924	0.043	

5.1.3 Models for Combination of Limited Length and Extended Type SCL

SPSS was run with different combination of the predictor variables listed in Section 5.1 in a modelling attempt for merging speed for the limited length and extended type SCL combined together in a single model. A binary variable, *typ* =1 for extended SCL and 0 for limited length SCL, was included to capture the effect of different configurations. The gore speed was the only statistically significant predictor while the length of the SCL had no significant effect on the merging speed. Modelling attempts excluding the gore speed produced a model similar to that of the limited length type. However, the SCL type showed a wrong correlation, indicating that merging speed decreases on the extended SCL.

5.2 MODELS FOR MERGING DISTANCES

This section provides the modelling attempts for estimating the 85th percentile passenger car merging distances, which is the effective length used for acceleration. The 85th percentile passenger car merging speed ($V_{85Merge}$) was added as a predictor variable in the modelling attempts, in addition to those listed in Section 5.1.

5.2.1 Merging Distance Model for Limited Length SCL

The summary of the merging distance model is shown in Table 5.5. As shown in the table, only the SCL length is included in the model as the statistically significant predictor for the 85th percentile passenger car merging distances ($D_{85Merge}$). The model is shown to be statistically significant at a 5% level of significance with a fairly good R^2

value of 0.675. The regression coefficients are also statistically significant at 5% level of significance. Out of all the modelling attempts for merging distances, the square function of SCL lengths produced the best model in terms of statistical significance, both of the intercept and the regression coefficient associated with the length.

Table 5.5: Summary of Merging Distance Model for Limited Length SCL

Model		Coefficients	t	p-value	R^2
1	(Constant)	79.61	2.426	0.046	0.675
	L_{Lim}^2	0.001	3.811	0.007	

The model for the estimation of effective acceleration length used by the 85th percentile passenger car may be given as:

$$D_{85Merge} = 79.61 + 0.001L_{Lim}^2 \qquad R^2 = 0.675 \qquad (5.3)$$

Where,

$D_{85Merge}$ = 85th percentile passenger car merging distance (m)

L_{Lim} = Length of limited length SCL (m)

The model in Equation 5.3 is applicable for limited length SCL ≥90 m in length, since the predicted merging distance is greater than the provided length of less than about 90 m. However, it should be cautioned that the SCL length in the range of 90 to 188 m is not covered in the database used in this study and application of the model will be an extrapolation. The model has good predictability for SCL length up to 500 m (including the taper). For example, for a SCL length of 468 m (the maximum length in this study)

the predicted effective acceleration distance ($D_{85Merge}$) is 299 m. The 85th percentile passenger car merging distance was observed to be 293 m maximum for the SCL cases used in this study. A SCL length of 90 m is the practical minimum for a freeway facility of this nature, as AASHTO (2004) recommended a taper length of up to 90 m at the end of parallel acceleration lane for a freeway design speed up to 100 km/h. Furthermore, a length of 545 m, from the end of controlling ramp curve was recommended for a freeway design speed of 120 km/h, when the entering vehicles speed at the end of controlling ramp curve is zero (stop condition). Accordingly, a length of 500 m from the point where the ramp and mainline pavement edges are 1.25 to the end of taper is the desired maximum length for all practical design of SCL at freeway interchanges. Therefore, the developed model may be regarded as a useful one in estimating the effective acceleration length used, and thereby the total SCL lengths to be provided.

5.2.2 Merging Distance Model for Extended SCL

Attempts have been made with different combinations of variables and their several transformations for developing merging distance model for the extended SCL as well as combined cases of limited length and extended SCL. None of the variables explained the variability in the 85th percentile passenger car merging distances, at 5% significance level.

5.3 MODELS FOR ACCELERATION RATES

This section provides the modelling attempts for estimating of the 85th percentile passenger car overall and maximum accelerations on the speed-change lane, and

description of the developed models. All the predictor variables mentioned in Section 5.2, except RL lane speed, merging speed, gore speed and number of basic freeway lanes, were entered in the modelling attempts.

5.3.1 Model for Overall Acceleration

The summary of the developed model for the prediction of overall acceleration rate (a_{85Over}) on the limited length SCL is shown in Table 5.6. As shown in the table, the quadratic function of SCL length, i.e. the length and the square of length of limited length SCL, is statistically significant at 5% level of significance for predicting the a_{85Over}. The model showed an excellent R^2 value of 0.951.

Table 5.6: Overall Acceleration Model Summary for Limited Length SCL

Model		Coefficients	t	p-value	R^2
1	(Constant)	1.086	4.413	0.005	0.627
	L_{Lim}	-0.002	-2.900	0.034	
2	(Constant)	2.605	8.386	0.001	0.951
	L_{Lim}	-0.012	-6.011	0.004	
	L_{Lim}^2	$1.596*10^{-5}$	5.128	0.007	

The regression model for the estimation of overall acceleration on the SCL then may be given as:

$$a_{85Over} = 2.605 - 0.012L_{Lim} + 1.596*10^{-5}L_{Lim}^2 \qquad R^2 = 0.951 \qquad (5.4)$$

Where,

a_{85Over} = 85th percentile passenger car overall acceleration on the SCL (m/s^2)

L_{Lim} = Length of limited length SCL (m)

The model in Equation 5.4 is valid for SCL length up to 375 m (including the taper), since the trend for overall acceleration rate was shown to be fairly constant for SCL lengths exceeding 375 m (Section 4.5). The model is useful in estimating the expected acceleration on the SCL to be provided at a merging site, and in trade-off between the geometry at upstream and downstream the gore. A shorter SCL is associated with higher acceleration rates on the SCL downstream the gore. In this situation, the provided geometry upstream the gore should be flatter, allowing a smooth transition from the ramp curve to the SCL, so that part of the acceleration occurs before reaching the gore.

5.3.2 Model for Maximum Acceleration

Modelling attempts for maximum acceleration rates (a_{85Max}) on the limited length SCL showed that regression coefficient associated with the length of SCL is statistically significant at a 5% level of significance, although the constant term is not statistically significant (Table 5.7). However, the model does not make practical sense as the sign associated with the L_{Lim} indicates that maximum acceleration rate increases with the increase in SCL lengths.

Table 5.7: Maximum Acceleration Model Summary for Limited Length SCL

Model		Coefficients	t	p-value	R^2
1	(Constant)	-0.228	-0.372	0.725	0.737
	L_{Lim}	0.006	3.741	0.013	

Modelling attempts for the extended SCL showed that only the binary variable for ramp grade is statistically significant at a 5% level of significance, indicating that a downgrade ramp assists in vehicle acceleration. However, without any quantitative variable in the model, the developed model would have no practical use. No predictor variable was shown to be statistically significant at a 5% level of significance for maximum acceleration rate when combining the limited length and extended type. This further indicates that the maximum acceleration rate is fairly constant, if not constrained by tight entrance terminal geometry.

5.4 MODELLING FREEWAY RIGHT LANE SPEED

This section presents the modelling attempts for the prediction of 85th percentile passenger car average speed (V_{85RL}) on freeway right lane downstream the gore, and description of the developed models. The explanatory variables entered in the modelling attempts are:

i. SCL equivalent hourly volumes (Q_{SCL}), pc/h/lane

ii. SCL HV % (*RampHV*)

iii. RL equivalent hourly volumes (Q_{RL}), pc/h/lane

iv. RL HV% (*RLHV*)

v. Merging speed of entering vehicles ($V_{85Merge}$)

The model summary for the prediction of V_{85RL} is shown in Table 5.8. As shown in the table, right lane equivalent hourly volume and merging speed of entering passenger vehicles are statistically significant at 5% level of significance, in predicting the freeway right lane speed along the SCL. The model showed a good coefficient of determination (R^2) of 0.777.

Table 5.8: Model Summary for Prediction of Right Lane Speed

Model		Coefficients	t	p-value	R^2
1	(Constant)	123.622	47.181	0.000	0.568
	Q_{RL}	-0.014	-5.259	0.000	
2	(Constant)	81.937	8.353	0.000	0.777
	Q_{RL}	-0.013	-6.722	0.000	
	$V_{85Merge}$	0.400	4.334	0.000	

The regression model for the estimation of right lane speed downstream the gore may be given as:

$$V_{85RL} = 81.937 - 0.013Q_{RL} + 0.400V_{85Merge} \qquad R^2 = 0.777 \qquad (5.5)$$

Where,

V_{85RL} = 85th percentile passenger car RL speed along the SCL (km/h)

$V_{85Merge}$ = 85th percentile passenger car merging speed (km/h)

Q_{RL} = Equivalent hourly volumes on the right lane (pc/h/lane)

The developed model shows that freeway right lane speed decreases as the right lane traffic volume increases and right lane speed increases as the merging speed increases. This in turn indicates that the through traffic is interrupted when the merging speed of entering vehicles is lower compared to the speed of through traffic. Therefore, provision of SCL and ramp geometrics that allow the entering vehicles to attain a speed close to the freeway right lane speed and merge in a suitable gap, without interrupting the through traffic, is important to maintain the concept of uninterrupted flow.

5.5 SAFETY PERFORMANCE MODELS

This section examines the effect of geometry of merge areas, traffic behaviour on and along the acceleration lanes, and traffic exposure or volume on the safety performance of the acceleration lanes. Attempts have been made to develop models for the prediction of collision frequencies expected to occur on a section along, and including, the acceleration lanes. Attempts have also been made to develop models for the number of collisions expected to occur on the acceleration lanes alone. Sarhan (2004) developed several models relating the geometry of the merging and diverging areas, and traffic exposure to the expected number of collisions on speed-change lanes as well as on sections within the merge and diverge areas of the freeway interchanges, using collision data for the same study area. A higher number of data points were used in the mentioned study in the modelling attempts as compared to this research study. However, speeds of the vehicles on the speed-change lanes as well as freeway mainline were unavailable, and were not captured in the modelling attempts or developed models in Sarhan (2004). Therefore,

collision modelling attempts in this study were limited to combining the speeds or accelerations with other geometric and traffic data as predictor variables. The following sections outline the modelling technique and modelling attempts together with description of the successful models.

5.5.1 Modelling Technique

Collisions are rare and discrete or random event with non-negative values. The traditional linear regression or least square technique assume continuous and normal distribution of data, and the resulting models may yield negative values whereas collisions are always positive or null values. Therefore, use of traditional linear regression techniques in modelling the collisions has been the subject of criticism among the researchers (Sarhan 2004). Poisson and negative binomial regressions accommodate the randomness of the collisions and therefore, are now well accepted techniques among the researchers for modelling the collisions expected to occur on a section of highway or at an intersection.

Both Poisson and negative binomial distributions are positively skewed with discrete distribution, and are based on common assumptions that the events are random and independent of each other. In the Poisson distribution, the variance is equal to the mean (μ) and does not take into account the over-dispersion of the data, resulting in an increased variation in the predicted collisions. Over-dispersion is a usual phenomenon in collision data where only few known and significant predictor variables are being included in all practical models with omission of other significant and unknown factors that cannot be captured or accommodated within the scope of research studies. The negative binomial regression adds an extra quadratic term ($\alpha\mu^2$) to the variance to capture

the over-dispersion and therefore, may be more appropriate in modelling the collisions. The coefficient associated with the quadratic term is called the dispersion parameter. Attempts have been made with both Poisson and negative binomial techniques, and the appropriate technique was selected based on statistical tests.

Pearson Chi-square test and scaled deviance are used in assessing the adequacy of the Poisson models in explaining the variability in collisions. If the value of over-dispersion parameter (τ) is significantly larger than 1, over-dispersion is indicated and Poisson model should be considered inappropriate for the predictive models. Negative binomial regression would be considered as the second option in this situation. Sarhan (2004) used a typical range of 0.8 to 1.2 for the value of over-dispersion parameter in determining the adequacy of the Poisson models. If the value falls outside this range, over-dispersion ($\tau >$ 1.2) or under-dispersion ($\tau < 0.8$) is indicated. Since this research is a second phase following Sarhan (2004), the same criteria were used in this study.

5.5.2 Data Points and Variables Used

Collision data were available for 22 sites, as collision records were not available for the new construction of Castlefrank NS-E. The dependent variable of collision models is the total collisions, which is a simple summation of different type of collisions (fatal, injury and PDO). Attempts have been made to develop models for total collisions on the section along the SCL (*TCol*), total collisions on the acceleration and through lanes, excluding collisions on deceleration lane (*TColThAcc*), and total collisions on the acceleration lane (*TColAcc*). The explanatory variables included in the modelling attempts are:

i. Traffic exposure (*TExpo*) in million vehicle-km (*Expo* = total exposure on section, *ExpoThAcc* = exposure on through lanes plus acceleration lane, and *ExpoAcc* = exposure on acceleration lane)

ii. Total (*AADT*), through plus entering (*ThEnAADT*), and entering (*EnAADT*) volumes in vehicles/day

iii. Length of the acceleration lanes (L_{acc}) in meters

iv. Length of the segments on which collisions were reported (*L*seg) in km

v. Speed disparities between right lane and entering vehicles (ΔV_{85}) in km/h

vi. 85th percentile right lane speed (V_{85RL}) in km/h

vii. Entering 85th percentile vehicles merging speed ($V_{85Merge}$) in km/h

viii. Number of basic through lanes (*BL*)

ix. 85th percentile passenger car accelerations on SCL (a_{85over} and a_{85Max})

x. SCL type (*Typ*): Coded as 1 for extended type and 0 for limited length type.

Traffic exposure is considered as one of the most important variable in collision modelling since it captures the effect of variable lengths and traffic volumes of different segments on which collisions are reported. The underlying assumption of exposure being an important variable is that the more a road section exposed to traffic the higher the risk of collision to the road users. Therefore, traffic exposure (or *AADT* plus segment length) was entered in each modelling attempts in addition to speed differences or merging speed and or right lane speed(s) or accelerations as predictor variable(s). The exposure was calculated as:

$$Expo = (Years) * (AADT) * (365) * (L_{seg}) * (10^6) \qquad (5.6.a)$$

$$ExpoThAcc = (Years) * (ThEnAADT) * (365) * (L_{seg}) * (10^6) \quad (5.6.b)$$

$$ExpoAcc = (Years) * (EnAADT) * (365) * (LACC) * (10^6) \quad (5.6.c)$$

Where,

Years = Study period (time period of collision data)

LACC = SCL length (km)

5.5.3 Model Forms, Parameter Estimation and Goodness of Fits

As mentioned earlier, the number of collisions is a positive event and therefore, the model forms should ensure that the predicted number is a positive one. The exponential relationship of explanatory variables in collision model meets the above criteria and hence, considered the appropriate form. Awatta (2003) and Sarhan (2004) successfully developed several statistically significant models assuming the mentioned form for the prediction of collisions on highway sections. The same form was assumed in modelling attempts in this research. In addition, attempts have also been made with natural log transformation of segment and acceleration lane length, and the annual average daily traffic, as used by Anderson et al. (1999) and Ng et al. (2004). The attempted model forms are shown in Equation 5.7 and 5.8.

$$Y = e^{\beta_0 + \beta_1 * TExpo + \sum \beta_i x_i} \quad (5.7)$$

$$Y = e^{\beta_0} * L^{\beta_1} * TVol^{\beta_2} * e^{\sum \beta_i x_i} \quad (5.8)$$

Where,

Y = Five-year total collisions

β_0 = Intercept

β_i = Regression parameters associated with the predictors

L = Length (*Lseg* or *Lacc*)

x_i = Other predictor variables

TExpo = Traffic Exposure (*Expo* or *ExpoThAcc* or *ExpoAcc*)

TVol = Traffic volume (*AADT* or *ThEnAADT* or *EnAADT*)

SAS statistical software was used in both Poisson and negative binomial regression modelling attempts. The generalized linear model procedures PROC GENMOD in SAS estimates the coefficients associated with the explanatory variables based on the maximum likelihood. The significance of model parameters (coefficients) was evaluated based on the *p*-values in the model output. If the *p*-value is less than an acceptable level (usually 0.05) the null hypothesis is rejected and the subject variable is considered as statistically significant in explaining the variability in collisions.

5.5.4 Collision Models

The adequacy of Poisson model was evaluated based on values of over-dispersion parameters in SAS outputs in several modelling attempts. The over-dispersion parameters based on both scaled deviance (D^m) and Pearson Chi-square (χ^2) were far above the generally accepted range of 0.8 to 1.2. Therefore, it was concluded that the collision data in this study are highly dispersed, and the Poisson regression is not appropriate in collision modelling with the available data. Collision modelling attempts using the negative binomial regression techniques showed substantial improvement in terms of

accommodating over-dispersed data. Appendix C and D show samples of negative binomial regression SAS outputs for sixteen modelling attempts with different combinations of predictor variables. The over-dispersion parameters based on both of deviance (D^m) and Pearson Chi-square (χ^2) ranged from 0.76 to 2.25 for the modelling attempts, which are much closer to the acceptable range of 0.8 to 1.2, compared to the Poisson model. Therefore, the negative binomial regression technique was considered as the better method, over the Poisson, for collision modelling in this study.

As mentioned earlier, collision modelling attempts in this research study were limited to combining the speeds, speed disparities, or acceleration rates with other geometric and traffic data as predictor variables. Separate attempts were made for limited length acceleration lanes as well as combined cases of limited length and extended acceleration lanes. Sets of two to four predictor variables were entered in the modelling attempts which resulted in a large number of model outputs. Only a single model for total collisions on acceleration lanes for combined case of limited and extended type (*TColAcc*) was found to be statistically significant at a 5% level of significance and practically meaningful as well. The summary of this statistically significant model is shown in Table 5.9. As shown in the table, the over-dispersion parameters based on deviance (D^m) and Pearson Chi-square (χ^2) are 1.2 and 1.1 respectively, which are within the acceptable range of 0.8 to 1.2. The model showed a perfect R^2 value of 1.0. It should be noted that R^2 value that is commonly used in linear regression is considered as an unsatisfactory measure of goodness of fit for Poisson and negative binomial regression models, as it does not seem to have a good statistical interpretation (Miaou 1996). Rather, the R^2 value shown in the table was calculated using the alternative criterion suggested by

Miaou (1996). However, the criterion does not reflect the number of covariates included in the model. Therefore, R^2 values for the Poisson and NB model should not be interpreted in a way similar to the linear regression model.

Table 5.9: Model Summary for Total Collision on Acceleration Lanes

Predictor	Coefficients	*p*-value	D^m/DF	χ^2/DF	R^2
Constant (β_0)	4.2025	<0.0001	1.2247	1.1267	1.0
ExpoAcc	0.0951	<0.0001			
$V_{85Merge}$	-0.0321	<0.0001			
Typ	-1.9927	<0.0001			

The model for five-year total collisions on acceleration lane then may be given as:

$$TColAcc = e^{(4.2025 + 0.0951 * ExpoAcc - 0.0321 V_{85Merge} - 1.9927 * Typ)} \quad (5.9)$$

Where,

TColAcc = Five-year total collisions on the acceleration lane

ExpoAcc = Traffic exposure on the acceleration lane, million veh-km

$V_{85Merge}$ = 85th percentile passenger car merging speed, km/h

Typ = Code for the SCL type, 1 for extended SCL, 0 for limited length SCL

Equation 5.9 shows that collisions on the SCL are expected to increase with the increase in traffic exposure, and decrease with the increase in merging speed. Therefore, higher merging speeds, closer to freeway speeds, would reduce collision potentials on the

acceleration lanes. The developed model in this study suggests that total collisions on extended SCL are expected to decrease when the acceleration lane is extended to exit gore at next interchange, as shown with negative sign for regression parameter associated with the SCL type code (*Typ*).

The speed difference between the freeway right lane adjacent to the acceleration lane and the merging speed was not statistically significant at a 5% level of significance, although the merging speed itself was shown to be significant one. This is probably due to the fact that the calculated speed differences underestimate the effect of merging vehicles, as they do not account for the impediments in speed of right lane vehicles. As shown in the right lane speed model (Equation 5.5), the merging speed of entering vehicle is statistically significant in predicting the right lane speed downstream the gore. In the developed model, 21% of the variability in right lane speed was shown to be explained by the merging speed of entering vehicles, and right lane speed was shown to increase with an increase in the merging speed. The actual speed difference between the RL and SCL would have been higher than that calculated for the analysis. For example, the observed V_{85RL} at a site is found to be 100 km/h with a $V_{85Merge}$ of 95 km/h. The calculated speed difference (ΔV_{85}) is then just 5 km/h. However, the right lane speed itself might have been subjected to a reduction due to the influence of the entering vehicles merging speed. For instance, say the impediment is 5 km/h. The actual right lane speed should have been 105 km/h, if not interrupted by the merging vehicles. Then the actual speed difference (ΔV_{85}) should have been 10 km/h, whereas the observed/ calculated difference is 5 km/h, indicating that the calculated ΔV_{85} is an under-estimate the actual speed difference.

6.0 APPLICATION OF THE DEVELOPED MODELS

A number of attempts have been made in this research to develop models for the prediction/ estimation of merging speeds, merging distances and acceleration behaviour on the entrance terminals, and speed on freeway right lanes along the acceleration lanes. Attempts have also been made to develop safety performance models for the acceleration lanes as related to speed, acceleration, traffic exposure and geometry of the SCL as well as through lanes. Four models have been successfully developed for the prediction of merging speed, overall acceleration on the SCL, and merging distance (from the beginning of the SCL) of entering vehicles. One model was developed for the estimation of freeway right lane speed along the SCL, and finally one more model was successfully developed for the safety performance of the acceleration lane. The merging speed, acceleration and merging distance models are expected to help the designers and highway professionals in selecting the geometry of the limited length acceleration lanes to be provided for comfortable and safe merging of the entering vehicles. The right lane speed model would help designers and highway professionals estimate the freeway right lane speeds at prevailing right lane traffic volume as well as the entering traffic behaviour. This would in turn also assist the designers to adjust the geometry of entrance ramps that allows the entering vehicles to attain higher speeds on the SCL and thereby, allow the mainline (right lane) traffic operation at higher speeds with no or minor interruptions by the entering vehicles. This section provides application examples of the models successfully developed and presented in previous chapter. It should be noted that the developed models reflect the off-peak hour conditions. Designers should also assess the

requirements to accommodate the peak-hour traffic before final selection of the geometric features. Furthermore, the developed models were based on data collected from a single freeway. The applicability of the models to other freeway and interchange geometric conditions should also be checked.

6.1 APPLICATION OF MERGING SPEED MODELS

Several graphs were created based on the models given in Equations 5.1 and 5.2 for the estimation of merging speed and length of acceleration lanes to a attain the desired merging speed. Figure 6.1 shows a graph for the determination of 85th percentile passenger car merging speed based on the provided length of the acceleration lane or the length of acceleration lane to be provided to attain the desired merging speed for different gore speeds (50 to 100 km/h).

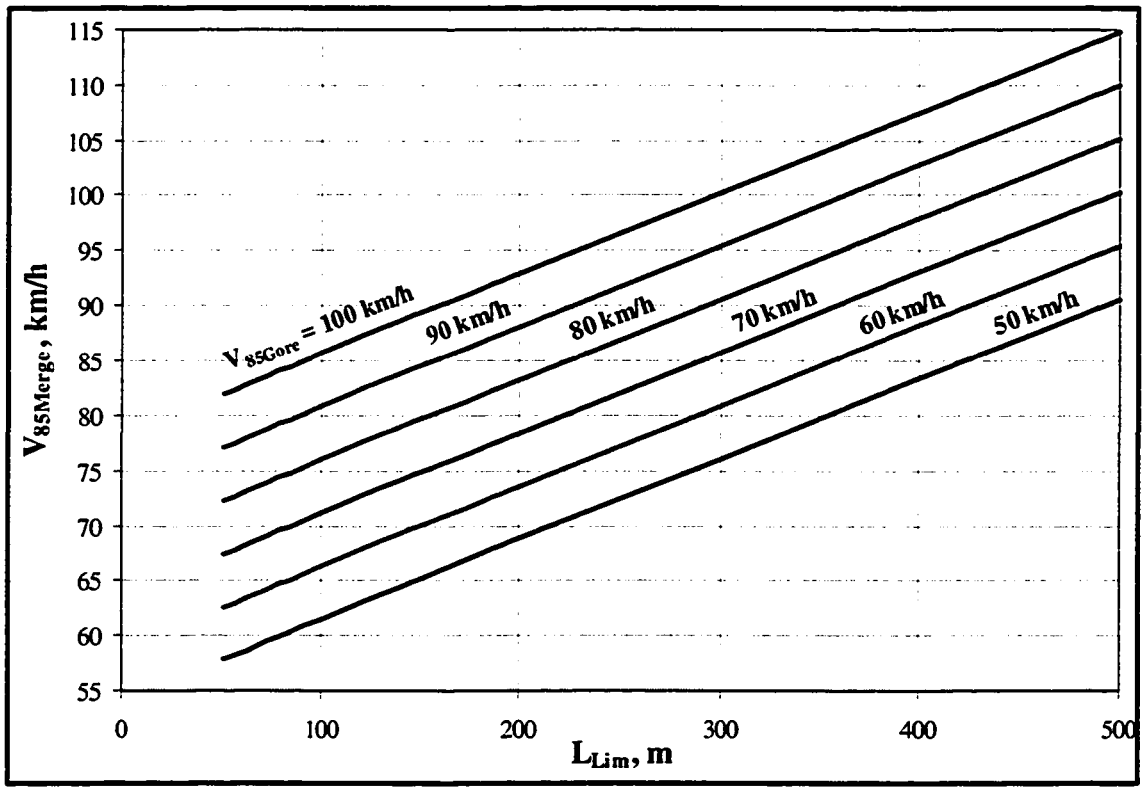


Figure 6.1: Application of Merging Speed Model based on Gore Speed and Length

For example, to attain a $V_{85Merge}$ of 100 km/h, the designer should provide an acceleration lane of about 425 m in length (including the taper), when the V_{85Gore} is 80 km/h. Alternatively, when the length of acceleration lane provided is 300 m and the V_{85Gore} is about 80 km/h, the expected $V_{85Merge}$ would be about 90 km/h. If the gore speed increases from 80 to 90 km/h, the expected merging speed increases from 90 to 95 km/h for the same length of acceleration lane.

Again, a 425 m long acceleration lane is desired to attain a $V_{85Merge}$ of 100 km/h for a V_{85Gore} of 80 km/h. Increasing the gore speed to 90 km/h, the desired length is 360 m to attain the same merging speed. Flatter ramp geometry or long transition curve preceding the gore would assist to attain a higher gore speed. The savings in length, 65 m in the

above example, may be compared with cost in improving the ramp geometry for higher gore speeds. However, the designers should keep in mind that tight ramp geometry may not be fully compensated with a very long acceleration lane, as they do not meet the driver expectancy. Consistency in the system that assists in smooth transition from the ramp to the acceleration lane is rather important. It should be noted that Model 5.1 and graphs in Figure 6.1 are valid for gore speeds lower than the expected merging speeds.

Model 5.2 was developed excluding the speeds at the beginning of the acceleration lanes and may be used to estimate the merging speed or length of the acceleration lane required when the gore speed is not available. Figure 6.2 shows graphs created based on Model 5.2 for different angles of convergence at physical nose (θ). The estimated merging speeds are plotted against the length of acceleration lane for different on-ramp (entering) equivalent hourly traffic volumes in pc/h/lane (Q_{SCL}) for each value of angle. Knowing the angle (θ) and expected volume (Q_{SCL}), the designers may estimate the $V_{85Merge}$ for acceleration lanes of known lengths. Alternatively, the designers may estimate the length of acceleration lane to be provided based on the desired $V_{85Merge}$, angle (θ) and expected on-ramp traffic volumes. For example, the expected $V_{85Merge}$ would be about 96 km/h for an angle (θ) of 3°, SCL length of 300 m, and ramp volume of 800 pc/h/lane (Figure 6.2.b). The expected $V_{85Merge}$ would increase from 96 to about 103 km/h as the SCL length increases from 300 to 400 m, with the same angle and ramp traffic volume.

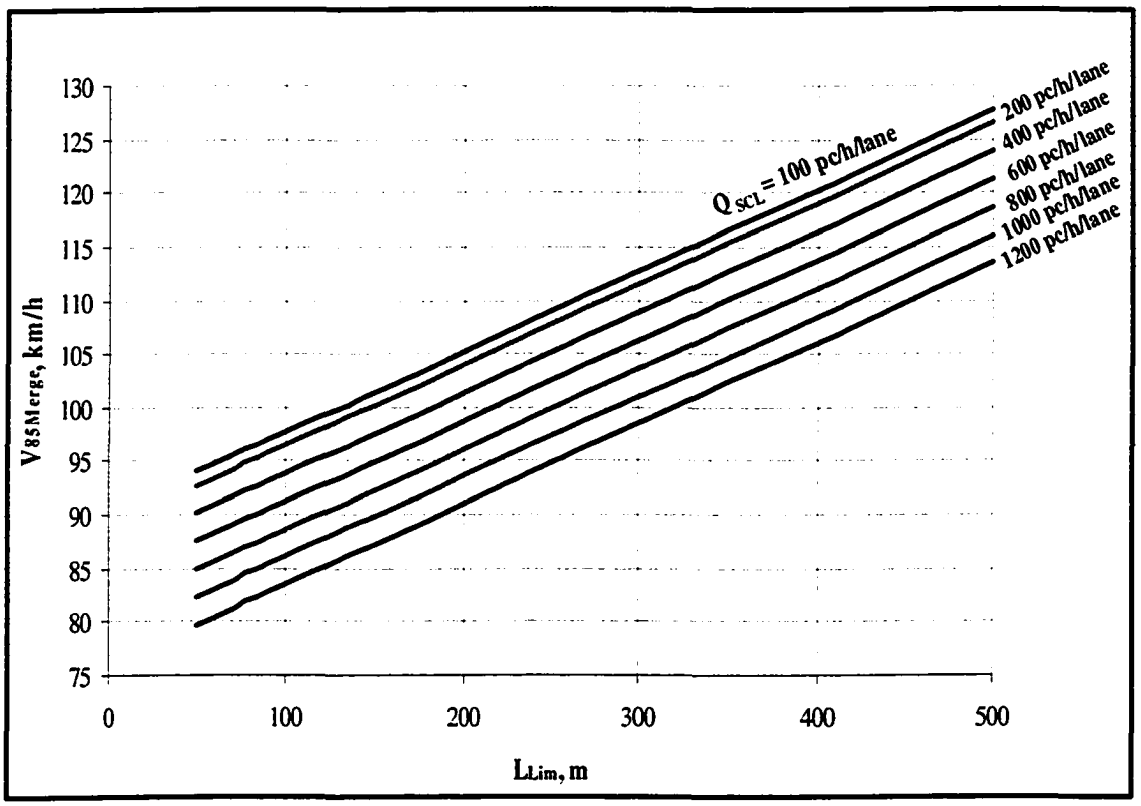
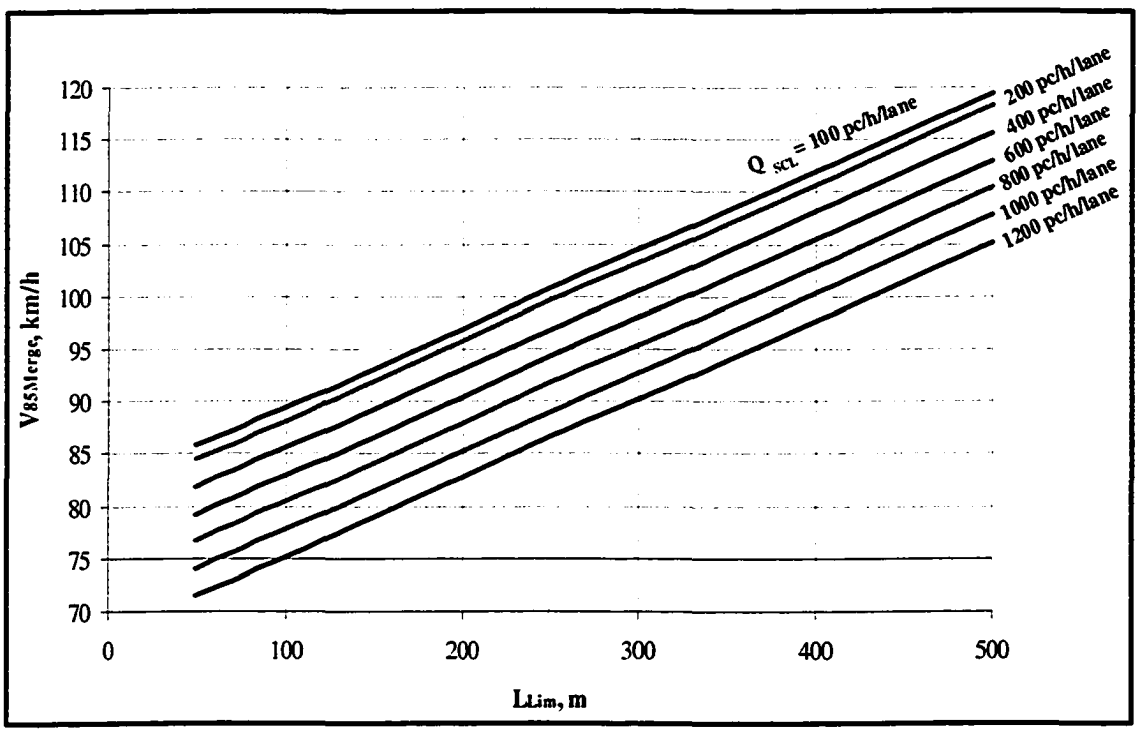


Figure 6.2.a: Merging Speed Model based on Geometry and Traffic Volume (θ = 2°)



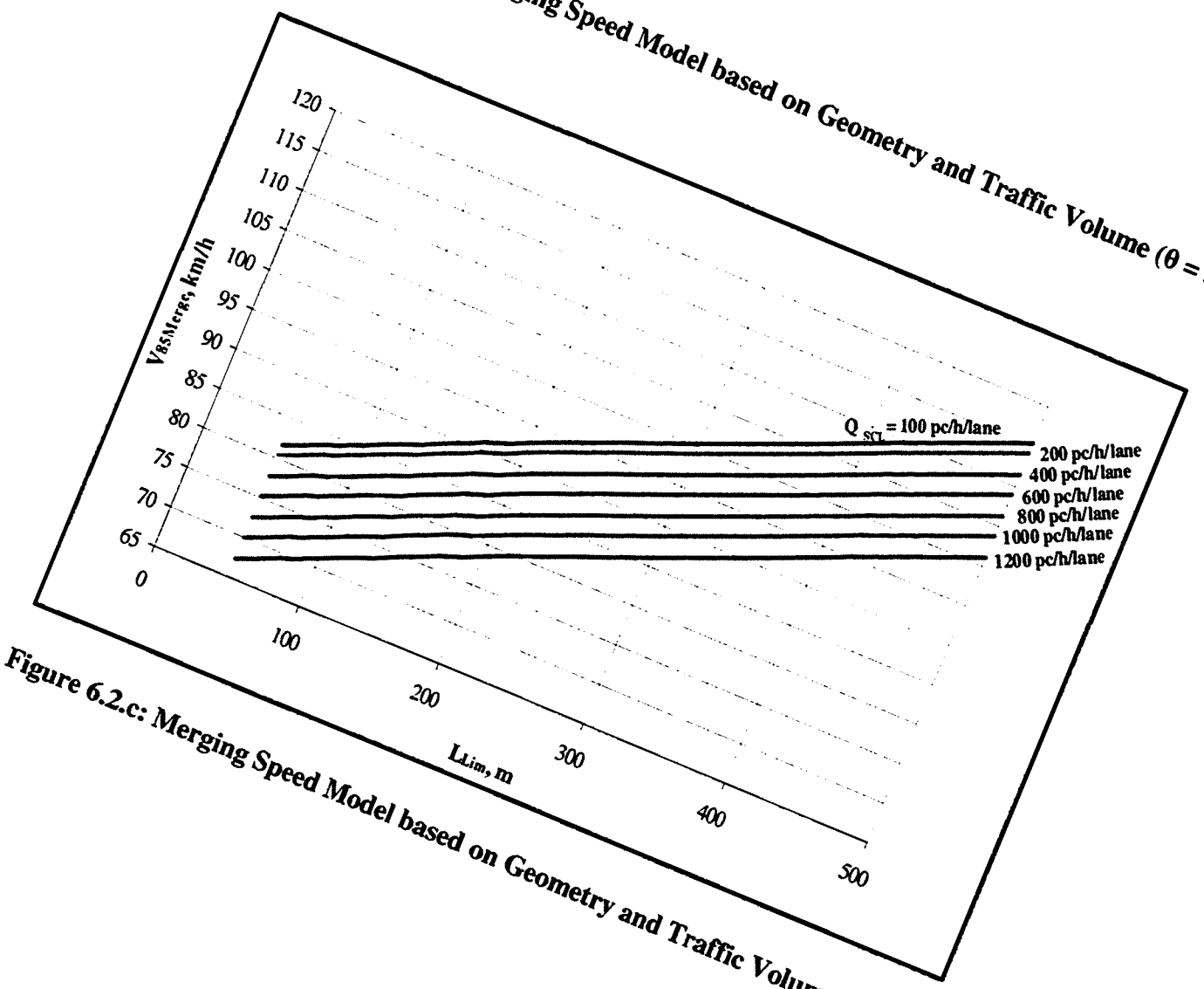


Figure 6.2.b: Merging Speed Model based on Geometry and Traffic Volume ($\theta = 3°$)

Figure 6.2.c: Merging Speed Model based on Geometry and Traffic Volume ($\theta = 4°$)

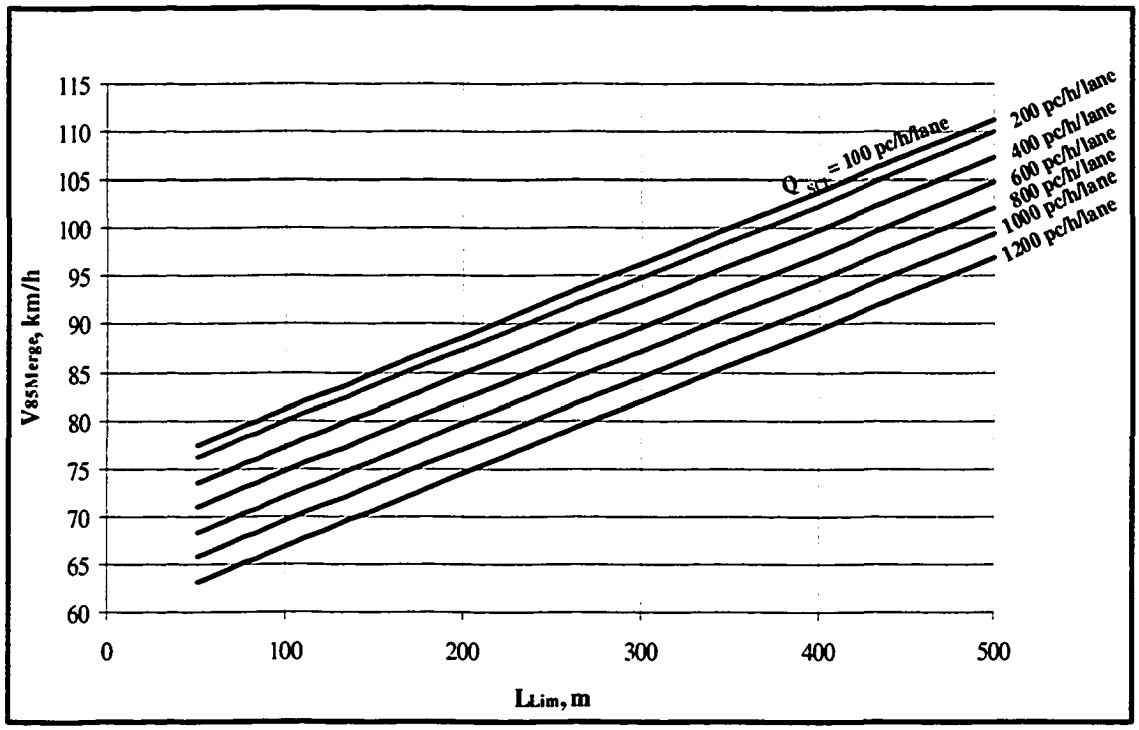


Figure 6.2.d: Merging Speed Model based on Geometry and Traffic Volume (θ = 6°)

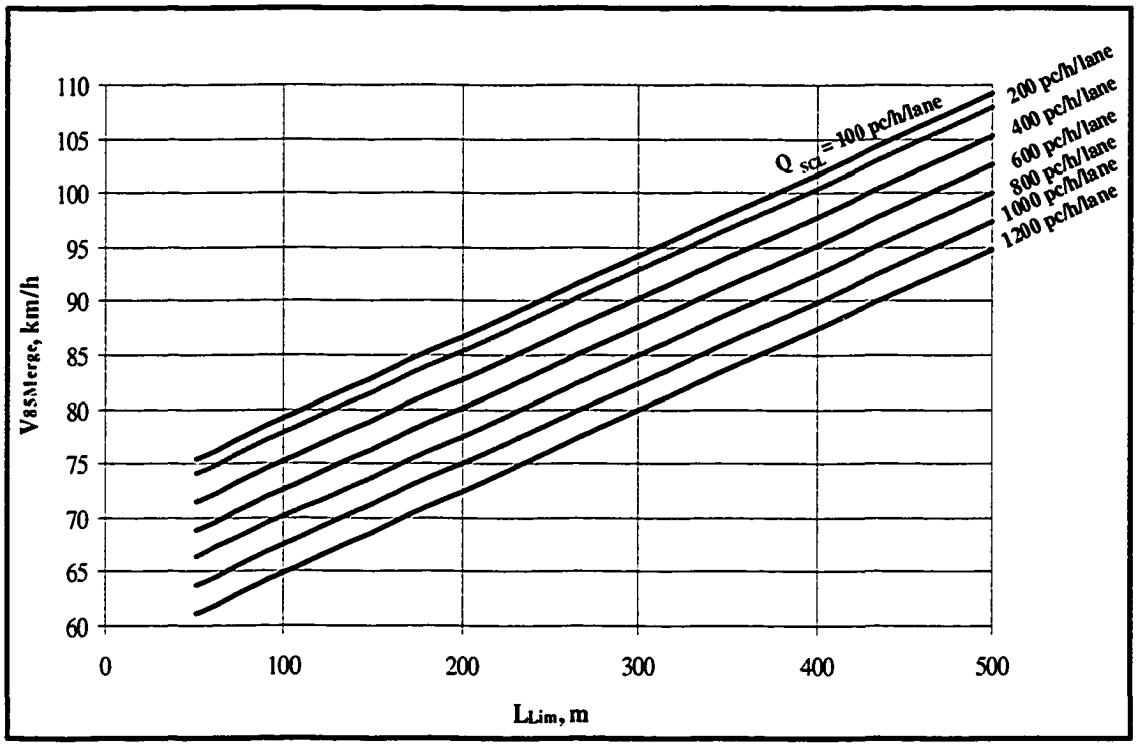


Figure 6.2.e: Merging Speed Model based on Geometry and Traffic Volume (θ = 8°)

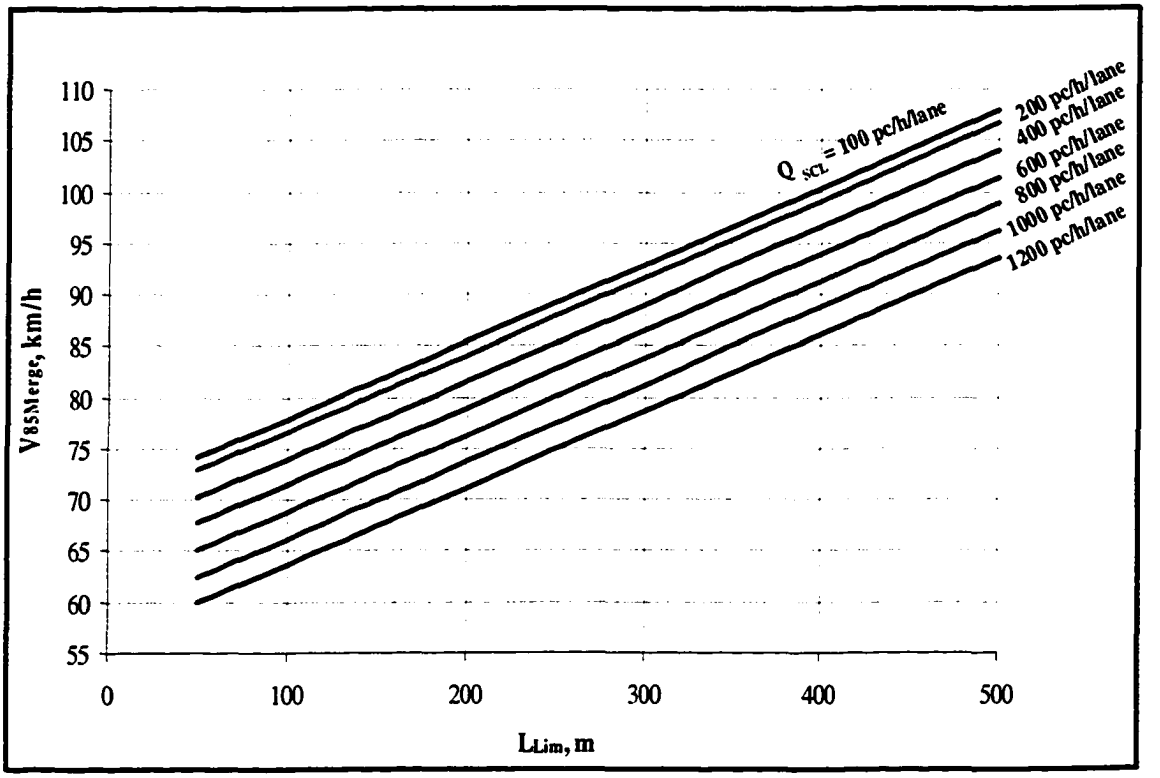


Figure 6.2.f: Merging Speed Model based on Geometry and Traffic Vol. ($\theta = 10°$)

The angle of convergence at physical nose should not be reduced at the expense of a sharp change in ramp curvature. Rather, the ramp curvature should provide a natural path for the entering vehicles for a smooth transition from the controlling ramp curve to the beginning of SCL (gore). Further, the models were developed using traffic data collected during the off-peak hours. The estimated merging speed for a given length of the SCL or the estimated length required in attaining a desired merging speed using the developed models and graphs [Models 5.1 and 5.2, Figures 6.1 through 6.2.f] reflect the off-peak hour conditions. The length of the acceleration lane required to accommodate the peak-hour traffic should be checked before final selection of the length to be provided.

6.2 APPLICATION OF MERGING DISTANCE MODEL

Equation 5.3 relates the provided length of acceleration lane to the effective acceleration distance used by the 85th percentile passenger car. Figure 6.3 provides the expected effective acceleration length as related to the SCL length. Model 5.3 is valid for limited length SCL of ≥ 90 m (including the taper) in length, and estimates better for a length up to about 500 m. From Figure 6.3 or Model 5.3, the designers may estimate the $D_{85Merge}$ for an acceleration lane of known length or a length of acceleration lane to be provided corresponding to a desired $D_{85Merge}$. For example, for a 350 m long acceleration lane, the expected 85th percentile merging distance is 205 m, provided that suitable gaps are available.

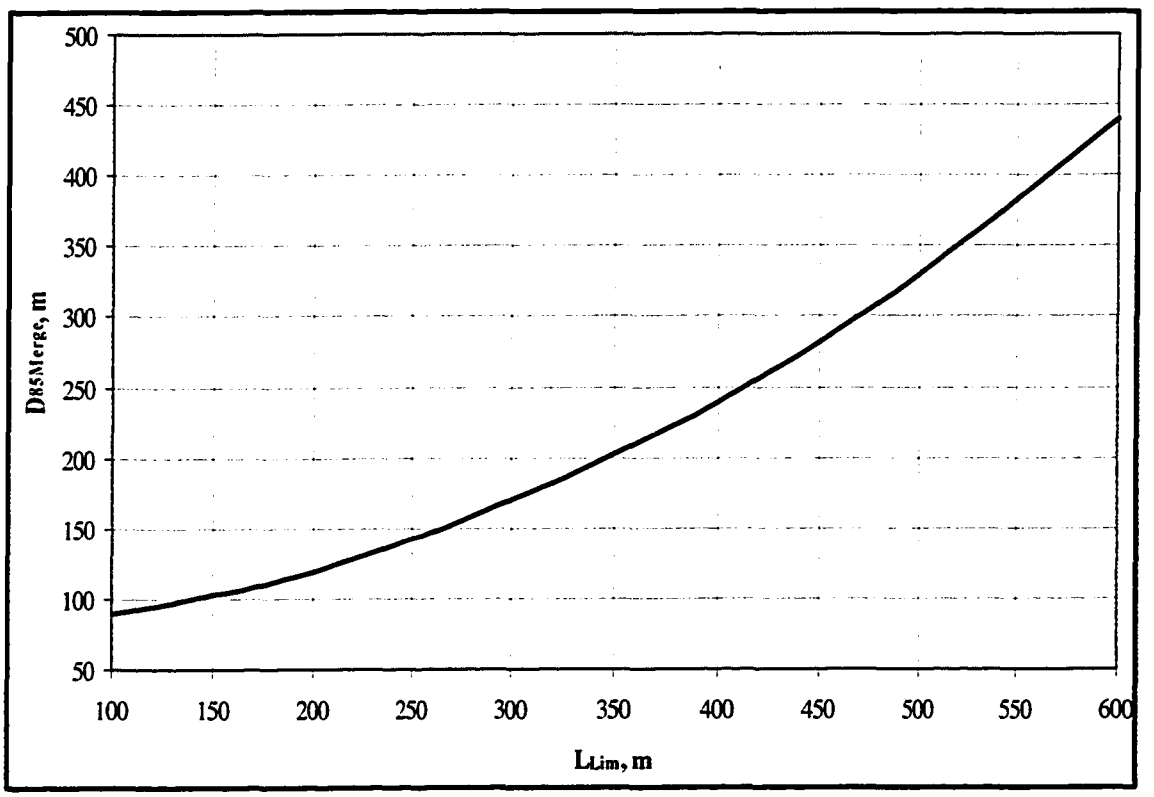


Figure 6.3: Application of Merging Distance Model

6.3 APPLICATION OF ACCELERATION MODEL

Figure 6.4 provides the application of Model 5.4, showing the relationship of the 85th percentile passenger car overall acceleration (a_{85Over}) on the SCL as related to the length of SCL. As mentioned earlier, Model 5.4 is valid for limited length SCL up to a length of 375 m, since the overall acceleration rate was shown to be fairly constant regardless of the provided length in excess of 375 m, from the point where ramp and mainline pavement edges are 1.25 to the end of taper. Model 5.4 or Figure 6.4 may be used to estimate the expected overall acceleration rate on a SCL of length ≤ 375 m, or the desired SCL length for a selected acceleration rate. For longer lanes, the acceleration rate is constant at about 0.4 m/s^2. For example, for a 150 m long acceleration lane the expected 85th percentile overall acceleration rate would be about 1.17 m/s^2. Increasing the length at downstream the gore to 250 m, the expected a_{85Over} decreases to 0.6 m/s^2, about 50% reduction in acceleration rate. Such a reduction would reduce stress on drivers to a great extent. When a shorter SCL is provided at downstream the gore, the ramp geometry should allow the vehicles to complete substantial acceleration on the section upstream the gore. In addition, the designer should keep is mind that the coordination between ramp and SCL geometry is important rather than providing a very long acceleration lane downstream the gore in compensation to tight ramp geometry.

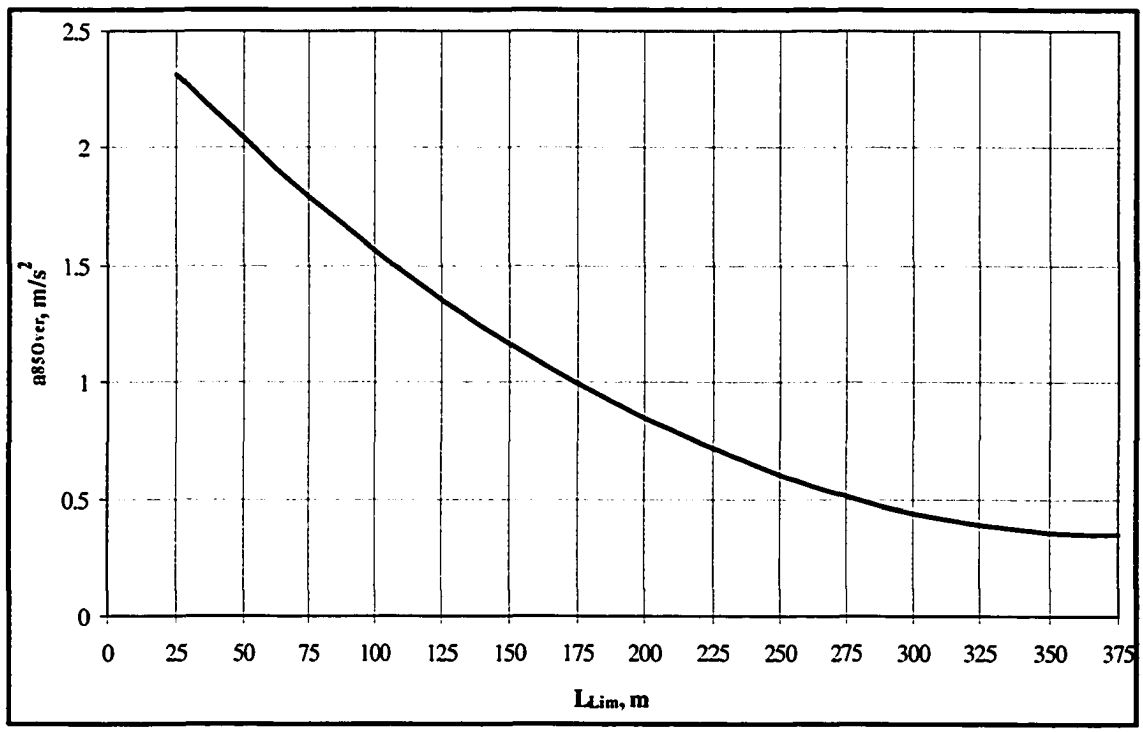


Figure 6.4: Application of Overall Acceleration Model

6.4 APPLICATION OF RIGHT LANE SPEED MODEL

The freeway right lane speed model presented in Equation 5.5 relates the 85th percentile passenger car right lane speed (V_{85RL}) to the right lane equivalent hourly traffic volume (Q_{RL}) and the 85th percentile merging speed of the entering vehicles ($V_{85Merge}$). In Figure 6.5, the estimated right lane speeds are plotted against the right lane equivalent hourly volumes for different merging speeds of entering vehicles. Knowing the traffic volumes on freeway right lanes, the designers and highway professionals may estimate from Figure 6.5 (or Model 5.5) the right lane speeds along the SCL for different merging speeds of the entering vehicles. For example, for a right lane volume (Q_{RL}) of 1000 pc/h/lane, the V_{85RL} along the acceleration lane is expected to be about 97 km/h when the

$V_{85Merge}$ is 70 km/h. With the same traffic volume, the right lane speed is expected to increase to 109 km/h when the merging speed increases to 100 km/h.

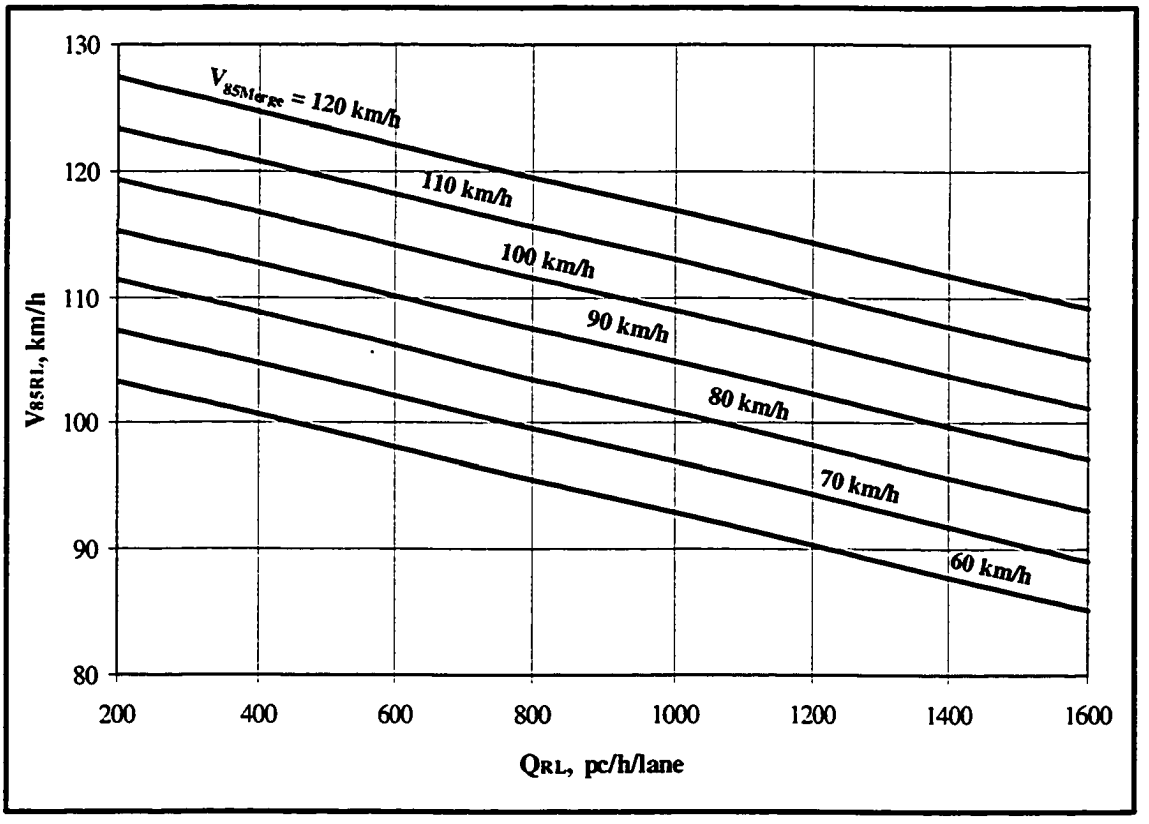


Figure 6.5: Application of Right Lane Speed Model

As shown in Figure 6.5, as the freeway traffic volume increases the right lane speed decreases, and when the merging speed increases, the right lane speed also increases. As shown earlier in the merging speed models, the merging speed of entering vehicles depend mainly on the ramp and acceleration lane geometry. Therefore, provision of entrance ramp geometry that allow the entering vehicles to attain higher merging speeds would improve traffic operations on the freeway merge areas. Alternatively, poor ramp and inadequate acceleration lane geometry restrain speed gain. The entering vehicles

merge at lower speeds with a large speed differences, interrupting through the traffic movement, and jeopardizing own and other traveller safety.

6.5 APPLICATION OF SAFETY PERFORMANCE MODEL

Figure 6.6 provides an application example of the collision model presented in Equation 5.9. Several curves are plotted for traffic exposures on the SCL in the range of 5 to 25 million vehicle-km and the 85th percentile passenger car merging speeds in the range of 60 to 120 km/h. Knowing the traffic exposure, the curves shown in the figure or Equation 5.9 may be used to estimate the expected number of total collisions on the acceleration lane for a safety explicit design of the entrance terminal. For example, for a limited length SCL with traffic exposure of 15 million vehicle-km, the expected number of total collisions on the acceleration lane is 22, when the $V_{85Merge}$ is 80 km/h. If the merging speed is increased from 80 to 90 km/h, the expected total collisions would decrease to 16 for the same exposure, a 27% reduction in total collisions. The traffic exposure may be calculated using Equation 5.6 for a selected SCL length and expected EnAADT. Extending acceleration lane from an entrance gore to the exit gore of next interchange shown to improve safety at freeway merge areas in terms of number of collisions on the acceleration lane. However, previous work by Sarhan (2004) showed that extending the acceleration lane may worsen the safety performance of the whole section. For the extended SCL, a longer length is available for the drivers to merge in a suitable/ large gap, as they are not confined by the limited length. This might have resulted in lower collisions on the extended SCL. On the other hand, the increased collisions on the whole section, in case of the extended SCL, might have resulted from higher lane changing

(weaving) manoeuvres and abrupt merging, especially by unfamiliar drivers, at the end of the extended SCL. The designer should examine trade-off between expected number of collisions on the whole section and on the SCL.

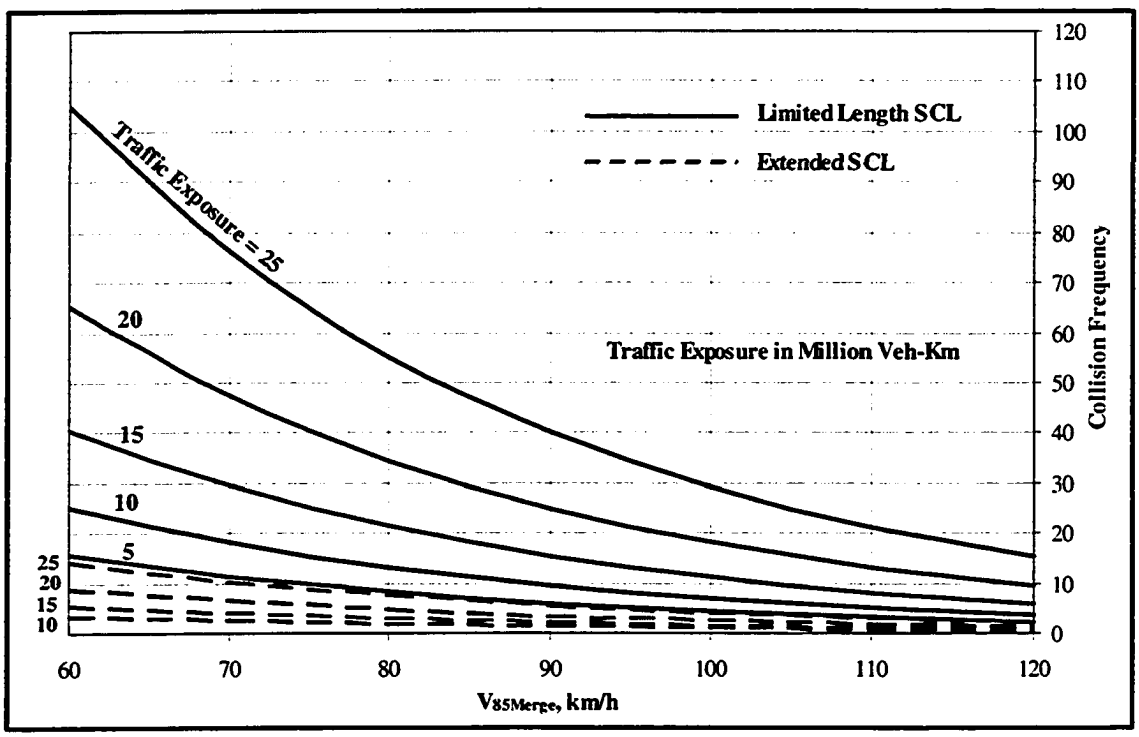


Figure 6.6: Application of Safety Performance Model

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

Safe and efficient operations of freeways are sensitive to the geometry and operations of merging areas at the interchanges. An innovative attempt has been made in this research to examine the effect of geometry and traffic conditions on the merging behaviour of entering vehicles and safety of merge areas. Vehicle speeds on acceleration as well as freeway right lanes were recorded at 23 entrance terminal sites during off-peak hours using two laser speed guns, simultaneously. Traffic data were collected, using video camera, at the same time of speed data collection to examine its effect on merging behaviours as well right lane speed variations at prevailing traffic conditions.

Several aspects of merging behaviours of entering vehicles as well as effect of merging vehicles on freeway right lane speeds in the vicinity of entrance terminals were examined. Attempts have been made to model merging speed of entering vehicles, accelerations on the speed-change lane, merging distance from the gore, freeway right lane speed along the acceleration lane, and number of collisions expected to occur on merging areas. Six statistically significant models were successfully developed.

Analysis of traffic behaviour showed that the merging speed varies with variation of speed of entering vehicles at gore, the length of the acceleration lane, angle of convergence at physical nose and volume of entering vehicles. Lower merging speeds were shown to be associated with higher collisions on the acceleration lanes. Right lane traffic volumes as well as merging speed of entering vehicles was shown to significantly

affect right lane speed along the acceleration lane. The findings and developed models would help designers and highway professionals in selecting the appropriate geometry of entrance terminals for safe and efficient operations of freeway merging areas. The findings and recommendations in this research study would also pave the way for future research in this important area of freeway research. The key findings of this research study are summarized in the following:

7.1.1 Descriptive Analysis Results

- ◊ The through lane speed is higher at sections away from the influence of merge area. Shorter SCL or lower merging speed causes higher interruption in the right lane speed along the SCL.
- ◊ Both ramp and SCL geometrics were shown to affect the merging speed of entering vehicles.
- ◊ The 85th percentile passenger car merging speed was shown to increase for SCL lengths up to about 425 m. An acceleration lane in excess of 425 m (including the taper) in length was shown not to be beneficial in increasing the merging speed.
- ◊ A length of about 375 m (including the taper) is required in completing the merging manoeuvre comfortably. Tighter ramp geometry and longer ramp curve restrain vehicle acceleration on the section upstream the gore and higher acceleration occur on downstream the gore within a short length. This emphasizes the importance of consistency in design and coordination among the geometric features.

- ◊ An acceleration rate of 2.0 m/s^2 appeared to be the maximum comfortable acceleration rate for the 85th percentile passenger car.
- ◊ An acceleration lane of 425 m (including the taper) was shown to be adequate for all geometric and traffic conditions to allow comfortable merging of 95% of vehicle population. The additional cost for the limited length acceleration lane in excess of 425 m in length is not justified. .

7.1.2 Regression Models

- ◊ For predicting the 85th percentile passenger car merging speed, the speed at the gore and the length of limited length SCL were shown to be statistically significant at a 5% level of significance. The model showed an excellent R^2 of 0.99.
- ◊ Inverse of convergence angle at physical nose, length of limited length SCL, and the entering volume were statistically significant at 5% level of significance for predicting merging speed, in absence of gore speed. Right lane traffic volume during off-peak hour was shown not to significantly affect the merging speed of entering vehicles. The model showed a very good R^2 of 0.85.
- ◊ For merging distance model, the square function of limited length SCL was shown to be statistically significant at a 5% level of significance. The model showed a fairly good R^2 of 0.68 and was shown to predict well the 85th percentile merging distances for SCL length in the range of 90 to 500 m.
- ◊ The model for estimation of overall acceleration rate on the limited length SCL showed that the quadratic function of SCL length is statistically significant at 5%

level of significance. The model is applicable for SCL length up to 375 m. The model showed an excellent R^2 value of 0.95.

- ◊ Right lane equivalent hourly volume and the 85th percentile passenger car merging speed were statistically significant, at 5% level of significance, in predicting the 85th percentile right lane speed along the SCL. The model showed a good R^2 value of 0.78. The right lane speed downstream the gore were shown to decrease with decrease in merging speed of entering vehicles, where the merging speed explained 21% of the variability in right lane speed.
- ◊ The 85th percentile merging speed of entering vehicles and traffic exposure on the acceleration lane were statistically significant at a 5% level of significance in predicting the number of five-year total collisions on the acceleration lane. This emphasizes the importance of providing entrance ramp and SCL geometrics that allow the entering vehicles to merge at closer to the through lane speed. Such a provision is vital for safe and efficient operation of merge areas, without interrupting the through traffic movement, and to maintain the freeway concept.
- ◊ Extending the acceleration lane from an entrance gore to the exit gore at next interchange shown to improve safety in terms of number of collisions expected to occur on the acceleration lane.

7.2 RECOMMENDATIONS FOR FUTURE STUDY

In light of the limitations of the data used in this study and findings of this research as described in preceding chapters, the following key recommendations are made. These recommendations would probably enhance the research findings and predictive models.

- ◊ Validity of the developed models should be checked using data from different freeway sites.
- ◊ Collection of speed and traffic data from merge areas with wider variation in lengths and configurations, especially inclusion of shorter ones than used in this study, would probably improve the reliability of the developed models.
- ◊ Inclusion of data from several freeways would probably enhance the models and would result in more reliable models.
- ◊ Inclusion of ramp radius with wider variations in curvatures, superelevation rates, and distances from downstream end of ramp curve to nose or end of acceleration lane, and quantitative values of ramp gradient may be helpful in developing more useful models.
- ◊ Adequate data for heavy vehicles would help to compare and evaluate differences in behaviours between truck and passenger car, rather than the comparison between all vehicles and passenger car.

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APPENDIX A: DISTRIBUTION OF VEHICLES AND THEIR MOVEMENT

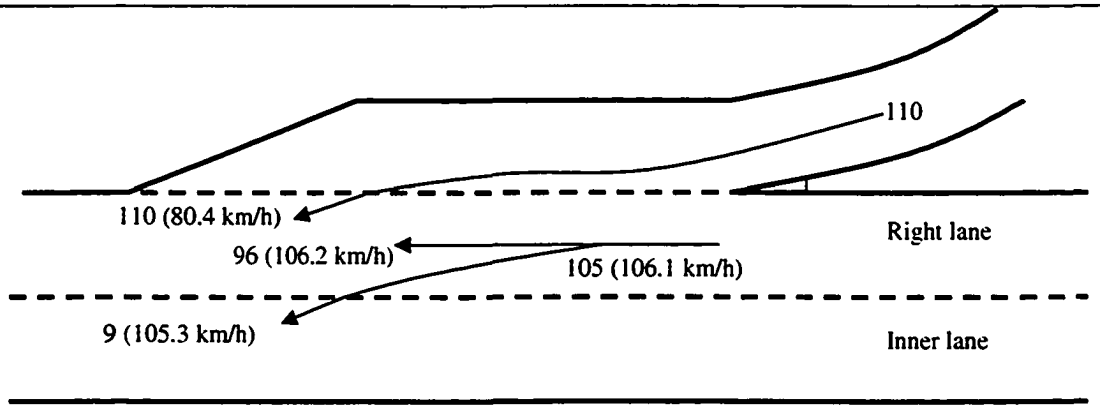


Figure A-1: Walkley Road W-W

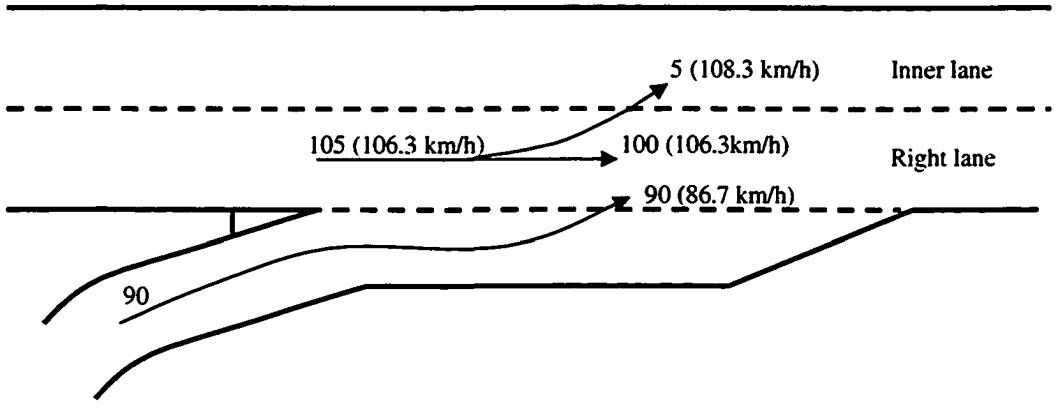


Figure A-2: Innes Road E-E

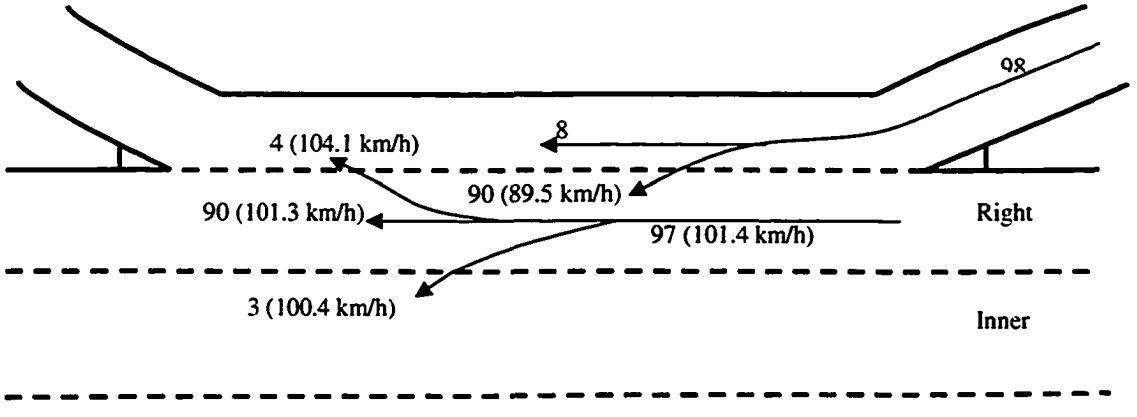


Figure A-3: St. Laurent N-W

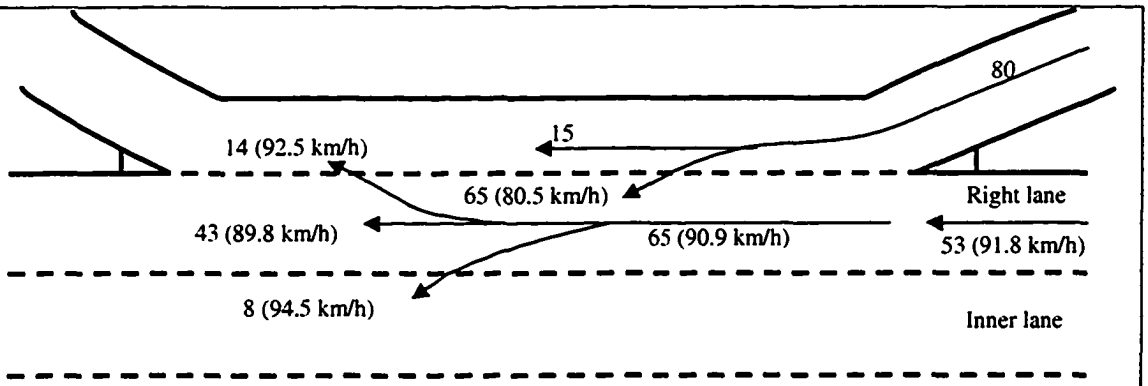
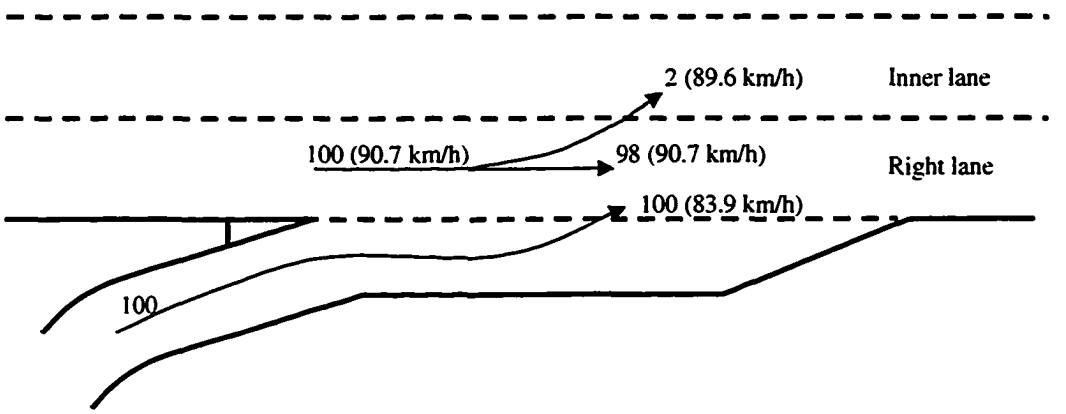


Figure A-4: Vanier N-W



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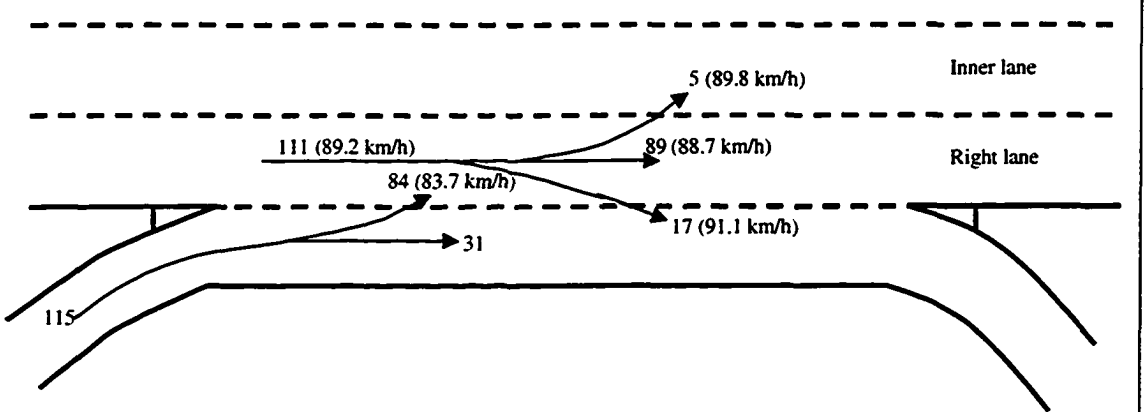
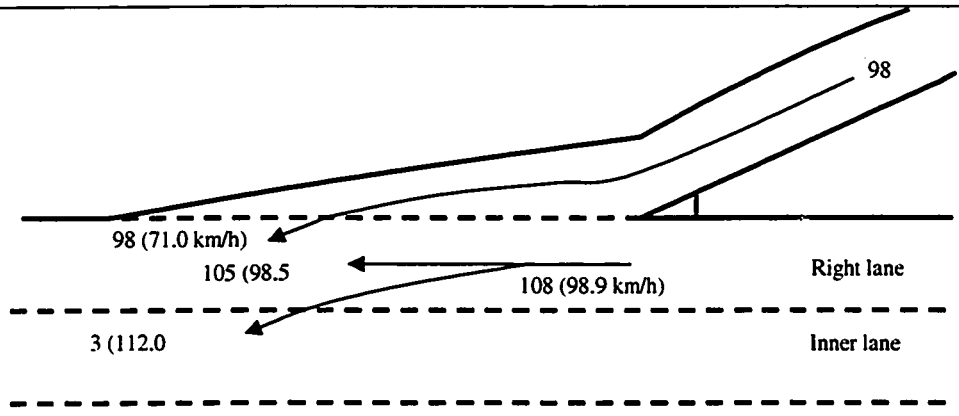


Figure A-6: Lees-E/ Nicholas N-E



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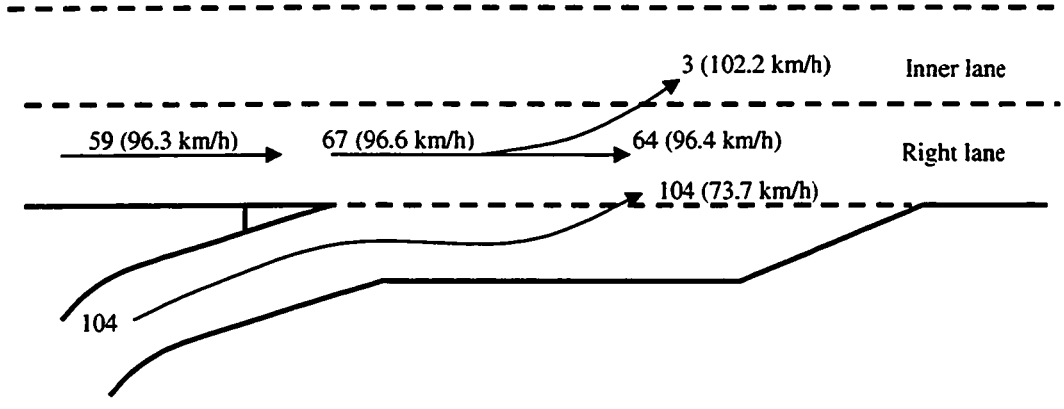

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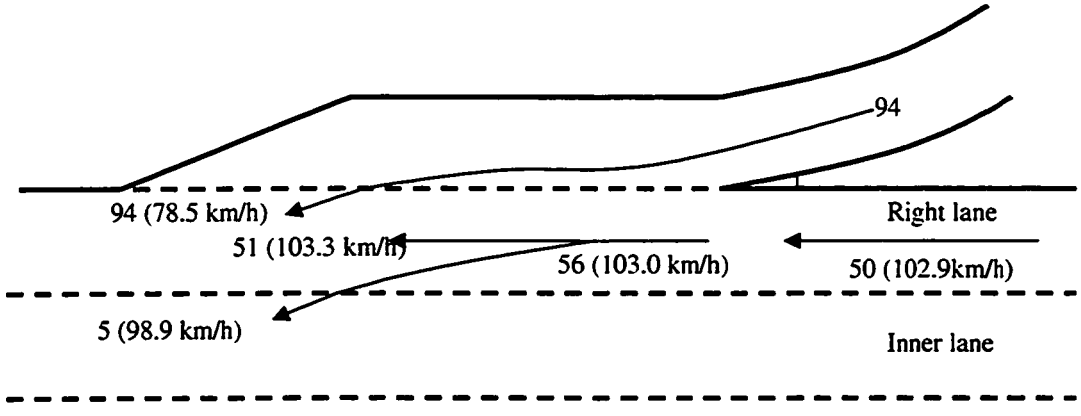
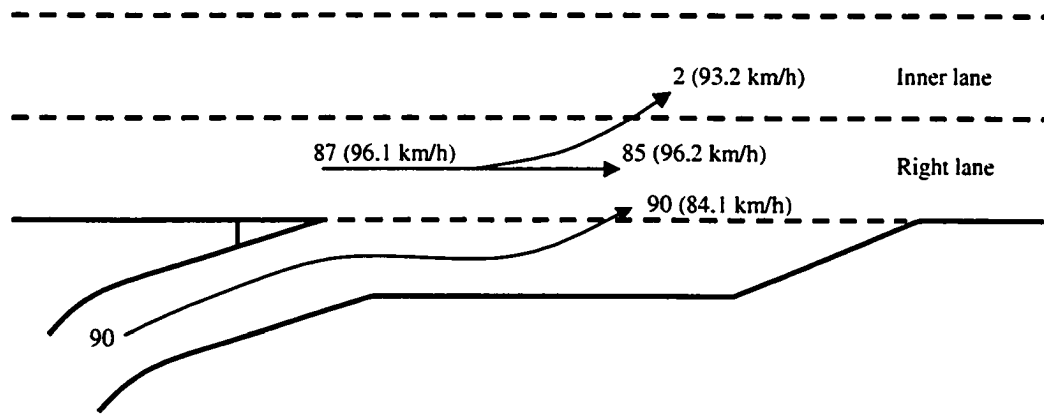


Figure A-9: Woodroffe NS-W



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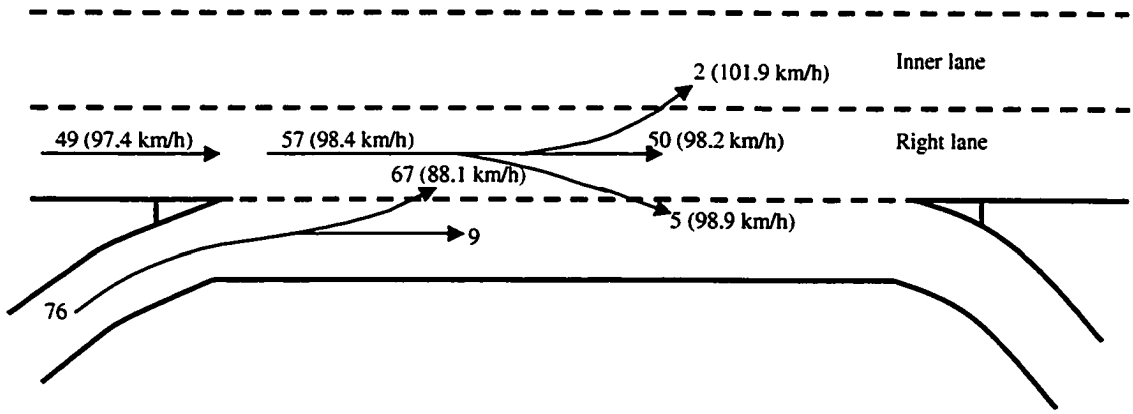


Figure A-11: Woodroffe S-E

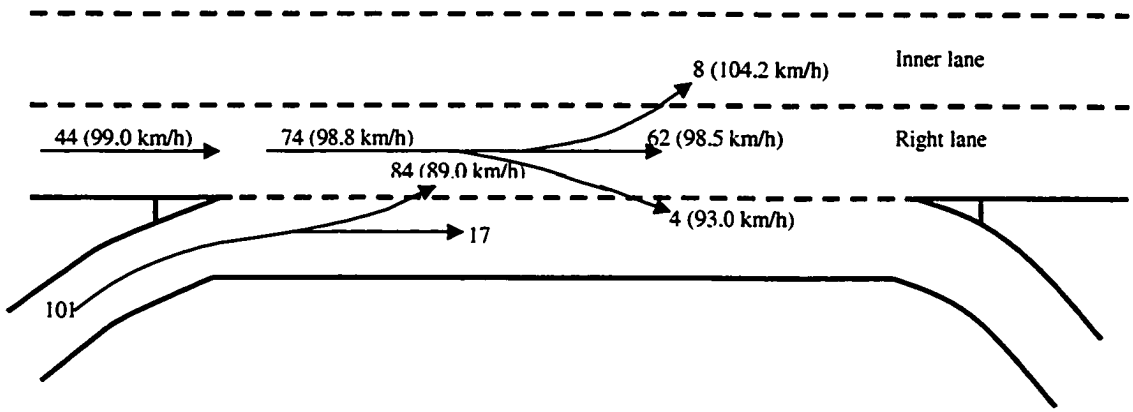


Figure A-12: Greenbank S-E

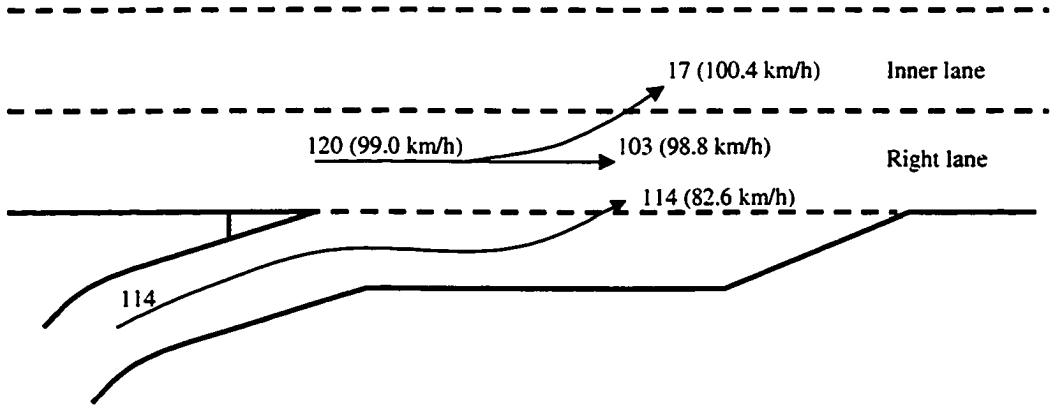
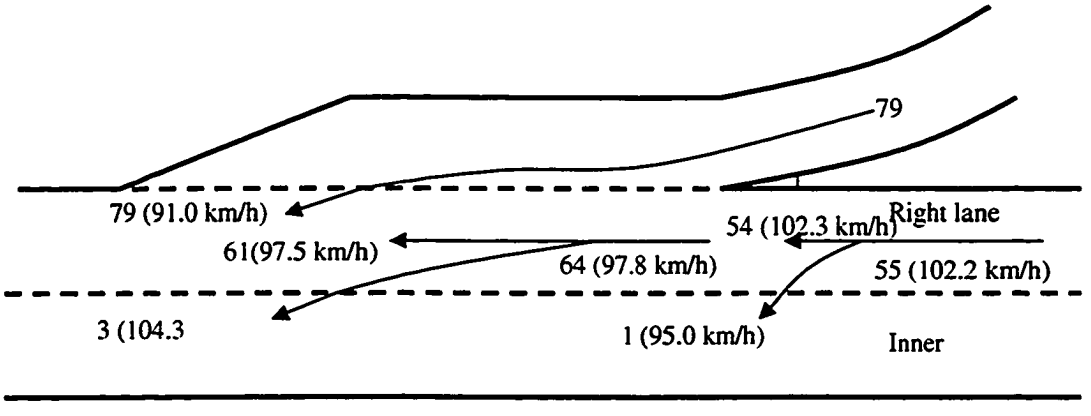


Figure A-13: Richmond S-E



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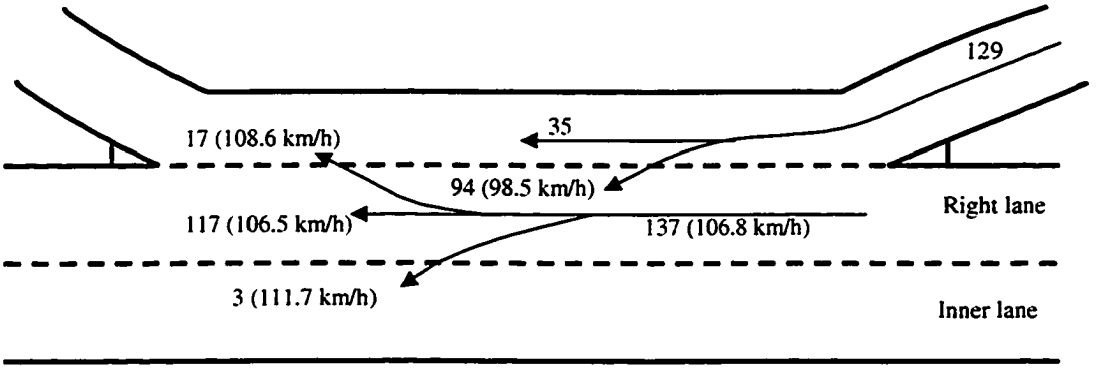
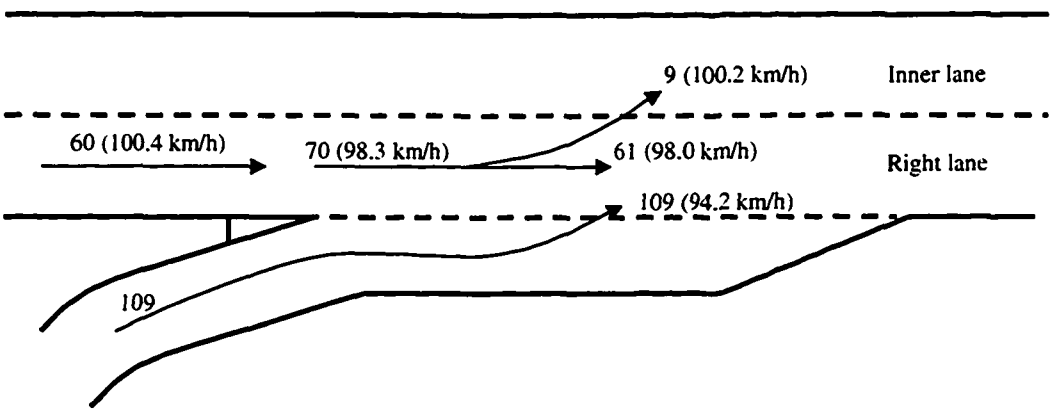


Figure A-15: March/ Eagleson N-W



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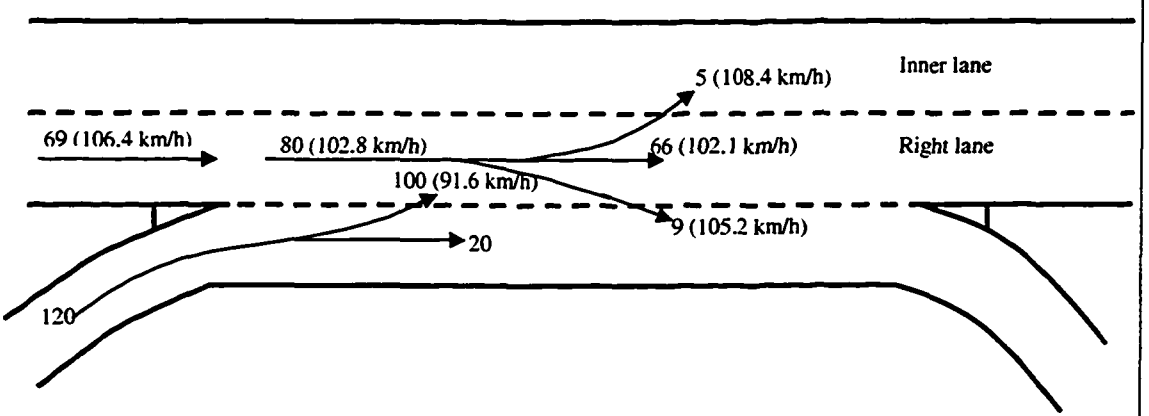

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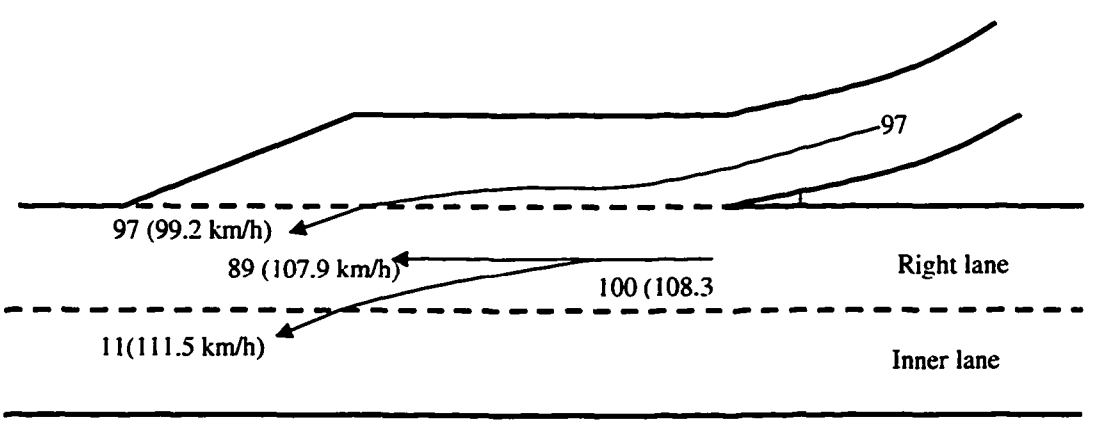


Figure A-18: Terry Fox N-W

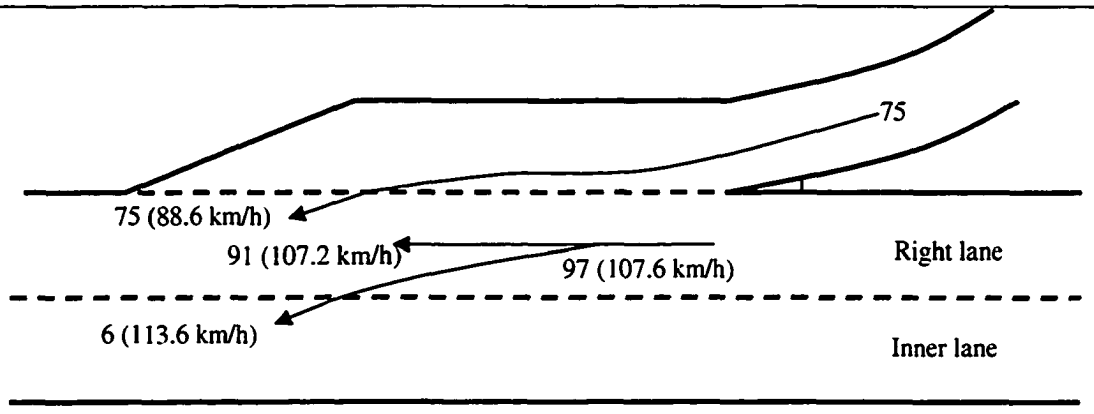
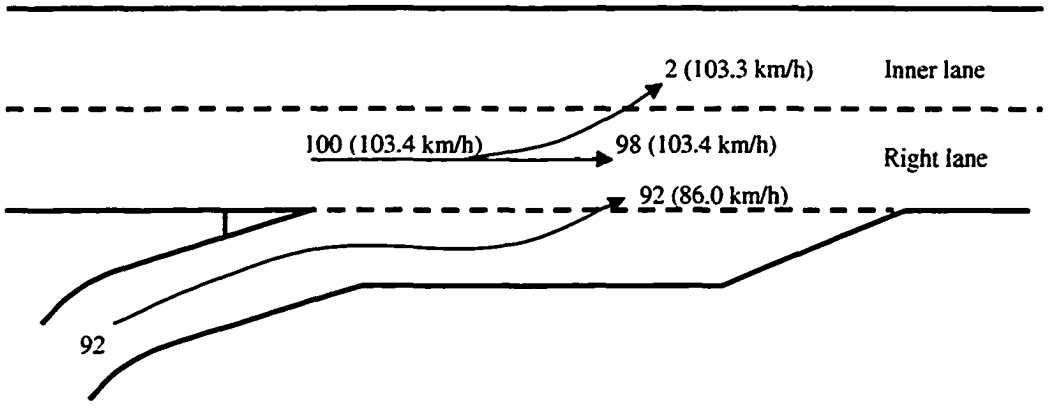


Figure A-19: Terry Fox S-W



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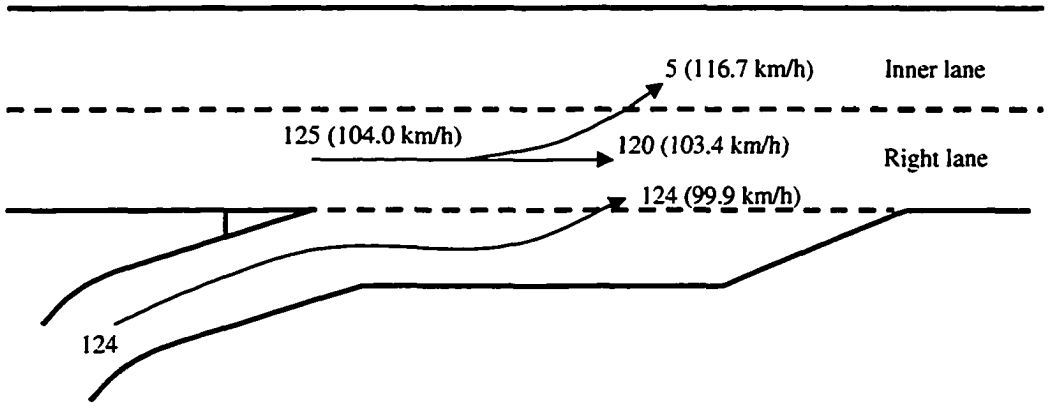

Figure A-21: Terry Fox S-E

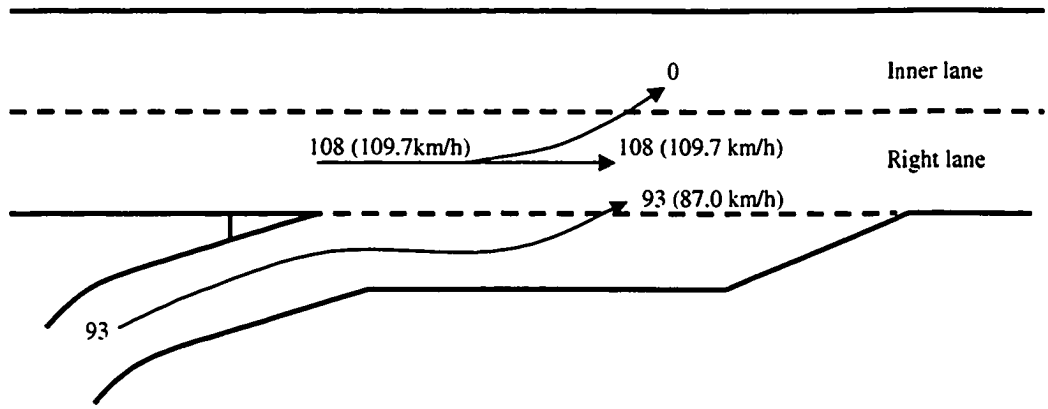

Figure A-22: Carp Road N-E

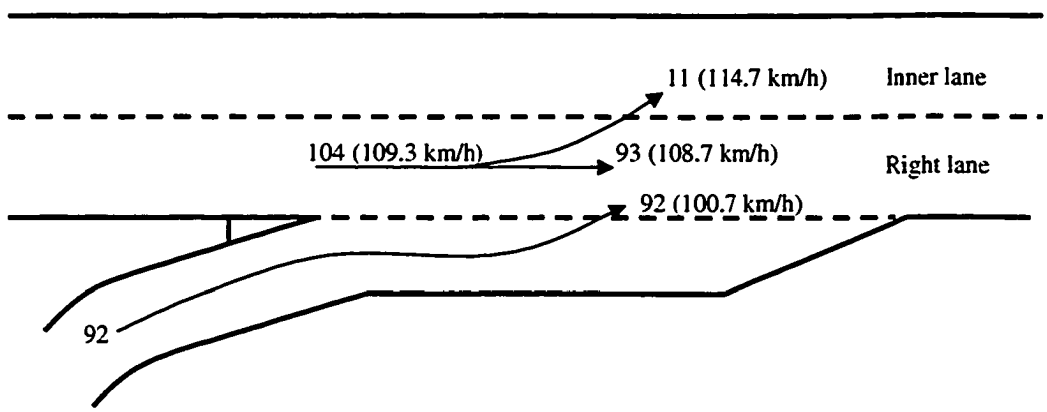

Figure A-23: Carp Road S-E

APPENDIX B: SUMMARY OF SPEED, ACCELERATIONS, AND MERGING DISTANCES

Table B-1: SUMMARY OF SPEEDS ON RIGHT LANES AND SCL

			RL: COMBINED PC & HV			SCL-COMB PC & HV		RIGHT LANE: PC ONLY			SCL-PC ONLY	
Site	**Statistics**		**All**	**Impd**	**UnImpd**	**Max Speed**	**Merge Speed**	**All**	**Impd**	**UnImpd**	**Max Speed**	**Merge Speed**
Walkley W-W	*Sample*	**N**	105		95		109	85		76		86
		85th	116.5		117.1		95.2	117.3		117.6		97.3
		Minimum	80.5		80.5		62.6	80.5		80.5		63.1
		Maximum	137.3		137.3		112.2	137.3		137.3		112.2
		Mean	106.1		107.8		83.9	107.4		109.4		86.2
		SD	10.84		9.93		10.64	11.17		9.83		10.10
	Population	**85th**	117.4		118.1		94.9	118.9		119.6		96.7
		95th	124.0		124.1		101.4	125.7		125.5		102.8
Innes E-E	*Sample*	**N**	105		104		89	95		94		83
		85th	112.4		112.5		100.3	113.1		113.2		100.4
		Minimum	86.8		86.8		64.5	86.8		86.8		69.4
		Maximum	122.2		122.2		116.0	122.2		122.2		116.0
		Mean	106.3		106.4		90.9	107.1		107.2		91.6
		SD	7.29		7.21		10.03	7.13		7.02		9.71
	Population	**85th**	113.9		113.9		101.3	114.5		114.5		101.6
		95th	118.3		118.3		107.4	118.8		118.8		107.6
St. Laurent N-W	*Sample*	**N**	97		69	90	90	85		61	81	81
		85th	111.8		112.2	106.2	104.4	112.7		113.3	106.8	105.1
		Minimum	81.8		81.8	75.4	75.4	87.8		87.9	75.4	75.4
		Maximum	121.0		121.0	120.4	116.6	121.0		121.0	120.4	116.6
		Mean	101.4		102.6	95.5	94.4	102.5		103.8	96.4	95.2
		SD	9.15		9.18	9.49	8.85	8.89		8.74	9.30	8.62
	Population	**85th**	110.9		112.1	105.4	103.5	111.7		112.9	106.0	104.1
		95th	116.4		117.7	111.2	108.9	117.1		118.2	111.7	109.3
Vanier N-W	*Sample*	**N**	65		43	65	65	61		40	61	61
		85th	101.0		102.6	94.4	94.4	101.2		103.2	94.5	94.4
		Minimum	71.2		71.2	66.8	57.0	71.2		71.2	66.8	57.0
		Maximum	112.9		112.9	112.9	108.9	112.9		112.9	112.9	108.9
		Mean	90.9		93.9	86.3	84.9	91.2		94.4	86.8	85.5
		SD	9.28		8.60	9.27	10.34	9.51		8.67	9.22	10.15
	Population	**85th**	100.6		102.8	95.9	95.7	101.1		103.4	96.4	96.1
		95th	106.2		108.0	101.6	102.0	106.8		108.6	102.0	102.2
Vanier N-E	*Sample*	**N**	99	39	60		97	91	36	55		93
		85th	98.9	93.8	101.1		93.4	99.1	93.3	101.2		93.6
		Minimum	73.7	73.7	78.3		69.1	73.7	73.7	79.6		71.9
		Maximum	112.8	98.9	112.8		103.1	112.8	98.9	112.8		103.1
		Mean	90.4	86.4	93.0		86.2	90.7	86.2	93.6		86.8
		SD	8.11	6.38	8.08		7.12	8.16	6.19	8.00		6.63
	Population	**85th**	98.8	93.0	101.4		93.6	99.1	92.6	101.9		93.7
		95th	103.7	96.9	106.3		97.9	104.1	96.4	106.8		97.7

Table B-1: SUMMARY OF SPEEDS ON RIGHT LANES AND SCL (Cont'd)

			RL: COMBINED PC & HV			SCL-COMB PC & HV		RIGHT LANE: PC ONLY			SCL-PC ONLY	
Site	**Statistics**		**All**	**Impd**	**UnImpd**	**Max Speed**	**Merge Speed**	**All**	**Impd**	**UnImpd**	**Max Speed**	**Merge Speed**
Nicholas NS-E	*Sample*	**N**	110		86		83	106		84		71
		85th	96.7		98.3		92.9	96.9		98.8		94.0
		Minimum	67.0		74.5		63.9	67.0		74.5		63.9
		Maximum	111.3		111.3		106.8	111.3		111.3		106.8
		Mean	89.4		91.2		85.3	89.6		91.2		85.7
		SD	8.73		8.27		8.75	8.82		8.34		9.05
	Population	**85th**	98.5		99.7		94.3	98.7		99.9		95.0
		95th	103.8		104.8		99.6	104.1		104.9		100.5
Parkdale NS-W	*Sample*	**N**	107	37	70	97	97	101	34	67	92	92
		85th	110.7	100.2	113.1	90.2	90.0	111.3	97.2	113.7	90.9	90.5
		Minimum	70.9	70.9	82.6	54.4	54.4	70.9	70.9	84.7	54.4	54.4
		Maximum	126.7	124.4	126.7	104.1	103.8	126.7	124.4	126.7	104.1	103.8
		Mean	98.5	91.2	102.3	80.2	79.9	98.6	90.3	102.7	80.5	80.1
		SD	11.75	11.03	10.26	9.84	9.96	11.92	10.95	10.13	9.95	10.09
	Population	**85th**	110.7	102.7	113.0	90.4	90.2	110.9	101.7	113.2	90.8	90.6
		95th	117.8	109.4	119.2	96.4	96.3	118.2	108.3	119.4	96.9	96.7
Maitland NS-E	*Sample*	**N**	67		42	103	103	65		41	99	99
		85th	106.4		108.6	96.6	96.6	106.7		108.7	97.3	96.9
		Minimum	78.2		82.9	64.2	63.7	78.2		82.9	64.2	64.2
		Maximum	119.7		119.7	111.2	111.2	119.7		119.7	111.2	111.2
		Mean	96.6		99.4	86.7	86.6	96.5		99.3	87.4	87.2
		SD	8.65		8.81	9.72	9.74	8.75		8.90	9.35	9.34
	Population	**85th**	105.6		108.5	96.8	96.7	105.6		108.6	97.1	96.9
		95th	110.9		113.9	102.7	102.6	110.9		114.0	102.7	102.6
Woodroffe NS-W	*Sample*	**N**	56		32	94	94	52		30	90	90
		85th	116.0		117.3	98.9	98.6	116.4		117.6	99.2	98.8
		Minimum	77.1		88.8	60.9	60.9	77.1		88.8	64.8	64.8
		Maximum	124.3		124.3	109.8	109.8	124.3		124.3	109.8	109.8
		Mean	102.9		106.3	88.6	88.5	103.0		106.5	89.1	89.0
		SD	10.42		9.02	9.38	9.35	10.74		9.25	9.0	8.9
	Population	**85th**	113.7		115.7	98.4	98.2	114.2		116.1	98.4	98.2
		95th	120.0		121.2	104.1	103.9	120.7		121.7	103.9	103.7
Woodroffe N-E	*Sample*	**N**	87		65		88	80		61	87	
		85th	105.2		106.1		94.9	105.2		106.6	95.0	
		Minimum	74.8		85.1		71.4	74.8		85.1	71.7	
		Maximum	117.2		117.2		106.5	117.2		117.2	106.5	
		Mean	96.1		98.7		87.5	96.5		99.1	87.6	
		SD	8.62		7.62		7.40	8.81		7.65	7.38	
	Population	**85th**	105.1		106.6		95.2	105.7		107.1	95.3	
		95th	110.3		111.3		99.7	111.1		111.7	99.8	

Table B-1: SUMMARY OF SPEEDS ON RIGHT LANES AND SCL (Cont'd)

			RL: COMBINED PC & HV			SCL-COMB PC & HV		RIGHT LANE: PC ONLY			SCL-PC ONLY	
Site	Statistics		All	Impd	UnImpd	Max Speed	Merge Speed	All	Impd	UnImpd	Max Speed	Merge Speed
Woodroffe S-E	*Sample*	N	57		32	66	66	56		32	66	66
		85th	107.9		110.5	103.8	103.6	108.0		110.5	103.8	103.6
		Minimum	75.3		89.5	74.5	74.3	75.3		89.5	74.5	74.3
		Maximum	121.0		121.0	114.5	114.2	121.0		121.0	114.5	114.2
		Mean	98.4		101.7	94.5	93.8	98.4		101.7	94.5	93.8
		SD	9.71		7.76	8.78	8.85	9.80		7.76	8.78	8.85
	Population	85th	108.5		109.8	103.6	102.9	108.6		109.8	103.6	102.9
		95th	114.4		114.5	108.9	108.3	114.5		114.5	108.9	108.3
Greenbank S-E	*Sample*	N	74		46	84	84	72		44	73	73
		85th	108.1		108.1	105.4	105.0	108.1		108.2	105.4	105.4
		Minimum	81.5		82.1	71.4	69.3	81.5		83.8	71.4	69.3
		Maximum	119.3		119.3	119.6	119.0	119.3		119.3	119.6	119.0
		Mean	98.8		99.5	95.8	94.8	99.2		100.2	96.8	95.6
		SD	9.00		8.81	9.90	10.31	8.80		8.39	9.84	10.41
	Population	85th	108.2		108.7	106.1	105.5	108.3		108.9	107.0	106.4
		95th	113.6		114.0	112.1	111.8	113.7		114.0	112.9	112.7
Richmond S-E	*Sample*	N	120		99	114	114	115		94	107	107
		85th	109.8		112.5	96.6	96.4	109.7		112.5	97.4	96.7
		Minimum	72.8		72.8	68.3	68.3	72.8		72.8	70.2	70.1
		Maximum	121.4		121.4	110.4	107.8	121.4		121.4	110.4	107.8
		Mean	99.0		99.9	88.2	87.7	99.0		100.0	88.5	88.1
		SD	10.30		10.15	8.45	8.44	10.28		10.13	8.30	8.28
	Population	85th	109.7		110.5	96.9	96.4	109.7		110.5	97.2	96.7
		95th	115.9		116.6	102.1	101.6	116.0		116.6	102.2	101.7
Moodie N-W	*Sample*	N	64		49	79	79	57		44	77	77
		85th	106.5		107.0	104.1	102.2	105.9		106.7	104.3	102.4
		Minimum	81.6		81.6	78.5	74.3	81.6		81.6	78.5	74.3
		Maximum	120.7		120.7	114.6	114.6	120.7		120.7	114.6	114.6
		Mean	97.8		98.7	95.1	93.2	97.6		98.4	95.3	93.4
		SD	7.85		8.04	8.62	8.78	8.08		8.26	8.55	8.77
	Population	85th	105.9		107.0	104.0	102.3	105.9		106.9	104.2	102.5
		95th	110.7		111.9	109.2	107.6	110.9		111.9	109.4	107.8
Eagleson N-W	*Sample*	N	135		129	92	92	119		114	83	83
		85th	115.5		115.5	110.9	109.1	116.2		116.0	111.9	109.3
		Minimum	86.5		86.5	79.3	77.7	90.8		92.4	80.7	79.6
		Maximum	127.8		127.8	124.6	124.0	127.8		127.8	124.6	124.0
		Mean	107.3		107.6	101.8	99.7	108.3		108.6	103.2	101.0
		SD	8.41		8.28	10.45	10.04	7.89		7.71	9.81	9.56
	Population	85th	116.0		116.1	112.6	110.2	116.5		116.6	113.2	110.9
		95th	121.1		121.2	119.0	116.3	121.3		121.3	119.3	116.7

Table B-1: SUMMARY OF SPEEDS ON RIGHT LANES AND SCL (Cont'd)

			RL: COMBINED PC & HV			SCL-COMB PC & HV		RIGHT LANE: PC ONLY			SCL-PC ONLY	
Site	**Statistics**		**All**	**Impd**	**UnImpd**	**Max Speed**	**Merge Speed**	**All**	**Impd**	**UnImpd**	**Max Speed**	**Merge Speed**
Eagleson S-E	*Sample*	**N**	70		56	108	107	66		53	102	101
		85th	106.9		107.4	109.6	108.3	106.9		107.5	109.7	109.3
		Minimum	81.8		81.8	82.2	78.6	81.8		81.8	82.2	78.6
		Maximum	117.2		117.2	127.1	123.2	117.2		117.2	127.1	123.2
		Mean	98.3		99.1	100.3	98.5	98.3		99.2	100.7	98.8
		SD	7.90		8.17	9.45	9.32	8.12		8.38	9.44	9.37
	Population	**85th**	106.5		107.6	110.0	108.2	106.7		107.9	110.4	108.6
		95th	111.3		112.6	115.8	113.9	111.7		113.0	116.2	114.3
Castlefrank NS-E	*Sample*	**N**	79		77	98	98	70		68	89	89
		85th	111.1		111.5	105.9	105.6	112.5		112.8	106.6	106.3
		Minimum	79.0		89.5	83.4	81.5	79.0		89.7	83.4	81.5
		Maximum	124.6		124.6	116.7	116.7	124.6		124.6	116.7	116.7
		Mean	103.2		103.6	98.1	97.4	103.5		104.0	98.9	98.2
		SD	8.15		7.61	7.76	8.01	8.26		7.63	7.59	7.88
	Population	**85th**	111.6		111.5	106.2	105.7	112.0		112.0	106.8	106.4
		95th	116.6		116.2	110.9	110.6	117.1		116.6	111.4	111.2
Terryfox N-W	*Sample*	**N**	100		85	97	97	90		78	91	91
		85th	116.2		115.9	115.0	114.8	116.5		116.3	115.4	114.8
		Minimum	92.8		92.8	81.2	79.7	92.8		92.8	85.5	85.5
		Maximum	129.8		129.8	128.6	128.2	129.8		129.8	128.6	128.2
		Mean	108.3		108.6	105.6	104.7	108.6		108.7	106.4	105.6
		SD	7.51		7.47	9.70	9.94	7.50		7.54	8.96	9.14
	Population	**85th**	116.0		116.3	115.7	115.0	116.4		116.6	115.8	115.1
		95th	120.6		120.9	121.6	121.1	121.0		121.1	121.2	120.6
Terryfox S-W	*Sample*	**N**	97		87		75	88		80		69
		85th	115.5		115.4		102.0	116.0		115.6		102.2
		Minimum	82.9		88.6		63.9	92.0		92.0		74.1
		Maximum	128.6		128.6		120.8	128.6		128.6		120.8
		Mean	107.6		107.8		92.6	108.4		108.4		94.2
		SD	8.40		7.74		10.80	7.75		7.35		9.37
	Population	**85th**	116.3		115.8		103.8	116.5		116.0		103.9
		95th	121.4		120.5		110.3	121.2		120.4		109.6
Terryfox N-E	*Sample*	**N**	100		90		91	88		80		84
		85th	111.9		112.5		100.0	112.8		112.9		100.6
		Minimum	82.9		82.9		64.2	82.9		82.9		73.0
		Maximum	123.3		123.3		113.5	123.3		123.3		113.5
		Mean	103.4		103.9		89.9	103.7		104.1		91.3
		SD	8.18		8.27		10.36	8.18		8.25		9.46
	Population	**85th**	111.9		112.4		100.7	112.2		112.7		101.1
		95th	116.9		117.5		107.0	117.2		117.7		106.8

Table B-1: SUMMARY OF SPEEDS ON RIGHT LANES AND SCL (Cont'd)

			RL: COMBINED PC & HV			SCL-COMB PC & HV		RIGHT LANE: PC ONLY			SCL-PC ONLY	
Site	**Statistics**		**All**	**Impd**	**UnImpd**	**Max Speed**	**Merge Speed**	**All**	**Impd**	**UnImpd**	**Max Speed**	**Merge Speed**
Terryfox S-E	*Sample*	**N**	124	38	86	124	124	115	35	80	115	115
		85th	113.6	110.9	114.6	115.9	114.6	113.8	111.6	114.5	116.9	115.1
		Minimum	82.2	87.0	82.2	82.7	80.2	82.2	90.4	82.2	82.7	80.2
		Maximum	128.3	116.2	128.3	129.0	129.0	128.3	116.2	128.3	129.0	129.0
		Mean	104.2	101.2	105.5	104.2	102.6	104.5	102.0	105.6	104.9	103.2
		SD	8.91	7.92	9.05	10.11	10.31	8.84	7.65	9.14	10.10	10.35
	Population	**85th**	113.5	109.4	114.9	114.7	113.3	113.6	109.9	115.0	115.3	113.9
		95th	118.9	114.3	120.4	120.8	119.6	119.0	114.6	120.6	121.5	120.2
Carp N-E	*Sample*	**N**	108		106		93	101		99		71
		85th	117.1		117.2		102.6	117.4		117.4		104.8
		Minimum	88.3		95.0		59.7	88.3		95.0		71.3
		Maximum	131.5		131.5		120.6	131.5		131.5		120.6
		Mean	109.7		110.0		91.3	110.2		110.5		94.9
		SD	8.17		7.88		11.16	8.24		7.91		9.06
	Population	**85th**	118.2		118.2		102.8	118.9		118.7		104.3
		95th	123.2		123.0		109.6	123.9		123.6		109.8
Carp S-E	*Sample*	**N**	102		93	91	91	93		84	85	85
		85th	117.5		117.7	114.7	112.3	117.7		117.8	114.8	112.7
		Minimum	87.4		87.4	89.4	87.2	87.4		87.4	89.4	87.2
		Maximum	127.3		127.3	120.4	120.4	127.3		127.3	120.4	120.4
		Mean	109.3		109.3	106.8	106.0	109.5		109.6	107.2	106.3
		SD	7.52		7.40	6.89	6.96	7.65		7.53	6.76	6.87
	Population	**85th**	117.1		117.0	114.0	113.2	117.5		117.4	114.2	113.4
		95th	121.7		121.5	118.2	117.4	122.1		122.0	118.3	117.6

Table B-2: SUMMARY OF GORE SPEEDS, ACCELERATIONS, AND MERGING DISTANCES-FULL PROFILE LANES

			SCL COMBINED PC & HV					SCL :PC ONLY				
Site	Statistics		Merge-Pt	Gore-Speed	Overall_Acc	Max_Acc	Mean_Acc	Merge-Pt	Gore-Speed	Overall_Acc	Max_Acc	Mean_Acc
St. Laurent N-W	*Sample*	N	90	90	90	90	89	81	81	81	81	80
		85th	315.9	95.5	0.449	2.215	0.491	318.4	96.0	0.414	2.215	0.495
		Minimum	49.2	64.6	-0.375	0.548	-0.226	49.2	70.8	-0.375	0.548	-0.226
		Maximum	450.6	112.4	0.714	3.080	0.890	450.6	112.4	0.714	3.048	0.890
		Mean	193.3	86.9	0.241	1.832	0.308	193.9	87.9	0.229	1.867	0.305
		SD	94.87	9.14	0.203	0.546	0.204	95.37	8.61	0.205	0.527	0.211
	Population	85th	291.6	96.4	0.452	2.398	0.518	292.8	96.9	0.441	2.413	0.524
		95th	349.4	102.0	0.575	2.730	0.642	350.8	102.1	0.567	2.734	0.652
Vanier N-W	*Sample*	N	64	64	64	65	64	60	60	60	61	60
		85th	199.7	87.5	0.459	1.651	0.454	201.2	87.7	0.460	1.650	0.462
		Minimum	16.9	66.2	-0.545	0.350	-0.500	16.9	66.2	-0.545	0.350	-0.500
		Maximum	272.7	100.6	0.950	2.201	0.942	272.7	100.6	0.950	2.201	0.942
		Mean	143.7	79.8	0.225	1.326	0.231	144.6	80.2	0.233	1.348	0.239
		SD	49.40	7.53	0.265	0.411	0.255	50.54	7.51	0.267	0.397	0.256
	Population	85th	194.9	87.6	0.499	1.752	0.496	197.0	88.0	0.509	1.759	0.505
		95th	225.0	92.2	0.660	2.002	0.651	227.7	92.6	0.671	2.000	0.661
Parkdale NS-W	*Sample*	N	98	97	96	97	95	93	92	91	92	90
		85th	106.7	78.0	0.876	2.805	0.965	109.5	78.5	0.875	2.823	0.971
		Minimum	33.1	48.3	-0.102	1.097	0.013	33.1	48.3	-0.102	1.097	0.013
		Maximum	157.2	91.6	1.318	3.917	1.256	157.2	91.6	1.318	3.917	1.256
		Mean	85.7	69.7	0.625	2.274	0.726	85.7	70.1	0.619	2.300	0.723
		SD	25.37	9.12	0.265	0.642	0.248	25.73	9.17	0.268	0.647	0.252
	Population	85th	112.0	79.2	0.899	2.940	0.983	112.4	79.6	0.897	2.971	0.984
		95th	127.5	84.7	1.061	3.331	1.133	128.1	85.2	1.060	3.365	1.137
Maitland NS-E	*Sample*	N	103	102	104	102	104	99	98	100	98	100
		85th	246.5	67.1	1.132	2.351	1.120	247.0	67.1	1.132	2.385	1.128
		Minimum	69.9	49.1	0.254	1.102	0.282	69.9	49.1	0.355	1.622	0.282
		Maximum	332.4	77.4	1.470	3.276	1.517	332.4	77.4	1.470	3.276	1.517
		Mean	173.3	62.5	0.843	2.093	0.843	175.6	62.7	0.855	2.111	0.855
		SD	63.06	5.03	0.253	0.413	0.255	62.979	4.993	0.246	0.405	0.251
	Population	85th	238.6	67.7	1.105	2.522	1.108	240.9	67.9	1.110	2.531	1.115
		95th	277.0	70.8	1.258	2.773	1.262	279.2	70.9	1.260	2.777	1.268
Woodroffe NS-W	*Sample*	N	94	94	92	94	91	90	90	88	90	87
		85th	149.8	86.6	0.839	1.922	0.849	150.6	87.2	0.851	1.907	0.855
		Minimum	27.5	58.3	0.036	0.554	0.039	27.5	63.5	0.036	0.554	0.039
		Maximum	201.4	103.1	1.451	2.213	1.360	201.4	103.1	1.451	2.206	1.360
		Mean	104.8	78.9	0.646	1.546	0.641	105.1	79.3	0.655	1.548	0.650
		SD	42.16	8.00	0.266	0.375	0.251	42.3	7.77	0.266	0.371	0.250
	Population	85th	148.5	87.0	0.922	1.936	0.901	148.9	87.3	0.931	1.933	0.909
		95th	174.2	92.0	1.084	2.164	1.053	174.7	92.0	1.093	2.159	1.061

Table B-2: SUMMARY OF GORE SPEEDS, ACCELERATIONS, AND MERGING DISTANCES-FULL PROFILE LANES (Cont'd)

			SCL COMBINED PC & HV					SCL :PC ONLY				
Site	Statistics		Merge-Pt	Gore-Speed	Overall_Acc	Max_Acc	Mean_Acc	Merge-Pt	Gore-Speed	Overall_Acc	Max_Acc	Mean_Acc
Woodroffe S-E	*Sample*	N	67	67	66	66	65	67	67	66	66	65
		85th	243.0	99.0	0.453	1.648	0.435	243.0	99.0	0.453	1.648	0.435
		Minimum	42.8	72.6	-0.191	0.553	-0.151	42.8	72.6	-0.191	0.553	-0.151
		Maximum	405.5	113.8	0.687	2.338	0.640	405.5	113.8	0.687	2.338	0.640
		Mean	169.8	88.2	0.256	1.296	0.264	169.8	88.2	0.256	1.296	0.264
		SD	82.19	8.91	0.166	0.352	0.162	82.19	8.91	0.166	0.352	0.162
	Population	85th	255.0	97.4	0.428	1.661	0.432	255.0	97.4	0.428	1.661	0.432
		95th	305.0	102.9	0.528	1.876	0.531	305.0	102.9	0.528	1.876	0.531
Greenbank S-E	*Sample*	N	80	84	83	83	82	70	73	72	72	71
		85th	234.3	98.5	0.394	1.656	0.405	239.1	98.6	0.406	1.659	0.412
		Minimum	61.6	68.5	-0.249	0.556	-0.174	61.6	68.5	-0.249	0.556	-0.174
		Maximum	391.6	113.9	0.770	2.195	0.731	391.6	113.9	0.770	2.195	0.731
		Mean	170.2	89.3	0.228	1.377	0.237	173.3	90.0	0.229	1.388	0.238
		SD	71.58	9.50	0.190	0.389	0.193	74.44	9.67	0.201	0.394	0.206
	Population	85th	244.4	99.1	0.425	1.780	0.437	250.5	100.0	0.437	1.797	0.451
		95th	287.9	104.9	0.541	2.016	0.555	295.8	105.9	0.559	2.037	0.576
Richmond S-E	*Sample*	N	110	113	113	113	113	103	106	106	106	106
		85th	124.2	91.7	0.420	1.647	0.505	123.2	91.7	0.423	1.649	0.507
		Minimum	26.1	63.8	-0.368	0.550	-0.353	26.1	66.9	-0.368	0.550	-0.155
		Maximum	191.0	100.5	0.715	2.194	0.839	191.0	100.5	0.715	2.194	0.839
		Mean	92.5	83.3	0.220	1.247	0.291	90.5	83.5	0.230	1.257	0.304
		SD	32.77	7.85	0.205	0.375	0.217	32.49	7.61	0.205	0.385	0.212
	Population	85th	126.4	91.4	0.433	1.635	0.516	124.2	91.4	0.442	1.656	0.524
		95th	146.4	96.2	0.558	1.863	0.648	143.9	96.1	0.567	1.890	0.653
Moodie N-W	*Sample*	N	77	79	78	79	78	75	77	76	77	76
		85th	171.0	101.0	0.320	1.633	0.327	165.5	101.1	0.322	1.630	0.328
		Minimum	57.3	73.3	-0.286	0.211	-0.356	57.3	73.3	-0.286	0.211	-0.356
		Maximum	260.1	111.9	0.492	2.201	0.522	260.1	111.9	0.492	2.201	0.522
		Mean	125.8	91.8	0.109	1.168	0.109	125.4	92.0	0.110	1.169	0.110
		SD	45.71	8.69	0.202	0.372	0.208	45.50	8.66	0.205	0.368	0.211
	Population	85th	173.2	100.8	0.318	1.554	0.325	172.6	101.0	0.323	1.551	0.329
		95th	201.0	106.1	0.442	1.781	0.452	200.3	106.2	0.447	1.774	0.457
Eagleson N-W	*Sample*	N	94	92	92	93	91	85	83	83	84	82
		85th	258.9	106.5	0.227	2.175	0.287	258.9	107.5	0.223	2.183	0.288
		Minimum	9.3	71.5	-0.528	0.531	-0.336	9.3	74.7	-0.528	0.531	-0.336
		Maximum	379.2	123.3	0.552	2.263	0.509	379.2	123.3	0.552	2.263	0.509
		Mean	193.4	96.7	0.028	1.475	0.136	191.2	98.3	0.014	1.489	0.126
		SD	69.35	10.56	0.209	0.466	0.160	70.70	9.58	0.213	0.481	0.164
	Population	85th	265.3	107.7	0.245	1.958	0.302	264.5	108.2	0.235	1.987	0.297
		95th	307.5	114.0	0.372	2.241	0.399	307.5	114.1	0.365	2.279	0.397

Table B-2: SUMMARY OF GORE SPEEDS, ACCELERATIONS, AND MERGING DISTANCES-FULL PROFILE LANES (Cont'd)

			SCL COMBINED PC & HV					SCL :PC ONLY				
Site	**Statistics**		**Merge-Pt**	**Gore-Speed**	**Overall_Acc**	**Max_Acc**	**Mean_Acc**	**Merge-Pt**	**Gore-Speed**	**Overall_Acc**	**Max_Acc**	**Mean_Acc**
Eagleson S-E	*Sample*	**N**	107	107	109	108	108	101	101	103	102	102
		85th	224.1	105.7	0.296	1.666	0.317	224.1	105.9	0.296	1.666	0.317
		Minimum	39.7	71.4	-0.389	0.529	-0.435	39.7	71.4	-0.389	0.529	-0.435
		Maximum	334.4	122.0	0.612	2.534	0.601	334.4	122.0	0.612	2.534	0.601
		Mean	157.2	95.7	0.110	1.305	0.130	156.9	96.1	0.110	1.312	0.129
		SD	60.23	9.43	0.189	0.446	0.199	60.06	9.39	0.193	0.449	0.205
	Population	**85th**	219.7	105.5	0.306	1.768	0.336	219.2	105.9	0.311	1.777	0.341
		95th	256.3	111.2	0.421	2.039	0.457	255.7	111.6	0.429	2.051	0.465
Castlefrank NS-E	*Sample*	**N**	98	99	99	100	96	89	90	90	91	87
		85th	242.7	99.4	0.455	2.196	0.456	243.5	100.3	0.453	2.196	0.466
		Minimum	3.4	73.7	-0.606	0.498	-0.103	3.4	77.2	-0.606	0.498	-0.103
		Maximum	351.7	106.8	0.751	3.274	0.685	351.7	106.8	0.751	3.274	0.685
		Mean	171.6	91.1	0.155	1.760	0.267	167.6	92.0	0.147	1.757	0.267
		SD	68.54	7.65	0.266	0.545	0.179	70.27	7.26	0.270	0.548	0.184
	Population	**85th**	242.6	99.0	0.430	2.325	0.452	240.5	99.6	0.427	2.325	0.458
		95th	284.3	103.6	0.592	2.657	0.560	283.2	104.0	0.591	2.658	0.570
Terryfox N-W	*Sample*	**N**	95	97	97	97	95	89	91	91	91	89
		85th	217.8	109.1	0.376	1.671	0.413	217.4	109.2	0.387	1.669	0.417
		Minimum	52.2	75.6	-0.384	0.554	-0.281	52.2	80.8	-0.384	0.554	-0.281
		Maximum	324.2	124.7	0.684	2.749	0.666	324.2	124.7	0.684	2.749	0.666
		Mean	170.0	100.0	0.173	1.442	0.221	167.8	101.0	0.175	1.445	0.223
		SD	54.65	9.48	0.203	0.433	0.185	51.88	8.74	0.207	0.433	0.188
	Population	**85th**	226.6	109.9	0.383	1.891	0.413	221.5	110.1	0.390	1.894	0.418
		95th	259.9	115.6	0.507	2.154	0.526	253.1	115.4	0.516	2.157	0.532
Terryfox S-E	*Sample*	**N**	124	124	124	124	124	115	115	115	115	115
		85th	226.5	109.3	0.317	2.548	0.348	230.1	110.4	0.317	2.568	0.349
		Minimum	62.8	78.0	-0.320	0.557	-0.302	62.8	78.0	-0.320	0.557	-0.302
		Maximum	301.1	127.5	0.632	3.550	0.631	301.1	127.5	0.632	3.550	0.631
		Mean	169.5	99.1	0.140	1.996	0.161	170.8	99.7	0.142	2.029	0.162
		SD	55.05	9.87	0.185	0.574	0.188	55.90	9.81	0.189	0.564	0.192
	Population	**85th**	226.6	109.3	0.332	2.591	0.355	228.8	109.9	0.338	2.614	0.362
		95th	260.1	115.3	0.444	2.940	0.470	262.8	115.8	0.453	2.957	0.479
Carp S-E	*Sample*	**N**	92	91	91	91	90	85	85	84	84	83
		85th	291.4	108.1	0.290	2.186	0.400	293.1	107.9	0.318	2.189	0.403
		Minimum	80.4	78.5	-0.340	0.000	-0.276	80.4	78.5	-0.340	0.000	-0.276
		Maximum	386.7	115.4	0.694	2.895	0.614	386.7	115.4	0.694	2.895	0.614
		Mean	219.7	99.8	0.137	1.451	0.229	220.8	100.1	0.142	1.460	0.234
		SD	67.94	7.62	0.195	0.579	0.174	69.72	7.53	0.198	0.592	0.178
	Population	**85th**	290.1	107.7	0.338	2.051	0.410	293.1	107.9	0.348	2.074	0.419
		95th	331.4	112.4	0.457	2.403	0.516	335.5	112.5	0.469	2.434	0.527

APPENDIX C: NB MODELLING ATTEMPTS FOR TOTAL COLLISIONS ON THE SEGMENT

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	12	16.4233	1.3686
Scaled Deviance	12	16.4233	1.3686
Pearson Chi-Square	12	9.8229	0.8186
Scaled Pearson X2	12	9.8229	0.8186
Log Likelihood		3206.3341	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	4.4823	0.4429	3.6143	5.3503	102.44	<.0001
Expo	1	-0.0017	0.0029	-0.0073	0.0039	0.35	0.5563
V85Merge	1	-0.0026	0.0039	-0.0102	0.0051	0.44	0.5074
Dispersion	1	0.6031	0.2120	0.3028	1.2010		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	12	16.3964	1.3664
Scaled Deviance	12	16.3964	1.3664
Pearson Chi-Square	12	9.7937	0.8161
Scaled Pearson X2	12	9.7937	0.8161
Log Likelihood		3206.5797	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	4.3803	0.3002	3.7918	4.9687	212.86	<.0001
Expo	1	-0.0015	0.0030	-0.0075	0.0044	0.25	0.6167
ΔV85	1	-0.0276	0.0279	-0.0822	0.0270	0.98	0.3217
Dispersion	1	0.5853	0.2067	0.2930	1.1693		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	12	16.4062	1.3672
Scaled Deviance	12	16.4062	1.3672
Pearson Chi-Square	12	10.2887	0.8574
Scaled Pearson X2	12	10.2887	0.8574
Log Likelihood		3206.4405	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	4.1298	0.3079	3.5263	4.7333	179.88	<.0001
Expo	1	-0.0011	0.0031	-0.0071	0.0049	0.13	0.7181
a850ver	1	0.0097	0.0129	-0.0156	0.0349	0.56	0.4540
Dispersion	1	0.5955	0.2096	0.2987	1.1873		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColThAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	12	16.4300	1.3692
Scaled Deviance	12	16.4300	1.3692
Pearson Chi-Square	12	14.2138	1.1845
Scaled Pearson X2	12	14.2138	1.1845
Log Likelihood		1292.8651	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	1.9603	0.8931	0.2099	3.7107	4.82	0.0282
ExpoThAcc	1	0.0006	0.0010	-0.0013	0.0024	0.36	0.5492
V85Merge	1	0.0134	0.0084	-0.0031	0.0300	2.54	0.1111
Dispersion	1	0.7773	0.2744	0.3892	1.5527		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColThAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	12	16.7101	1.3925
Scaled Deviance	12	16.7101	1.3925
Pearson Chi-Square	12	12.1739	1.0145
Scaled Pearson X2	12	12.1739	1.0145
Log Likelihood		1291.8156	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	3.0956	0.4565	2.2009	3.9903	45.99	<.0001
ExpoThAcc	1	0.0008	0.0011	-0.0014	0.0031	0.52	0.4721
ΔV85	1	0.0245	0.0353	-0.0447	0.0936	0.48	0.4879
Dispersion	1	0.8748	0.3039	0.4428	1.7281		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	10	16.0416	1.6042
Scaled Deviance	10	16.0416	1.6042
Pearson Chi-Square	10	11.4432	1.1443
Scaled Pearson X2	10	11.4432	1.1443
Log Likelihood		3209.2586	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	0.3067	3.8584	-7.2557	7.8690	0.01	0.9366
LnAADT	1	0.3214	0.3538	-0.3721	1.0149	0.83	0.3637
a85Over	1	-0.0241	0.1083	-0.2363	0.1882	0.05	0.8241
LnLseg	1	0.2293	0.3793	-0.5141	0.9728	0.37	0.5455
Typ	1	0.4454	0.2267	0.0011	0.8896	3.86	0.0494
Dispersion	1	0.4211	0.1552	0.2045	0.8671		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	11	16.3069	1.4824
Scaled Deviance	11	16.3069	1.4824
Pearson Chi-Square	11	11.2571	1.0234
Scaled Pearson X2	11	11.2571	1.0234
Log Likelihood		3207.4447	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	1.2147	3.8636	-6.3578	8.7872	0.10	0.7532
LnAADT	1	0.2432	0.3564	-0.4553	0.9418	0.47	0.4949
a85Over	1	0.1152	0.0878	-0.0570	0.2873	1.72	0.1898
LnLseg	1	0.5647	0.3830	-0.1860	1.3154	2.17	0.1404
Dispersion	1	0.5262	0.1888	0.2605	1.0630		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	11	16.3458	1.4860
Scaled Deviance	11	16.3458	1.4860
Pearson Chi-Square	11	11.3042	1.0277
Scaled Pearson X2	11	11.3042	1.0277
Log Likelihood		3206.9451	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	3.8443	3.7137	-3.4345	11.1231	1.07	0.3006
LnAADT	1	0.0514	0.3448	-0.6244	0.7272	0.02	0.8815
V85Merge	1	-0.0030	0.0035	-0.0099	0.0039	0.71	0.3990
LnLseg	1	0.4834	0.3879	-0.2769	1.2437	1.55	0.2127
Dispersion	1	0.5602	0.1989	0.2793	1.1236		

APPENDIX D: NB MODELLING ATTEMPTS FOR TOTAL COLLISIONS ON THE SCL

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	12	14.2862	1.1905
Scaled Deviance	12	14.2862	1.1905
Pearson Chi-Square	12	11.7218	0.9768
Scaled Pearson X2	12	11.7218	0.9768
Log Likelihood		313.4934	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	1.0515	0.3352	0.3946	1.7084	9.84	0.0017
ExpoAcc	1	0.0277	0.0050	0.0179	0.0376	30.30	<.0001
ΔV85	1	0.0015	0.0325	-0.0621	0.0651	0.00	0.9630
Dispersion	1	0.2279	0.1434	0.0664	0.7825		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	12	14.2547	1.1879
Scaled Deviance	12	14.2547	1.1879
Pearson Chi-Square	12	11.2707	0.9392
Scaled Pearson X2	12	11.2707	0.9392
Log Likelihood		313.7572	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	1.2966	0.3861	0.5398	2.0534	11.28	0.0008
ExpoAcc	1	0.0290	0.0046	0.0200	0.0380	39.70	<.0001
V85Merge	1	-0.0029	0.0040	-0.0108	0.0049	0.55	0.4603
Dispersion	1	0.2135	0.1395	0.0593	0.7687		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	11	18.6477	1.6952
Scaled Deviance	11	18.6477	1.6952
Pearson Chi-Square	11	20.5960	1.8724
Scaled Pearson X2	11	20.5960	1.8724
Log Likelihood		317.7620	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	1.0069	0.1783	0.6575	1.3563	31.91	<.0001
ExpoAcc	1	0.0162	0.0036	0.0092	0.0233	20.43	<.0001
a85Over	1	-0.0346	0.0088	-0.0519	-0.0173	15.34	<.0001
Typ	1	0.5343	0.1388	0.2622	0.8064	14.81	0.0001
Dispersion	1	0.0000	0.0001	0.0000			

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	11	13.6916	1.2447
Scaled Deviance	11	13.6916	1.2447
Pearson Chi-Square	11	12.3939	1.1267
Scaled Pearson X2	11	12.3939	1.1267
Log Likelihood		320.2466	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	4.2025	0.6410	2.9471	5.4599	43.00	<.0001
V85Merge	1	-0.0321	0.0063	-0.0444	-0.0198	26.22	<.0001
ExpoAcc	1	0.0951	0.0144	0.0668	0.1234	43.46	<.0001
Typ	1	-1.9927	0.4496	-2.8747	-1.1123	19.66	<.0001
Dispersion	0	0.0000	0.0000	0.0000	0.0000		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	11	14.2950	1.2995
Scaled Deviance	11	14.2950	1.2995
Pearson Chi-Square	11	11.7927	1.0721
Scaled Pearson X2	11	11.7927	1.0721
Log Likelihood		313.4956	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	1.0629	0.3759	0.3260	1.7997	7.99	0.0047
ExpoAcc	1	0.0281	0.0080	0.0125	0.0438	12.36	0.0004
ΔV85	1	0.0007	0.0346	-0.0671	0.0685	0.00	0.9837
Typ	1	-0.0181	0.2728	-0.5527	0.5165	0.00	0.9472
Dispersion	1	0.2276	0.1433	0.0662	0.7820		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColAcc
Observations Used	10

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	6	11.6651	1.9442
Scaled Deviance	6	11.6651	1.9442
Pearson Chi-Square	6	13.5082	2.2514
Scaled Pearson X2	6	13.5082	2.2514
Log Likelihood		104.4213	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	0.8225	0.4538	-0.0670	1.7120	3.28	0.0699
EnAADT	1	0.0001	0.0001	-0.0000	0.0003	2.34	0.1262
Lacc	1	0.0001	0.0000	0.0000	0.0001	20.56	<.0001
a85Over	1	-0.0474	0.0125	-0.0720	-0.0228	14.30	0.0002
Dispersion	1	0.1110	0.1775	0.0048	2.5504		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	10	13.5815	1.3582
Scaled Deviance	10	13.5815	1.3582
Pearson Chi-Square	10	12.3261	1.2326
Scaled Pearson X2	10	12.3261	1.2326
Log Likelihood		311.9328	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	-2.3825	2.2954	-6.8814	2.1164	1.08	0.2993
LnEnAADT	1	0.3952	0.3499	-0.2907	1.0811	1.28	0.2588
V85Merge	1	0.0053	0.0063	-0.0071	0.0176	0.69	0.4046
LnLacc	1	0.3775	0.3439	-0.2966	1.0516	1.20	0.2724
Typ	1	0.3838	0.6809	-0.9507	1.7183	0.32	0.5730
Dispersion	1	0.3574	0.1769	0.1355	0.9427		

The GENMOD Procedure

Model Information

Data Set	WORK.TOT
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	TColAcc
Observations Used	15

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	10	13.7283	1.3728
Scaled Deviance	10	13.7283	1.3728
Pearson Chi-Square	10	13.5077	1.3508
Scaled Pearson X2	10	13.5077	1.3508
Log Likelihood		311.6041	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	-2.9946	3.5906	-10.0320	4.0427	0.70	0.4043
LnEnAADT	1	0.5525	0.4283	-0.2869	1.3920	1.66	0.1970
a85Over	1	-0.0217	0.1675	-0.3499	0.3065	0.02	0.8970
LnLacc	1	0.5315	0.3436	-0.1419	1.2049	2.39	0.1219
Typ	1	0.2047	0.6481	-1.0656	1.4750	0.10	0.7521
Dispersion	1	0.3773	0.1913	0.1396	1.0193		