Safety Performance of Freeway Merge and Diverge Areas

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ABSTRACT

Freeways have a distinctive characteristic of offering the highest level of mobility with full control of access. Access to freeways is provided through interchanges with a system of loops and ramps. Adequate lengths between ramp terminals, with enough length for speed change lanes, give the road users enough time to find acceptable gaps, accelerate, decelerate, and merge or diverge properly. Relatively short spacing forces drivers to perform unsafe manoeuvres and hence increase the probability of collisions. Most of previous research has focused on analyzing the operational conditions of the merge and diverge areas using a speed-based analysis. However, little research has handled the effect of merging and diverging from a safety-explicit point of view. Twenty-six interchanges along Highway 417 within the City of Ottawa were selected to quantify the effects of ramp terminal spacing and traffic volumes on safety performance through regression analysis. Poisson and negative binomial models were developed using collision data covering five years from 1998 to 2002. Modelling attempts resulted in 23 statistically significant models relating factors such as traffic volumes and geometric features to collision frequency.

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LIST OF ABBREVIATIONS

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
<i>AdvSp</i>	Advisory Speed
ETR	Engineering and Title Records
<i>Expo</i>	Exposure
HCM	Highway Capacity Manual
IC	Interchange
<i>LAcc</i>	Length of Acceleration Lane
<i>LDec</i>	Length of Deceleration Lane
<i>LSeg</i>	Length of Segment
MTO	Ontario Ministry of Transportation
<i>NR1</i>	Number of Lanes of the First Ramp
<i>NR2</i>	Number of Lanes of the Second Ramp
<i>NSeg</i>	Number of Basic Lanes of the Segment
<i>NV</i>	Entering Volume
<i>NXV</i>	Entering and Exiting Volume
PDO	Property Damage Only
TAC	Transportation Association of Canada
<i>WeaveType</i>	Weaving Type
<i>XV</i>	Exiting Volume

1. INTRODUCTION

1.1 BACKGROUND

The function of the road network is to move people and goods from one location to another in a safe and comfortable manner. According to the highway design classification system, roads have traditionally been classified into four groups; namely: locals, collectors, arterials, and freeways. In addition, two more classifications have been included in the new Canadian design guide by the Transportation Association of Canada (TAC 1999) and are applicable only to urban roads; namely: public lanes and expressways. Each group can provide a different level of the two fundamental functions of mobility and land access. While public lanes provide the highest level of land access in the road network, freeways provide the highest level of mobility. Since freeways are more associated with high speeds, which better suite the mobility function, a collision on a freeway will be more likely to cause a fatality and a serious injury.

As defined in the Highway Capacity Manual (HCM 2000), a freeway is a divided highway with full control of access and two or more lanes for the exclusive use of traffic in each direction (HCM 2000). The two directions of the freeway must be completely separated along its entire length either by a raised barrier or a physical median. Moreover, no signalized, stop controlled, yield controlled, or uncontrolled at grade intersections are allowed along the freeway. Any access to or from the freeway is limited to the use of interchanges to guarantee uninterrupted flow. Generally, interchanges may connect two freeways, a freeway to an arterial, or two arterials roadways. For interchanges that

connect two freeways, TAC (1999) suggested different interchange types, such as a cloverleaf type (for four-leg interchanges) and a trumpet type (for three-leg interchanges). Besides, another set of suggested types was recommended for freeway-arterial interchanges such as the parclo type. There are many factors that affect selecting the type of the interchange including traffic volumes, movement directions, roadway alignment, terrain, and surrounding features such as adjacent rivers, railways or roadways.

The connecting links within an interchange that allow traffic to move from one roadway to another are known as ramps. From the freeway viewpoint, there are two types of ramps: (1) on-ramp or entrance ramp, by which drivers can enter the freeway, and (2) off-ramp or exit-ramp, by which drivers can leave the freeway. The two types are almost similar in shape but different in their function and operating speeds. Off-ramps are always carrying vehicles with higher speeds than the on-ramps with higher probability of collisions. Typically, a merge area would always follow an on-ramp while a diverge area would always precede an off-ramp.

The ramp terminals along the freeway divide it into segments. Three different segments are traditionally used in the analysis process; each has a unique operating conditions. The three segments are *basic freeway* segments, *freeway weaving* segments, and *ramps and ramp junctions*. A *basic freeway* segment is the one that is not influenced by ramps or weaving areas. On the other hand, a *freeway weaving* segment is usually formed when a merge area is closely followed by a diverge area, or when an on-ramp is closely followed by an off-ramp and the two are joined by an auxiliary lane (HCM 2000). When the freeway weaving segment length exceeds a specific distance, 750 m according

to HCM (2000), it is recommended to treat it as isolated merge and diverge areas, called *ramp and ramp junction* segments.

There are many common factors affecting safety performance on freeways. However, within the merge and diverge areas, the traffic operation becomes more complicated with more factors affecting traffic operation and safety performance such as distances available for acceleration, deceleration, and weaving. Relatively short distances between ramp terminals under normal traffic operating conditions would make it difficult for vehicles to cross the path of each other to get to their appropriate lanes. Therefore, drivers might have to make unsafe lane change manoeuvres, which might subsequently lead to higher collision frequencies. Road users in the merge and diverge areas are required to select and adjust speeds without causing undue hazards to themselves or to other road users.

Such hazardous manoeuvres would normally increase in terms of frequency and intensity with increasing the volumes of vehicles entering and exiting a specific segment on the freeway. These traffic volumes would increase the number of lane changes required over this segment. In addition, finding a gap between moving vehicles to perform a merge or diverge operation will be more difficult under congested traffic conditions. Drivers are then forced to raise the risk level that they are willing to accept and may perform unsafe lane changes. Therefore, segments that experience high entering and exiting traffic volumes should be expected to experience higher collisions frequencies.

On the other hand, adequate length of speed change lanes might mitigate the complexity of the problem and give the drivers enough time to find suitable gaps, accelerate or decelerate, and merge or diverge in a proper way. Ideally, drivers should be able to complete their manoeuvres in the merging process before reaching the end of the acceleration lane. In cases where the length of acceleration lane is inadequate, drivers would find themselves having to merge with the main traffic whether or not appropriate gaps are available. Such a scenario would also result in more congestion and would increase the disturbance to the main traffic. Moreover, collisions would be more likely to occur within a merge area with a relatively short acceleration lane.

On the other hand, safety performance on the diverge areas might be more critical due the fact that deceleration lanes have higher speeds compared with acceleration lanes. The length of the deceleration lanes should enable the driver to perform the deceleration process within the deceleration lane. If the length of the deceleration lane is inadequate, a driver would start decelerating before occupying the deceleration lane. This could cause a reduction in the speed, and hence in the level of service of the freeway. Moreover, it may result in a collision if the lead vehicle's deceleration level is underestimated as mentioned by Park et al. (2001). Well-designed signs, with enough spacing between signs and the following exits, should help mitigate the insufficient deceleration length.

Figure 1.1 shows the current state of road safety in Ontario, according to the *Ontario Road Safety Annual Report (ORSAR 2001)*. The figure shows the number of collisions that occurred over fourteen years from 1988 to 2001. Although the total collisions including property damage only (PDO) do not follow a specific trend but rather fluctuate over the presented period, the figure shows a slight improvement in collisions

involving fatalities and injuries over the entire road network. However, the figure shows that the number of fatal and injury collisions are almost levelling off, and therefore there are still in deed needs for further research on safety improvements.

Figure 1.1: Property Damage, Personal Injury, and Fatal Collisions Occurring in Ontario, (ORSAR, 2001)

In addition, Figure 1.2 shows the general trend of the total number of persons involved fatal, injury and PDO collisions in Ontario. The figure also shows the contribution of the errors due to improper lane change. The figure shows a trend of increasing the total number of persons involved in collisions as well as persons involved in collisions caused by improper lane changes especially after year 1998. Although this chart represents the collisions that happened on the entire road network in Ontario, it

reflects the necessity of devoting more effort to improve the safety performance where intensive lane change manoeuvres usually take place.

Figure 1.2: Persons Involved in Collisions in Ontario and Contribution of Improper Lane Change (ORSAR, 2001).

Analysis of safety performance on freeways is a major concern because of the high probability of more severe collisions due to the high operating speeds as well as because of the traffic congestion that would follow a collision. Current design guides, such as TAC (1999), recommend empirical values for ramp terminal spacing, which are based on implicit-safety criteria, in terms of the speed of traffic. For example, they recommend the distances between successive exits to be enough for adequate signing without explicitly quantifying the expected safety benefits from changing the spacing between successive exits. As stated in TAC (1999), *“In general, reliable, specific, and explicit quantitative correlations between interchange parameters and safety are not*

available. The design domain concept is therefore applied in terms of empirical information such as, “best practices”, which are provided as guidance...”

1.2 OBJECTIVES

Based on the discussions presented in the previous section, the objectives of this thesis were set as follows:

- Summarizing the available literature on factors contributing to safety performance on freeway section.
- Reviewing the previous research on safety performance of the merge and diverge areas.
- Quantifying the relationship between merge and diverge parameters and safety performance.
- Estimating the expected safety improvements and collision reduction opportunities using modelling techniques.

1.3 SCOPE OF RESEARCH

The scope of this research is limited to Highway 417 within the City of Ottawa, Ontario, Canada. Twenty-six interchanges along Highway 417 between Interchange 145 west (intersection with Highway 7), and Interchange 110 east (intersection with Walkley Road), were selected for this study. The reason behind this selection is to have enough variation in land use and interchange characteristics with the selected interchanges for a

reliable analysis. Highway 417 has urbanized characteristics within the core of the City of Ottawa with relatively short distances between ramp terminal spacing. As Highway 417 goes east and west the spacing between the interchanges increases giving a wide range of ramp terminal spacing that can help in statistically capturing the safety effects due to this variation.

1.4 THESIS ORGANISATION

After presenting the introduction of the thesis in this first chapter, Chapter 2 provides the design criteria and definitions that were mentioned in the current design guidelines. Chapter 3 contains a literature review of the factors influencing safety performance with a focus on safety of merge and diverge areas. Chapter 4 discusses the details of the data collection, including site selection and the collection of geometric data, traffic volume, and collision history. Chapter 5 provides a descriptive analysis of the data collected in this study. Chapter 6 summarizes the common steps to develop a good model with an acceptable goodness of fit. Chapter 7 applies the modelling steps to examine the relationship between collisions and candidate merge and diverge parameters such as lengths, volumes, and speeds. Finally, Section 8 presents the main conclusions and recommendations of the research results.

2. REVIEW ON DESIGN CRITERIA

This chapter provides some of the design criteria and definitions that were mentioned in the current design guidelines such as the Transportation Association of Canada (TAC 1999), the Highway Capacity Manual (HCM 2000), and the American Association of state Highway and Transportation Officials (AASHTO 2001). Section 2.1 discusses the interchange design criteria, including interchange spacing, ramp terminal spacing, ramp uniformity, ramp speed and width, and speed change lanes. In Section 2.2, the merge and diverge areas are defined. Finally, Section 2.3 discusses the definition and different types of weaving areas.

2.1 INTERCHANGE DESIGN CRITERIA

An interchange is defined in TAC (1999) as a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roads and for any range of design speeds. The main difference between an interchange and a simple at grade intersection is that the first permits high traffic volumes with relatively safer operations and smooth manoeuvres. Typically, interchanges eliminate crossing conflicts and minimize turning conflict between traffic volumes for different movement directions.

It is highly recommended in TAC (1999) to consider a consistent design for a series of interchanges in conjunction with individual interchange design. This supports the important relationship between the design consistency of different roadway elements

and safety. *In general, reliable, specific and explicit quantitative correlations between interchanges parameters and safety are not available* (TAC 1999, 2.4.1.1). However, the interest in explaining the explicit relationship between design criteria and safety has recently become an attractive point in transportation fields as was found in the recent research.

2.1.1 Interchange Spacing

The location of the interchanges along any freeway should be selected based on two criteria. The first criterion is to provide an adequate service that is needed by the surrounding communities along the freeway. This could be interpreted as a need for more interchanges near business and congested areas. The second criterion is to minimize the disturbance in the traffic operation that might result from high density of access points. The balance between these two criteria would result in acceptable interchange spacing under the prevailing conditions. As mentioned in TAC (1999), interchanges are normally located 3 to 8 km apart in rural areas at which population density is not high. However, in urban areas, this relatively long distance cannot serve the community in a proper manner. Therefore, a recommendation to use interchange spacing of 2 to 3 km was mentioned in TAC (1999). Besides, designers need to note that *"Freeway collision rates tend to increase as interchange spacing decreases in urban areas. This effect on collision rates is an important consideration in urban area interchange spacing."* (TAC 1999, 2.4.3.1)

AASHTO (2001) design guide proposed lower values of interchange spacing than the values mentioned in TAC (1999). It was recommended in AASHTO (2001) to use a

distance of at least 3 km in rural areas. In addition, this distance is recommended to be 1.5 km in urban areas. In cases that require a relatively short distance between interchanges than these minimum values, freeway could be provided with collector-distributor roads.

2.1.2 Ramp Terminal Spacing

The spacing between each two successive ramps along the highway represents one segment. Ramp terminal spacing could be located between interchanges or within one interchange. Typically, highway segments can be divided into four main types; namely: (1) *entrance-entrance*; (2) *entrance-exit*; (3) *exit-entrance*; and (4) *exit-exit*. The classification is based mainly on the type of the first ramp and second ramp for a specific segment. The length of each segment is stated to be measured from the physical nose at the gore of the first ramp to the physical nose at the gore of the second ramp. TAC (1999) sets empirical values, as shown in Figure 2.1, for ramp terminal spacing that are based on different criteria as follow:

1. Distances between an entrance and exit were recommended to be based on weaving requirements.
2. Distances between two successive exits should be based on the provision of adequate signing to enable vehicles to do the required manoeuvre properly.
3. Distances between two successive entrances should be enough for vehicles joining the freeway from the first entrance to merge safely.

4. Distances between an exit and entrance should be enough for vehicles to be prepared for the next merge.

Spacing	Mainline design speed (km/h)					
(m)	80	90	100	110	120	130
L ₁	300	325	350	375	400	425
L ₂	200	225	250	275	300	325
L ₃	Based on Weaving requirements					
L ₄	Sufficient to allow for acceleration and merging length before second entrance					

Figure 2.1: Ramp Terminal Spacing (TAC 1999: 2.4.6.16).

2.1.3 Ramp Uniformity

Ramp uniformity or consistency has been stressed in the TAC guide, and can be considered a special application of the general design consistency concept. It was mentioned that, the consistency between the ramps would guarantee safe and smooth traffic operation on freeways. Ramp uniformity could be provided through maintaining single exits ahead of structures on a specific freeway length. Ramps located ahead of underpass structures are more preferable to be used than ones with exits beyond the structures. They are more visible to the driver and they help the drivers to select the

appropriate speed and adjust the vehicle position to perform safe manoeuvres. Moreover, as mentioned in TAC (1999), the left-hand entrance of the high speed roadways is not a preferable design. On the other hand, using right-hand side exits is more suitable in terms of consistency and uniformity.

Figure 2.2 shows the consistency concept as mentioned in TAC (1999). In the first design scheme in this figure, it is obviously indicated that the interchanges have a mix of single and double exits. In addition, some of the exits are beyond the structure and the rest are before it. This design is considered inconsistent and would cause a kind of confusion for the driver. On the other hand, the second design scheme consists of a consistent design in which all interchanges have single exits before the structure. This kind of consistency would keep the driver concentration at a uniform level and hence improve operating conditions.

AASHTO (2001) emphasised the same concept in which designing a series of interchanges should consider the consistency of the interchanges as well as individual interchange. Expression *interchange uniformity* was used by AASHTO (2001) to apply the concept of using consistent exits and entrances. Generally, *"Considering the need for high capacity, appropriate level of service, and maximum safety in conjunction with freeway operations, it is desirable to provide uniformity in exit and entrance patterns"* (AASHTO 2001, p 811).

Figure 2.2: Consistency of Exits (TAC 1999, 2.4.4.3).

In the study area selected for this research, all interchanges located on highway 417 are consistent with exits located before structures except only one interchange (Interchange 126 – intersection with Maitland Avenue) that had an exit after the structure. It was very interesting to note that the segment just preceding this interchange experienced the highest collision frequency of 78 property damage only (PDO) collisions compared to other segments in the study area. The consistency concept could be one of the main factors that caused this relatively high collision frequency observed at this location.

2.1.4 Ramp Speed and Width

For further achievement of the consistency concept along the highway, design speeds for terminal ramps are recommended to be carefully selected in order to keep the

expectancy of the driver at reasonable values. Table 2.1 shows the recommended design speeds for ramps with the domain concept as mentioned in TAC (1999). Conceptually, it is desirable to have ramps with design speeds relatively close to the highway speed. Therefore, the upper limit in this table is more recommended. For suitable ramp widths, it was mentioned in TAC (1999) that ranges from 4.8 m to 5.0 m would be sufficient for single ramps. However, for ramps with two lanes or more, it is recommended to use lanes with a minimum width of 3.7 m.

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

Roadway Design Speed (km/h)	Ramp Design Speed Domain (km/h)
<i>60</i>	<i>50-40</i>
<i>70</i>	<i>60-40</i>
<i>80</i>	<i>70-40</i>
<i>90</i>	<i>80-50</i>
<i>100</i>	<i>90-50</i>
<i>110</i>	<i>100-60</i>
<i>120</i>	<i>110-60</i>
<i>130</i>	<i>110-70</i>

2.1.5 Speed Change Lanes

Speed change lane is defined in TAC (1999) as an added lane adjoining the travelled way of the roadway and does not necessarily imply a definite lane of uniform width. It was also mentioned that speed change lanes should have enough length to guarantee collision-free and comfortable operation. TAC (1999) stated that the length of a speed change lane is based on three factors; (1) the running speed on the through lanes; (2) the control speed of the ramp; (3) the manner of decelerating or accelerating on the speed change lanes.

Typically, there are two main types of speed change lanes; namely: parallel type and direct taper type. The parallel type is an auxiliary lane that consists of two parts; (1) L_d : that has a constant width for gradual speed change between the freeway and the ramp and (2): L_t : that has a gradual change in width to guarantee adequate lateral movement rate. On the other hand, the direct taper type of speed change lanes consists of an auxiliary lane that has a gradual change in lane width. *"This type works on the principle of a direct entry or exit at a flat angle"* (TAC 1999). Figure 2.3 shows the different speed change lane types as illustrated in the TAC (1999).

Another definition for the speed change lane was mentioned in AASHTO (2001) as an auxiliary lane, including tapered area, primarily for acceleration or deceleration of vehicles entering or leaving the through-traffic lanes. Generally, the purpose of the speed change lane was stated as: *"An auxiliary lane may be provided to comply with the concept of lane balance, to comply with capacity needs, or to accommodate speed changes, weaving, and maneuvering of entering and leaving traffic"* (AASHTO 2001, 818).

To minimize the disturbance of the through traffic operation, speed change lanes are recommended to be included in the design especially for highways having expressway characteristics. A sufficient width and length for speed change lanes are required to facilitate the traffic manoeuvres at the merge and diverge areas in a proper way. In addition, it was mentioned in AASHTO (2001) that the auxiliary lane should be provided with lane width equal to the through lane width. Moreover, the shoulder width should be ranged from 2.4 to 3.6 m with minimum of 1.8 m (AASHTO 2001). Some considerations regarding speed change lanes were also mentioned as follow:

- Speed change lanes are more reasonable to be used in highways with high speeds and high volumes.
- Some of the drivers use little of the available speed change lanes. However, these lanes are necessarily to improve traffic operations on highways.
- Extent of drivers' use of speed change lanes depends on the traffic volumes.

Figure 2.3: Speed Change Lanes (TAC, 1999: 2.4.6.4).

- Using speed change lanes with a long taper fits more the behaviour of most drivers.
- Deceleration lanes before off-ramps may function as storage lanes besides the main function of changing speeds.

In addition, using continuous auxiliary lanes between the entrance and exit ramps are recommended by AASHTO (2001) in three cases: (1) at interchanges that are close to each other; (2) when the distance between the end of the taper on the entrance and the beginning of the taper on the exit terminal is relatively short; (3) when local frontage roads do not exist. Figure 2.4 shows the different types of continuous auxiliary lanes according to AASHTO (2001).

Moreover, the auxiliary lane could be terminated in two-lane exit (Figure 2.4.A), or in single-lane exit (Figure 2.4.B). Another method of terminating the auxiliary lane is to carry the full width auxiliary lane to the nose before tapering it into the mainline roadway (Figure 2.4.C). In case that a single-lane exit experiences disturbance with vehicles attempting to recover on the through lane, the recovery lane should be extended 150 to 300 m before being tapered (Figure 2.4.D). For large interchanges, designers might need to extend the auxiliary lane for approximately 750 m downstream the influence of the interchange (Figure 2.4.E).

Figure 2.4: Alternative Methods of Dropping Auxiliary Lanes (AASHTO 2001, 819).

2.2 MERGE AND DIVERGE AREAS

2.2.1 Definition

Generally, the merge areas are the areas that follow on-ramps while the diverge areas are the ones that precede off-ramps. Merge areas were defined in HCM (2000) as the area at which individual merging vehicles attempt to find gaps in the adjacent freeway

lane traffic stream. The merging vehicles coming from the on-ramp initially occupy the acceleration lane and then seek an acceptable gap to safely join the freeway. This process requires accelerating the vehicle from a lower speed at the ramp to a higher one on the freeway before reaching the end of the acceleration lane. The entering vehicles cause other vehicles moving on the freeway to push themselves away from the lanes adjacent to the on-ramp. On the other hand, a diverge area is defined as the area at which a single traffic stream splits into two streams (HCM 2000). Vehicles that separate from the freeway stream first occupy the deceleration lane and then reduce their speed from the freeway speed to the ramp speed. Providing adequate length for acceleration and deceleration lanes can give drivers time to complete their manoeuvres safely.

2.2.2 Lane Balance at Merge and Diverge Areas

The balance between the numbers of lanes is a vital factor that should be considered at any segment on the freeway. It was mentioned in the TAC (1999) as well as the AASHTO (2001) that, at merge areas, the summation of the number of lanes of the freeway upstream the merge area and the number of lanes of the on-ramp should be equal to or one more than the number of lanes downstream the merge section (see Figure 2.5). In addition, at diverge areas; the number of the lanes upstream a diverge area should be less one than the summation of the number of lanes of the off-ramp and the number of lanes downstream a diverge area. In cases "*Where an auxiliary lane is extended beyond an entrance to maintain both lane balance and basic lanes, the lane should be continued for 700 m to 900 m to permit entering traffic to disperse into the through lanes*" (TAC 1999, 2.1.6.5).

Figure 2.5: Lane Balance Concept (TAC 1999, 2.1.6.3).

2.3 WEAVING AREA

2.3.1 Definition

Although merge and diverge manoeuvres are always required with all ramp terminals joining the freeway, weaving sections are only formed when a merge area is directly followed by a diverge area. Weaving sections have been defined in TAC (1999) and AASHTO (2001) as roadway segments where the pattern of traffic entering and leaving at contiguous points of access result in vehicle paths crossing each other. In the HCM (2000), a weaving section was defined as the crossing of two or more traffic streams travelling in the same general direction along a significant length of highway, without the aid of traffic control devices.

Typically, weaving section could be located within interchanges, between an on-ramp followed by an off-ramp of successive interchanges, or on segments of overlapping roads. Short distances available for the drivers to do the appropriate manoeuvres in

weaving sections create a potentially hazardous location and increase the probability of collisions. With increasing the weaving length, merge and diverge manoeuvres tend to segregate over the entire length between the on-ramp and the following off-ramp. The term *out of the realm of weaving* has been used by TAC (1999) to point to a section where frequency of changing lanes is similar to that on an open road. It was mentioned also in TAC (1999, 2.1.7.6) that a distance of 1000 m or more is considered a criterion for this type of weaving. However, in HCM (1994), 762 m (2500 ft) is the maximum weaving length for their methodology to be valid. This length was revised to be 750 m in HCM (2000). On the other hand, weaving volume was defined in HCM (2000) as the portion of total volume in the weaving section that has to change their lanes in order to reach their destination.

2.3.2 Weaving Types

Based on their operational features, weaving areas are generally classified into three types (*A*, *B*, and *C*) in almost all references including TAC (1999) and HCM (2000) according to the operational features. The three types are summarized as follows (see Figure 2.6):

1. *Type A*: each weaving vehicle in this type makes one lane change to reach the desire terminal or exit. Type A could be one of two forms. First form is *ramp weave* or *one-sided weave* at which on-ramp is connected to off-ramp by an auxiliary lane. Second form is the *major weaving section* which is characterized by three or more entry and exit roadways having multiple lanes (see Figure

- 2.6.i). A unique characteristic of *Type A* is that it has a *crown line* connecting the nose of the entrance gore area and the nose of the exit gore area. All weaving volumes should cross the crown line in order to execute the required weaving manoeuvres.
2. *Type B*: one of the two weaving manoeuvres in this type could be accomplished without making any lane change, while a maximum of one lane change is required by the other weaving volume (see Figure 2.6.ii). *Type B* would normally tend to carry larger weaving volumes than *Type A*.
  3. *Type C*: the same as *type B* except the number of lane changes required for the other volume is at least two lanes (see Figure 2.6.iii).

In terms of the number of entrances and exits, Figure 2.7 shows two types of weaving sections as mentioned in AASHTO (2001). A single weaving section occurs when single entrance is followed by single exit. On the other hand, a multiple weaving section is defined as a portion of a one-way roadway that has two successive entrances followed closely by one or more exits; or has one entrance followed by two or more exits (AASHTO 2001).

Figure 2.6: Types of Weaving Sections (TAC 1999, 2.1.7.5).

Figure 2.7: Simple and Multiple Weaving Sections (AASHTO 2001, exhibit 2-34).

2.3.3 Weaving Length

A criteria for measuring weaving length has been set by TAC (1999) as the length from the point that has the lane edges apart by 0.5 m at the merge to the point that has the lane edges apart by 3.7 m at the diverge (see Figure 2.8). The weaving length is recommended in TAC (1999) to be at least 800 m to 1000 m if the section is between freeway interchange and arterial interchange. In addition, a distance of at least 550 m to 700 is recommended if the weaving section is between two arterial interchanges. Another similar weaving length configuration was mentioned in HCM (2000) but instead of the 0.5 m distance between lane edges at the merge area, a distance of 0.6 m was recommended.

Figure 2.8: Method of Measuring Weaving Length (TAC 1999, 2.1.7.7).

3. LITERATURE REVIEW

After reviewing the design criteria in the current design guides, this chapter presents the previous research that addressed the effect of these criteria on safety performance either implicitly or explicitly. Generally, freeway safety is function of a huge number of factors. Many of these factors are very difficult to be controlled, measured, or analyzed. However, others can be modelled and can eventually be manipulated to reduce the number of collisions, and will also be covered in this chapter. Section 3.1 discusses the safety performance of highways in general. Section 3.2 focuses on safety performance at merge and diverge areas. Finally, section 3.3 shows the previous research at weaving sections with freeway safety interaction.

3.1 SAFETY PERFORMANCE

Prediction of collision frequencies is not an easy task due to the complexity of the interaction between the factors that affect the collision frequency on the road network. In order to discover which of them are more significant and can be used to improve safety, researchers have attempted to examine different combinations of safety predictors that are believed to be effective in safety evaluation. Generally, factors that affect highway safety can be categorized into one of four categories; namely: environmental factors, road users characteristics, traffic characteristics, and the geometric features of the road. The variation of these four categories results in different safety levels and causes variations in the collision frequency on a specific highway section.

Miaou (1996) mentioned that there are two types of collision variations. First type is *between-sites variation* or systematic variation which explains the variation of collision frequency from site to another. One can theoretically account for the systematic variation if all possible factors that affect the collisions are considered. The second type is the *between-time variation* or random variation. Simply, if all possible variables for a specific site are kept constant, a difference in the collision frequency over equal time periods will still be noticed for this site. The random part of collision variation statistically follows a specific probability function. Miaou (1996) mentioned that Poisson and Gamma distributions are superior to explain this part as will be discussed later. Figure 3.1 shows a diagram to clarify the variation in collision frequency as mentioned by Miaou (1996).

Figure 3.1: Total Explained, Unexplained, and Random Variation of Collision Frequencies (Miaou 1996).

Among the significant factors, there are factors that are difficult to be included in the analysis because of the difficulty of measuring them such as the driver behaviour and the car condition immediately before the collision. The following discussion presents

more details about safety performance to explain the systematic variation of the collisions.

3.1.1 Environmental Factors

One of the main factors that can affect operating conditions on a road network is the environmental conditions. Many researches have examined the effect of environmental factors on the operating conditions, such as operating speeds, while others attempted to relate their effect to the explicit safety performance. For example, Liang et al. (1998) mentioned that there are many factors that can affect vehicle's speed, among which visibility is a main factor besides other factors such as speed limit, geometric design, traffic density, and roadway surface condition. Reduced visibility might be caused by darkness, rain, fog, snow, or dust. Mean speed reduction for all vehicles during fog and snow events was observed to be 8.0 km/h and 19.2 km/h, respectively. They also found a reduction of 1.1 km/h of mean speed for every kilometre per hour that the wind speed exceeds 40 km/h.

The problem with speed reductions during bad environmental conditions, as mentioned by Liang et al. (1998), is that speed reductions accompanied by high variances in speeds which are believed to affect safety. However, an interesting observation from this study is that almost no collisions occurred during the periods of poor environmental conditions. They interpreted that drivers were more careful, and were able to avoid each other safely during these periods.

Brown and Baass (1997) studied the effect of seasonal variation on frequencies and rates of highway collisions. The analysis included all police-reported collisions associated with numbered roads in the Monteregion region of Quebec. It was found in this study that a percentage of 30 % to 50 % of highway collisions was associated with bad weather conditions such as rain, snow, hail, and icy conditions. It was also found that the winter season experienced a notable increase in minor collisions and a reduction in severe and fatal collisions. In addition, a negative correlation between monthly rates of both severe and fatal injuries and material-damage-only collisions was observed.

Golob and Recker (2003) used linear and nonlinear multivariate statistical analysis to determine how the types of collisions that occur on freeways are related to weather, traffic volume, and lighting condition. The study was carried out on six freeways located in Southern California. Data of approximately 1200 collisions during 1998 were used in this research, and collision data were characterized by the type and the location of the primary collision, the number of vehicles involved, and the collision severity in terms of injury versus property damage only.

The study revealed that collisions resulting from hitting a side-object and collisions involving multiple vehicles because of weaving manoeuvres are more likely to occur on wet roads. These two kinds of collisions were also found to be associated with relatively stable traffic characterized by low volumes and high steady speeds. In addition, rear-end collisions were more likely to occur on dry roads during daylight where there is a high variation in relatively low speeds. It was also mentioned that collision severity is influenced more by volume than by speed. In general, it was stated that moderate volumes with relatively high speeds produced conditions that increased collision severity.

3.1.2 Road Users

The characteristics of road users can significantly affect safety. Dissanayake et al. (1999) used regression analysis to forecast highway collision rates of selected population subsets in Florida. The subsets used in this study included older drivers (older than 65), young drivers, international drivers, and school-age children as non-motorists. They used a wide range of data in the modelling starting from 1975 to 1996. Among the statistical methods used in this study, log linear modelling was the one that fit best with all subsets. The dependent variables for the developed models were fatality rate per 100 million vehicle miles travelled, fatality rate per 100,000 populations, and fatality rate per 100,000 licensed drivers. The models were calculated by taking the natural logarithm of the actual fatality rates. The coefficient of determination " R^2 " value for the forecasting models ranged from 0.784, in the model predicting collisions for older drivers, to 0.881, in predicting collisions for international tourists.

Another study which handled roadside effect on run-off-road collisions was carried out by McGinnis et al. (2001). They collected collision data from 1975 to 1997 from the Fatality Analysis Reporting System (FARS). The purpose of this study was to investigate the effect of driver characteristics, such as gender, age, and alcohol usage, on run-off-road collision rates. It was found that males aged 20 to 24 had run-off-road collision rates 3.3 times those of females of the same age. In addition, using run-off-road collision rates for females aged 40 to 49 as a base; the collision rate was 20 times higher for teenage males and 9 times higher for teenage females. This ratio for drivers aged 70 and older was found to be 4.5 for males and 4.0 for females. For the effect of alcohol, it was found to be a factor in run-off-road collisions for nearly 50 % of the males aging 20

to 39. Also, more than 50% alcohol involvement was reported for all drivers during dark conditions. It was finally concluded that this research has identified types of drivers, vehicles, and highways where efforts are needed to reduce collisions.

Mercier et al. (1997) studied the effect of age and gender on injury severity in head-on highway vehicular collisions. They used data from the Iowa Department of Transportation from 1986 to 1993. There were approximately 66,000 to 68,000 total vehicular collisions reported each year. The study confirmed the common belief that injuries to older drivers and passengers are more severe than injuries to younger drivers and passengers. In addition, results of logistic regression analysis indicated that four factors were found to be significant; age of drivers and passengers, position in the vehicle, form of protection such as air bags, and a set of interactive variables such as age and position.

3.1.3 Traffic Characteristics

Awad and Janson (1998) used both regression analysis and artificial intelligence techniques to predict truck collisions on freeway ramps in Washington State. The collisions that occurred at intersections with arterials were excluded from this study, while all collisions that took place on ramps, acceleration/deceleration lanes, or in the lanes adjacent to the acceleration/deceleration lanes were included. The total number of interchanges was 391 interchanges among which 82 experienced no collisions during the study period. Finally, a dataset containing 1,030 truck collisions associated with 581 ramps was used to model truck collisions. A total of ten factors were found to capture

more than 75 % of the total variation in the data. Among those ten factors, only four factors were found sufficiently independent and of some explanatory power. These four factors are:

1. Gore to taper distance.
2. Ramp traffic volume.
3. Main road traffic volume.
4. Truck percentage on the main road.

Other factors such as ramp length, truck percentage on the ramp, weather conditions, urban/rural location, visibility, and road surface conditions were not significant. In this study, the Poisson or negative binomial regression was recommended to improve the fit rather than traditional linear regression models. In order to approximate the existing data to fit Poisson distribution, a square root transformation of the truck collision frequency data was used. Four categories were used in the analysis, one for each known ramp type named; diamond, loop, outer connector, and directional ramps. The coefficient of determination " R^2 " of the developed models ranged from 0.062 to 0.233 which was considered statistically very low. In addition, it was mentioned that the ability of artificial intelligence techniques as another methodology to predict truck collisions was greater than regression procedures. However, it was finally recommended increasing the training data to reach reasonable results. The study did not give any satisfactory results with the rest of the collision data.

The relationship between traffic volumes and collision rates was also examined by Zhou and Sisiopiku (1997). The study area included a 26 km segment of interstate I-

94 in Detroit. The ratio between the average hourly traffic volume collected in 1993 and 1994 to capacity (v/c) was taken as an indication of the traffic condition. The study revealed a relationship between (v/c) ratio and total collision rate described by a U-shape pattern. However, rates of collisions involving injury and fatality were found to decrease as (v/c) ratio increased. The finding was interpreted to mean that the high collision rates for a high range of (v/c) was due to the traffic conflicts being a major contributing factor. It was also found that the developed models had a weak relationship between (v/c) ratio and collision rates, with R^2 values ranging between 0.27 and 0.79. Finally, it was mentioned that the study was primarily concerned with the trend of the relationship more than the significance of the regression parameter.

The relationship between collisions and traffic flow was discussed by Mensah and Haur (1998). These researchers tested the availability of using average flow to predict the collision frequency. First, they mentioned that collisions should be related to the flows prevailing near the time of collision occurrence. Two situations that do not affect taking the average flow in the analysis, as stated by Mensah and Hauer (1998), are:

1. A constant flow during the averaging period, and
2. A linear relationship between traffic flow and expected collision frequency.

Otherwise, an error in considering average traffic flow needs to be considered. In this study, procedures were developed to account for the fluctuation in traffic flow over a specific period in order to eliminate the effect of averaging. Assuming a known distribution of the traffic flow over a period of time T , that is $q_1, q_2, \dots, q_n$ during proportion $p_1, p_2, \dots, p_n$ of time T , a correction factor W can be calculated as follows:

$$W = \frac{E\{\mu(q)\}}{\mu(E\{q\})} \quad \text{Equation 3.1}$$

Where;

$$E\{\mu(q)\} = \sum_1^n \mu(q_i) p_i \equiv \text{Equivalent expected collision frequency over a period of time } T$$

$$\mu(E\{q\}) = \mu\left(\sum_1^n q_i p_i\right) \equiv \text{Average collision frequency over a period of time } T \text{ multiplied by the equivalent traffic volume over the same period of time } T.$$

Charts of W have been developed by Mensah and Hauer (1998) based on the fact that the relationship between average flow and expected collisions is not a linear relationship. A recommendation was mentioned to use the developed charts and dividing the estimated collisions calculated in any time period by the W values to account for the variation of the traffic over that period of time.

Hauer (2001) mentioned that, in calculating collision rates, using Equation 3.2 does not give equal rates for different vehicle types if these different vehicle types are assumed equally likely to be involved in a collision. Alternatively, Hauer (2001) recommended using Equation 3.3 that has the same vehicle type in both, numerator and denominator.

$$\text{Accident rates} = \frac{\text{number of collisions involving a given vehicle(driver)}}{\text{Exposure by vehicles (drivers) of the given type(group)}} \quad \text{Equation 3.2}$$

$$\text{Truck involved rates} = \frac{\text{Number of trucks involved in accidents}}{\text{Total number of trucks}} \quad \text{Equation 3.3}$$

It was mentioned that, if vehicle exposure is in the denominator, the count of vehicles of the same type must be in the numerator, and a similar approach must be followed when dealing with drivers. In addition, Hauer (2001) tried to answer the question of what meaning can be attributed to a finding of overrepresentation. It was recommended that, one should not conclude that a certain vehicle type is more likely to be involved in collision if this type was found overrepresented in reported collisions of a specific severity. This is because that conclusion may be caused by a combination of three factors:

1. The probability to be in a collision per unit of exposure.
2. The probability of the collision to be reported.
3. The probability of the collision to be of the specific severity.

3.1.4 Geometric Characteristics

Many researches have studied the effect of the geometric characteristics from a safety-explicit point of view. All geometric elements along any highway should provide a smooth operation that does not violate the driver expectancy and guarantees a safer road network through the design consistency concept. Most of the previous research studied the design consistency concept on two-lane highways. However, similar benefits should be expected from applying the concept to all road types.

3.1.4.a Definition

Design consistency, as defined by Al-Masaeid et al. (1995), is the degree to which highway systems are designed and constructed to avoid critical manoeuvres that may lead to collision risk. Anderson et al. (1999) suggested another definition of design consistency as the ability of highway geometry to conform to driver expectancy. In the last few decades, it was found through research that an inconsistent roadway is more prone to collisions than a consistent one. A road design that violates the driver expectancy increases the probability of relatively long response time as well as unsafe driving manoeuvres. As stated by Ng and Sayed (2004), when design consistency is ensured, all abrupt changes in geometric features for contiguous highway elements are eliminated, preventing critical driving manoeuvres and minimizing collision risk.

3.1.4.b Common Consistency Measures

Design consistency measures were classified in TAC (1999) as one of three main categories; namely: cross section, operating speed, and driver workload consistency. These categories were then revised by Easa (2002) to include another main category; named alignment geometry consistency. First, cross section elements such as lane and shoulder widths should be kept to the same width everywhere as mentioned by TAC (1999). A sudden change in the configuration of the cross section might violate the expectancy of the drivers and cause them to do unsafe manoeuvres. It is recommended to introduce a gradual change with tapers as practically possible.

As for the second measure, operating speed is considered a visible indicator of inconsistencies. Drivers tend to reduce their speed if the roadway violates their expectancy. Typically, operating speed is defined as the speed selected by the drivers when not restricted by other users and is normally represented by the 85th percentile speed which will not be exceeded by 85 % of the drivers. The difference between the design speed V_d and operating speed V_{85} is a good indicator of the consistency for a single element. Moreover, the difference of operating speeds between successive elements (ΔV_{85}) is used to measure the inconsistency by a driver moving from one element to another.

The third measure is driver workload which was defined in TAC (1999) as the demands placed on a driver by the road. Driver workload was also defined by Messer (1980) as the time rate at which drivers must perform the driving task which changes continuously until it is completed. This term represents the demands required by the driver to interact with the information transferred from the surrounding features. If the workload drops too low or rises too high, the collision rate can increase (TAC 1999). Easa (2002) mentioned that consistent road geometry allows a driver to predict the correct path while using little visual information processing.

Finally, alignment indices are quantitative measures used to describe the character of an alignment. Consistency indicators could be one of the following commonly used alignment indices shown in Table 3.1.

Table 3.1: Commonly Used Alignment Indices.

Average radius of curvature	$AR = \sum \frac{R_i}{n}$
Ratio between maximum and minimum radius	$RR = \frac{R_{\max}}{R_{\min}}$
Curve radius to average radius	$CRR_i = \frac{R_i}{AR}$
Average rate of vertical curvature	$AVC = \frac{\sum A_i }{n}$

3.1.4.c Design Consistency and Safety Relationship

Among the research found addressing the relationship between safety and design consistency is that by Anderson et al. (1999). They investigated the relationship between safety and five candidate measures of design consistency; namely: speed reduction on curves relative to the proceeding element either tangent or curve (SR), average radius (AR), ratio between maximum and minimum radius (RR), average rate of vertical curvature (AVC), and ratio between individual curve radius and average radius (CRR). The statistical techniques used were Poisson, negative binomial, and lognormal regression. Three equations, Equation 3.4 through Equation 3.6, show the resulting models to evaluate the safety on individual curves.

$$Y = e^{-7.1977} * AADT^{0.9224} * CL^{0.8419} * e^{0.0662SR} \quad \text{Equation 3.4}$$

$$Y = e^{-0.8571} * MVKT * e^{0.078SR} \quad \text{Equation 3.5}$$

$$Y = e^{-5.932} * AADT^{0.8265} * CL^{0.7727} * e^{-0.3873CRR} \quad \text{Equation 3.6}$$

Where: Y = the number of collisions that occurred on the horizontal curve during a 3-year period; AADT = Annual Average Daily Traffic (veh/d); CL = horizontal curve length (km); MVKT = exposure (million veh-km of travel); SR = speed reduction on

horizontal curve from adjacent tangent or curve (km/h); and CRR = the ratio of individual horizontal curve radius to the average radius for a road section. All the predictors in the three models were found significant at 95 %confidence level. In order to measure the goodness of fit of the models, Anderson et al. (1999) found that the over-dispersion parameter based on the Pearson χ^2 values equal to 1.21, 1.344, and 1.296 for Equation 3.4, Equation 3.5, and Equation 3.6, respectively. Although these values are just outside the generally acceptable range of 0.8 to 1.2, Anderson et al. (1999) mentioned that this is often the case with regression relationships between geometric design elements and safety.

In addition, three more models were developed in the same research to evaluate safety of entire road sections. The three models included average radius of curvature "AR", ratio between maximum and minimum radius "RR", and average rate of vertical curvature "AVC" as predictors of the number of collisions. Anderson et al. (1999) used R^2 values to measure the goodness of fit for the three models indicated in Equation 3.7 through Equation 3.9.

$$Y = e^{-7.845} * AADT^{0.995} * L_{section}^{1.108} * e^{-0.000137 AR} \quad \text{Equation 3.7}$$

$$Y = e^{-7.859} * AADT^{0.988} * L_{section}^{1.058} * e^{0.0043 RR} \quad \text{Equation 3.8}$$

$$Y = e^{-8.297} * AADT^{1.052} * L_{section}^{1.167} * e^{-0.0028 AVC} \quad \text{Equation 3.9}$$

Where; L = segment length; AR = average radius (m); RR = ratio of maximum radius to minimum radius, and AVC = average rate of vertical curvature (m/%grade). They found that all the models are statistically significant at the 95 %confidence level with R^2 values equal to 0.6726, 0.6651, and 0.6880 for the three equations respectively.

Another study was done by Ng, and Sayed (2004) to quantify the safety benefits of each of the consistency measures. Data for 319 horizontal curves and 511 tangents located in Okanagan and Kootenay regions of the province of British Colombia were examined. They grouped the models into two main groups. The first group predicts the collisions in terms of individual consistency measures in order to investigate how design consistency is represented by individual measures relates to safety. The second group tended to include as many design consistency measures as are statistically significant in order to improve the prediction accuracy and develop quantitative relationships that can serve as evaluation tools to investigate road safety.

For the first group, six models, as shown in Table 3.2, were developed. The models revealed that the number of collisions is positively correlated to the difference between operating and design speeds ($V_{85}-V_d$) of the road section. In addition, the larger the drop in operating speed (ΔV_{85}) when moving from one section to another, the more the collisions that would be expected on that section. Moreover, increasing the curve radius relative to the average radius of the section (CRR) causes a significant reduction in collision frequency. Also, the higher the visual demand, the more the collisions that would be expected to occur. Finally, as the difference between assumed and demanded side friction (Δf_R) increases, the number of collisions decreases.

For the second group, as shown in Table 3.3, two models were developed. The first one is applicable for horizontal curves only. This model includes three design consistency measures ($V_{85}-V_d$, ΔV_{85} , and Δf_R). On the other hand, the second model is applicable to horizontal curves as well as tangents and includes only two design consistency measures (ΔV_{85} and Δf_R).

Table 3.2: Models Relating Consistency and Safety (Ng and Sayed, 2004).

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

Note: L , section length (km); V , annual average daily traffic (vehicles/d); $V_{85} - V_d$, difference between operating and design speed (km/h) of a single element; $\Delta V_{85} = |V_{85i} - V_{85i+1}|$ where V_{85i} and V_{85i+1} are operating speeds on elements i and $i + 1$, respectively; $\Delta f_R = f_R - f_{RD}$, where f_R and f_{RD} = side friction assumed and demanded on element i , respectively; CRR, ratio of radius of an individual element i to the average radius of the alignment; VD_{LU} , visual demand of unfamiliar drivers; VD_{LF} , visual demand of familiar drivers.

Table 3.3: Models for Evaluating Overall Consistency (Ng and Sayed, 2004).

No.	Model form		t-ratio	κ	Pearson χ^2 (χ^2 test) Standard deviation
7a*	Accidents per 5 years = $\exp(-3.369) L^{0.8858} V^{0.5841}$ $\times \exp[0.0049(V_{g5} - V_d) + 0.0253\Delta V_{g5} - 1.177\Delta f_R]$	a_0	-1.894	1.734	248.66
		a_1	6.480		(355.26)
		a_2	2.749		339.26
		a_3	2.085		
		a_4	2.022		
		a_5	-1.932*		
7b†	Accidents per 5 years = $\exp(-2.338) L^{1.092} V^{0.4629}$ $\times \exp[IC (0.022\Delta V_{g5} - 1.189\Delta f_R)]$ where IC = dummy variable (0 for tangents and 1 for horizontal curves)	a_0	-2.25	2.511	828.68
		a_1	22.46		(889.81)
		a_2	3.81		841.06
		a_3	2.06		
		a_4	-1.64‡		

* developed based on horizontal curve data only.

† developed based on horizontal curve and tangent data.

‡ Statistically significant at 10% significance level.

It is noteworthy to mention that TAC (1999) design guide included the effect of the speed reduction on safety into the design process. It was mentioned that vehicles travelling with speed much higher or lower than the average speed would be expected to have a higher probability of involving in a collision. Figure 3.2 indicates the relationship between speed reduction from one segment to another along the highway and mean collision rate as stated by TAC (1999). It was recommended to deal numerically with this figure with caution as designers should use local data (TAC 1999). Rather, the chart establishes a trend for the effect of inconsistency on potential for collisions.

The most recent study that evaluated the geometric design consistency from an explicit safety point of view is that by Awatta (2003). She examined four consistency measures; namely: operating speed, vehicle stability, alignment indices, and driver

workload. The purpose of her research was to quantify the relationship between the previous mentioned consistency measures and actual number of collisions through regression analysis. It should be noted that the study was limited to horizontal curves of two-lane rural highways. Her study included testing 70 curves of seven two-lane highways with applying both Poisson and negative binomial techniques. The study showed that all design consistency measures, except the design speed, provided good relationship with safety performance.

Figure 3.2: Mean Collision Rate versus Mean Speed Difference between Geometric Elements (TAC, 1999: 1.4.3.5).

Moreover, using sensitivity analysis, Awatta (2003) ranked the consistency measures in terms of the strength of their relationship to safety. It was found that the difference in the 85th percentile speeds between each two successive highway elements

(ΔV_{85}) had the strongest relationship with safety, followed by CRR, visual demand, ΔV_{85} , and finally the vehicle stability.

3.1.5 Other Factors

3.1.5.a Signs

One of the factors that might strongly contribute to the performance of hazardous manoeuvres on freeways is bad sign locations and arrangement. Dutta et al. (2002) assessed the effect of overhead sign design on the driver behaviour during weaving manoeuvres. They evaluated the safety effect of the overhead exit signs located on freeways around Logan International Airport in Boston, Massachusetts. They mentioned that bad design for exit signs may lead to either missing the exit and wasting time moving around the city or making a decision at the last moment to change lanes leading to safety problems during the manoeuvres. The existing signs had non-centered positions over the lanes. In addition, the down arrows in the signs did not point directly to the relevant lane centerline. Dutta et al. (2002) used a simulator to simulate the performance of drivers with four different types of overhead signs. Generally, Dutta et al. (2002) mentioned that a better performance can be achieved on weaving sections by using more clear and understandable signs.

3.1.5.b Highway Marking

Highway marking is considered also a significant factor controlling the vehicle's speeds and hence safety. Retting et al. (2000) discussed the effect of marking on freeway exit speeds. They studied the effect of narrowing the lane width of the curve and a portion of the tangent section leading into the curve by using a gradual inward taper of the existing edge line. Speeds were measured six weeks before and two weeks after installing the pavement marking that narrowed the lane width in the ramp approach. Retting (2000) measured the changes that took place in the mean speed, 90th percentile speeds, and percentage of passenger vehicles exceeding posted advisory speeds by more than a specific value (16.1 km/h and 8 km/h). Installing the marking was found to result in a drop in the proportion of passenger vehicles travelling more than 16.1 km/h above the posted speed by 6 %, 13 %, and 17 % for three sites out of the four sites examined in this research while the fourth site was not observed. In addition, the proportion of large trucks that was found exceeding the posted speeds by 8 km/h decreased by 22 %, 17 %, and 19 % for three out of the four sites after installing the marking. This brief discussion shows the significant effect of the marking in reducing vehicles speeds.

3.2 MERGE AND DIVERGE AREAS

A merge or diverge area, as discussed in Chapter 2, is the area that follows an on-ramp or precedes an off-ramp, respectively. There are two types of analysis found in the previous research at these areas; (1) implicit safety analysis that considers the indirect effect of design on safety and is presented in Section 3.2.1; (2) explicit safety analysis

that evaluates the direct improvement of safety in the analysis methodology and is presented in Section 3.2.2.

3.2.1 Implicit Safety Analysis

The most recent and widely used methodology for analyzing merge and diverge areas is provided by HCM (2000). The methodology is based mainly on mitigating the turbulence in the mainline traffic caused by the entering and exiting volumes. It was mentioned that the operational effects due merge and diverge manoeuvres are heaviest in the first two lanes adjacent to the speed change lane besides the speed change lane itself. The effect extends 450 m downstream the physical gore in the merge areas and 450 m upstream the physical gore of the diverge areas. In both situations, vehicles approaching from the mainline attempt to move away from the lanes adjacent to the ramps as long as the capacity permits to do so.

According to TAC (2000), there is no evidence so far that merge and diverge manoeuvres affect the capacity of the freeway mainline. The only effect is adding or subtracting demand to the freeway. The capacity of the merge area is the same as the capacity downstream the merge area (C_1 in Figure 3.3) as if the segment was a freeway basic segment. Besides, the capacity of the freeway at the diverge area is either the capacity of the upstream basic freeway segment (C_1 in Figure 3.4) or the capacity of the downstream basic freeway segment and the ramp itself (C_2, C_3 in Figure 3.4). If the demand of exit volumes through the off-ramp exceeds the capacity, a breakdown is most likely to occur. On the other hand, HCM (2000) stated that the flow rate occupying the

first two lanes adjacent to the speed change lane (C_2 in Figure 3.3) cannot exceed 4600 pc/h for on-ramps. In addition, for off-ramps, the flow occupying the first two lanes including the off-ramp flow (C_4 in Figure 3.4) cannot exceed 4400 pc/h.

Figure 3.3: Capacity at Merge Areas (HCM 2000).

Figure 3.4: Capacity at Diverge Areas (HCM 2000).

3.2.2 Explicit Safety Analysis

Investigating the relationship between safety and geometric elements at merge and diverge areas has been an active area of research since 1965. Lundy (1965) studied 722 freeway ramps with a total volume of two billion vehicles moving over these ramps. It was found that 32 % from the 722 ramps have experienced zero collisions while a total of 1643 collisions were included in the study for the rest of the ramps. The 722 ramps

have experienced 18 % of the collisions on the freeways they serve. The purpose of Lundy's study was to classify the geometric features by ramp type and investigate the relationship between each type and safety. It was concluded finally that there is no statistically significant difference between ramps that experienced zero collisions and those that experienced collisions for each ramp type. It was also mentioned that off-ramps were associated with higher collision rates than on-ramps. He recommended using an acceleration lane length of 243 m (800 ft) or greater and a deceleration lane length of 274 m (900 ft) in order to reduce collision frequencies. Two drawbacks are evident in this study. First, collisions were assumed to be related to traffic volumes only, regardless of the ramp length. Second, using the collision rate has recently been considered unsatisfactory since it is based on the false assumption of linear relationship between number of collisions and traffic volumes.

Later, Cirillo (1970) investigated the relationship between the geometry of the interstate highway system and safety. The study included the effect of weaving length, acceleration lanes, and deceleration lanes on safety performance. It was concluded that increasing the weaving length would generally help reduce collision rates. For merging and diverging vehicles greater than 6 % of the mainline volumes, increasing the length of both acceleration and deceleration lanes was also found to reduce the collision rates. However, the sensitivity of safety improvement to the increase of the length of acceleration lanes was higher than the sensitivity to the increase of the length of deceleration lanes.

Michaels and Fazio (1989) studied the gap acceptance concept on 120 merge locations to investigate the relationship between acceleration lane length and probability

of finding a gap within the moving vehicles on the freeway section. They found a need to increase the length of the acceleration lane with decreasing the ramp speed and with increasing the entering traffic volume. Using an acceleration length of 243 m (800 ft) was recommended to allow drivers find acceptable gaps 85 % of the time. Generally, this length of 243 m was suggested to be sufficient for freeway volumes over 1,200 pc/ln/h (passenger cars per lane per hour) and ramp design speeds over 48 km/h (30 mph).

Fazio et al. (1993) suggested using a new term to assess the safety performance of weaving sections referred to as traffic conflict rate. Two types of traffic conflicts on freeways were defined: (1) rear-end conflict which happens when a vehicle slows or stops on a freeway lane and the immediately following vehicles apply the break to avoid a collision, and (2) lane change conflict in which a vehicle changes lanes and the immediately following vehicle reacts to avoid a collision. The main two objectives for this study were to develop a tool to determine the conflict rate for an existing or proposed freeway, and to discover the relationship between conflict rate and crash rate and the validity to use the first as safety indicator. Ten weaving sites were selected on Interstate I-294, the Tri-State Tollway, to achieve the study objectives, where the INTRAS program distributed by the Federal Highway Administration (FHWA), under its technology transfer program, was used to obtain the conflict rate. In addition, rear-end and angle/sideswipe collisions were extracted to represent the collisions due to the two conflict types. For both conflict types, the study considered only conflict rates that occurred while the immediately following vehicle had a deceleration rate greater than or equal to 0.61 m/sec^2 . A correlation of +0.74 was found between lane change conflict rates and reported angle/sideswipe crash rates, and a +0.95 correlation coefficient was found

between rear-end conflict rate and rear-end crash rates. It was therefore concluded that "conflict rate enables highway engineers to obtain an operation evaluation as well as safety evaluation for alternatives to determine which design is less hazardous" (Fazio et al. 1993).

Hunter et al. (2001) evaluated the ramp design speed criteria by observing six (non-loop) ramps in three Texas cities. In addition to the accepted time gap sizes and the freeway time headways, ramp and freeway speeds were used to describe the traffic operations at merge locations. It was mentioned that drivers were observed not to start accelerating until they obtain a clear view of the freeway. Therefore, it was recommended that the AASHTO acceleration length model should be revised to include only the lane portion from which the driver can see vehicles in the freeway right lane. Finally, it was mentioned that a good ramp design would produce relatively low acceleration rates ranging from a zero value to 3.2 km/h/s, while a bad ramp design would produce higher acceleration rates ranged from zero to 6.4 km/h/s.

Bauer and Harwood (1997) focused on the safety performance of freeway interchanges. The objectives of this study were to develop statistical models to investigate the relationships between traffic collisions, highway geometric elements, and traffic volumes for interchange ramps and speed-change lanes. The data used in this study included geometric design and traffic volume data for ramps and speed-change lanes in the State of Washington. Due to the lack of necessary data to obtain the speed change lane length as mentioned in AASHTO, the taper length was included with the speed change lane length used in the analysis. Regression models were then developed using negative binomial technique and explained between 10 % and 42 % of the variability in

the collision data. This relatively low goodness of fit was interpreted by the fact that between 50 % and 80 % of the ramps experienced no collisions or only one collision in the 3-year study period. It was also noted that modelling the total number of collisions that occurred on the ramp as well as the adjacent speed change lane gave better results than disaggregating the collisions for the two elements.

Khan et al. (1999) investigated the relationship between fatal, injury, and PDO collisions and traffic volumes, segment length, and vehicle miles travelled. Highway geometric data, traffic characteristics, and ten years of collision data were collected for 258 km of Interstate I-25 between Denver and Pueblo. A number of statistical methods were used including linear regression, Poisson, negative binomial, log-normal distributions to achieve optimum prediction level. Aggregate collisions data were used as well as disaggregate data including a set of variable combinations including urban or rural, higher or lower traffic volumes, long or short segment, and high or low vehicle-miles-travelled. The resulting models of this study revealed a generally good performance of Poisson regression models in predicting collisions based on Pearson's χ^2 statistics. By disaggregating the collisions, the injury and PDO collisions mostly followed the Poisson distribution while the fatal collisions followed the log-normal distribution. In addition, Khan et al. (1999) mentioned that segment length and traffic volumes were found to be more significant variables for injury and PDO collisions than they are for fatal collisions. They interpreted that observation by assuming a less severe collision for short segments with high volumes.

3.3 WEAVING SECTIONS

Similar to the two analysis types that were found in the literature for merge and diverge areas, several other research works have addressed the performance of weaving segments. It is worth to remind that the weaving segments are those between an on-ramp and an off-ramp. In this type, a merge area is directly followed by a diverge area. Section, 3.3.1 discusses the implicit safety analysis while the explicit safety analysis will be covered in Section 3.3.2.

3.3.1 Implicit Safety Analysis

It was mentioned in AASHTO (2001) that weaving section should be designed to achieve a constant level of service within the entire highway. The level of service of the weaving section depends mainly on the segment length, number of lanes, congestion degree, and traffic volumes for individual movement moving over this segment. The traditional and most commonly used method in analyzing weaving section is the one stated in the HCM (2000). The method contains models for predicting the space mean speed for both weaving and non-weaving volumes within the weaving section. As mentioned before, the weaving volume is the portion of total volume in the weaving section that has to change their lanes in order to reach their destination, while the rest of the volume is called non-weaving volume. The space mean speeds are then used to calculate the portion of the section lanes which is occupied by the weaving volumes. If this required portion of a given number of lanes exceeds a specific maximum value provided, the weaving vehicles are constrained to use the maximum value. Subsequently,

the weaving vehicles cannot occupy as much of the roadway as would be needed to establish equilibrium operations between weaving and non-weaving traffic. This case is then called a constrained section. On the contrary, if the number of lanes needed by the weaving volumes was less than the maximum values the section is called unconstrained section.

The space mean speed for the overall volume in the weaving section is then calculated in terms of both weaving and non-weaving space mean speed as well as their volumes. Finally, the density is calculated as a function of the overall speed and total volumes. One can evaluate the freeway segment through the level of service (LOS) according to the previous calculated density which takes a maximum value of 27 pc/km/ln. Again, for this methodology to be valid a maximum length of 750 m is specified for all types of weaving sections, while merging and diverging section would be analyzed separately for longer sections.

Fitzpatrick and Nowlin (1996) studied the weaving operations on one-way frontage roads using six field sites ranging from three to four lanes and approximately from 137 m to 335 m in length. The relationship developed between the speed on weaving sections and weaving length is shown in Figure 3.5. The figure indicates that the slope change of the relationship decreases with increasing weaving lengths, and the 300 m weaving length was estimated as the point after which increasing the weaving length does not greatly affect the speeds. Another recommendation was to avoid using weaving lengths less than 200 m.

Figure 3.5: Relationship between Speed and Weaving Length (Fitzpatrick, 1996: 49)

Kwon et al. (2000) studied the relationship between speeds and traffic volumes for weaving sections. The study focused on simple ramp-weave areas defined as on-ramp followed by an off-ramp with the two ramps connected by an auxiliary lane. A total of 226 areas were identified as weaving sections including 119 classified as simple ramp-weave areas located in Minneapolis-St. Paul, Minnesota. However, no clear relationship between speed and weaving flow rate was found in the data. The explanation given was that free merging-diverging speeds range from 48-64 km/h and a wide range of weaving volumes can be observed within that speed range. In addition, they mentioned that at free flow conditions, the diverge speed is mainly function of the geometric conditions of the exit ramp. The main conclusion from their study was that the maximum possible weaving volume in a simple ramp-weave section is equal to the maximum through volume that the

auxiliary lane can carry, which emphasized the HCM (2000) that recommended a maximum weaving flow rate of 2800 pc/h for *type A* weaving sections.

Fredericksen and Ogden (1994) proposed a new analytical technique for analyzing slow-speed weaving sections *Type A*, while focusing on frontage road facilities with ramps on the left side. The term Lane Change Intensity "*LCI*" was used as an indication of traffic disturbance on the weaving sections. *LCI* was defined as the number of lane changes per hour per lane per mile and can be calculated from Equation 3.10.

$$LCI = \frac{\text{number of lane changes per hour}}{(\text{number of lanes})(\text{length of weaving section})} \quad \text{Equation 3.10}$$

The data collected for this study were grouped into three categories: (1) weaving section lengths from 122 to 183 m (400 to 599 ft), (2) weaving section lengths from 183 m to 274 m (600 to 899 ft), and (3) weaving section lengths from 274 to 366 m (900 to 1200 ft). The resulting models from this study were:

$$\text{Group(1)} : LCI = 10.46(V / n) + 372 \dots \dots R^2 = 0.94 \quad \text{Equation 3.11}$$

$$\text{Group(2)} : LCI = 8.52(V / n) + 79 \dots \dots R^2 = 0.78 \quad \text{Equation 3.12}$$

$$\text{Group(3)} : LCI = 391(V / n) + 590 \dots \dots R^2 = 0.82 \quad \text{Equation 3.13}$$

Where;

LCI = Lane changes per hour per lane per mile

V = Hourly volume entering weaving section, and

n = Number of lanes in weaving sections

Based on this analysis, a weaving section length ranging from 274 to 366 m (900 to 1200 ft) was recommended to help ensure that weaving problems are minimized. Also, it was mentioned that weaving sections less than 600 ft will most likely experience operational problems. Finally, a recommendation of three lanes, two for through traffic and one auxiliary lane connecting the two ramps, was also mentioned. It should be noted, however, that one of the drawbacks of the models developed in this study is using the total volume to predict *LCI*. Although R^2 values were reasonably acceptable, weaving manoeuvres are obviously a function of the composition of the traffic within the weaving section.

Westland (1998) conducted a study on the effect of highway geometry on freeway queuing at merge sections using a data set from three merge locations on freeways near Amsterdam and Rotterdam. The Geometric Factor defined as the ratio of the total number of exiting traffic lanes to the total number of incoming traffic lanes for a given section was used as indication of queuing characteristics. It was mentioned that the main cause of queuing is the lane drop following a merge where the on-ramp joins the mainline or when two main freeways merge. The common problem was that traffic demand downstream of this merge is always greater than the capacity. It was also mentioned that the section will function as a geometric bottleneck if the geometric factor is less than one. Therefore, the number of lanes downstream was recommended to be at least equal to the number of lanes upstream to avoid having bottleneck problems.

In further explanation of the effect of the traffic conflict on the drivers' behaviour and hence on the safety, a study was carried out by Park et al. (2001) to investigate the drivers' characteristics in the perception of a lead vehicle's deceleration using film clips

of two cars following each other in a car-following situation. One of the main conclusions of this study that can help researchers interpret the cause of collisions in weaving sections is that drivers generally underestimate the lead vehicle's deceleration level. This underestimation increases with increasing the decelerating level. The relationship between the actual and perceived deceleration level is shown in Figure 3.6. As shown in this figure, the relationship between perceived and actual deceleration levels was a straight line with a coefficient of determination of 0.997 and generally below the equality line.

Sarvi et al. (2002) performed a study on modelling vehicle acceleration-deceleration behaviour during freeway and ramp merging manoeuvres while focusing on the manoeuvres under congested traffic situation. The objective of the study was to design safer and less congested merging points as well as to control efficiently the bottleneck sections. The models developed in this study could predict ramp vehicles acceleration-deceleration behaviour as a function of the following parameters:

1. Location of ramp vehicle, freeway lead vehicle, and freeway lag vehicle.
2. Speed of ramp vehicle, freeway lead vehicle, and freeway lag vehicle.
3. Desired spacing between vehicles.

Figure 3.6: Relations between Perceived and Actual Deceleration Level by Driving Experience (Park et al., 2001).

It was mentioned that, generally, searching for gaps to do acceptable manoeuvres for merging does not occur under heavy traffic conditions. For this congested situation, a squeeze merging can be observed at the end of the merging section in a way referred to as *zip merging*. Finally, no relationship was found between acceleration lane length and maximum flow rate of merging sections.

Later, Sarvi et al. (2003) studied the effect of geometric characteristics on the merging capacity. The conversion angle of entrance ramps, defined as the angle between the ramp tangent and freeway, was used as a significant factor that affects merging

capacity. Increasing the conversion angle to 10 degrees was found to increase the merging capacity to 1925 veh/h/lane, but any further increase causes negative effect in the merging capacity. In addition, a constant value of merging capacity with conversion angles greater than 40 degree has been observed.

Kou and Machemehl (1997) presented a methodology to model ramp vehicle acceleration-deceleration behaviour at freeway on-ramps by capturing data using a videotape and manual image processing. The two main types of entrance ramps, parallel and tapered, were considered in this study. A main conclusion was that, ramp vehicle merging position is not significantly related to any traffic parameter included in the study such as traffic flow volumes, ramp vehicle approach speeds, relative speeds to freeway lag vehicle. The reason behind the insignificant effect of the traffic parameter was interpreted by Kou and Machemehl (1997) as insufficient data. They recommended developing their models using more freeway merge data from different sites.

Ostrom et al. (1993) suggested a procedure, named total point flow, for analyzing freeway weaving sections as most speed-oriented methodologies were suggested to be unsatisfactory in describing freeway weaving section behaviour. The total point flow method attempted to predict the flows at a set of points within the weaving section. The points at which the analysis was done are 76.2 m (250 ft) downstream of the merge and 152.4 m (500 ft) increments from the merge to the end of the section. Thus, this method should give an idea about the distribution of movements and lane changing within the analysis area. The proposed models used the following equation:

$$Flow = \theta_0 + \theta_1 * FF + \theta_2 * FR + \theta_3 * RF + \theta_4 * RR \quad \text{Equation 3.14}$$

Where

$\theta_i \equiv$ Coefficients

$FF \equiv$ Freeway-to-Freeway movement,

$FR \equiv$ Freeway-to-off-ramp movement,

$RF \equiv$ on-ramp-to-Freeway movement, and

$RR \equiv$ on-ramp-to-off-ramp movement,

It should be noted that the study included only simple weave in which one on-ramp and one off-ramp are connected by one or more auxiliary lane and isolated from the influence of other ramps. However, the study covered both types of simple weave, namely ramp weave in which one-lane on-ramp is connected to one-lane off-ramp by an auxiliary lane and major weave in which an on-ramp is connected to an off-ramp by one or more auxiliary lanes and at least one of the ramps has two lanes and the other ramp has one or two lanes. The proposed method was then validated and compared with the traditional speed-based analysis methods. Using the proposed method, approximately 70 % of the volumes were predicted within 10 % of the actual volumes.

3.3.2 Explicit Safety Analysis

As mentioned by Bared et al. (1999), deceleration lanes were shown to exhibit higher collision rates than acceleration lanes. The purpose of this research was to provide the highway planner or designer a tool to assess the benefits of improving safety of interchanges. Based on the information documented by Bauer et al. (1997), collision

frequencies for entire ramp and speed change lanes were modelled as shown in Equation 3.15.

$$N = RAADT^{0.78} FAADT^{0.13} e^{(-7.27+0.45DIA+0.78PAR-0.02FF+0.69OC-0.37RUR+0.37DECEL-2.59SCLN+1.62RLEN)} \quad \text{Equation 3.15}$$

Where;

- N = Expected number of total collisions in a three-year period on entire ramp combined with speed-change lane
- RAADT = Ramp AADT (veh/day)
- FAADT = Mainline freeway AADT for the direction of travel in which the ramp is located
- DIA = 1 if the ramp is diamond ramp and zero otherwise
- PAR = 1 if the ramp is parclo loop ramp and zero otherwise
- FF = 1 if the ramp is free-flow ramp and zero otherwise
- OC = 1 if the ramp is outer connection ramp and zero otherwise
- RUR = 1 if the area type is rural and zero otherwise
- DECEL = 1 if the area is an off-ramp and zero otherwise
- SCLN = Speed-change lane length (miles)
- RLEN = Ramp length (in miles)

These factors were found to predict collision frequencies with coefficient of determination $R^2 = 0.43$. Finally, the length of speed-change lanes was recommended to be increased beyond the critical lengths in the AASHTO design guide. This was particularly true for the deceleration lanes, which were found to have higher collision frequencies.

Another study by Mountain et al. (1996) had a main objective of developing and validating a method for predicting expected collisions on a main road with minor

junctions using empirical bayes method. The operational conditions on freeways are not the same to those on the roads with minor or major junctions, but the researchers focused generally on the weaving effect on safety. Based on the developed non-linear relationship between collision frequency and traffic flow, the main conclusion from this study was that using collision rates can be misleading when used in comparing the safety levels on different locations. In addition, Mountain et al. (1996) investigated the relationship between the mean number of minor junctions and collision frequency. Figure 3.7 shows the general trend of the relationship between the traffic conflicts due to weaving manoeuvres, presented as the number of minor junctions, and observed mean annual collisions per kilometre. Obviously, increasing the number of junctions resulted in short distances for vehicle manoeuvres. The situation would be worst if the traffic volumes approach capacity with stop-and-go operating conditions.

Figure 3.7: Variation of Annual Collisions with the Number of Minor Junctions per Kilometre (Mountain et al., 1996).

A similar relationship was mentioned in AASHTO (2002) to that developed by Mountain et al. (1996) as shown in Figure 3.8. The relationship between total access points per kilometre and collision rate has a slope that decreases with increasing the number of minor junctions, which is more realistic to explore the effect of minor junctions on the collision frequencies.

Figure 3.8: Effect of the Access Points per Kilometre on Collision Frequency (AASHTO, 2001: 93).

Another research was carried out by Mouskos et al. (1999) to discuss the effect of access density on traffic collisions on New Jersey state highways, where access density was defined as the number of access points per kilometer on a road section in each direction. The effect of geometric and environmental factors was taken in consideration in addition to the traffic effect on collision rates. The access-related collisions were

extracted from the data by counting all collisions involving two vehicles where one of them is either in left turn action or right turn action. Collisions reported at intersections while crossing the road segment were eliminated. It was found that 7 % from the collisions were due to manoeuvring to and from access points, and subsequently this 7% reflected the weaving effect on highway safety.

4. DATA COLLECTION AND PREPARATION

After identifying the goals of this study, and based on the literature review, data were acquired to achieve these goals. The data collected included the highway geometric data for the selected study area as well as traffic and collision data. First, Section 4.1 describes the selected study area and the rationale for this selection. Then, Section 4.2 presents the maps used to extract the needed geometric data, while Section 4.3 discusses the traffic data. Finally, section 4.4 presents the collision data.

4.1 SITE SELECTION

As mentioned previously in Section 1.3, the study area was selected to be the portion of Highway 417 from Interchange 145 west (intersection with Highway 7) to Interchange 110 east (intersects with Walkley Road). The total number of interchanges in the selected study area is 26 interchanges with 48 exits and 65 entrances in both directions. Highway 417 within the study area is serving provincial inter-city travel as well as commuter traffic. In addition, traffic volumes are high, and already exceed the theoretical design capacity during several hours of the day (MTO 2004). This increase in the traffic volumes would normally result in increasing delays and collisions.

One of the main reasons for selecting the candidate area of study is due the variation of land use along the route of Highway 417. Figure 4.1 shows the selected study area which is characterized as urbanized commercial and industrial at the eastern and

western part while the central part can be characterized more as an urban area. More details about these interchanges are shown in Table 4.1.

Figure 4.1: The Selected Study Area of Highway 417.

The nature of the central part with restricted right-of-way and larger density of interchanges forced the designers to reduce the spacing between each two successive ramp terminals in order to properly serve the core of the city. However, there was no such restriction on the spacing in the eastern and western part. This obvious difference in the interchange spacing which accompanied the variety of the land use should help reveal any existing trends for the variation of safety performance with the variation of interchange spacing.

Table 4.1: Summary of Interchanges under Study.

Interchange Number	Intersecting Road	Total Number of	
		Exits	Entrances
IC145	Highway 17	2	2
IC144	Carp Road	2	3
IC142	Palladium Road	2	3
IC140	Terry Fox Drive	2	4
IC138	Eagleson Road	2	4
IC134	Moodie Drive	2	4
IC132	Highway 416	3	2
IC130	Richmond Road	2	4
IC129	Pinecrest Road	2	4
IC127	Woodroffe Avenue	3	2
IC126	Maitland Avenue	2	2
IC124	Kirkwood Avenue	2	3
IC123	Island Park Road	1	0
IC122	Parkdale Avenue	2	2
IC121B	Rochester Street	1	1
IC121A	Bronson Avenue	2	1
IC120B	Lyon Street	0	1
IC120A	Kent Street	1	0
IC119B	O'Connor Street	1	1
IC119A	Metcalf Street	1	1
IC118	Nicholas Street	2	3
IC117	Riverside Drive	2	4
IC115	St. Laurent Boulevard	2	4
IC113	Highway 174	3	2
IC112	Innes Road	2	4
IC110	Walkley Road	2	4

4.2 GEOMETRIC DATA

The geometric data that were required for the fulfillment of this research included lengths of all segments and speed change lanes within the study area. Subsequently, the first step was to seek the maps that cover the selected part of Highway 417. Two map formats were found; (1) digitized maps available in the Carleton University's library; and (2) scanned images of *Engineering and Title Records* (ETR) for the projects carried out on Highway 417 provided by Ontario Ministry of Transportation (MTO). Section 4.2.1 and 4.2.2 discuss the format of the obtained maps. Then, Section 4.2.3 illustrates the site investigation carried out to validate the obtained maps. Finally, Section 4.2.4 describes the procedures that were followed to obtain the required measurements.

4.2.1 Digitized Maps

Digital maps for the City of Ottawa were obtained from the Maps, Data and Government Information Centre (MADGIC) at the Carleton University library. The maps were available in AutoCAD format digitized to scale 1:2000. This digitization scale means that any error in digitization of 1 millimetre can cause a difference in the actual distance of 2 meters. The acceptance of this precision depends mainly on the extracted lengths. For example, determining the width or number of lanes of the highway using this kind of maps was rejected. However, the precision of the obtained spacing between ramp terminals was considered acceptable since the lengths ranged from 160 m to 5,116 m.

The entire collection of the available maps, which is approximately 420 maps, covers the entire City of Ottawa as shown in Figure 4.2, where each small rectangle in

this figure represents one map. The maps that cover Highway 417 from IC 142 west (intersection with Palladium Drive) to IC 110 east (intersection with Walkley Road) are 46 maps. Each map covers an area that is two kilometres horizontally by one kilometre vertically. A sample of the maps is shown in Figure 4.3 covering IC 140.

Figure 4.2: AutoCAD Maps Covering City of Ottawa.

Figure 4.3: A Sample of AutoCAD Maps for IC 140.

4.2.2 MTO Engineering and Title Records

Four townships were identified to cover the portion of Highway 417 selected for the study. As shown in Figure 4.4 these four townships are: Huntley, March, Nepean, and Gloucester. Accordingly, ETR drawings that cover these four townships were requested from MTO. A total of 261 maps were then provided as "TIF" images. The maps included plans, profiles, and cross section data. A total of 82 maps were found to contain the required data and were therefore isolated and printed. The maps were assumed to be more accurate than the digitized ones. They contained stations with continuity through two townships of Nepean and Ottawa, while Gloucester Township contained only one map for IC 112 (intersection with Innes Road) with a different stationing system. However, the two maps covered Huntley were found without stations. Table 4.2 shows more details about the obtained maps.

Figure 4.4: Four Townships Covering the Selected Area of Study.

Table 4.2: Summary of Maps Obtained from Ontario Ministry of Transportation.

Township	Number of maps	Stations	
		From	To
Huntley	2	NA	NA
Nepean	28	10+125	17+650
Ottawa	51	17+650	32+500
Gloucester	1	12+600	13+600

Along with the maps shown in Table 4.2, there were another 4,957 maps for all projects that were carried out in the selected study area since 1973. A total of 118 maps out of these 4,957 were found covering the study area and were then isolated. Some of the projects were pavement reconstruction of existing lanes with adding new lanes or extending speed change lanes. In addition, few projects involved a completely new design for few interchanges. Specifically, IC 132 (intersection with Highway 416) was redesigned in 1990; IC 117 (intersection with Riverside Drive) experienced shifting of

few ramps in 1987; and for IC 129 (intersection with Pinecrest Drive) the ramp serving traffic coming from east and going to south was closed in 1990. Figure 4.5 shows the entire collection of the projects covering the study area with the project number written beside each one. Data were extracted from the projects' maps while portions of the highway that were not found under any projects were extracted from the original maps.

Figure 4.5: Projects Carried out in the Study Area Since 1973.

4.2.3 Site Investigation

Visiting the area of study and comparing it with the obtained maps was necessary to validate the maps and ensure that they were up to date. A video clip was taken for the area of study for both directions east and west to create a permanent log for the current state of the highway. The maps were then compared to the video clip to ensure an updated design. All maps were found to be up to date with the current geometric

characteristics of the highway. The video clip was also used to obtain the number of lanes of the mainline as well as the number of lanes of all ramps. In addition, advisory speeds posted on all exits for both directions were also gathered in order to evaluate the effect of the operating conditions at the freeway exits on safety performance.

4.2.4 Length Measurements

The distance between each two successive ramp terminals was measured from the printed MTO maps. It was found that some of the maps were available with the edge of the pavement and lane marking (see Figure 4.6), and were perfect for extracting the length. However, other maps were provided with only the general pavement boundary (see Figure 4.7). Therefore, an approximation in identifying the start and end of the segment was necessary to obtain the segment length as well as the length of speed change lanes. In the cases that road marking is provided, measuring a segment length was from the point that the pavement edges of the ramp and mainline are 1.25 m apart, which is almost the typical value in TAC (1999) to the similar point of the next ramp. According to AASHTO (2001) design guide, the criterion is still acceptable as the recommendation is that start measuring from the point that the edges are from 0.6 m to 3.0 m.

Figure 4.6: A Sample of MTO Map for IC 121B: With Marking.

Figure 4.7: A Sample of MTO Map for IC 118: Without Marking.

The AutoCAD files were used in extracting eight lengths within and between IC 134 (intersection with Moodie Drive) and IC 138 (intersection with Eagleson Road). Although the lengths obtained from the AutoCAD files may not be as accurate as those obtained from MTO maps, it was considered acceptable for segment length measurement as mentioned before. In order to measure distances from the AutoCAD files, two steps were used as follow:

1. The 50 maps covering the study area were merged in one AutoCAD file.
2. An arbitrary stationing system was implemented for the centreline of the two directions of the highway using *Softdesk* software.

With these two steps complete, it became easier to determine the station of each exit and entrance and determine the relative distance between each two successive access points for both directions. Figure 4.8 shows a sample of the stationing system used in order to measure the distances. As mentioned earlier, the precision of the measured distance is equal to the precision of the digitized map (1:2000). This relatively low precision should be acceptable for measuring the distance between ramp terminals since the distances are relatively long. However, getting the number of lanes or widths was determined from MTO maps and confirmed by the site visit.

Figure 4.8: Arbitrary Stationing System for Highway 417.

4.3 TRAFFIC DATA

The traffic volumes for Highway 417 were obtained from the MTO traffic volume database. The obtained data included traffic counts for the mainline as well as traffic counts for all ramps forming the selected interchanges within the study area. The data, however, did not have the same format for the entire study area. Part of the data was formatted and presented as a map on which all traffic volumes are shown, while the other part was obtained in a tabular format. All the obtained data were prepared and converted to present equivalent passenger car counts. The following is a more detailed description for the data obtained.

4.3.1 From IC 132 to IC 145

For the first part, data from IC 132 east (intersection with Highway 416) to IC 145 west (intersection with Highway 7) were available in a map format that include all traffic volumes for both the mainline and interchange ramps. A sample of the obtained data is shown in Figure 4.9. The volumes shown in this map correspond to the peak hour volumes. It was noted that some of the freeway segments within IC 132 (intersection with Highway 416) were provided with both peak hour volumes and AADT. Due to the lack of needed information to convert from peak hour volumes to AADT, volumes on IC 132 were used to calculate a conversion factor to convert from peak hour volume to AADT.

Figure 4.9: Sample of Traffic Volumes Data in Map Format.

Based on this assumption, the peak hour volume was found equal to 9.5 % of the AADT. The assumed conversion factor was checked against the typical values mentioned in the HCM (2000) and was found close to the typical default value of 9.3 % for urban roads. Finally, entering and exiting traffic volumes were extracted and converted for all freeway segments within the candidate study area.

4.3.2 From IC 110 to IC 132

The second part included traffic volume data from IC 110 east (intersection with Walkley Road) to IC 132 west (intersection with Highway 416). The data were obtained in two separate tables. One of the two tables included the traffic volumes of the interchange ramps as shown in Table 4.3. Fields that were used to describe the traffic counts in this file are:

1. *Interchange name* in which the interchanges are named by the intersecting road.
2. *Interchange number* as indicated previously in Table 4.1.
3. *Ramp direction* to indicate which ramp is specified by this volume. Each record in this field consists of two directions. The first indicates the direction from which the traffic is coming and the other one indicates the direction to which the traffic is going (*e.g.* W-S means a ramp serving traffic coming from west and going to south)
4. *Count Duration* which shows the number of days used to perform this counting process, where a period of three or seven days was used to count the traffic.

5. *AWD* "Average Weekday Traffic" field contains an estimate for the average traffic volume in equivalent passenger car units on a typical weekday.

Table 4.3: Sample Traffic Volume Data in Tabular Format: Volumes on Ramps.

I/C Name	I/C #	Ramp	Count Duration	AWD	ADT*	Average Weekday AM Peak	Average Weekday PM Peak	AADT**
Nicholas St.	118	W-N	7 days	9,829	9,329	807	719	8,750
Lees Ave.	118	W-E/W	7 days	5,544	5,055	416	599	4,950
Nicholas St.	118	N-E	7 days	11,180	10,203	690	990	9,950
Lees Ave.	118	E/W-E	7 days	12,282	12,573	703	971	10,950
Nicholas St.	118	E-N	7 days	13,882	13,485	1,196	1,005	12,350
Lees Ave.	118	E-N/S	7 days	7,211	7,108	443	618	6,400
Lees Ave.	118	N/S-W	7 days	7,452	7,080	488	608	6,650
Greenfield Ave.	118	N/S-W	7 days	9,377	8,888	909	694	8,350
Riverside Dr.	117	W-N/S	3 days	22,926	N/A	1,851	1,683	20,400
Riverside Dr.	117	N-E	3 days	4,516	N/A	255	547	4,000
Riverside Dr.	117	S-E	3 days	5,745	N/A	382	590	5,100
Vanier Pkwy.	117	E-N/S	3 days	9,909	N/A	793	716	8,800
Vanier Pkwy.	117	S-W	3 days	11,017	N/A	999	988	9,800
Vanier Pkwy.	117	N-W	3 days	11,767	N/A	1,422	988	10,450
St. Laurent Blvd.	115	W-N/S	7 days	17,317	16,330	1,255	1,224	15,400
St. Laurent Blvd.	115	N-E	7 days	4,337	4,445	275	373	3,850
St. Laurent Blvd.	115	S-E	7 days	2,453	2,176	153	245	2,200
St. Laurent Blvd.	115	E-N/S	7 days	7,328	7,157	659	559	6,500
St. Laurent Blvd.	115	S-W	7 days	5,664	5,114	371	454	5,050
St. Laurent Blvd.	115	N-W	7 days	12,748	12,104	1,293	889	11,350
Ottawa 174	113	W-E	7 days	36,113	34,563	1,940	3,286	32,150
Ottawa 174	113	S-E	7 days	7,710	7,111	561	731	6,850
Aviation Pkwy.	113	S-N	3 days	2,694	N/A	255	250	2,400
Ottawa 174	113	E-S	7 days	6,879	6,375	497	567	6,100
Ottawa 174	113	E-W	7 days	34,495	33,482	2,900	2,162	30,700
Aviation Pkwy.	113	N-S	3 days	3,531	N/A	335	283	3,150

6. *ADT* "Average Daily Traffic" calculated using the seven-day counts only.
7. *Average Weekday AM Peak* includes the average traffic counts during the AM peak period.
8. *Average Weekday PM Peak* includes the average traffic counts during the PM peak period.
9. *AADT* "Annual Average Daily Traffic" contains the average traffic counts during the year and estimated using MTO 2000 AWD adjustment factor for urban commuter pattern.

On the other hand, the second table contained the traffic counts for the mainline within the study area. Traffic volumes for both directions between each two successive

interchanges were provided in this table. A sample of the data presented in this file is shown in Table 4.4. More illustration for the fields used to describe volumes for each section is shown below:

Table 4.4: Sample Traffic Volume Data in Tabular Format: Volumes on Mainline.

Mainline Section (between interchanges)		A.M. Peak Hour Volume		P.M. Peak Hour Volume		AWD			AADT **	DHY **
From:	To:	EB	WB	EB	WB	EB	WB	Total		
Highway 416	Richmond Road	4,950	3,440	3,900	3,500	48,430	56,040	104,470	93,000	9,500
Richmond Road	Pinecrest Rd./ Greenbank Rd. *	5,700	4,200	4,650	5,450	59,200	65,300	124,500	112,000	11,400
Pinecrest Rd./ Greenbank Rd. *	Woodroffe Avenue	5,540	5,600	5,100	5,960	62,650	69,680	132,330	118,000	12,050
Woodroffe Avenue	Maitland Avenue *	6,000	5,900	5,300	6,100	72,600	74,100	146,700	130,500	13,300
Maitland Avenue	Kirkwood Avenue	6,540	5,500	5,600	6,150	78,040	74,500	152,540	136,000	13,850
Kirkwood Avenue	Island Park Drive	7,490	5,850	6,070	6,320	85,830	77,230	163,060	145,000	14,890
Island Park Drive	Parkdale Avenue	7,490	6,340	6,070	6,510	85,830	80,670	166,500	148,000	15,100
Parkdale Avenue	Rochester Street	7,670	6,840	6,360	6,760	88,220	82,910	171,130	152,500	15,550
Rochester Street	Bronson Avenue	7,050	6,400	5,870	6,220	81,370	76,770	158,140	140,500	14,350
Bronson Avenue	Lyon Street *	6,250	6,600	5,200	6,350	70,850	81,750	152,600	136,000	13,850
Lyon Street	Kent Street	6,250	6,820	4,630	6,090	62,190	73,150	135,340	120,500	12,300
Kent Street	O'Connor Street	5,150	6,820	4,630	5,500	62,190	73,150	135,340	120,500	12,300
O'Connor Street	Metcalfe Street *	5,100	6,150	4,550	5,600	61,550	73,400	134,950	120,000	12,250
Metcalfe Street	Nicholas Street	6,350	7,180	6,840	6,510	79,260	79,250	158,510	141,000	14,400
Nicholas Street	Vanier Parkway	6,430	7,370	7,760	6,680	87,350	83,510	170,860	152,000	15,500
Vanier Parkway	St. Laurent Boulevard	4,140	6,410	6,160	4,460	71,580	70,630	142,210	126,500	12,900
St. Laurent Boulevard	Ottawa Rd. 174 IC *	3,100	5,500	5,650	3,600	61,050	59,550	120,600	107,500	10,950
Ottawa Rd. 174 IC	Innes Road *	2,250	3,750	3,650	2,750	36,850	37,350	74,200	66,000	6,750
Innes Road	Wakley Road *	1,550	3,350	3,150	2,200	29,550	29,100	58,650	52,000	5,300
Wakley Road	Russell Road *	550	3,000	2,550	700	17,200	16,450	33,650	30,500	3,100

1. *Mainline section* (between interchanges) indicates the location by identifying the first and second interchanges bounding the section.
2. *A.M. Peak Hour Volume* includes AM peak hour volumes for both directions separately: eastbound "EB" and westbound "WB".
3. *P.M. Peak Hour Volume* includes PM peak hour volumes for the same two directions named EB and WB.
4. *AWD "Average Weekday Traffic"* for both directions as well as the total volume.

5. *AADT* "Annual Average Daily Traffic" based on MTO 2000 adjustment factors
6. *DHV* "Design Hourly Volumes" adjusted also by MTO 200 adjustment factors.

4.4 COLLISION DATA

Collision data for a five-year period from 1998 to 2002 were obtained from the City of Ottawa. The data format was 5 separate Excel files; one for each year. All the files have common field names that describe each collision. Figure 4.10 shows a total of 17 fields that were used to describe each collision with a sample of four collisions occurring within the study area. It should be noted that the fields shown in Figure 4.10 were rearranged to fit in one figure. For each collision record, *Date*, *Time* and *DayofTheWeek* were used to define the time of the occurrence. *ClassificationofAccident* contains the type of the accidents as fatal injury, non-fatal injury, or property damage only (PDO).

Generally, a collision is classified as fatal if the collision resulted in the death of at least one person within 30 days of the collision. Injury collision classification is used with collisions that resulted in the bodily injury of at least one person, and not resulting in death. Finally, PDO collisions are the ones in which no person suffered a bodily injury and the damage was reported. All collisions fatal and injury are reported, while PDO collisions are reported only if the property damage exceeded a specific threshold. It should be noted that the collision-reporting threshold was fixed at \$ 1000 over the study period from 1998 to 2002.

ImpactType describes the type of collision as angle, sideswipe, rear-end, turning movement, or single types. Two more fields were provided to describe the visibility and operational conditions. The first one, *Environment1*, describes the weather condition at the time of the collision as clear, rain, freezing rain, snow, drifting snow, strong wind, and fog. The other field, *Light*, describes the light condition as: dark, dawn, daylight, and dust. The number of persons, vehicles, and pedestrians involved in the collision were shown in *NumberOfPersons*, *NumberOfVehicles*, and *NumberOfPedestrians*, respectively. *TrafficControl* describes the intersection control type at the location of the collision. It should be noted here that the files contained collisions that occurred on the freeway junction along with collisions that occurred on all segments between each two junctions.

AccidentID	Date	Time	DayOfTheWeek	ClassificationOfAccident	ImpactType
018001018	2001/12/11	17:35	Tuesday	P.D. only	Sideswipe
018001016	2001/12/11	06:40	Tuesday	Non-fatal injury	Sideswipe
018000591	2001/07/25	16:40	Wednesday	P.D. only	Rear end
018000745	2001/09/25	11:25	Tuesday	P.D. only	Other
Environment1	Light	TrafficControl	NumberOfPersons	NumberOfVehicles	NumberOfPedestrians
Clear	Dark	No control	0	2	0
Clear	Dark	No control	1	3	0
Clear	Daylight	No control	0	2	0
Clear	Daylight	No control	0	2	0
Municipality	STREET1	STREET2	STREET3	MapLocationID	
Ottawa	HIGHWAY 417	HWY 417 CARLING IC124R76		3641175027133	
Ottawa	HIGHWAY 417	HWY 417 CARLING IC124R76		3641175027133	
Ottawa	HIGHWAY 417	HWY 417 CARLING IC124R67	HWY 417 CARLING IC124R76	S18539	
Ottawa	HIGHWAY 417	HWY 417 CARLING IC124R67	HWY 417 CARLING IC124R76	S18539	

Figure 4.10: Sample of Collision Data.

The location of the collision is defined in five fields. The first field, *Municipality*, defines the municipality in which the collision occurred. The value in this field was Kanata, Nepean Ottawa, Gloucester, Cumberland, or West Carleton. The following three

fields (*street 1*, *street 2*, and *street 3*) were used to identify the location of the section on the highway. *Street 1* is the name of the highway on which the collision took place; *Street 2* is the first access point, either exit or entrance; and *Street 3* is the second access point. Access points were identified using both interchange number and ramp number within this interchange. Both *street 2* and *street 3* were used to identify the two vertices of the segments. Any collision that occurred at an access point was identified using *street 1* and *street 2* only with blank value in *street 3*. Finally, *MapLocationID* is the last field that was used to identify the collision location depending on whether the collision occurred on a specific segment or in the vicinity of an on-ramp or off-ramp.

A collision on a specific segment was given an arbitrary code in *MapLocationID* field (see the second two records in Figure 4.10), while a collision in the vicinity of on-ramp or off was provided with a coordinate system that was placed in the *MapLocationID* (see the first two records in Figure 4.10). This coordinate system was used to plot all these collisions on the AutoCAD maps with a unique identifier given to all collisions on each point. Hence, it was possible to separate the collisions on Highway 417 from those on the intersecting roads. For example, in Figure 4.11, collisions given the identifiers 70, 71, 75, 78, and 79 were added to those collisions on the corresponding segment of Highway 417, while collisions with the identifiers 72, 74, and 76 were omitted. All these procedures were used with each file corresponding to each year. The database software Microsoft Access was then used to match and count the collisions that occurred on each segment over the five years. After combining the records of the five files, the total collisions over the entire period for each segment formed the final collision database for the analysis process.

Figure 4.11: Collisions on Junctions at IC 129.

5. DESCRIPTIVE STATISTICS

This chapter provides a descriptive analysis of the data collected in this study. Sections 5.1, 5.2, and 5.3 present a statistical description of the extracted segment types, lengths, and number of basic lanes of Highway 417, respectively. Section 5.4 describes the collision history, while Section 5.5 provides a summary of the traffic volumes operating on the Highway 417 mainline as well as the exit and entrance volumes. Finally, Section 5.6 shows the frequencies of different advisory speeds posted on all exits.

5.1 SEGMENT TYPES

Basically, a segment is used to refer to the spacing between each two successive ramps on the freeway. There are four types of segments analyzed in this study; namely: *entrance-entrance*, *entrance-exit*, *exit-entrance*, and *exit-exit*. The type of each segment depends mainly on the type of the two bounding ramps. The total number of segments of all types is 94 in both directions; EB and WB. The distribution of these segments is shown in more details in Table 5.1.

Table 5.1: Summary for Segment Counts Included in the Analysis.

	<i>Entrance-Entrance</i>	<i>Entrance-Exit</i>	<i>Exit-Entrance</i>	<i>Exit-Exit</i>
EB	13	16	16	3
WB	10	18	16	2
Total	23	34	32	5

5.2 LENGTHS OF SEGMENTS AND SPEED CHANGE LANES

As mentioned before, the segment length was measured from the physical gore of the two ramps bounding the segment at which the pavement edges of the ramp and the mainline are 1.25 m apart. The segments are either between interchanges, and are relatively long, or within an interchange, and are relatively short. This variety in segment length would help capture the effect of length on safety performance. As shown in Table 5.2, the lengths ranged from a minimum of 160 m to a maximum of 5116 m with an average of 795 m and a standard deviation equal to 792 m. Figure 5.1 shows the frequency distribution of the 94 segment lengths within the study area. The maximum frequency of 41 segments was associated with a range of length from 375 to 625 m (500 m in the graph). In addition, only four segments had a length equal to or higher than 2000 m.

Table 5.2: Summary of Segment Lengths in the Study Area.

Direction	Number	Minimum (m)	Maximum (m)	Mean (m)	Standard Deviation (m)
EB	48	229	4982	794	779.43
WB	46	160	5116	796	814.45
Total	94	160	5116	795	792.46

Figure 5.1: Frequency Distribution of Segment Lengths.

On the other hand, speed change lanes (acceleration and deceleration lanes) within the study area had either a limited length with a physical end or an extended length to the next ramp. As mentioned before, the length of the speed change lane used in the analysis is the distance from the point at which the ramp and the mainline pavement edges are 1.25 m apart and the point that the speed change lane pavement edge intersects with the mainline edge. The total number of acceleration lanes within the study area was 57 lanes in both directions including both limited and extended types. It should be noted that the length of only one acceleration lane could not be identified due to the lack of information. For the limited type, a total of 41 acceleration lanes had lengths ranging from 150 m to 500 m with an average of 348 m and a standard deviation of 96.53 m,

while, for the extended type, the lengths of 15 acceleration lanes ranged from 180 m to 1802 m with an average of 747 m and a standard deviation of 321.21 m. Figure 5.2 shows the frequency distribution for the 56 lengths of acceleration lanes. The figure indicates that 28 lengths out of 56 (34 %) had lengths between 250 m to 450 m, while a total of 10 lengths had lengths higher than 550 m.

Figure 5.2: Frequency Distribution for Lengths of Acceleration Lanes.

In addition, the total number of deceleration lanes was 39 for both directions including both types; limited and extended. Two lanes out of these 39 deceleration lanes also could not be identified due to the lack of information. For the limited type, a total of 21 deceleration lanes had lengths ranged from 72 m to 509 m with an average of 287 m and a standard deviation of 116.18 m, while, for the extended type, a total of 16

deceleration lanes had lengths ranging from 171 m to 1302 m with an average of 703 m and a standard deviation of 309.68 m. Figure 5.3 shows the frequency distribution for lengths of deceleration lanes. The maximum frequency of eight deceleration lanes, as shown in this figure, was associated with lengths ranged from 250 m to 350 m.

Figure 5.3: Frequency Distribution for Lengths of Deceleration Lanes.

5.3 NUMBER OF BASIC LANES

As shown schematically in Figure 5.4, the portion of Highway 417 within the study area varies in terms of the number of basic lanes. The mainline consists of four basic lanes in the area from IC 145 west (intersection with Highway 7) to IC 134 east (intersection with Moodie Drive). Then, the number increases to be eight lanes from IC

134 west to IC 129 east (intersection with Pinecrest Road). Six basic lanes are then observed in the area from IC 129 west to IC 124 east (intersection with Carling Avenue). As Highway 417 approaches the core of the city, the number increases again to be eight lanes for the distance between IC 124 west to IC 120A east (intersection with Kent Street). Six basic lanes are then observed after that from IC 120A west to IC 113 east (intersection with Highway 174). Finally, a number of four basic lanes are observed from IC 113 west to the limit of the study area at IC 110 (intersection with Walkley Road).

Figure 5.4: Number of Basic Lanes of Highway 417 within the Study Area.

5.4 COLLISIONS

The collected collisions, as indicated earlier, included five years from 1998 to 2002 for all segments. The data included three types of collisions; namely: property damage only (PDO), Injury, and fatal collisions. Collisions were classified into two main classes; (1) Class I: sum of total collisions for each segment (PDO+IN+FA); (2) Class II: equivalent property damage only (EPDO). The first class is a simple summation of all

types of collisions. In this class, safety improvement efforts attempt to reduce the number of collisions regardless of the collision type. On the other hand, in the second case, the variation in severity between collisions is acknowledged through using a weighted sum of collisions. Zuehlke (2002) recommended weights of fatal and injury collisions equal to 12 and 6, respectively, while the weight of a PDO collision is equal to 1 by definition. These weighting factors were used to convert all collisions to EPDO collisions.

For Class I, as shown in Figure 5.5, the collision frequencies ranged from zero to 130 collisions with an average of 33 collisions and a standard deviation of 31.46 collisions. The highest frequency of 20 segments was observed for segments with 10 collisions over five years. On the other hand, for class II, the collision frequencies ranged from zero to 260 collisions with an average of 69 collisions and a standard deviation of 64.26 collisions. The frequency distribution, as shown in Figure 5.6, indicated that the maximum frequency of Class II remained the same as Class I at segments with 20 equivalent collisions. It should be noted that the overall distributions for Class I and Class II are positively skewed as shown in the two figures.

Figure 5.5: Frequency Distribution for Sum of Collisions.

Figure 5.6: Frequency Distribution for *EPDO* Collisions.

5.5 TRAFFIC VOLUMES

As mentioned before, the obtained data included all AADT for the mainline as well as all ramps; off-ramps and on-ramps. For each segment, there are three traffic volumes; namely: *entrance volumes*, *exit volume*, and *through volume*. The summation of the entering and exit volumes for each segment was also suggested as a potential factor that might affect the safety performance on any segment. It should be noted that *entrance volume* is only included if the first ramp is an on-ramp. Similarly, *Exit volume* is considered only if the second ramp is an off-ramp.

For example, as the first case shown in Table 5.3, the segments between an on-ramp and an off-ramp had the two volumes, entrance and exit, added together. The other extreme is segments between an off-ramp and an on-ramp (fourth case in Table 2.1), where both *entrance* and *exit volumes* were not included. The number of segments that had the first ramp as an on-ramp was equal to the summation of *entrance-entrance* plus *entrance-exit* segments and was equal to 57 segments (34+23). Besides, the number of segments that had the second ramp as an off-ramp was equal to 39 segments (34+5). In addition, the number of segments experienced *entrance+exit* volume was 62 (34+23+5). Table 5.4 shows in more details the minimum, maximum, mean, and standard deviation of all types of traffic volumes.

Table 5.3: The Number of Peak Hour Volumes Included in the Study.

<i>Segment Type</i>	<i>Entrance Volume</i>	<i>Exit Volume</i>	<i>Entrance+Exit Volume</i>
	34	34	34
	23	NA	23
	NA	5	5
	NA	NA	NA
Total	57	39	62

Table 5.4: Summary of Peak Hour Volumes for All Segments.

Traffic type	Number	Minimum (veh/h)	Maximum (veh/h)	Mean (veh/h)	Standard Deviation (veh/h)
<i>Through</i>	94	960	6944	3811	1912
<i>Entrance</i>	57	5	2216	555	435
<i>Exit</i>	39	100	2050	820	519
<i>Entrance+Exit</i>	62	1070	7901	4449	2176

5.6 ADVISORY SPEEDS

The advisory speeds posted on all exits for both directions were collected as previously mentioned during the site investigation. These speeds were assumed to have an effect on the deceleration behaviour of the driver in the vicinity of off-ramps. The posted speeds on the available 39 exits ranged from 40 km/h to 80 km/h. Table 5.5 shows

the frequencies of the advisory speeds for both direction within the study area. It is obvious that the majority of the exits had low advisory speeds, while only five exits had 70 and 80 km/h which implies a significant drop from the freeway speed to the exit speed.

Table 5.5: Summary of Advisory Speed Frequencies in the Study Area.

Advisory Speed (km/h)	40	50	60	70	80
Frequency	12	10	12	3	2

6. REGRESSION ANALYSIS

This chapter summarizes the common steps to develop a good model with an acceptable goodness of fit with a focus on the appropriate techniques for modelling collision data. Section 6.1 shows briefly the common steps of modelling collisions. Following, Sections 6.2 to 6.6 presents a detailed discussion of each of these steps including selecting a probability distribution, a model form, and variables, besides estimating the regression parameters and assessing the goodness of fit.

6.1 BACKGROUND

In safety analysis, regression models can be used to study the expected safety improvement in terms of reduction of collision frequencies that might result from revising some design elements. Miaou (1996) set five general tasks to develop a collision model. These five steps are:

1. Probability distribution (Section 6.2): finding a good probability distribution that accommodates the random part of collision variation,
2. Model form (Section 6.3): choosing a suitable function form,
3. Variables selection (Section 6.4): selecting the set of predictive and dependent variables.
4. Parameters estimation (Section 6.5): estimating the coefficients of the regression parameters, and

5. Goodness to fit (Section 6.6): assessing the goodness of fit for the developed model.

These five steps are discussed in more details as follow:

6.2 PROBABILITY DISTRIBUTION

Reviewing the collision modelling for the last few decades revealed the inadequacy of the traditional linear regression techniques to describe a collision data. Conceptually, linear models do not match the reality of collision data that are known to be discrete data and cannot be well described by a continuous distribution. In addition, using linear regression modelling may result in negative values which in turn violate the fact that collision data are always positive values. Moreover, the normal distribution assumed in linear modelling suggests a symmetric distribution around the mean which is not true for collision data. The characteristics of the collision distribution in hand, as previously indicated in Section 5.4, obviously violate the main assumptions in linear regression techniques.

For these reasons, it has become necessary to replace the traditional linear regression by more suitable techniques. Poisson and negative binomial distributions have proven to be promising approaches adequately describe the variation in collision data. The positively skewed distribution with non-negative and discrete nature makes both distributions valid and statistically acceptable techniques for collision modelling. The ability of the model to predict collision frequency generally increases with increasing the

number of predictors. That is, the more variables included in the model, the more systematic variation of the collisions that can be captured.

However, with taking into account only few significant predictors, as it is in collision modelling, other significant factors are omitted resulting in an increase in collision variation. This can be described statistically as an over-dispersion of the data. One of the main distinctive characteristics of negative binomial distribution over Poisson distribution is adding an extra term to the variance to accommodate this over-dispersion. Nevertheless, both techniques should be tried to determine the most appropriate approach. Both Poisson and negative binomial distributions have a common assumption that the events must be random besides being independent on each other. The following is a brief discussion of Poisson and negative binomial distributions followed by the criteria to choose between them.

6.2.1 Poisson Distribution

$$P(y) = \frac{\mu^y \exp(-\mu)}{y!} \quad \text{Equation 6.1}$$

Where; $E(y) = \text{Var}(y) = \mu$, $P(y)$ = probability of obtaining y successes.

Poisson distribution assumes that the event (y) is a random and independent event with discrete distribution. Conceptually, the distribution gives the probability of having a specific number of successes (y) in terms of only the mean ($E(y)$). In general, there are two conditions that can encourage using the Poisson distribution; (1) increasing the number of trials (n); and (2) decreasing the probability of having a success in each trial. If

collisions follow the Poisson distribution, the exponential relationship would be considered suitable to guarantee positive values for collisions under all circumstances (see Equation 6.2).

$$y_i = e^{\sum \beta_0 + \beta_i * x_i} \quad \text{Equation 6.2}$$

Where y_i =collision frequency; β_0 =intercept, β_i =regression coefficients; x_i =predictors.

In addition, the exponential relationship results in a skewed distribution that can closely match the actual data. Theoretically, as indicated earlier, the mean and variance for this distribution are assumed to be equal. This might be taken as one of the main drawbacks of applying the Poisson regression. This assumption puts constraints on the suitability of collision modelling and encourages seeking other techniques that can have more flexibility to the relationship between the mean and the variance.

6.2.2 Negative Binomial Distribution

$$P(y) = \binom{N-1}{n-1} p^n (1-p)^{N-n} \quad \text{Equation 6.3}$$

Where; $P(y)$ = probability of obtaining y successes; N = the number of successes; n = the number of trials; and p = the probability of having a success.

The over-dispersion of collision data, as mentioned earlier, results from omitting some significant variables that are beyond the research goals of the model. It can also result from omitting other variables that cannot be accounted for, such as vehicle

conditions and driver behaviour. The absence of all these omitted variables could make the data to disperse and the distribution to follow a Gamma distribution instead of Poisson. This dispersion can be accommodated using negative binomial distribution as it has an added term in the variance called the dispersion parameter (α). As shown in Equation 6.4, the added term ($\alpha\mu^2$) gives an advantage over the Poisson distribution and fit better in cases of over-dispersed collision data. The negative binomial distribution would approach a Poisson distribution when the dispersion parameter (α) approaches zero.

$$E(y) = \mu \text{ \& \; } \text{Var}(y) = \mu + \alpha\mu^2 \quad \text{Equation 6.4}$$

6.2.3 Selecting the Appropriate Technique

Choosing between Poisson and negative binomial should be based on statistical tests for the examined data. Several tests can be used to determine which distribution can better fit the available data. The most common one is based on Pearson Chi-Square (χ^2) test. Conceptually, the application of this test helps comparing observed counts resulting from a particular test to the expected ones using a specific regression technique. The Chi-Square (χ^2) value can be simply calculated using Equation 6.5; where (y_i) is the observed value and ($\hat{y}$) is the predicted one. The ratio between the χ^2 and the number of degrees of freedom is called the over-dispersion parameter (τ). It should be noted here that the number of degrees of freedom is equal to the number of observations (n) minus the number of parameters (k) in the model. The more different τ is from one, the less the

ability of the model to predict the collisions using the proposed technique (see Equation 6.6).

$$\chi^2 = \sum \frac{(y_i - \hat{y}_i)^2}{\hat{y}_i} \quad \text{Equation 6.5}$$

$$\tau = \frac{\chi^2}{n - k} \quad \text{Equation 6.6}$$

Where; χ^2 = Pearson chi-squares value; n = number of observations; and k = number of parameters.

The over-dispersion parameter (τ) can be used to evaluate the validity of the proposed technique to capture the variation in the observations. Due to the simplicity of using the Poisson distribution, compared to the negative binomial distribution, the first will be assumed and tested for its validation to represent the actual data using the over-dispersion parameter (τ). The typical value for this parameter as mentioned by Miaou (1996) is between 0.8 and 1.2. If the value violates this range, an over- dispersion or under-dispersion is then evident depending on whether τ is higher or lower the acceptable range, respectively. The relationship between the over-dispersion parameter (τ) and the dispersion parameter (α) previously mentioned in Equation 6.4 is described by Miaou (1996) as shown in Equation 6.7. As the τ approaches 1, assuming a Poisson regression technique, the α reaches zero causing the variance to be equal to the mean thus perfectly match the Poisson distribution.

$$\tau = 1 + \alpha \quad \text{Equation 6.7}$$

Similar to Chi-square test, the scaled deviance (D^m) can be used to measure the over-dispersion of the data. As shown in Equation 6.8, the D^m represents double the

difference between the log-likelihood achieved by a model that perfectly fits the data (L^f) and the assumed model with the candidate variables (L^m). Dividing this term by the number of degrees of freedom, as shown in Equation 6.9, gives another measure to evaluate the dispersion in the data. Similar to Chi-square test, values greater than one warrant using negative binomial distribution to overcome the over-dispersion in the data.

$$D^m = 2(L^f - L^m) \quad \text{Equation 6.8}$$

$$\tau = \frac{D^m}{n-k} \quad \text{Equation 6.9}$$

6.3 MODEL FORM

The step that follows the identification of an appropriate distribution is to find a valid model form to capture this distribution. The assumed model form should guarantee non-negative values in the predicted collisions besides being able to capture the non-normality shape of the distribution. Qin et al. (2004) mentioned that using the exponential relationship in collision modelling is considered to be a satisfactory form that meets the mentioned characteristics. The suggested model is shown in Equation 6.10.

$$Y = e^{\beta_0} * e^{\beta_1 x_1} * e^{\beta_2 x_2} * \dots * e^{\beta_n x_n} \quad \text{Equation 6.10}$$

The exposure to traffic is a considerable term that should be included in collision models to capture the effect of the variation in length and traffic volumes between different segments. Risk and Shaoul (1982) mentioned that the importance of involving the exposure term in collision models is due to the assumption that the more a road user is involved with the road and traffic system, the more collisions he is liable to have.

There are two methods to account for traffic exposure in the model. The first method considers indirectly the variation in the amount of exposure by using collision rate in the units of collision/million vehicle-km. Collision rate can be calculated by dividing the collision frequency by the traffic volume and length for each highway segment. This method has been considered unsatisfactory since any volume change would reflect an improvement in safety performance even if the collision frequency remains constant. On the other hand, the second method involves using traffic volume and segment length as predictor variables in the safety model. This approach should reveal the actual relationship between traffic exposure and collisions.

Since long highway segment would have greater chance to experience more collisions, the segment length should be considered along with the traffic volume in the exposure term. There are two methods to consider both variables in the collision model. The first method is to consider the segment length as a separate variable from the traffic volume. In such a case, both volume and length variables would have different coefficient in the model. Alternatively, segment length and traffic volume could be combined in the model as one term as shown in Equation 6.11. Possible models that were suggested in the previous research to account for the exposure term as well as the explanatory variables are shown in Equation 6.11 through Equation 6.15.

$$Exposure = (years)(AADT)(365)(L)(10^{-6}) \quad \text{Equation 6.11}$$

$$y = e^{\beta_1 * LSeg} e^{\beta_2 * AADT} e^{\sum \beta_i * x_i} \quad \text{Equation 6.12}$$

$$y = e^{\beta_1 * Expo} e^{\sum \beta_i * x_i} \quad \text{Equation 6.13}$$

$$y = LSeg^{\beta_1} AADT^{\beta_2} e^{\sum \beta_i * x_i} \quad \text{Equation 6.14}$$

$$y = Expo^{\beta_1} e^{\sum \beta_i * x_i} \quad \text{Equation 6.15}$$

Where; $years$ = the study period in years; Y = collision frequency in the study period; $AADT$ = Annual Average Daily Traffic; L = Segment length in (m); $Expo$ = exposure term (million vehicle-km); β_1 , and β_2 are the variable coefficients.

6.4 VARIABLES SELECTION

After identifying different model forms with their possible combinations of the exposure term, the next step is to present the available predictors that need to be evaluated in terms of their effect on safety performance. Section 6.4.1 discusses the selection of the dependent variable represented by different collision types. In Section 6.4.2, the candidate explanatory variables will be illustrated.

6.4.1 Dependent Variable

As mentioned in Section 5.4, the collision data used in this study were classified into two main groups; (1) collisions that occurred on a specific segment and identified by the highway name (Highway 417) and the two bounding junctions; and (2) collisions that occurred on a speed change lane and identified by a coordinate system. Both groups also included the collision frequency for three collision types (PDO, injury, and fatal). Based on the available data and the approaches that were found in the literature, six dependent variables were formed and prepared for the analysis. The six variables are:

1. *TCol*: Total collisions for each segment that were calculated as a simple summation of all three collision types including collisions that occurred on the segment or speed change lane.
2. *NCol*: The simple summation of all collisions that took place on the acceleration lane just following an entrance (on-ramp).
3. *XCol*: The simple summation of all collisions that took place on deceleration lane just preceding an exit or off-ramp.
4. *TEPDO*: The total equivalent PDO collisions for each segment based on weighting factors of 6 and 12 for injury and fatal collisions, respectively (see Section 5.4)
5. *NEPDO*: The total equivalent PDO for collisions that occurred on an acceleration lane, calculated using the same weighting factors.
6. *XEPDO*: The total equivalent PDO for collisions occurred on a deceleration lane using the same conversion factors.

Figure 6.1 shows an example of a highway segment between an on-ramp and off-ramp. The highway experienced collisions on the segment as well as on the acceleration and deceleration lanes with types and frequencies shown in the figure. Sample calculations for the six dependent variables on that assumed segment are illustrated in the figure.

6.4.2 Explanatory variables

The next step after identifying the collision sets that may be used as a dependent variable in the model is to identify the predictors that could be used to capture the variation in collision frequencies among the different segments. It should be mentioned that traffic exposure that is represented by the traffic volume and the segment length is essential to each model developed and must be included according to one of the four forms in Equation 6.12 through Equation 6.15. In addition, there are several other explanatory variables that were assumed to have an effect on safety performance of the freeway. Some of these factors are associated with the composition of the traffic volume entering and exiting from each segment, and others are related to the geometry of the road.

Figure 6.1: A Sample Calculation of the Six Dependent Variables.

The Annual Average Daily Traffic (*AADT*) for each segment represents the total volume on that segment, and as explained earlier is essential in all developed models. More traffic volumes that might also affect the safety performance are *entrance volume* (*NV*), *exit volume* (*XV*), and summation of entrance and exit volumes (*NXV*). The methodology of calculating these three factors for each segment type was discussed in more details in Section 5.5. It is worth to mention here that, each of these three traffic volumes represents only a percentage of the *AADT* on the freeway segment. This percentage ranges theoretically from zero to hundred. The purpose of involving these factors is to evaluate the effect of the number of lane changes associated with the manoeuvres by the entering and exiting volumes on the freeway safety performance.

Factors that can be classified as geometric characteristics include first the length of the segment (*LSeg*) measured from the gore of the first ramp to the gore of the second ramp for each segment. Similar to *AADT*, this variable is involved in all models according to one of the four forms in Equation 6.12 through Equation 6.15. In addition, the length of the acceleration lane (*LAcc*) and the length of the deceleration lane (*LDec*) were measured from the physical gore of the ramp to the point at which the pavement edge of the ramp intersects with the pavement edge of the mainline. In cases that the acceleration or deceleration lane has unlimited length and is extended to the next ramp, a code *AccCo* or *DecCo* was created. Either *AccCo* or *DecCo* was set to be zero for the limited lane and was set to be one for extended length of the lane. These codes would help distinguish between the two types of acceleration and deceleration lanes in the analysis.

The basic number of lanes (*NSeg*) for each segment was identified using the video clip that was taken for the study area. Besides, for each segment, the number of lanes of the first ramps (*NR1*) and the second ramp (*NR2*) were also identified. In addition, the advisory speed (*AdvSp*) posted on each exit was collected. The weaving type (*WeavType*) for *entrance-exit* segments was identified and was thought to have potentially affected the safety performance on the weaving sections. Table 6.1 summarizes all the explanatory variables that were included in the analysis.

Table 6.1: All Candidate Explanatory Variables.

Category	Explanatory variables	Description
Traffic	<i>AADT</i>	Annual Average Daily Traffic (veh/d)
	<i>NV</i>	Entering traffic volume through on-ramp (veh/h)
	<i>XV</i>	Exiting traffic volume through off-ramp (veh/h)
	<i>NXV</i>	Summation of entering and exiting traffic volumes (veh/h)
Lengths	<i>LSeg</i>	Segment length between ramp terminals (km)
	<i>LAcc</i>	Length of the acceleration lane (m)
	<i>LDec</i>	Length of the deceleration lane (m)
	<i>AccCo</i>	0 if limited length of acceleration lane, 1 for extended
	<i>DecCo</i>	0 if limited length of deceleration lane, 1 for extended
Widths	<i>NSeg</i>	The basic number of lanes
	<i>NR1</i>	The first ramp number of lanes
	<i>NR2</i>	The second ramp number of lanes
Others	<i>AdvSpeed</i>	Advisory speeds of off-ramps (km/h)
	<i>WeavType</i>	0 for weaving <i>Type A</i> , 1 for weaving <i>Type B</i>

6.5 PARAMETERS ESTIMATION

After identifying the, probability distribution, model form, and candidate predictors to evaluate a certain dependent variable in an assumed model, the next step is to estimate the parameters (β_i) in the models. Since the linear regression technique was rejected in this study, the traditional least square principle to estimate the parameters was deemed invalid. Alternatively, the maximum log-likelihood function $L(\beta)$ is more suitable for Poisson and negative binomial techniques. The maximum log-likelihood function $L(\beta)$ is shown in Equation 6.16.

$$\log L(\beta) = \sum [y_i \log(\mu_i) - \mu_i - \log(y_i!)] \quad \text{Equation 6.16}$$

Where; y_i = the observed variable; μ_i = the variable mean. The coefficients of the model variables (β_i), for both Poisson and negative binomial, have been estimated in this study using the generalized linear model procedures PROC GENMOD in the SAS statistical software, which is one of the most widely used statistical software in North America. The results provided by the software also include the standard error and p -value for each parameter. The standard error is the standard deviation of the sampling distribution while, p -value can be used to evaluate a null hypothesis that a specific parameter is not significantly different from zero. The null hypothesis is rejected if the p -value is less than a specific level of significance, usually taken as 5 %. Finally, the results include also the Pearson chi-square (χ^2), scaled deviance (D^m), and dispersion parameter (α) previously discussed in Section 6.2.3.

6.6 GOODNESS OF FIT

The coefficient of determination R^2 , traditionally used in linear regression, is an unsatisfactory measure of goodness of fit for Poisson and negative binomial regression. To demonstrate the pitfalls of using R^2 , Miaou (1996) assumed two simple linear relationships between y and both x_1 and x_2 . The first relationship $[y = \beta_0 + \beta_1 * x_1]$ has a lower slope value (β_1) than the slope of the second relationship. He created some points with equal offsets from the two regression lines and calculated the R^2 values for both relationships. It was then shown that increasing the slope of the relationship increases the value of R^2 even for the same offsets between the created points and the regression line. It was therefore concluded that this widely used R^2 value would represent a false measure for goodness of fit.

In this study, the goodness of fit was assessed by comparing the values D^m and χ^2 against the maximum value of the χ^2 distribution corresponding to the model's number of degrees of freedom at the 95 % confidence level. Other secondary parameters included the over-dispersion parameters (τ) based on D^m and χ^2 statistics. As mentioned in Section 6.2.3, the values of τ for a specific model compared to the acceptable range of 0.8 to 1.2 should help assess the validation of each model.

Miaou (1996) suggested using another goodness of fit measure that is valid for Poisson and negative binomial models. The new approach describes the expected relative improvement in collision estimation due to added predictors to the model. The comparison between the estimation ability of the model without predictors and the model with predictors depends on the dispersion parameter (α) previously discussed in Section

6.2.2. The maximum value for this parameter (α_{max}) is typically obtained from models without any predictors, and with adding predictors to the model α would decrease. This new measure of goodness of fit (R_α^2) can then be calculated using Equation 6.17. It was stated by Miaou (1996) that, although this measure appears to be extremely simple, it does not seem to have a good statistical interpretation. In addition, it does not reflect the number of covariates included in the model.

$$R_\alpha^2 = 1 - \frac{\alpha}{\alpha_{max}}$$

Equation 6.17

7. DATA PROCESSING

This chapter represents the application of the traditional steps of modelling that were illustrated in Chapter 6 to examine the relationship between collisions and candidate merge and diverge parameters such as lengths, volumes, and speeds. These steps were applied on all investigated models in order to obtain the most reliable ones with acceptable goodness of fit. The results are divided into three main sections: Section 7.1 includes the modelling attempts to evaluate the safety of all freeway segments, Section 7.2 discusses collision modelling of the weaving segments only, and finally Sections 7.3 focuses on collision modelling at freeway entrances and exits. In addition, Section 7.4 summarizes the results of all modelling attempts, while Section 7.5 presents application examples of the developed models.

7.1 ANALYSIS OF ALL SEGMENTS COMBINED

This section presents the modelling attempts for all segments combined. As explained earlier, collision experience on a specific segment may be estimated using *TCol*, which represents the simple summation of all collisions, or *TEPDO*, which represents the total equivalent property damage-only collisions. In addition, both Poisson and negative binomial were considered as two different modelling techniques. Moreover, four approaches to include the traffic exposure in the collision model had to be evaluated. Table 7.1 shows a summary of the alternative factors considered in the modelling attempts. Furthermore, different combinations of eleven explanatory variables had to be

tested to reach the most reliable models; these variables are NV , XV , NXV , $LAcc$, $AccCo$, $LDec$, $DecCo$, $NR1$, $NR2$, $NSeg$, $AdvSp$ (refer to Table 6.1 for description).

Table 7.1: A summary of Collision Modelling in Terms of the Exposure.

Parameter	Number of cases	Description
Regression Technique	2	(1) Poisson (2) Negative binomial
Collision Set	2	(1) <i>TCol</i> : Sum of all collisions on segments (2) <i>TEPDO</i> : Equivalent Property damage-only
Traffic Exposure	4	(1) Untransformed AADT and <i>LSeg</i> (2) Untransformed <i>Expo</i> (3) Natural logarithm of AADT and <i>LSeg</i> (4) Natural logarithm of <i>Expo</i>

In order to simplify the complicated modelling process associated with the huge number of trials required to test all possible combinations, the more appropriate regression technique, collision set, and traffic exposure were first identified. This process was conducted through analyzing and comparing the results of all possible trials of collision modelling based on traffic exposure only. Subsequently, a total of 16 models resulted from all possible combinations of two collision sets, two regression techniques, and four traffic exposure approaches were generated and tested. The SAS output for these attempts is shown in Appendix A and the main results are discussed in Section 7.1.1 through 7.1.4.

7.1.1 Selection of Regression Technique

To select between Poisson and negative binomial, the results of eight models using Poisson regression for the two collision sets and four exposure approaches were compared with the results of the corresponding models using the negative binomial regression. For the eight models using the Poisson, the over-dispersion parameters based on both D^m and χ^2 ranged from 9.9888 to 40.7048. Therefore, the results of the Poisson technique were extremely violating the acceptable range of 0.8 to 1.2. Obviously, the results reflect a tremendous dispersion in the data that could not be captured by the Poisson distribution. On the other hand, the results of the negative binomial regression results were found more suitable with a notable improvement in the over-dispersion parameters that ranged from 0.6053 to 1.2515. Subsequently, it was concluded that negative binomial regression was more appropriate to accommodate the high level of over-dispersion in the data, and hence was selected for final model development.

7.1.2 Selection of Collision Set

Since using the Poisson regression in this study was deemed much more inferior to the negative binomial regression, the comparison between the two sets of collisions (*TCol* and *TEPDO*) was conducted using the negative binomial regression only. Subsequently, a total of eight models for the two collision sets and four exposure approaches were compared with the main focus being selecting the most appropriate collision set. The over-dispersion parameters were found to range from 0.6506 to 1.2439

in the case of modelling the *TCol* collision set compared to a range of 0.6053 to 1.2421 in the case of *TEPDO* collision set.

Because of the very slight difference between the results of the over-dispersion parameter, it was not possible to identify the more appropriate collision set at this stage. For this purpose, six more models were developed excluding the exposure and including the explanatory variables that were expected to yield the most significant relationships; namely *NV*, *XV*, *NXV*, *AdvSp*, *LAcc&AccCo*, and *LDec&DecCo*. The six models were generated twice, once for each collision set. As shown in Table 7.2, the over-dispersion parameters were found closer to the recommended range in the case of using *TCol* rather than the case of using *TEPDO* (see Appendix B for SAS output). Therefore, it was found more appropriate to use *TCol* in the final model development.

Table 7.2: *TCol* versus *TEPDO* based on the over-dispersion parameters.

Explanatory Variable	TCol		TEPDO	
	$D^m/(n-k)$	$\chi^2/(n-k)$	$D^m/(n-k)$	$\chi^2/(n-k)$
<i>NV</i>	1.2149	0.7909	1.2344	0.7001
<i>XV</i>	1.2488	0.6542	1.2531	0.5874
<i>NXV</i>	1.2036	0.7951	1.2225	0.7386
<i>AdvSp</i>	1.2491	0.6856	1.2540	0.5866
<i>LAcc&AccCo</i>	1.2418	0.8533	1.2620	0.7345
<i>LDec&DecCo</i>	1.2950	0.6138	1.2995	0.5436

7.1.3 Selection of Traffic Exposure

Using the negative binomial regression technique and the *TCol* collision set, the four models for the individual traffic exposure approaches were re-evaluated in order to

select the most suitable one (see Appendix A for SAS output). The over-dispersion parameters were considered the main criteria of the selection. Table 7.3 shows the four models using the four different exposure approaches besides the goodness of fit measures for each model.

The first approach to include the traffic exposure was applied using traffic volume (*AADT*) and segment length (*LSeg*) separately. The model was found to be statistically significant at 5 % level of significance with a relatively high value of R_{α}^2 of 0.4883. In addition, the sign for the two regression coefficients corresponding to *AADT* and *LSeg* were found to be positive which reflects the logical trend of increasing the collision frequency with increasing *AADT* and *LSeg*. On the other hand, the over-dispersion parameters based on D^m and χ^2 were 1.2515 and 0.6506, respectively. That is the two parameters were outside the recommended range of 0.8 to 1.2.

The second approach for involving the traffic exposure in the collision model assumed a single exposure term (*Expo*) which combines *AADT* and *LSeg* for each segment as mentioned previously in Equation 6.11. In this approach, the regression coefficients were found to be statistically significant at 5 % level of significance. In addition, the direction of the relationship represented by the estimated parameter sign was also logical. Moreover, the over-dispersion parameters of this model based on D^m and χ^2 were 1.2139 and 0.7954, respectively. Although these values are relatively very close to the acceptable range, the value of R_{α}^2 was relatively low.

Table 7.3: The Suggested Four Approaches to Accommodate the Traffic Exposure.

$TCol = e^{(b_0+b_1(AADT)+b_2(LSeg))}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		113.8873
b ₀	1.2663	<0.0001	Pearson chi-square (χ^2), [max]		59.2026, [114.22]
b ₁	0.0348	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.2515
b ₂	0.5059	<0.0001		χ^2/DF	0.6506
Dispersion (α)	0.6106		R_α^2		0.4883
$TCol = e^{(b_0+b_1(Expo))}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		111.6786
b ₀	2.4883	<0.0001	Pearson chi-square (χ^2), [max]		73.1762, [115.34]
b ₁	0.0143	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.2139
Dispersion (α)	0.8230			χ^2/DF	0.7954
			R_α^2		0.3103
$TCol = e^{b_0(AADT)^{b_1}(LSeg)^{b_2}}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		113.1943
b ₀	-0.5897	0.2485	Pearson chi-square (χ^2), [max]		60.4336, [114.22]
b ₁	1.1347	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.2439
b ₂	0.6053	<0.0001		χ^2/DF	0.6641
Dispersion (α)	0.6806		R_α^2		0.4296
$TCol = e^{b_0(Expo)^{b_1}}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		112.2327
b ₀	0.0984	0.8252	Pearson chi-square (χ^2), [max]		69.0843, [115.34]
b ₁	0.8473	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.2199
Dispersion (α)	0.7603			χ^2/DF	0.7509
			R_α^2		0.3628

The third approach that was used to involve traffic exposure depended on calculating the natural logarithm of the two variables; $AADT$ and $LSeg$. The reason behind this transformation is to force the collision frequency to equal zero if either $AADT$ or $LSeg$ is also equal to zero. In this case, all regression coefficients were found

significant at 5 % level of significance except the intercept which had a p -value of 0.2485. The over-dispersion parameters based on D^m and χ^2 were 1.2439 and 0.6641, respectively, and were outside the accepted range. On the other hand, the R_a^2 value was lower than the first approach but generally acceptable. In addition, the direction of the relationships was found acceptable with positive signs. It was concluded finally that the model may not provide satisfactory predictive capability due to the insignificance of the intercept.

Finally, the fourth approach suggested including the transformed single exposure term. The positive sign for the exposure term was acceptable as the case with the first three approaches. In addition, the model had a relatively satisfactory R_a^2 value of 0.3628. Moreover, the estimated coefficient of *Expo* had a p -value that is less than 0.05. On the other side, the over-dispersion parameters for this model for D^m and χ^2 were 1.2199 and 0.7509, respectively, that are slightly outside the acceptable range. However, the intercept p -value of 0.8252 was found extremely high and indicates that the model was statistically unacceptable.

Based on the observed results of the four exposure approaches, the third and fourth ones were rejected due to the insignificance of the intercepts. Accordingly, the selection became limited to the first and second approaches. Although the R_a^2 value for the first approach (0.4883) is higher than that for the second approach (0.3103), the second approach was accepted because of the more reasonable values of the over-dispersion parameters based on D^m and χ^2 (1.2515, 0.6506 for the first and 1.2139, 0.7954 for the second). Subsequently, it was concluded that using untransformed single

exposure parameter provided superior results and was therefore incorporated in all models. However, the suggested exponential relationship between the traffic exposure and collision frequency should be used with caution since having a zero traffic exposure would still yield a non-zero number of collisions which is not logical. Therefore, the developed models should be limited to traffic exposures ranging from 6.22 to 199.12 million-vehicle-kilometre which is the range available in the database for model development.

7.1.4 Significant Combinations

After identifying the best regression technique (negative binomial), the suitable collision set (*TCol*), and the appropriate exposure approach (*Expo*), explanatory variables were then tested along with the exposure term in different modelling attempts. A total of eleven explanatory variables were evaluated, which are *NV*, *XV*, *NXV*, *LAcc*, *AccCo*, *LDec*, *DecCo*, *NR1*, *NR2*, *NSeg*, *AdvSp*. The methodology that was followed to incorporate the explanatory variables consisted of three stages; (1) *Stage I*: models that included individually a single variable in addition to *Expo*; (2) *Stage II*: models that contained different combinations of two variables in addition to *Expo*; and (3) *Stage III*: models with different combinations of more than two variables in addition to *Expo*. In the three stages, the inclusion of two explanatory variables was contingent on another variable being included. Namely, *AccCo* (code of acceleration lane) and *DecCo* (code of deceleration lane) could only be included in a model if *LAcc* (length of acceleration lane) and *LDec* (length of deceleration lane) were also included, respectively. The SAS output

for the models with significant combinations is shown in Appendix C and the results are demonstrated in the coming discussion.

In *Stage I*, nine models were attempted through the inclusion of individual variables and *Expo*. The results of the nine models indicated that six models were statistically significant, while the other three were insignificant. The six models relate *NV*, *XV*, *NXV*, *NSeg*, *LAcc&AccCo*, and *LDec&DecCo* to the *TCol* with values of D^m and χ^2 that were less than the maximum values, as the case with all other significant models in this study. Although the first three models that contained the three traffic volumes *NV*, *XV*, and *NXV* were significant at 5 % level of significance, their results indicated more reasonable values of the over-dispersion parameters (τ) associated with *NXV* of 1.2248 and 0.9186 compared to 1.2304 and 0.8380 for *NV* and 1.3047 and 0.6136 for *XV*. Besides, the inclusion of the *NXV* takes into account the effect of both entering and exiting traffic volume on segment collisions. This model also had a coefficient of determination R_a^2 of 0.4183, as shown in Table 7.4. Moreover, the direction of the relationship of the three volumes was found as expected with a positive sign which indicates increasing collisions with increasing volumes.

Table 7.4 also shows the two models that described the effect of the type and length of both acceleration and deceleration lane on safety performance. For the acceleration lane model, only one length out of 57 could not be identified due to the lack of data. Therefore, a total of 56 segments were involved in producing a significant model at 5 % level of significance. In addition, the values of D^m and χ^2 of 66.1740 and 44.6212 were less than the maximum value of 69.816. Moreover, the R_a^2 value of 0.5816 reflects

the ability of the type and length of the acceleration lane to capture 58.16 % of the variation in collisions between different freeway segments.

Table 7.4: Significant Models Resulting from Individual Explanatory Variables.

$TCol = e^{(b_0 + b_1(Expo) + b_2(NXV))}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		72.2606
b_0	2.1442	<0.0001	Pearson chi-square (χ^2), [max]		54.1991, [77.922]
b_1	0.0107	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.2248
b_2	0.0005	0.0020		χ^2/DF	0.9186
Dispersion (α)	0.6983		R_α^2		0.4183
$TCol = e^{(b_0 + b_1(Expo) + b_2(LAcc) + b_3(AccCo))}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		66.1740
b_0	2.5389	<0.0001	Pearson chi-square (χ^2), [max]		44.6212, [69.816]
b_1	0.0168	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.2726
b_2	-0.0020	0.0009		χ^2/DF	0.8581
b_3	1.6269	<0.0001	R_α^2		0.5816
Dispersion (α)	0.5520				
$TCol = e^{(b_0 + b_1(Expo) + b_2(LDec) + b_3(DecCo))}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		45.7752
b_0	2.7238	<0.0001	Pearson chi-square (χ^2), [max]		24.0219, [47.367]
b_1	0.0136	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.3871
b_2	-0.0015	0.0148		χ^2/DF	0.7279
b_3	1.2143	0.0011	R_α^2		0.4123
Dispersion (α)	0.4142				

As discussed previously, increasing the length of the acceleration lane should give the road users more time to take and execute right decisions in merge areas. The direction of the relationship between the length of the acceleration lane and collision frequency in the developed model supports the advantage of increasing the length in improving the

overall safety. In addition, the positive sign of the *AccCo* coefficient indicates that, for the same length of acceleration lane, acceleration lanes with limited length would provide safer operating conditions than the extended ones. This might be due to the interaction between the merge area at the entrance and the next merge or diverge area at the next ramp which would negatively affect the driver behaviour. Furthermore, drivers might have a false impression that the number of lanes has been increased and that would encourage them to continue occupying the auxiliary lane until the next ramp is reached and then be forced to make the merge manoeuvres quickly.

On the other hand, in order to quantify the effect of the type and length of the deceleration lane, a total of 39 segments were involved in the fourth model. It should be noted that 2 segments out of the 39 had the deceleration lane length missing and hence were excluded. Only 37 segments were modelled and resulted in a significant relationship at 5% level of significance. The values of D^m and χ^2 of 45.7752 and 24.0219 were less than the maximum value of 47.367. Moreover, the coefficient of determination R_a^2 of 0.4123 was also acceptable. Similar to the model of acceleration lane, the direction of the relationship reflects the safety improvement resulting from increasing the length of the deceleration lane having it separate from the preceding acceleration lane. This would help separate each two successive operations of either merge or diverge and provide enough time to drivers to make both manoeuvres properly.

Finally, the last model in *Stage I* described the effect of the number of basic lanes (*NSeg*) on safety performance. The model had *p*-values that were less than 0.05. The direction of the relationship indicated safer operational conditions with decreasing *NSeg*

which would be interpreted that the number of lane changes on a specific segment normally decreases with relatively low number of *NSeg*.

In *Stage II*, the collision models included different combinations of two variables along with the exposure variable. Modelling attempts included trying all possible combinations of nine variables out of the eleven variables besides including the *AccCo* and the *DecCo* along with *LAcc* and *LDec*, respectively in all models. A total of 36 models were generated and evaluated in this stage. However, only three models were found statistically significant at 5 % level of significance. The first two models included the combination of both the *AdvSp* and each of two volumes *XV* and *NXV*. Although the two models were statistically significant, the model based on *NXV* was preferred since it accommodates the effect of both entering and exiting volumes.

Table 7.5 shows the model that relates the *AdvSp* and the *NXV* to the collision frequency with R_{α}^2 value equal to 0.4462. The values of D^m and χ^2 of 47.7475 and 24.7148, respectively, were less than the maximum value of 49.765. In addition, the direction of the relationship between *NXV* and collision frequency was as obtained before as a direct proportional relationship. For the *AdvSp*, a negative sign was also observed which can be interpreted as that increasing the speed of the vehicles at the exit means less reduction in the speeds from the freeway to the exit. With reducing this drop in speed between the freeway and the exit, vehicles could smoothly exit the freeway without disturbing the through traffic, even if the length of the deceleration lane is relatively short. It was concluded from this model that providing exits that can safely accommodate vehicles with relatively higher speeds would help improve the safety performance of the preceding diverge areas.

Table 7.5: Significant Combinations of Two Explanatory Variables.

$TCol = e^{(b_0+b_1(Expo)+b_2(NXV)+b_3(AdvSp))}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		47.7475
b ₀	3.5979	<0.0001	Pearson chi-square (χ^2), [max]		24.7148, [49.765]
b ₁	0.0082	0.0004	Over-dispersion parameter (τ)	D^m/DF	1.3642
b ₂	0.0004	0.0048		χ^2/DF	0.7061
b ₃	-0.0194	0.0429	R_α^2		0.4462
Dispersion (α)	0.4016				
$TCol = e^{(b_0+b_1(Expo)+b_2(NXV)+b_3(NSeg))}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		72.6645
b ₀	0.8593	0.0465	Pearson chi-square (χ^2), [max]		57.9263, [76.764]
b ₁	0.0104	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.2528
b ₂	0.0003	0.0153		χ^2/DF	0.9987
b ₃	0.5062	0.0011	R_α^2		0.5170
Dispersion (α)	0.5799				

Table 7.5 also shows the third significant model in *Stage II* that related both the NXV and $NSeg$ to collision frequency. The model had values of D^m and χ^2 of 72.6645 and 57.9263 that are less than the maximum value of 76.764 with relatively high R_a^2 of 0.5170. In addition, the direction of the relationships between collision frequency and both NXV and $NSeg$ were logically acceptable as obtained previously in *Stage I*.

In *Stage III*, due to the huge number of all trials, a total of 20 combinations were selected based on the results of *Stage I* and *Stage II*. The 20 models that are shown in Table 7.6 were generated and evaluated with considering few aspects in the selection. Among them, some variables had to be compiled in the same model, such as the $LAcc$ and $LDec$ with their codes; $AccCo$ and $DecCo$ respectively. In addition, combining NXV and either NV or XV was rejected due to the correlation between them. However, all

suggested models were found insignificant at 95 % significance level. The finding might have resulted from the relatively low number of data points available for modelling.

Subsequently, no more combinations were tested and all other trials were assumed to be insignificant.

Table 7.6: Different Combinations of More Than Two Explanatory Variables.

Model	LAcc	AccCo	NR1	NV	NXV	NSeg	LDec	DecCo	NR2	XV	AdvSp
1	•	•	•				•	•			
2	•	•		•			•	•			
3	•	•			•		•	•			
4	•	•					•	•			
5	•	•				•	•	•			
6	•	•					•	•			
7	•	•					•	•	•		
8	•	•					•	•		•	
9	•	•					•	•			•
10	•	•	•	•							
11							•	•	•	•	
12							•	•		•	•
13							•	•	•		•
14	•	•			•		•	•			
15	•	•				•	•	•			
16	•	•			•	•	•	•			
17				•		•				•	
18				•		•					•
19						•				•	•
20				•		•				•	•

7.2 ANALYSIS OF WEAVING SEGMENTS

As mentioned earlier, a weaving segment consists of a merge area followed closely by a diverge area, thus experiencing an interaction between merge and diverge manoeuvres that would make the operational condition more complicated than other types of segments. To evaluate the effect of weaving manoeuvres on safety performance, total of 34 *entrance-exit* segments within the study area were isolated and analyzed. Although all these 34 segments were a subgroup of all segments analyzed in Section 7.1, the analysis of weaving segments only should shed more light on the relationship between the candidate explanatory variables and collision frequency on such segments that share unique and similar characteristics. The same methodology that was followed in the previous section to reduce the number of models was again followed in this section.

First, a total of 16 models (two regression techniques, two collision sets, and four traffic exposure approaches) were applied on the 34 segments similar to Section 7.1. The results were evaluated in order to select the appropriate regression technique, collision set, and traffic exposure. Then, different combinations of twelve explanatory variables, which were the eleven variables listed in Section 7.1 plus *WeaveType*, were then attempted. Section 7.2.1 shows briefly the results of the first 16 models and the rationale of the selection. Then, Section 7.2.2 presents several models that were developed using different combinations of the explanatory variables.

7.2.1 Regression Technique, Collision Set, and Exposure Selection

The over-dispersion parameters were found to range from 8.1492 to 20.9961 with Poisson regression and from 0.6442 to 1.4314 with negative binomial regression. These results indicated that Poisson regression was again extremely outside the recommended range of 0.8 to 1.2 and was therefore unable to capture the variation of the data. Alternatively, the negative binomial regression provided more reasonable values and hence was applied in all modelling attempts.

In order to select the appropriate collision set, the results of eight models including the two collision sets (*TCol* and *TEPDO*) and the four exposure approaches mentioned in Section 7.1 were compared. Similar to the case of modelling all segments, there was a small difference between the over-dispersion parameter of the two collision sets. However, slight improvement in the over-dispersion parameter was observed with using *TCol*. Therefore, *TCol* was considered more appropriate and was used with all models. Finally, the untransformed single exposure term (*Expo*) was found satisfactory in terms of the results of the over-dispersion parameter. The over-dispersion parameters based on D^m and χ^2 were 1.2775 and 0.6930, respectively, and were the closest values to the recommended range. Subsequently, it was concluded that the untransformed single exposure term was considered the best approach, and was included in all models.

7.2.2 Significant Combinations

After selecting the negative binomial, the *TCol* collision set, and the untransformed single exposure term to be used in the modelling attempts, all variables

were tried to reach the most significant models. First, all explanatory variables were modelled individually excluding *AccCo* and *DecCo* which were always combined with the *LAcc* and *LDec*, respectively, resulting in five significant models. Then, a total of 45 models were attempted by examining all possible combinations of two variables plus *Expo*, resulting in four significant models. Finally, twenty models of combining more than two variables (similar to those previously mentioned in Section 7.1.4) were analyzed by adding *WeaveType* to each of these models; however, no models were significant at this stage. All attempts of modelling weaving segments resulted in a total of nine significant models at 5 % level of significance (see Appendix D for SAS output).

Seven models out of the nine significant ones indicated similar trends to those corresponding models in Section 7.1.4, but with different regression coefficient. These seven models included the *NV*, *NXV*, *NSeg*, *LAcc&AccCo*, and *LDec&DecCo* individually, in addition to *Expo*, as well as the combination of *NXV&NSeg* and *XV&AdvSp*, in addition to *Expo*. Models number eight and nine were different from those obtained in Section 7.1.4 and contained the combination of *NV&NSeg*, in addition to *Expo*, and the combination of *AdvSp&WeaveType*, in addition to *Expo*.

From the first seven models, and because of the importance of the two models that contains the type and length of acceleration and deceleration lane as they could be controlled, these two models were re-presented in Table 7.7 with their goodness of fit measures. The first model in this table relates the length of acceleration lane with its code to collision frequency. The coefficient of determination R_{α}^2 had a value of 0.4297 which is relatively less than that for modelling all segments. However, this would be due to the relatively lower value of the number of degrees of freedom (53 for all segments and 29

for weaving segments). In addition, the values of D^m and χ^2 of 40.6712 and 22.1770 were less than the maximum value of 42.56. On the other hand, the second model relates the length and type of deceleration lane to collision frequency and also had D^m and χ^2 values of 39.8475 and 19.3142 that were less than the maximum value of 41.34. The coefficient of determination R_α^2 of 0.4179 was also acceptable.

Table 7.7: Significant Models from Modelling Attempts of Weaving Segments.

$TCol = e^{(b_0 + b_1(Expo) + b_2(LAcc) + b_3(AccCo))}$				
Parameters			Goodness of Fit	
Variable	Estimate	p-value	Scaled Deviance (D^m)	40.6712
b_0	2.7803	<0.0001	Pearson chi-square (χ^2), [max]	22.1770, [42.56]
b_1	0.0132	<0.0001	Over-dispersion parameter (τ)	D^m/DF 1.4025
b_2	-0.0014	0.0470		χ^2/DF 0.7647
b_3	1.2324	0.0033	R_α^2 0.4297	
Dispersion (α)	0.4728			
$TCol = e^{(b_0 + b_1(Expo) + b_2(LDec) + b_3(DecCo))}$				
Parameters			Goodness of Fit	
Variable	Estimate	p-value	Scaled Deviance (D^m)	39.8475
b_0	2.6870	<0.0001	Pearson chi-square (χ^2), [max]	19.3142, [41.34]
b_1	0.0140	<0.0001	Over-dispersion parameter (τ)	D^m/DF 1.4231
b_2	-0.0016	0.0069		χ^2/DF 0.6898
b_3	1.3760	0.0006	R_α^2 0.4179	
Dispersion (α)	0.4637			
$TCol = e^{(b_0 + b_1(Expo) + b_2(AdvSp) + b_3(WeaveType))}$				
Parameters			Goodness of Fit	
Variable	Estimate	p-value	Scaled Deviance (D^m)	42.0868
b_0	3.7594	<0.0001	Pearson chi-square (χ^2), [max]	24.3129, [46.98]
b_1	0.0102	<0.0001	Over-dispersion parameter (τ)	D^m/DF 1.4029
b_2	-0.0413	0.0003		χ^2/DF 0.8104
b_3	0.9827	0.0007	R_α^2 0.5142	
Dispersion (α)	0.3963			

The third model in Table 7.7 shows the effect of the advisory speed and the weaving type on collision frequency. The model had regression coefficients that are significant at 5 % level of significance and acceptable value of R_a^2 of 0.5142. In addition, the values of D^m and χ^2 of 42.0868 and 24.3129 were less than the maximum value of 46.98. The direction of the relationship between the *AdvSp* and collisions was similar to the one that was obtained before and was discussed in Section 7.1.4. The model also provides the effect of the weave type on safety performance. The positive sign of *WeaveType* indicated that weaving Type *A* would be associated with safer operational conditions than weaving Type *B* (see Section 2.3.2 for weaving types). This is due to the fact that, in weaving Type *B*, one of the two weaving volumes has a chance to exit the freeway while sharing the same lane with the through traffic. That could give a right to exiting volume to decelerate while still occupying the through lane of the freeway.

7.3 ANALYSIS OF ENTRANCES AND EXITS

As previously indicated in Section 6.4.1, four variables were suggested to represent the collisions experience at entrances and exits; that are *NCol* and *NEPDO* for entrances and *XCol* and *XPDO* for exits. For modelling attempts of collisions that occurred at entrances, all possible combinations of the two collision sets and four variables (*NV*, *LAcc*, *AccCo*, and *NRI*) were examined. Having *AccCo* contingent on *LAcc*, a total of 14 models were evaluated. Only one model, which relates the entrance volume to collision frequency, was found significant at the 5 % level of significance (see Appendix E for SAS output). The values of D^m and χ^2 of 58.8535 and 58.5938 were less

than the maximum value of 73.29. In addition, the direction of the relationship between the entering volume and collision frequency was logical and indicated increasing collisions with increasing entering volumes.

As for modelling attempts of collisions at exits, all possible combinations of two collision sets and five explanatory variables (XV , $LDec$, $DecCo$, $NR2$, and $AdvSp$) resulted in a total of 30 models. Appendix E shows the SAS output of the four significant models that resulted from all attempts. Two of these models related the XV to $XCol$ and $XEPDO$ and were considered logical. Two other models represented the relationship between the two explanatory variables XV and $AdvSp$ and the two collision sets $XCol$ and $XEPDO$. Although these two models were found significant at 5 % level of significance, the case of $XCol$ resulted in more reliable values of the over-dispersion parameters of 1.3037 and 0.7409 (see first model in Table 7.8). As mentioned previously, the direction of the relationship between the advisory speed and the collision frequency suggests that safer operational conditions are associated with higher exit speeds.

Table 7.8: Significant Models from Modelling Attempts of Entrances and Exits.

$XCol = e^{(b_0 + b_1(XV) + b_2(AdvSp))}$					
Parameters			Goodness of Fit		
Variable	Estimate	p-value	Scaled Deviance (D^m)		46.9320
b_0	3.3681	<0.0001	Pearson chi-square (χ^2), [max]		26.6722, [50.964]
b_1	0.0012	<0.0001	Over-dispersion parameter (τ)	D^m/DF	1.3037
b_2	-0.0427	0.0021		χ^2/DF	0.7409
Dispersion (α)	0.6343		R_α^2		0.4412

7.4 SUMMARY OF MODELLING ATTEMPTS

The previous sections discussed the effect of the traffic volume composition (NV , XV , NXV), the geometric features (type and length of speed change lanes), and other factors such as the number of lanes, the weaving type, and advisory speed on collision frequency. A total of 228 models were attempted, as shown in Table 7.9. All trials resulted in a total of 23 significant models. In these 23 models, and except $NR1$ and $NR2$, all the explanatory variables that were believed to affect the safety performance at merge and diverge areas were found significant at 5 % level of significance.

There are two specific significant models that were considered useful in improving the safety performance of freeways. These two models, as shown in Equation 7.1 and Equation 7.2, relate the length and type of speed change lanes to the five-year collision frequency. Compared to all other significant models, these two models have the unique characteristic that the explanatory variables, other than the traffic exposure, are design elements that can be controlled and managed to provide safer operational conditions.

$$TCol = e^{(2.5389+0.0168(Expo)-0.0020(LAcc)+1.6269(AccCo))} \quad \text{Equation 7.1}$$

$$TCol = e^{(2.7238+0.0136(Expo)-0.0015(LDec)+1.2143(DecCo))} \quad \text{Equation 7.2}$$

As mentioned earlier, the exponential relationship between the traffic exposure and collision frequency in these two models might result in non-zero collisions on a specific segment that has a zero traffic exposure, which is not logical. Therefore, the power form of the exposure along with the type and length of speed change lanes was again re-generated. For this purpose, the natural logarithm of $Expo$ as well as the natural

logarithm of both *LSeg* and AADT were tried resulting in four other models; two models for acceleration lanes and two models for deceleration lanes (see Appendix *F* for SAS output). The four models were found to have insignificant intercepts which led to a conclusion that generally the exponential form shown in Equation 7.1 and Equation 7.2 provides more reliable performance. However, using these two models should consider the limits of the traffic exposures of 6.22 to 199.12 as previously mentioned.

Table 7.9: A Summary of the Overall Models in the Study.

Category	No. of All Trials	No. of Significant Models	Description
Total Segments	28	NA	Selection of regression technique, collision set, and traffic exposure.
	9	6	Individual explanatory variables.
	36	3	Combinations of 2 explanatory variables.
	20	0	Combinations of more than 2 explanatory variables.
Weaving Segments	16	NA	Selection of regression technique, collision set, and traffic exposure.
	10	5	Individual explanatory variables.
	45	4	Combinations of 2 explanatory variables.
	20	0	Combinations of more than 2 explanatory variables.
Entrances	14	1	All possible combinations.
Exits	30	4	All possible combinations.
Total	228	23	

7.5 APPLICATION EXAMPLE OF DEVELOPED MODELS

As previously mentioned, the two models that involve the type and the length of speed change lanes for both acceleration and deceleration lanes should help designers evaluate the expected safety performance on freeways. This section presents the benefits of using the developed models as a tool in order to reduce the number of collisions especially on segments experience a high collision frequency. Bared et al. (1999) studied the expected reduction in collision frequency in terms of a specific increase in the speed change lanes. Their study included the three-year collision frequency that on ramps connected to the speed change lanes. They found that increasing the length of the speed change lane causes a reduction in collision frequency. Although the direction of the relationship between the length of the acceleration lane and collision frequency were logical, the collisions on on-ramps are most probably affected by the design of the ramp itself rather than the design of the acceleration lane.

As previously shown in this chapter, the developed models that describe the relationship between the length of speed change lanes and collision frequencies for five years had similar directions to those mentioned by Bared et al. (1999). In order to facilitate the task of safety-explicit design, two graphs were created as potential design aids. The first graph, as shown in Figure 7.1, summarizes the relationship between the type and length of acceleration lanes and five-year collision frequency for different levels of traffic exposure. It should be noted that a collision frequency obtained from this graph would represent the mean expected collision frequency at a specific location. The figure presents the trend of the relationship that indicates a general reduction in collision

frequency with increasing the length of the acceleration lanes for both types; extended and limited acceleration lanes.

Figure 7.1: Length of Acceleration Lane versus Five-Year Collisions.

It is obvious from the figure that, increasing the traffic exposure increases the probability of having more collisions. Furthermore, the graph shows the tremendous difference between collision frequencies of limited and extended segments for the same traffic exposure and the length of acceleration lane. For example, an acceleration lane with limited length of 400 m and traffic exposure of 25 million vehicle-km is expected to result in a total of 9 collisions, while an extended lane with the same length and traffic exposure might experience a higher number of 44 collisions. It is worth to mention here that the figure considered the limits of the lengths and traffic exposure that were used to develop the model. For example, the lengths of the acceleration lanes ranged from 150 to

488 m for limited length lanes and ranged from 180 to 1302 m for extended lanes.

Besides, the range of the traffic exposure was from 6.22 to 199.12 million vehicle-km.

The second graph, as shown in Figure 7.2, illustrates the relationship between the type and length of the deceleration lane and collision frequency of five years period with different levels of traffic exposure. Similar to the case of acceleration lanes, increasing the length of the lane indicates a notable improvement in safety performance and reduces the number of collisions. In addition, deceleration lanes with limited type are associated with safer performance compared to extended lanes with the same length. The graph also considered the concept of using valid limits of all measures. The two charts can be used as a tool to evaluate the expected safety improvement that would result from changing the length of a speed change lane.

Figure 7.2: Length of Deceleration Lane versus Five-Year Collisions.

In order to illustrate the benefits of these charts, a case study was selected within the study area which is located between IC 115 and IC 117 in the WB direction. The segment is of the extended type with a length of 807 m and a traffic exposure of 92.53 million vehicle-km. The estimated number of collisions was equal to 61 collisions using the chart of the acceleration and was equal to 54 using the chart of the deceleration lane. The average value of 58 collisions based on the two expected collision frequencies resulting from the two models was assumed to be the best representative of the expected collisions on a specific segment. It should be noted here that the segment experienced an actual number of 66 collisions during the five-year study period, from 1998 to 2002.

According to the developed models and corresponding graphs, using an acceleration lane that is separated from the deceleration lanes would help reduce the

number of collisions on this segment. It was assumed that an acceleration lane with length of 300 m and a deceleration lane with length of 200 m are appropriate and satisfy the average lengths of speed change lanes within the study area. In order to numerically support the suggested alternative, using a traffic exposure of 92.53, a length of 300, and a lane with limited type in Figure 7.1 results in 33 collisions while, using length of 200 and a lane with limited type in Figure 7.2 results in 40 collisions. The average of these two numbers, which is equal to 37 collisions, represents the expected number of collisions after the suggested modification. Obviously, there is a notable improvement in the safety performance of this segment associated with the expected reduction of the collision frequency from 58 to 37.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

The safety performance of freeways at merge and diverge areas is generally affected by a large number of factors that influence the driver behaviour in such areas. In order to develop a safety tool which is capable of capturing the variation in collision frequency for different merge and diverge areas, a total of twelve factors were examined. The selected study area for this research included 26 interchanges along Highway 417 with 57 merge areas and 39 diverge ones. The required geometric data, traffic volumes, and collision data were gathered and analyzed resulting in 23 statistically significant models out of a total of 228 attempts. The 23 significant models should help maximize the freeway safety and provide a useful tool for decision makers to reduce the number of collisions. The following are the main conclusions that can be drawn from the study:

- After rejecting the traditional linear regression techniques, the negative binomial was found superior to the Poisson in modelling collisions since it accommodated the over-dispersion in the collision data.
- The simple summation of all collisions provided more reliable models than using the equivalent property damage-only collision set.
- The results of collision modelling using four different exposure approaches indicated that the exponential relationship between a single exposure term and the collision frequency resulted in the best model fit and was hence included along with other combinations of explanatory variables in all developed models.

- Increasing the number of vehicles that enter and exit the freeway at a specific segment would cause an increase in the number of collisions on this segment.
- The general trend of the relationship between the length of the speed change lane and the collision frequency of the associated segment suggests decreasing number of collisions with increasing the length of speed change lane.
- Carrying the full width of the speed change to the gore of the following ramp might increase the number of collisions on the segment between these two ramps. Therefore, it is recommended to provide the speed change lane with a tapered portion at the end and beginning of acceleration and deceleration lanes, respectively.
- In cases that warrant an increase or decrease in the basic number of lanes to satisfy the capacity needs of the freeway, extending the speed change lane is not recommended. Alternatively, changing the number of lanes should be implemented within the basic section and away from the influence of speed change lanes.
- Off- ramps with design criteria that provide relatively high speeds for vehicles exiting the freeway would allow safer operational conditions compared to off-ramps with relatively lower speeds.
- The direction of the relationship between the weaving type and collision frequency indicates that weaving type *A* was associated with relatively lower collisions compared to weaving type *B*.

8.2 RECOMMENDATIONS AND FUTURE WORK

In order to enhance the obtained relationships between merge and diverge elements and the safety performance of freeways, and based on the findings of this research, the following is recommended.

- Including more data and increasing the number of segments would provide a wider variation of the design criteria and would therefore strengthen the developed models and improve the collision prediction.
- Obtaining more accurate geometric data of speed change lanes such as the length of the parallel and tapered portions and the actual width might help investigate the individual effect of such factors on safety performance.
- Using a data set with more accurate and detailed collision locations would be helpful to accurately locate the accident on the freeway segment and therefore isolate collisions within the influence of merge and diverge areas.
- Measuring the operating speeds instead of using the advisory ones for vehicles exiting the freeway will be more reliable. Besides, studying the acceleration and deceleration behaviour of vehicles occupying the speed change lanes should help strengthen the relationship between speeds and collision frequency.
- More research is required to evaluate the safety performance of freeways in terms of:
  - The spacing between the acceleration and deceleration lanes within weaving segments.

- The spacing between the physical nose at the gore of on-ramp and the end of a preceding deceleration lane if available.
- The spacing between the physical nose at the gore of off-ramp and the beginning of the following acceleration lane if available.

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**APPENDIX A: SAS OUTPUT: RESULTS OF 16 MODELS FOR
SELECTION OF REGRESSION TECHNIQUE, COLLISION SET,
AND TRAFFIC EXPOSURE.**

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Poisson						
Link Function	Log						
Dependent Variable	Tcol	Tcol					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	91	1052.0729	11.5612				
Scaled Deviance	91	1052.0729	11.5612				
Pearson Chi-Square	91	908.9784	9.9888				
Scaled Pearson X2	91	908.9784	9.9888				
Log Likelihood		8639.6784					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	1.0372	0.0769	0.8864	1.1880	181.78	<.0001
AADT	1	0.0392	0.0011	0.0371	0.0414	1252.49	<.0001
Lseg	1	0.5087	0.0202	0.4690	0.5483	631.52	<.0001
Scale	0	1.0000	0.0000	1.0000	1.0000		

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Poisson						
Link Function	Log						
Dependent Variable	Tcol	Tcol					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	91	1207.3472	13.2676				
Scaled Deviance	91	1207.3472	13.2676				
Pearson Chi-Square	91	1072.3764	11.7844				
Scaled Pearson X2	91	1072.3764	11.7844				
Log Likelihood		8562.0412					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	-1.2658	0.1642	-1.5876	-0.9440	59.44	<.0001
LNAADT	1	1.3193	0.0418	1.2374	1.4013	996.07	<.0001
LnLseg	1	0.6685	0.0292	0.6113	0.7257	525.17	<.0001
Scale	0	1.0000	0.0000	1.0000	1.0000		

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Poisson						
Link Function	Log						
Dependent Variable	Tcol	Tcol					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	92	1684.1215	18.3057				
Scaled Deviance	92	1684.1215	18.3057				
Pearson Chi-Square	92	1641.3079	17.8403				
Scaled Pearson X2	92	1641.3079	17.8403				
Log Likelihood		8323.6541					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.7430	0.0322	2.6798	2.8061	7253.98	<.0001
Expo	1	0.0107	0.0003	0.0101	0.0113	1200.67	<.0001
Scale	0	1.0000	0.0000	1.0000	1.0000		

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Poisson						
Link Function	Log						
Dependent Variable	Tcol	Tcol					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	92	1453.2687	15.7964				
Scaled Deviance	92	1453.2687	15.7964				
Pearson Chi-Square	92	1388.4295	15.0916				
Scaled Pearson X2	92	1388.4295	15.0916				
Log Likelihood		8439.0805					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	0.0793	0.1073	-0.1311	0.2896	0.55	0.4602
LnExpo	1	0.8530	0.0249	0.8043	0.9018	1175.63	<.0001
Scale	0	1.0000	0.0000	1.0000	1.0000		

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Poisson						
Link Function	Log						
Dependent Variable	TEPDO	TEPDO					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	91	2428.8348	26.6905				
Scaled Deviance	91	2428.8348	26.6905				
Pearson Chi-Square	91	2132.2126	23.4309				
Scaled Pearson X2	91	2132.2126	23.4309				
Log Likelihood		22466.6539					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	1.8933	0.0522	1.7909	1.9956	1315.50	<.0001
AADT	1	0.0374	0.0008	0.0359	0.0388	2433.84	<.0001
Lseg	1	0.4871	0.0140	0.4597	0.5146	1210.69	<.0001
Scale	0	1.0000	0.0000	1.0000	1.0000		

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Poisson						
Link Function	Log						
Dependent Variable	TEPDO	TEPDO					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	91	2758.7298	30.3157				
Scaled Deviance	91	2758.7298	30.3157				
Pearson Chi-Square	91	2592.6992	28.4912				
Scaled Pearson X2	91	2592.6992	28.4912				
Log Likelihood		22301.7065					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	-0.2612	0.1113	-0.4794	-0.0430	5.51	0.0189
LNAADT	1	1.2468	0.0284	1.1911	1.3026	1923.26	<.0001
LnLseg	1	0.6374	0.0202	0.5979	0.6769	999.75	<.0001
Scale	0	1.0000	0.0000	1.0000	1.0000		

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Poisson						
Link Function	Log						
Dependent Variable	TEPDO	TEPDO					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	92	3706.1244	40.2840				
Scaled Deviance	92	3706.1244	40.2840				
Pearson Chi-Square	92	3744.8373	40.7048				
Scaled Pearson X2	92	3744.8373	40.7048				
Log Likelihood		21828.0091					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	3.5141	0.0222	3.4707	3.5576	25079.6	<.0001
Expo	1	0.0102	0.0002	0.0098	0.0106	2226.65	<.0001
Scale	0	1.0000	0.0000	1.0000	1.0000		

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Poisson						
Link Function	Log						
Dependent Variable	TEPDO	TEPDO					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	92	3214.2794	34.9378				
Scaled Deviance	92	3214.2794	34.9378				
Pearson Chi-Square	92	3239.7032	35.2142				
Scaled Pearson X2	92	3239.7032	35.2142				
Log Likelihood		22073.9316					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	0.9676	0.0737	0.8231	1.1121	172.30	<.0001
LnExpo	1	0.8153	0.0172	0.7817	0.8489	2256.62	<.0001
Scale	0	1.0000	0.0000	1.0000	1.0000		

The GENMOD Procedure						
Model Information						
Data Set	WORK.T					
Distribution	Negative Binomial					
Link Function	Log					
Dependent Variable	Tcol					
Observations Used	94					
Criteria For Assessing Goodness Of Fit						
Criterion	DF	Value	Value/DF			
Deviance	91	113.8873	1.2515			
Scaled Deviance	91	113.8873	1.2515			
Pearson Chi-Square	91	59.2026	0.6506			
Scaled Pearson X2	91	59.2026	0.6506			
Log Likelihood		8986.3663				
Algorithm converged.						
Analysis Of Parameter Estimates						
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	1.2663	0.2295	0.8166 1.7160	30.45	<.0001
AADT	1	0.0348	0.0038	0.0273 0.0424	82.19	<.0001
Lseg	1	0.5059	0.1168	0.2770 0.7347	18.77	<.0001
Dispersion	1	0.6106	0.1029	0.4388 0.8496		

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Negative Binomial						
Link Function	Log						
Dependent Variable	Tcol	Tcol					
Observations Used	94						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	91	113.1943	1.2439				
Scaled Deviance	91	113.1943	1.2439				
Pearson Chi-Square	91	60.4336	0.6641				
Scaled Pearson X2	91	60.4336	0.6641				
Log Likelihood		8982.0094					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	-0.5897	0.5110	-1.5913	0.4118	1.33	0.2485
LNAADT	1	1.1347	0.1399	0.8605	1.4088	65.80	<.0001
LnLseg	1	0.6053	0.1286	0.3531	0.8574	22.14	<.0001
Dispersion	1	0.6806	0.1116	0.4935	0.9386		

The GENMOD Procedure						
Model Information						
Data Set		WORK.T				
Distribution		Negative Binomial				
Link Function		Log				
Dependent Variable		Tcol	Tcol			
Observations Used		94				
Criteria For Assessing Goodness Of Fit						
Criterion	DF	Value	Value/DF			
Deviance	92	111.6786	1.2139			
Scaled Deviance	92	111.6786	1.2139			
Pearson Chi-Square	92	73.1762	0.7954			
Scaled Pearson X2	92	73.1762	0.7954			
Log Likelihood		8974.3207				
Algorithm converged.						
Analysis Of Parameter Estimates						
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.4883	0.1724	2.1504 2.8261	208.40	<.0001
Expo	1	0.0143	0.0025	0.0095 0.0191	33.88	<.0001
Dispersion	1	0.8230	0.1280	0.6068 1.1161		

The GENMOD Procedure						
Model Information						
Data Set		WORK.T				
Distribution		Negative Binomial				
Link Function		Log				
Dependent Variable		Tcol	Tcol			
Observations Used		94				
Criteria For Assessing Goodness Of Fit						
Criterion	DF	Value	Value/DF			
Deviance	92	112.2327	1.2199			
Scaled Deviance	92	112.2327	1.2199			
Pearson Chi-Square	92	69.0843	0.7509			
Scaled Pearson X2	92	69.0843	0.7509			
Log Likelihood		8977.6003				
Algorithm converged.						
Analysis Of Parameter Estimates						
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	0.0984	0.4458	-0.7753 0.9721	0.05	0.8252
LnExpo	1	0.8473	0.1151	0.6217 1.0730	54.16	<.0001
Dispersion	1	0.7603	0.1208	0.5569 1.0381		

The GENMOD Procedure						
Model Information						
Data Set		WORK.T				
Distribution	Negative Binomial					
Link Function	Log					
Dependent Variable	TEPDO	TEPDO				
Observations Used	94					
Criteria For Assessing Goodness Of Fit						
Criterion	DF	Value	Value/DF			
Deviance	91	113.2935	1.2450			
Scaled Deviance	91	113.2935	1.2450			
Pearson Chi-Square	91	50.0182	0.5497			
Scaled Pearson X2	91	50.0182	0.5497			
Log Likelihood		23458.2355				
Algorithm converged.						
Analysis Of Parameter Estimates						
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.1139	0.2584	1.6074 2.6205	66.91	<.0001
AADT	1	0.0331	0.0044	0.0245 0.0416	57.36	<.0001
Lseg	1	0.4851	0.1360	0.2186 0.7516	12.73	0.0004
Dispersion	1	0.8621	0.1314	0.6394 1.1622		

The GENMOD Procedure						
Model Information						
Data Set	WORK.T					
Distribution	Negative Binomial					
Link Function	Log					
Dependent Variable	TEPDO	TEPDO				
Observations Used	94					
Criteria For Assessing Goodness Of Fit						
Criterion	DF	Value	Value/DF			
Deviance	91	113.0303	1.2421			
Scaled Deviance	91	113.0303	1.2421			
Pearson Chi-Square	91	55.0858	0.6053			
Scaled Pearson X2	91	55.0858	0.6053			
Log Likelihood		23454.7731				
Algorithm converged.						
Analysis Of Parameter Estimates						
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	0.4637	0.5610	-0.6358 1.5633	0.68	0.4085
LNAADT	1	1.0499	0.1540	0.7481 1.3517	46.48	<.0001
LnLseg	1	0.5856	0.1493	0.2931 0.8782	15.40	<.0001
Dispersion	1	0.9304	0.1395	0.6936 1.2482		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial	Log			
Link Function				TEPDO	TEPDO		
Dependent Variable				94			
Observations Used							
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	92		112.6266			1.2242	
Scaled Deviance	92		112.6266			1.2242	
Pearson Chi-Square	92		69.8876			0.7596	
Scaled Pearson X2	92		69.8876			0.7596	
Log Likelihood			23449.4648				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	3.2550	0.1921	2.8784	3.6316	287.02	<.0001
Expo	1	0.0139	0.0028	0.0085	0.0193	25.24	<.0001
Dispersion	1	1.0441	0.1524	0.7843	1.3899		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial	Log			
Link Function				TEPDO	TEPDO		
Dependent Variable				94			
Observations Used							
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	92		112.7831			1.2259	
Scaled Deviance	92		112.7831			1.2259	
Pearson Chi-Square	92		64.1885			0.6977	
Scaled Pearson X2	92		64.1885			0.6977	
Log Likelihood			23452.1072				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	0.9705	0.4905	0.0092	1.9319	3.92	0.0478
LnExpo	1	0.8135	0.1273	0.5641	1.0630	40.86	<.0001
Dispersion	1	0.9866	0.1459	0.7384	1.3182		

**APPENDIX B: SAS OUTPUT: RESULTS OF 12 MODELS FOR
SELECTION OF COLLISION SET.**

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			57				
Missing Values			37				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	55		66.8168			1.2149	
Scaled Deviance	55		66.8168			1.2149	
Pearson Chi-Square	55		43.5007			0.7909	
Scaled Pearson X2	55		43.5007			0.7909	
Log Likelihood			6450.8756				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.9468	0.2419	2.4727	3.4209	148.42	<.0001
NV	1	0.0011	0.0004	0.0004	0.0018	8.75	0.0031
Dispersion	1	1.1213	0.2054	0.7831	1.6057		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			39				
Missing Values			55				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	37		46.2053			1.2488	
Scaled Deviance	37		46.2053			1.2488	
Pearson Chi-Square	37		24.2047			0.6542	
Scaled Pearson X2	37		24.2047			0.6542	
Log Likelihood			6251.9186				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	3.6501	0.2425	3.1748	4.1254	226.56	<.0001
XV	1	0.0003	0.0003	-0.0002	0.0008	1.76	0.1845
Dispersion	1	0.6917	0.1615	0.4377	1.0930		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			62				
Missing Values			32				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	60		72.2165			1.2036	
Scaled Deviance	60		72.2165			1.2036	
Pearson Chi-Square	60		47.7030			0.7951	
Scaled Pearson X2	60		47.7030			0.7951	
Log Likelihood			7046.5904				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	2.7455	0.2312	2.2924	3.1986	141.05	<.0001
NXV	1	0.0008	0.0002	0.0004	0.0011	14.80	0.0001
Dispersion	1	0.9371	0.1694	0.6576	1.3354		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			56				
Missing Values			1				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	53		65.8179			1.2418	
Scaled Deviance	53		65.8179			1.2418	
Pearson Chi-Square	53		45.2229			0.8533	
Scaled Pearson X2	53		45.2229			0.8533	
Log Likelihood			6399.3371				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	3.5566	0.3238	2.9219	4.1912	120.64	<.0001
Lacc	1	-0.0007	0.0008	-0.0023	0.0010	0.64	0.4241
AccCo	1	1.0958	0.5041	0.1077	2.0839	4.72	0.0297
Dispersion	1	1.1818	0.2165	0.8252	1.6923		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			37				
Missing Values			2				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	34		44.0301			1.2950	
Scaled Deviance	34		44.0301			1.2950	
Pearson Chi-Square	34		20.8697			0.6138	
Scaled Pearson X2	34		20.8697			0.6138	
Log Likelihood			6198.4677				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	3.9435	0.2633	3.4274	4.4596	224.30	<.0001
Ldec	1	-0.0001	0.0007	-0.0015	0.0013	0.02	0.8861
DecCo	1	0.1655	0.4277	-0.6728	1.0038	0.15	0.6988
Dispersion	1	0.7008	0.1682	0.4379	1.1217		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			39				
Missing Values			55				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	37		46.2152			1.2491	
Scaled Deviance	37		46.2152			1.2491	
Pearson Chi-Square	37		25.3685			0.6856	
Scaled Pearson X2	37		25.3685			0.6856	
Log Likelihood			6252.1135				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	4.8888	0.6535	3.6080	6.1696	55.97	<.0001
AdvSp	1	-0.0184	0.0121	-0.0420	0.0053	2.32	0.1275
Dispersion	1	0.6847	0.1601	0.4329	1.0828		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial	Log			
Link Function				TEPDO	TEPDO		
Dependent Variable				57			
Observations Used				37			
Missing Values							
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	55		67.8907			1.2344	
Scaled Deviance	55		67.8907			1.2344	
Pearson Chi-Square	55		38.5042			0.7001	
Scaled Pearson X2	55		38.5042			0.7001	
Log Likelihood			16177.0202				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	3.6566	0.2591	3.1487	4.1645	199.12	<.0001
NV	1	0.0011	0.0004	0.0003	0.0018	7.49	0.0062
Dispersion	1	1.2970	0.2291	0.9175	1.8335		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial	Log			
Link Function				TEPDO	TEPDO		
Dependent Variable				39			
Observations Used				55			
Missing Values							
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	37		46.3638			1.2531	
Scaled Deviance	37		46.3638			1.2531	
Pearson Chi-Square	37		21.7349			0.5874	
Scaled Pearson X2	37		21.7349			0.5874	
Log Likelihood			15159.0997				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	4.4039	0.2514	3.9113	4.8966	306.95	<.0001
XV	1	0.0003	0.0003	-0.0003	0.0008	0.97	0.3244
Dispersion	1	0.7358	0.1666	0.4722	1.1467		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial	Log			
Link Function				TEPDO	TEPDO		
Dependent Variable				62			
Observations Used				32			
Missing Values							
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	60		73.3521			1.2225	
Scaled Deviance	60		73.3521			1.2225	
Pearson Chi-Square	60		44.3137			0.7386	
Scaled Pearson X2	60		44.3137			0.7386	
Log Likelihood			17638.9324				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	3.4692	0.2570	2.9654	3.9730	182.18	<.0001
NXV	1	0.0007	0.0002	0.0003	0.0012	11.38	0.0007
Dispersion	1	1.1359	0.1956	0.8105	1.5919		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial	Log			
Link Function				TEPDO	TEPDO		
Dependent Variable				56			
Observations Used				1			
Missing Values							
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	53		66.8845			1.2620	
Scaled Deviance	53		66.8845			1.2620	
Pearson Chi-Square	53		38.9299			0.7345	
Scaled Pearson X2	53		38.9299			0.7345	
Log Likelihood			15996.7786				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	4.2306	0.3369	3.5703	4.8909	157.70	<.0001
Lacc	1	-0.0006	0.0009	-0.0022	0.0011	0.43	0.5115
AccCo	1	1.0408	0.5255	0.0108	2.0709	3.92	0.0476
Dispersion	1	1.3607	0.2408	0.9620	1.9248		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			TEPDO		TEPDO		
Observations Used			37				
Missing Values			2				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	34		44.1833			1.2995	
Scaled Deviance	34		44.1833			1.2995	
Pearson Chi-Square	34		18.4830			0.5436	
Scaled Pearson X2	34		18.4830			0.5436	
Log Likelihood			15006.6954				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	4.5897	0.2621	4.0760	5.1033	306.72	<.0001
Ldec	1	0.0001	0.0007	-0.0013	0.0015	0.02	0.9000
DecCo	1	0.0587	0.4343	-0.7926	0.9099	0.02	0.8926
Dispersion	1	0.7245	0.1695	0.4581	1.1459		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			TEPDO		TEPDO		
Observations Used			39				
Missing Values			55				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	37		46.3980			1.2540	
Scaled Deviance	37		46.3980			1.2540	
Pearson Chi-Square	37		21.7026			0.5866	
Scaled Pearson X2	37		21.7026			0.5866	
Log Likelihood			15159.7989				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	5.6487	0.6724	4.3307	6.9666	70.57	<.0001
AdvSp	1	-0.0198	0.0124	-0.0441	0.0045	2.54	0.1106
Dispersion	1	0.7102	0.1617	0.4545	1.1097		

**APPENDIX C: SAS OUTPUT: RESULTS OF 9 SIGNIFICANT
MODELS OF ALL SEGMENTS.**

The GENMOD Procedure

Model Information

Data Set	WORK.T
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	57
Missing Values	37

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	54	66.4413	1.2304
Scaled Deviance	54	66.4413	1.2304
Pearson Chi-Square	54	45.2539	0.8380
Scaled Pearson X2	54	45.2539	0.8380
Log Likelihood		6463.7051	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	1.9004	0.2480	1.4143	2.3864	58.73	<.0001
Expo	1	0.0133	0.0025	0.0083	0.0183	27.51	<.0001
NV	1	0.0009	0.0003	0.0004	0.0014	10.43	0.0012
Dispersion	1	0.7148	0.1447	0.4807	1.0630		

The GENMOD Procedure

Model Information

Data Set	WORK.T
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	39
Missing Values	55

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	36	46.9686	1.3047
Scaled Deviance	36	46.9686	1.3047
Pearson Chi-Square	36	22.0912	0.6136
Scaled Pearson X2	36	22.0912	0.6136
Log Likelihood		6258.2242	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.5885	0.3249	1.9517	3.2252	63.48	<.0001
Expo	1	0.0095	0.0025	0.0046	0.0144	14.44	0.0001
XV	1	0.0005	0.0002	0.0000	0.0009	4.22	0.0399
Dispersion	1	0.4879	0.1220	0.2988	0.7965		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial	Log			
Link Function				Tcol		Tcol	
Dependent Variable							
Observations Used				62			
Missing Values				32			
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	59		72.2606			1.2248	
Scaled Deviance	59		72.2606			1.2248	
Pearson Chi-Square	59		54.1991			0.9186	
Scaled Pearson X2	59		54.1991			0.9186	
Log Likelihood			7055.4739				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.1442	0.2237	1.7058	2.5825	91.91	<.0001
Expo	1	0.0107	0.0025	0.0057	0.0157	17.71	<.0001
NXV	1	0.0005	0.0002	0.0002	0.0008	9.51	0.0020
Dispersion	1	0.6983	0.1350	0.4781	1.0199		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial	Log			
Link Function				Tcol		Tcol	
Dependent Variable							
Observations Used				94			
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	91		112.5503			1.2368	
Scaled Deviance	91		112.5503			1.2368	
Pearson Chi-Square	91		73.3726			0.8063	
Scaled Pearson X2	91		73.3726			0.8063	
Log Likelihood			8981.5436				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	1.1256	0.3728	0.3950	1.8563	9.12	0.0025
Expo	1	0.0114	0.0022	0.0072	0.0157	27.62	<.0001
Nseg	1	0.5385	0.1380	0.2680	0.8090	15.23	<.0001
Dispersion	1	0.6930	0.1125	0.5042	0.9525		

The GENMOD Procedure

Model Information

Data Set	WORK.T
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	56
Missing Values	1

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	52	66.1740	1.2726
Scaled Deviance	52	66.1740	1.2726
Pearson Chi-Square	52	44.6212	0.8581
Scaled Pearson X2	52	44.6212	0.8581
Log Likelihood		6419.7277	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.5389	0.2503	2.0484 3.0295	102.90	<.0001
Expo	1	0.0168	0.0024	0.0120 0.0215	47.99	<.0001
Lacc	1	-0.0020	0.0006	-0.0032 -0.0008	10.93	0.0009
AccCo	1	1.6269	0.3306	0.9790 2.2748	24.22	<.0001
Dispersion	1	0.5520	0.1211	0.3590 0.8485		

The GENMOD Procedure

Model Information

Data Set	WORK.T
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	37
Missing Values	2

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	33	45.7752	1.3871
Scaled Deviance	33	45.7752	1.3871
Pearson Chi-Square	33	24.0219	0.7279
Scaled Pearson X2	33	24.0219	0.7279
Log Likelihood		6207.0678	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.7238	0.3017	2.1324 3.3152	81.49	<.0001
Expo	1	0.0136	0.0029	0.0080 0.0193	22.29	<.0001
Ldec	1	-0.0015	0.0006	-0.0027 -0.0003	5.94	0.0148
DecCo	1	1.2143	0.3725	0.4842 1.9444	10.63	0.0011
Dispersion	1	0.4142	0.1114	0.2444 0.7017		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			39				
Missing Values			55				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	35		47.5919			1.3598	
Scaled Deviance	35		47.5919			1.3598	
Pearson Chi-Square	35		23.2109			0.6632	
Scaled Pearson X2	35		23.2109			0.6632	
Log Likelihood			6261.5991				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	3.9272	0.5772	2.7959	5.0584	46.30	<.0001
Expo	1	0.0090	0.0023	0.0045	0.0135	15.52	<.0001
XV	1	0.0006	0.0002	0.0002	0.0010	8.84	0.0029
AdvSp	1	-0.0273	0.0099	-0.0467	-0.0078	7.56	0.0060
Dispersion	1	0.3996	0.1040	0.2400	0.6654		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			39				
Missing Values			55				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	35		47.7475			1.3642	
Scaled Deviance	35		47.7475			1.3642	
Pearson Chi-Square	35		24.7148			0.7061	
Scaled Pearson X2	35		24.7148			0.7061	
Log Likelihood			6261.4296				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	3.5979	0.5780	2.4651	4.7307	38.75	<.0001
Expo	1	0.0082	0.0023	0.0037	0.0127	12.66	0.0004
NXV	1	0.0004	0.0001	0.0001	0.0006	7.94	0.0048
AdvSp	1	-0.0194	0.0096	-0.0382	-0.0006	4.10	0.0429
Dispersion	1	0.4016	0.1048	0.2408	0.6699		

The GENMOD Procedure

Model Information

Data Set	WORK.T
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol Tcol
Observations Used	62
Missing Values	32

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	58	72.6645	1.2528
Scaled Deviance	58	72.6645	1.2528
Pearson Chi-Square	58	57.9263	0.9987
Scaled Pearson X2	58	57.9263	0.9987
Log Likelihood		7060.6528	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	0.8593	0.4316	0.0134	1.7053	3.96	0.0465
Expo	1	0.0104	0.0022	0.0060	0.0148	21.42	<.0001
NXV	1	0.0003	0.0001	0.0001	0.0006	5.88	0.0153
Nseg	1	0.5062	0.1556	0.2013	0.8111	10.59	0.0011
Dispersion	1	0.5799	0.1172	0.3902	0.8619		

**APPENDIX D: SAS OUTPUT: RESULTS OF 9 SIGNIFICANT
MODELS OF WEAVING SEGMENTS.**

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol		Tcol		
Observations Used			34				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	31		41.7620			1.3472	
Scaled Deviance	31		41.7620			1.3472	
Pearson Chi-Square	31		21.6151			0.6973	
Scaled Pearson X2	31		21.6151			0.6973	
Log Likelihood			5666.7932				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.4246	0.3087	1.8195	3.0296	61.69	<.0001
Expo	1	0.0095	0.0026	0.0044	0.0146	13.21	0.0003
NV	1	0.0008	0.0003	0.0003	0.0013	8.35	0.0039
Dispersion	1	0.4597	0.1273	0.2672	0.7911		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol		Tcol		
Observations Used			34				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	31		41.6184			1.3425	
Scaled Deviance	31		41.6184			1.3425	
Pearson Chi-Square	31		19.2690			0.6216	
Scaled Pearson X2	31		19.2690			0.6216	
Log Likelihood			5666.3984				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.3039	0.3337	1.6498	2.9580	47.65	<.0001
Expo	1	0.0100	0.0026	0.0048	0.0151	14.46	0.0001
NXV	1	0.0004	0.0001	0.0001	0.0007	7.84	0.0051
Dispersion	1	0.4733	0.1301	0.2762	0.8112		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol		Tcol		
Observations Used			34				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	31		41.5432			1.3401	
Scaled Deviance	31		41.5432			1.3401	
Pearson Chi-Square	31		22.8450			0.7369	
Scaled Pearson X2	31		22.8450			0.7369	
Log Likelihood			5666.3352				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	1.3941	0.5810	0.2553	2.5329	5.76	0.0164
Expo	1	0.0090	0.0027	0.0037	0.0142	11.22	0.0008
Nseg	1	0.5544	0.1922	0.1777	0.9310	8.32	0.0039
Dispersion	1	0.4763	0.1305	0.2784	0.8149		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol		Tcol		
Observations Used			33				
Missing Values			1				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	29		40.6712			1.4025	
Scaled Deviance	29		40.6712			1.4025	
Pearson Chi-Square	29		22.1770			0.7647	
Scaled Pearson X2	29		22.1770			0.7647	
Log Likelihood			5615.9093				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.7803	0.3597	2.0753	3.4853	59.75	<.0001
Expo	1	0.0132	0.0029	0.0075	0.0190	20.43	<.0001
Lacc	1	-0.0014	0.0007	-0.0028	-0.0000	3.94	0.0470
AccCo	1	1.2324	0.4190	0.4111	2.0537	8.65	0.0033
Dispersion	1	0.4728	0.1330	0.2723	0.8206		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			32				
Missing Values			2				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	28		39.8475			1.4231	
Scaled Deviance	28		39.8475			1.4231	
Pearson Chi-Square	28		19.3142			0.6898	
Scaled Pearson X2	28		19.3142			0.6898	
Log Likelihood			5612.9294				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.6870	0.3574	1.9865	3.3874	56.53	<.0001
Expo	1	0.0140	0.0031	0.0079	0.0200	20.31	<.0001
Ldec	1	-0.0016	0.0007	-0.0030	-0.0002	5.28	0.0215
DecCo	1	1.3760	0.4509	0.4922	2.2597	9.31	0.0023
Dispersion	1	0.4637	0.1339	0.2633	0.8168		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			34				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	30		42.3733			1.4124	
Scaled Deviance	30		42.3733			1.4124	
Pearson Chi-Square	30		21.5996			0.7200	
Scaled Pearson X2	30		21.5996			0.7200	
Log Likelihood			5668.7854				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	1.5652	0.5082	0.5692	2.5612	9.49	0.0021
Expo	1	0.0089	0.0024	0.0041	0.0136	13.40	0.0003
NV	1	0.0006	0.0003	0.0001	0.0011	4.74	0.0295
Nseg	1	0.3678	0.1826	0.0100	0.7257	4.06	0.0440
Dispersion	1	0.3978	0.1138	0.2270	0.6971		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			34				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	30		42.2568			1.4086	
Scaled Deviance	30		42.2568			1.4086	
Pearson Chi-Square	30		19.6206			0.6540	
Scaled Pearson X2	30		19.6206			0.6540	
Log Likelihood			5668.3699				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	1.4844	0.5069	0.4909	2.4778	8.58	0.0034
Expo	1	0.0091	0.0024	0.0043	0.0139	13.85	0.0002
NXV	1	0.0003	0.0001	0.0000	0.0005	4.06	0.0438
Nseg	1	0.3727	0.1867	0.0068	0.7385	3.99	0.0459
Dispersion	1	0.4099	0.1166	0.2348	0.7159		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			34				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	30		41.7257			1.3909	
Scaled Deviance	30		41.7257			1.3909	
Pearson Chi-Square	30		19.7219			0.6574	
Scaled Pearson X2	30		19.7219			0.6574	
Log Likelihood			5667.3798				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	4.0115	0.6887	2.6616	5.3614	33.92	<.0001
Expo	1	0.0091	0.0026	0.0040	0.0142	12.23	0.0005
XV	1	0.0006	0.0002	0.0002	0.0010	7.45	0.0063
AdvSp	1	-0.0290	0.0112	-0.0510	-0.0070	6.68	0.0097
Dispersion	1	0.4437	0.1233	0.2573	0.7650		

The GENMOD Procedure

Model Information

Data Set	WORK.T
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	Tcol
Observations Used	34

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	30	42.0868	1.4029
Scaled Deviance	30	42.0868	1.4029
Pearson Chi-Square	30	24.3129	0.8104
Scaled Pearson X2	30	24.3129	0.8104
Log Likelihood		5668.9874	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	4.7421	0.6692	3.4304	6.0538	50.21	<.0001
Expo	1	0.0102	0.0024	0.0054	0.0150	17.45	<.0001
AdvSp	1	-0.0413	0.0115	-0.0638	-0.0187	12.87	0.0003
WeaveType	1	0.9827	0.2896	0.4152	1.5502	11.52	0.0007
Dispersion	1	0.3963	0.1126	0.2271	0.6916		

**APPENDIX E: SAS OUTPUT: RESULTS OF 5 MODELS OF
ENTRANCES AND EXITS.**

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			NEPDO		NEPDO		
Observations Used			57				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	55		58.8535			1.0701	
Scaled Deviance	55		58.8535			1.0701	
Pearson Chi-Square	55		58.5938			1.0653	
Scaled Pearson X2	55		58.5938			1.0653	
Log Likelihood			427.1498				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	0.8244	0.4042	0.0322	1.6165	4.16	0.0414
NV	1	0.0013	0.0006	0.0001	0.0026	4.78	0.0288
Dispersion	1	2.5905	0.5915	1.6559	4.0527		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Xcol		Xcol		
Observations Used			39				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	37		46.5234			1.2574	
Scaled Deviance	37		46.5234			1.2574	
Pearson Chi-Square	37		26.6281			0.7197	
Scaled Pearson X2	37		26.6281			0.7197	
Log Likelihood			610.9220				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	1.4002	0.2973	0.8174	1.9829	22.17	<.0001
XV	1	0.0010	0.0003	0.0003	0.0016	9.43	0.0021
Dispersion	1	0.8456	0.2370	0.4882	1.4646		

NOTE: The negative binomial dispersion parameter was estimated by maximum likelihood.

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Xcol	Xcol			
Observations Used			39				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	36		46.9320			1.3037	
Scaled Deviance	36		46.9320			1.3037	
Pearson Chi-Square	36		26.6722			0.7409	
Scaled Pearson X2	36		26.6722			0.7409	
Log Likelihood			615.1432				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	3.3681	0.6998	1.9965	4.7397	23.16	<.0001
XV	1	0.0012	0.0003	0.0007	0.0018	18.49	<.0001
AdvSp	1	-0.0427	0.0139	-0.0700	-0.0155	9.43	0.0021
Dispersion	1	0.6343	0.1926	0.3499	1.1500		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			XEPDO	XEPDO			
Observations Used			39				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	37		46.9205			1.2681	
Scaled Deviance	37		46.9205			1.2681	
Pearson Chi-Square	37		22.7190			0.6140	
Scaled Pearson X2	37		22.7190			0.6140	
Log Likelihood			1814.9603				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	2.2242	0.3337	1.5702	2.8782	44.43	<.0001
XV	1	0.0008	0.0004	0.0001	0.0015	5.55	0.0185
Dispersion	1	1.2160	0.3003	0.7494	1.9730		

The GENMOD Procedure

Model Information

Data Set	WORK.T
Distribution	Negative Binomial
Link Function	Log
Dependent Variable	XEPDO
Observations Used	39

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	36	47.2539	1.3126
Scaled Deviance	36	47.2539	1.3126
Pearson Chi-Square	36	22.6914	0.6303
Scaled Pearson X2	36	22.6914	0.6303
Log Likelihood		1817.2401	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	3.8094	0.7890	2.2631 5.3557	23.31	<.0001
XV	1	0.0010	0.0003	0.0004 0.0017	10.11	0.0015
AdvSp	1	-0.0342	0.0150	-0.0636 -0.0048	5.18	0.0228
Dispersion	1	1.0613	0.2718	0.6425 1.7531		

**APPENDIX F: SAS OUTPUT: RESULTS OF 4 MODELS USING
NATURAL LOGARITHM TRANSFORMATION FOR THE TRAFFIC
EXPOSURE.**

The GENMOD Procedure							
Model Information							
Data Set	WORK.T						
Distribution	Negative Binomial						
Link Function	Log						
Dependent Variable	Tcol	Tcol					
Observations Used	56						
Missing Values	1						
Criteria For Assessing Goodness Of Fit							
Criterion	DF	Value	Value/DF				
Deviance	52	67.0948	1.2903				
Scaled Deviance	52	67.0948	1.2903				
Pearson Chi-Square	52	42.5462	0.8182				
Scaled Pearson X2	52	42.5462	0.8182				
Log Likelihood		6423.8252					
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald	95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	-1.1464	0.5343	-2.1936	-0.0992	4.60	0.0319
LnExpo	1	1.2189	0.1396	0.9452	1.4925	76.22	<.0001
Lacc	1	-0.0020	0.0005	-0.0030	-0.0009	13.69	0.0002
AccCo	1	1.4288	0.2916	0.8573	2.0004	24.01	<.0001
Dispersion	1	0.4589	0.1060	0.2919	0.7217		

Model Information						
Data Set		WORK.T				
Distribution		Negative Binomial				
Link Function		Log				
Dependent Variable		Tcol	Tcol			
Observations Used		56				
Missing Values		1				
Criteria For Assessing Goodness Of Fit						
Criterion	DF	Value	Value/DF			
Deviance	51	67.4875	1.3233			
Scaled Deviance	51	67.4875	1.3233			
Pearson Chi-Square	51	41.0472	0.8048			
Scaled Pearson X2	51	41.0472	0.8048			
Log Likelihood		6424.5235				
Algorithm converged.						
Analysis Of Parameter Estimates						
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits	Chi-Square	Pr > ChiSq
Intercept	1	-1.1098	0.7349	-2.5502 0.3306	2.28	0.1310
LnLSeg	1	1.1154	0.1593	0.8032 1.4276	49.03	<.0001
LNAADT	1	1.3779	0.1914	1.0027 1.7532	51.81	<.0001
Lacc	1	-0.0017	0.0006	-0.0028 -0.0005	8.00	0.0047
AccCo	1	1.1922	0.3481	0.5099 1.8746	11.73	0.0006
Dispersion	1	0.4423	0.1036	0.2795 0.7000		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			37				
Missing Values			2				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	33		46.7422			1.4164	
Scaled Deviance	33		46.7422			1.4164	
Pearson Chi-Square	33		24.6860			0.7481	
Scaled Pearson X2	33		24.6860			0.7481	
Log Likelihood			6210.3887				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	-1.7404	0.9111	-3.5261	0.0453	3.65	0.0561
LnExpo	1	1.3331	0.2178	0.9063	1.7600	37.46	<.0001
Ldec	1	-0.0020	0.0006	-0.0031	-0.0009	13.11	0.0003
DecCo	1	1.4620	0.3383	0.7989	2.1251	18.67	<.0001
Dispersion	1	0.3319	0.0936	0.1910	0.5767		

The GENMOD Procedure							
Model Information							
Data Set				WORK.T			
Distribution			Negative Binomial				
Link Function			Log				
Dependent Variable			Tcol	Tcol			
Observations Used			37				
Missing Values			2				
Criteria For Assessing Goodness Of Fit							
Criterion	DF		Value			Value/DF	
Deviance	32		47.2732			1.4773	
Scaled Deviance	32		47.2732			1.4773	
Pearson Chi-Square	32		24.1839			0.7557	
Scaled Pearson X2	32		24.1839			0.7557	
Log Likelihood			6211.8643				
Algorithm converged.							
Analysis Of Parameter Estimates							
Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	-1.6242	0.8400	-3.2706	0.0222	3.74	0.0532
LnLseg	1	1.0743	0.2503	0.5837	1.5648	18.42	<.0001
LNADT	1	1.4877	0.2251	1.0466	1.9289	43.69	<.0001
Ldec	1	-0.0014	0.0006	-0.0026	-0.0001	4.51	0.0337
DecCo	1	0.9328	0.4429	0.0647	1.8009	4.44	0.0352
Dispersion	1	0.2992	0.0864	0.1700	0.5268		