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Safety performance assessment of freeway interchanges, ramps and ramp terminals

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**SAFETY PERFORMANCE ASSESSMENT OF FREEWAY INTERCHANGES,
RAMPS AND RAMP TERMINALS**

by

Bishnu Prasad Parajuli, B. Eng., Nagpur, 1992

A thesis

Presented to Ryerson University

**in partial fulfillment of the
requirements for the degree of
Master of Applied Science
in the Program of Civil Engineering**

Toronto, Ontario, Canada, 2005

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Safety Performance Assessment of Freeway Interchanges, Ramps and Ramp Terminals

Master of Applied Science, 2005

By Bishnu Prasad Parajuli

**Department of Civil Engineering
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Abstract

Existing safety performance functions for mainline interchanges and ramps of Ontario freeways are updated using negative binomial regression. The functional forms of the updated models are different from the existing models. In addition, new safety performance functions for ramp terminal sites are developed.

Network screening to identify sites in need of safety treatment has been illustrated using two different methods, one based on a potential for safety improvement (PSI) index and, the other based on an index of a high proportion of a specific accident type. A comparison of rankings for 3-legged signalized ramp terminals by the two methods indicates reasonably consistent results, with some key differences. The method of screening for high proportion of specific accidents can be a possible alternative to PSI index method where safety performance functions and/or traffic volumes are not available since, unlike the PSI Index method, it does not require these inputs.

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Dedication

For my parents

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List of Acronyms

AADT	Average Annual Daily Traffic
ADT	Average Daily Traffic
AWD	Average Weekday Traffic
CURE	Cumulative Residuals
DES	Detailed Engineering Study
EB	Empirical Bayes
FHWA	Federal Highway Administration (USA)
FI	Fatal and Injury (Collisions)
MTO	Ministry of Transportation, Ontario
ML	Maximum likelihood
SPF	Safety Performance Function
PDO	Property Damage Only
PSI	Potential for Safety Improvement

1. Introduction

In 2002, Ontario experienced 1,328 deaths on the road; as many as 107,004 suffered bodily injuries and there were 343,891 highway accidents related to property damage (MTO website). The cost of such accidents turns out to be in billions of dollars. This indicates that the economic benefit of highway construction is marred by the cost associated with highway accidents. Authorities are more and more concerned about such accidents and attempts are being made to reduce them.

The first step in the process of reducing the accidents is the identification of hazardous locations. A detailed investigation can be carried out on these locations to find remedial measures. Conventional methods, which are still commonly used by many authorities in North America, are based on accident counts or accident frequency (Persaud, 2001), where sites with high accident count over certain period or high accidents per year is considered as hazardous locations. Highway safety researchers are in agreement that these methods of identification of hazardous location are flawed because of the fact that highway accident occurrence is a random phenomenon. A detailed discussion of the reasons is presented in Chapter 2 under literature review.

An increasingly popular method used to identify sites for potential safety improvement (PSI) is the Empirical Bayes method that uses actual accident counts and expected accident frequency derived from a safety performance function (SPF) to identify the sites for improvement.

A safety performance function is an empirically derived Equation using historical data related to traffic accidents. Such an Equation estimates the accident frequency (usually accidents per year) as a function of one or several variables such as average annual daily traffic (AADT), road segment length, turning radius, surface condition, type of signal control and so on. The actual SPF depends on availability of data and level of significance of the variables in the model. These are discussed in detail in Chapter 2. Hauer (2004) identified the main purposes of developing SPFs as follows:

- To estimate the safety of road elements based on its traits.
- To estimate the safety effect of a change in a trait of a road element.

Generally, SPFs are separately developed for different entities due the inherent difference in characteristics of such entities with respect to safety. For example, four-legged signalized intersections have a different SPF from the one for three-legged signalized intersections. Likewise, three-legged signalized intersections have a different SPF from three-legged stop-controlled intersections and so on. This is because even though the variables used in the different cases are the same, the effect of these variables depends on the types of sites.

Hauer (2002) defined network screening as *“the activity of administering a low cost examination of all entities of a population”*. In a road network, the entities consist of freeways, intersections, ramps, interchanges, etc. The outcome of network screening is a ranked list of sites with most unsafe site at the top of the list. Those sites at the top of list qualify for detailed engineering study (DES) to identify countermeasures for accident reduction.

Methods of ranking of sites differ from jurisdiction to jurisdiction. Some rank them based on accident counts and some use accident rates. An increasingly popular method is potential for safety improvement (PSI) index method, which is based on Empirical Bayes method. But this method requires huge historical database that needs resources that may not be available in small municipalities. Heydecker and Wu (1991) proposed an alternative method for screening that may be applied in the absence of extensive database to develop SPFs. This method flags sites with high proportion of particular types of accidents and also serves as a pre-diagnostic tool in this respect. Theoretical detail of the method is discussed in Chapter 3 under model formulation.

1.1 Objectives of Study

The Ministry of Transportation, Ontario (MTO) developed SPFs for mainline freeway interchanges and ramps in 2000 using data from 1989 to 1996 (Bahar and Persaud, 2000). These models need to be updated and expanded using new data set. There were no separate models developed for ramp terminals since the ramp terminal related accidents were included in ramp models. As the ramp terminals have different characteristics from the ramps, different models for ramps and ramp terminals would give better estimates of safety. This belief was one primary motivation for this project.

The main objectives of this research were to use data from 1997 to 2003 from Central, Southwest and Northeast Regions of Ontario for the followings:

1. To update the SPFs for interchanges
2. To update / develop SPFs for different types of ramps as follows
 - i. Flared on-ramps
 - ii. Flared off-ramps
 - iii. Loop on-ramps
 - iv. Loop off-ramps
 - v. Freeway to freeway ramps
 - vi. Flared and loop combined on-ramps
 - vii. Flared and loop combined off-ramps
 - viii. Other on-ramps
 - ix. Other off-ramps

Note that ramps that split from the main ramps are placed under the other category.

3. To develop SPFs for ramp terminals, which are the intersections where off-ramps meet the side road. The aim was to develop models disaggregated at the following levels.
 - i. Three-legged signalized ramp terminals
 - ii. Three-legged stop controlled ramp terminals
 - iii. Four-legged signalized ramp terminals where the fourth leg is the origin of entrance ramp

- iv. Four-legged stop-controlled ramp terminals where the fourth leg is origin of entrance ramp

Various model categories are illustrated in Fig. 1.1.

- 4. To investigate the method of screening ramp terminals to flag them based on high proportions of specific accident types
- 5. To compare the ranking of ramp terminal sites based on PSI index method and high proportions of specific accident types method.

1.2 Organization of Report

This report is organized into seven Chapters. Chapter 1 outlines the background and objectives of the research. The literature review is discussed in Chapter 2. Research methodology is provided in Chapter 3. Data collection and preparation is presented in Chapter 4. Results and discussions of SPF calibration are presented in Chapter 5. Results of screening for high proportions of specific accident types are discussed in Chapter 6. Conclusions of the research and recommendations are presented in Chapter 7.

Fig 1.1 Illustrations of Model Categories

2. Literature Review

Before making a costly decision for safety improvement of road elements, it is important to identify the road elements where there is a high potential for safety improvement. Persaud (2001) surveyed 30 highway authorities in the US and Canada to find out the state of practice in methods in identifying sites for safety improvement. The majority of them used the conventional method of collision counts or collision rates as an indicator of hazardous location. He and many other researchers (Hauer, 1997, Miaou, 1996) illustrated that such conventional methods are seriously flawed. As accident frequency is non-linear function of traffic flow, using collision counts produces a bias in favor of high-volume sites and using collision rate produces a bias in favor of low volume sites. Further, regression-to-mean effect could result the sites with temporarily high collision counts being incorrectly identified as unsafe.

In order to overcome the shortcomings of conventional methods, Hauer (1997) proposed a method that uses safety performance functions. In this method, the weighted average of the actual count of accidents and the expected number of accidents estimated by a safety performance function (SPF) is calculated as an estimate of safety. This method, which compensates for the random fluctuations in collision occurrence, is called the Empirical Bayes method of safety estimation. As the SPF is the key in safety estimation using this method, a detailed discussion of this concept is presented in the following section.

2.1 Safety Performance Function (SPF)

The highway traffic system is a complex one that involves drivers, vehicles, road, traffic and environment. An ideal safety performance function summarizes the relationship among these variables where accident frequency is expressed as a function of the system's traits. Many of these variables can be measured physically but for some of them, it is practically impossible to get a direct measurement. In addition, traffic accident occurrence is a random phenomenon that follows a certain statistical distribution. Hence, a traffic accident model is composed of the following three components:

- A systematic component, which can be measured

- Missed variables, which can not be measured. These variables involve mainly the driver and vehicle related variables.
- A random component arising from the randomness of traffic accident occurrence.

Regression analysis is done using measurable variables to establish the relationship between them and accident frequency. The effect of randomness and missed variables is reflected in the estimated parameters.

Hauer (2004) identified the steps involved in developing a SPF as

- Assumption of probability distribution of accident counts
- Identification of functional form
- Parameter estimation
- Test for goodness of fit

A detailed discussion on the identification of functional form is presented in section 3.2 under model specification. There is an agreement among safety researchers that the probability distribution of highway accident is either Poisson or negative binomial (Hauer, 2004, Bauer and Harwood, 1997, Miaou, 1996). These researchers pointed out that the conventional method of assuming a normal probability distribution is flawed mainly because of following reasons:

- Accidents are discrete random events that are sporadic. It is not reasonable to assume them as continuous random events by assuming a normal distribution.
- It is quite common that accident counts have zero or small numbers during a study period. Small numbers which are zero or close to zero do not follow a normal distribution.

Miaou (1994) compared the effectiveness of Poisson and negative binomial models applied to traffic accident counts and concluded that if the variance is greater than the mean, the data are said to be overdispersed and negative binomial models give better results. In case data are not overdispersed, both models give consistent results. Hauer (2004) showed that the Poisson model is a special case of negative binomial model.

The most common method used to estimate parameters is to maximize the likelihood function (Hauer, 2004). More conveniently, a log likelihood function is used because of its simplicity. Likelihood functions used in this study are discussed under model formulation in sections 3.1.1 and 3.1.3.

No study to date has modeled ramp terminal accidents. However, there is one related research study. Bauer and Harwood (1997) excluded rear-end collisions occurring on the ramps in their research on “accident on freeway ramps and speed change lanes”, considering these rear-end collisions as ramp terminal related. For the current project, a preliminary study of the sketches on hard copy Ontario provincial police reports of accidents and the electronic ramp accident database revealed that the accidents coded as “intersection related” and “in intersection” in MTO’s Accident Information System (AIS) are mostly rear-end collisions at the ramp terminals. Thus, these two types of accidents from the electronic database were assumed to be the ramp terminal related in this research.

Ramp terminal accidents resemble intersection accidents with some differences. One of the differences is that in the case of ramp terminals, there is high chance of rear end accidents at the exit ramps as the vehicles from the high speed freeway approach the ramp terminal. On the other hand, normal intersection accidents are influenced by several other factors such as driveway density in the vicinity of the intersection and pedestrian flow, among others. Poch and Mannering (1996) studied urban intersection accidents in Washington area and found that total opposing traffic volume at the intersection, total left turning volume and total right turning volumes are significant variables contributing the urban intersection accidents. Persaud and Nguyen (1998) modeled intersection accidents on provincial roads of Ontario and found that total approaching volume from all approaches were the major contributing factor for accident occurrence on three-legged and four-legged intersections. There were no geometric variables involved in the models, possibly due to unavailability of such data.

The Ministry of Transportation, Ontario (MTO) developed safety performance functions (SPFs) for mainline freeway interchanges and ramps using accident data from Central

Region from 1989 to 1996 (Bahar and Persaud, 2000). Several models were developed for freeway interchanges and ramps at a disaggregate level based on their configurations and severities of accidents. They identified the average annual daily traffic (AADT) as a main factor contributing the accidents in mainline freeways and regular ramps. Ramp length was included only in the freeway-to-freeway ramp models. The accident influence length of an interchange was assumed to be one kilometer on either side of the interchange irrespective of the spacing of successive interchanges. This assumption resulted in overlapping of influence length where the spacing of two interchanges was less than two kilometers. In order to avoid the overlapping of influence length and double counting of accidents, the influence length assumption was slightly modified in this study. Accident influence length was assumed to be one kilometer on either side of the interchange but where the spacing of two interchanges is less than two kilometers, accidents within half the distance between the interchanges were assigned to one interchange and accidents occurring on the other half of the segment were assigned to the other interchange. Unlike the previous study, the influence length was not constant.

The operational characteristics of a traffic system are dynamic. They change over time due to the change in operational environment. Because of this the existing MTO SPFs needed to be updated and expanded using a wider database and data from more recent years. Also, it is felt necessary to develop separate models for the ramp terminals in order to better understand the nature of accidents occurring at such sites.

2.2 Network Screening

Network screening is done to identify the road locations that have a high promise for safety improvement. Two methods of relevance to this research are discussed in the following sections.

2.2.1 Network Screening by the PSI Index Method

In this method, sites are ranked by a PSI index. Sites having high values of this index qualify for a detailed engineering study (DES) to investigate possible treatment options. In MTO's existing methodology, the PSI is defined as the difference between the site's

expected number of collisions and those expected at sites with similar characteristics. However, Hauer (2002) has suggested that the PSI is simply the site's expected number of collisions.

Persaud and Lyon (1999) used the Empirical Bayes (EB) method to estimate the expected number of collisions of individual intersection and road segment sites on Ontario provincial roads. The number of collisions expected at sites with similar characteristics is simply the estimate from an SPF. They pointed out that EB method smoothes the random fluctuation of accident counts. The EB method of estimating safety is widely used and considered to be a state-of-the-art method.

In the EB method, the accident count (x) of any highway location in most recent n years is combined with an estimate of expected annual accidents (P) based on accident history of similar locations. Mathematically, it can be written as follows (Persaud and Lyon, 1999).

$$m = w_1(x) + w_2(P) \quad (2.1)$$

where,

m is long term mean of accident count

P is the estimate of annual accident calculated from the safety performance function; w_1 and w_2 are estimated as follows.

$$w_1 = \frac{P}{\phi + nP} \quad (2.2)$$

$$w_2 = \frac{\phi}{\phi + nP} \quad (2.3)$$

where,

n : number of years the accident count is done

$\phi = \frac{P^2}{\text{var}(P)}$, is a constant for a given model and is estimated during modeling as the dispersion parameter of the negative binomial distribution assumed for the SPF. The variance of m is given by

$$\text{var}(m) = \frac{x + \phi}{\left(n + \frac{\phi}{P}\right)^2} \quad (2.4)$$

2.2.2 Network Screening for High Proportions of Specific Accident Types

The estimation of PSI index requires large amounts of data such as AADT, and consequently resources and time. In many cases, particularly in small municipalities, these data may not be available. An alternative method has been proposed by Heydecker and Wu (1991). This method is called screening for high proportions of specific accident types. The main advantage of this method is that it does not require a large amount of data, specifically AADTs. Only the accident counts by type for a few years are enough to rank a site. The so called ‘pattern score’ of each site for each accident type is estimated to rank the site. Sites with a high pattern score are considered to have a high potential for safety improvement. A detailed discussion on the theoretical background of this method is presented in section 3.5.

3. Research Methodologies

3.1 Model Formulation

Accident counts are considered to follow Poisson or negative binomial distributions. Miaou (1996) compared the Poisson and negative binomial models and concluded that Poisson models are appropriate when mean of the accident counts is equal to variance. When they are not equal, it is appropriate to assume negative binomial distribution. Formulation of models assuming Poisson and negative binomial distributions is discussed in the following sections.

3.1.1 Poisson Model

Let Y_i be the number of accidents at i^{th} element of highway, X_{ij} be the j^{th} predictor variable at i^{th} element, β_j be the parameter associated with the j^{th} explanatory variable, then the relationship between the expected number of accidents, μ_i , and its predictor variables X can be established as follows.

Assuming accident counts, Y_i , follow the Poisson distribution with mean, μ_i , probability that a highway element with set of predictor variables $X_{i1}, X_{i2}, \dots, X_{ij}$ experiences y_i accidents can be expressed as

$$P(Y_i = y_i, \mu_i) = \frac{(\mu_i)^{y_i} e^{-\mu_i}}{y_i!} \quad (3.1)$$

This gives

$$P(Y_i = y_i, \mu_i) = \exp[y_i \ln(\mu_i) - \mu_i - \ln(y_i!)] \quad (3.2)$$

Comparing the Equation (3.2) with the canonical form of generalized linear model as described by McCullagh and Nelder (1999), Dobson (2002) and Gill (2000), the first term in the parenthesis on the right hand side of the Equation is called interaction component and the coefficient of y_i (i. e. $\log \mu_i$) is a canonical link, also called the link function. This

link function is expressed as a linear function of independent variables as shown in the following Equation.

$$\ln(\mu_i) = \beta_0 + \sum_{j=1}^q \beta_j X_{ij} \quad (3.3)$$

where, β is a vector of parameters and X is a vector of independent variables.

3.1.2 Parameter Estimation of Poisson Model

If there are n locations and the accidents at each location are independent of each other, the joint probability of accidents at all locations is given by the product of all individual probabilities. This product is called likelihood function, L , which is given by

$$L = \prod_{i=1}^n \frac{(\mu_i)^{y_i} e^{-\mu_i}}{y_i!} \quad (3.4)$$

For ease of calculation, a log likelihood function is generally used instead of the likelihood function. The log likelihood function, l , is given by

$$l = \sum_{i=1}^n y_i \ln(\mu_i) - n \mu_i - \sum_{i=1}^n \ln(y_i!) \quad (3.5)$$

This gives

$$l_m = \sum_{i=1}^n y_i (\beta_0 + \sum_{j=1}^q \beta_j X_{ij}) - n e^{(\beta_0 + \sum_{j=1}^q \beta_j X_{ij})} \quad (3.6)$$

The parameters β can be estimated such that the log likelihood function (3.6) is maximized. Note that the third term on the right hand side of Equation (3.5) has been ignored for maximization, as it is constant. l_m is the revised log likelihood function.

PROC GENMOD procedure of the SAS software can be used to estimate the parameters β .

3.1.3 Overdispersion and Negative Binomial Model

Miaou (1996) and others showed that the assumption of a Poisson distribution does not hold in accident data as the variance is generally greater than the mean. When variance is greater than mean, the data are said to be overdispersed. The solution to this problem is to assume that accident counts over a set of similar sites follow the negative binomial distribution, which is a discrete distribution that accounts for overdispersion in data. This distribution is an extension of the Poisson distribution where the overdispersion is represented by overdispersion parameter, ϕ . As $\phi \rightarrow 0$, the distribution becomes Poisson.

Overdispersion in accident counts is thought to be due to the inability of the assumed independent variables to explain the variance of the model. This is because the actual number of independent variables may be more than the number of variables considered in the models. Model formulation assuming the negative binomial distribution is discussed below.

Let Y_i be the number of accidents at i^{th} element of highway, assuming that Y_i follows negative binomial distribution with parameters α and ϕ , the probability that the element experiences $Y_i = y_i$ accidents is given by

$$P(Y_i = y_i, \alpha, \phi) = \frac{(y_i + \phi - 1)!}{y_i! (\phi - 1)!} \alpha^\phi (1 - \alpha)^{y_i} \quad (3.7)$$

Equation (3.7) can be expressed as

$$P(Y_i = y_i, \alpha, \phi) = \exp[y_i \ln(1 - \alpha) + \phi \ln(\alpha) + \ln\left(\frac{(y_i + \phi - 1)!}{y_i! (\phi - 1)!}\right)] \quad (3.8)$$

Comparing (3.8) with canonical form of generalized linear model by McCullagh and Nelder (1999), the link function can be written as

$$\ln(1 - \mu_i) = \beta_0 + \sum_{j=1}^q \beta_j X_{ij} \quad (3.9)$$

where, X_{ij} : the j^{th} explanatory variable at i^{th} element

β_j : the parameter associated with the j^{th} variable

The mean is given by

$$\mu_i = \frac{\phi(1-\alpha)}{\alpha} \quad (3.10)$$

Likewise, the variance is given by

$$\text{Var}(Y) = \frac{\phi(1-\alpha)}{\alpha^2} \quad (3.11)$$

3.1.4 Parameter Estimation of Negative Binomial Model

If there are n locations and the accidents at each location are independent of each other, the joint probability of accidents at all locations is given by the likelihood function, L_N , as follows

$$L_N = \prod_1^n \frac{(y_i + \phi - 1)!}{y_i! (\phi - 1)!} \alpha^\phi (1 - \alpha)^{y_i} \quad (3.12)$$

The log likelihood function, l_N , is given by

$$l_N = \sum_1^n [\ln(y_i + \phi - 1)! - \ln(y_i!) - \ln(\phi - 1)! + \phi \ln(\alpha) + y_i \ln(1 - \alpha)] \quad (3.13)$$

$$l_N = \sum_1^n y_i \ln(1 - \alpha) + n\phi \ln(\alpha) + f(y_i, \phi) \quad (3.14)$$

The parameters can be estimated by maximizing the log likelihood function given by Equation (3.14) subject to the condition given by the Equation (3.9).

3.2 Model Specification

For mainline interchanges and ramps, two types of models can be specified as given by Equations (3.15) and (3.16). Suitable models can be selected based on the results of goodness of fit test.

$$1. (Acc/year)_i = a(AADT_i)^b e^{\sum_j c_j X_j} \quad (3.15)$$

$$2. (Acc/year)_i = a(AADT_i)^b \prod_j (X_j)^{c_j} \quad (3.16)$$

For ramp terminals, the model can be specified as a function of approaching AADTs and the dummy variable as shown in the Equation (3.17).

$$(Acc/year)_i = a(AADTramp)_i^b (AADTcross)_i^c e^{\sum_j d_j X_j} \quad (3.17)$$

where,

i : the site identifier

j : the geometric variable identifier

a , b , c and d : coefficients to be estimated

e : base of natural logarithm.

X : vector of geometric variables

$(AADTramp)_i$: total AADT of the ramp approach and its opposite approach (where applicable) of i^{th} ramp terminal

$(AADTcross)_i$: total approach AADT on the cross road of the i^{th} ramp terminal

Applying the boundary conditions to model (3.15), when AADT is zero, accidents per year become zero no matter what the values of model parameters are. The second component in the model (the exponent) yields a non-negative value in all the cases. The effect of this component is positive if additive terms result in a value greater than one, and negative if it is less than one. In case it becomes zero, it has no effect in the model. In model (3.16), if any of the geometric variables is zero (i.e. road is nonexistent), accidents per year is zero. The same holds for model (3.17). So, all the model forms are intuitively meaningful.

Actual models used for mainline interchanges, ramps and ramp terminals are discussed in sections 5.1, 5.2 and 5.3 respectively.

3.3 Model Calibration

For calibration purpose, total accidents over the study period of 7 years were used rather than the average accident. For this, the model (3.15) can be modified as

$$(Total\ accident)_i = (N)(a)(AADT_i)^b e^{\sum_j c_j X_j} \quad (3.18)$$

where, N is the study period in years.

Equation (3.18) can be reduced to

$$\ln(Total\ accident)_i = \ln(N) + \ln(a) + b \ln(AADT)_i + \sum_j c_j X_j \quad (3.19)$$

Likewise, model (3.16) can be written as

$$\ln(Total\ accident)_i = \ln(N) + \ln(a) + b \ln(AADT)_i + \sum_j c_j \ln(X_j) \quad (3.20)$$

Model (3.17) can be reduced to

$$\begin{aligned} \ln(Total\ accident)_i &= \ln(N) + \ln(a) + b \ln(AADTramp)_i \\ &+ c \ln(AADTcross)_i + \sum_j d_j X_j \end{aligned} \quad (3.21)$$

In the SAS GENMOD procedure used for model fitting, the link function is log and $\ln(N)$ is an offset.

3.4 Goodness of Fit Test

Once the model parameters are estimated, their statistical significance has to be tested to make sure these estimates are sensible. In the published literature, there is no unique way to test the significance of the estimated parameters. The first and obvious test is to see whether the estimated parameters show an intuitively correct direction. For example, in a

road segment model, the longer the segment length, the higher should be the accident frequency. That means the coefficient of segment length should be a positive number.

Poch and Mannering (1996) suggested the likelihood ratio test for goodness of fit. Bauer and Harwood (1996) used two main criteria to test the goodness of fit of accident models. They were mean deviance (i.e. the deviance per degree of freedom) and Pearson Chi-square per degree of freedom, also called as Chi-square ratio. The first one is the measure of degree of dispersion (overdispersion or underdispersion). The dispersion parameter estimated with mean deviance close to one suggests that the data are neither overdispersed nor underdispersed. Likewise, a Pearson Chi-square ratio between 0.8 and 1.2 was assumed to be an indication of a good fit of a model. Miaou (1996) evaluated various methods to test the goodness of fit and concluded that the conventional R^2 method is seriously flawed to test the goodness of fit of accident models. More recently, Hauer and Bamfo (1997) proposed a graphical method to test the overall goodness of fit of accident models where cumulative residuals are plotted against the AADT arranged in ascending order. An oscillation of the resulting curve about X-axis within the range of ± 2 times the standard deviations is an indication of good fit at 95% confidence level. Council and Stewart (1999) used the probability value (P-value) to test the goodness of fit of parameters. They ignored the variables where P-value was greater than 0.05.

Summarizing the goodness of fit criteria, different researchers used different criteria. In this research, overall goodness of fit was tested using mean deviance and Chi-square ratio. The parameters were assessed by their P-value. These tests were further supplemented by cumulative residual (CURE) plots.

3.5 Theoretical Background of the Method of Screening for High Proportions of Specific Accident Types

This method of site screening is based on identification of unusual types of accidents, which have occurred at a site and quantifying and ranking the degree to which they are unusual.

This method was first proposed by Heydecker and Wu (1991). In this method, if true proportion of target accidents at a site i is μ_i , then the probability of observing x_i target accidents at the site with n_i accidents is given by the Binomial distribution as follows

$$f(x_i / n_i, \mu_i) = \binom{n_i}{x_i} \mu_i^{x_i} (1 - \mu_i)^{n_i - x_i}, 0 \leq x_i \leq n_i \quad (3.22)$$

where, $\binom{n_i}{x_i}$ is a binomial coefficient defined as

$$\binom{n}{x} = \frac{n!}{x!(n-x)!} \quad (3.23)$$

The parameter μ_i is constant for a given site and varies randomly from site to site. Heydecker and Wu (1991) assumed that μ_i follows Beta distribution, which is defined as

$$g(\mu / \alpha, \beta) = \frac{\mu^{\alpha-1} (1-\mu)^{\beta-1}}{B(\alpha, \beta)}, \quad 0 < \mu < 1 \quad (3.24)$$

where,

$$B(\alpha, \beta) = \frac{\Gamma(\alpha)\Gamma(\beta)}{\Gamma(\alpha + \beta)} \quad (3.25)$$

where, α and β are the parameters of Beta prior distribution and $\Gamma(.)$ defined as

$$\Gamma(z) = \int_0^\infty e^{-t} t^{z-1} dt \quad (3.26)$$

The mean and variance of the Beta prior distribution can be expressed as a function of parameters α and β as follows.

$$E(\mu) = \frac{\alpha}{\alpha + \beta} \quad (3.27)$$

$$Var(\mu) = \frac{\alpha\beta}{(\alpha + \beta)^2 (\alpha + \beta + 1)} \quad (3.28)$$

where, $E(\mu)$ is the prior estimate of μ_i .

Combining the Binomial distribution (3.22) and Beta distribution (3.25) results in an unconditional Binomial-Beta distribution, which can be written as follows

$$h(x_i / n_i, \alpha, \beta) = \binom{n_i}{x_i} \frac{B(\alpha + x_i, \beta + n_i - x_i)}{B(\alpha, \beta)} \quad (3.29)$$

Using Bayes theorem the prior Beta distribution is combined with site specific collision data (n_i, x_i) for each site to derive the adjusted posterior beta distribution which can be written as

$$g(\mu_i / \alpha', \beta') = \frac{\mu^{\alpha'-1} (1 - \mu)^{\beta'-1}}{B(\alpha', \beta')} \quad 0 < \mu < 1 \quad (3.30)$$

α' and β' are posterior parameters and can be expressed as

$$\alpha' = \alpha + x_i \quad (3.31)$$

$$\beta' = \beta + n_i - x_i \quad (3.32)$$

Equation (3.30) is also a Beta Distribution

For the posterior distribution, the expected value and variance for site i can be written as follows:

$$E(\mu_i) = \frac{\alpha'}{\alpha' + \beta'} \quad (3.33)$$

$$Var(\mu_i) = \frac{\alpha' \beta'}{(\alpha' + \beta')^2 (\alpha' + \beta' + 1)} \quad (3.34)$$

The pattern score is defined as the probability that expected value of μ_i is greater than a certain value of Beta prior distribution, μ_m . Mathematically, it can be expressed as

$$P(\mu_i > \mu_m) = 1 - B(\mu_m, \alpha', \beta') \quad (3.35)$$

where,, μ_m is such that

$$\int_{\mu_m}^1 g(\mu) / \alpha, \beta) d\mu = \pi \quad (3.36)$$

where, π can have value from 0 to 1. In this study π was assumed to be 0.5, which is considered to be a neutral value. That means μ_m was the median of Beta prior distribution.

Pattern scores estimated using Equation (3.35) and arranged in descending order rank sites based on the assumed collision type.

3.6 Parameter Estimation of Beta Prior Distribution

The parameters of the Beta distribution have to be estimated to apply Bayes theorem, which can be stated as follows.

Suppose that a parameter μ has prior distribution, $P_b(\mu)$ and that for a specified value of μ , the joint probability of making observations $x_1, x_2, x_3, \dots, x_n$ ($=x$) is given by $P_0(x/\mu)$. The posterior distribution, $P_0(\mu)$, given the observations x is given by

$$P_a(\mu/x) = \frac{P_0(x/\mu)P_b(\mu)}{\int_{\mu} P_0(x/\mu')P_b(\mu')d\mu'} \quad (3.37)$$

For any value of μ the value of the posterior is proportional to that of prior and to the probability that the observations would occur if that value of μ is obtained.

Heydecker and Wu (1991) proposed three methods to estimate the parameters as follows.

- Maximum likelihood method
- Method of moment – First method
- Method of moment-Second method

Mollet (2004) presented a parameter estimation method using Microsoft Excel's worksheet function. The second method of moment and the maximum likelihood method were used in this study for parameter estimation and theoretical background of these methods are discussed in the following sections.

3.6.1 Method of Moment

The parameters α and β of the Beta distribution can be expressed in terms of moments (mean and variance) of the Beta distribution as shown in Equations (3.27) and (3.28). If we can find the mean and variance from the observed data, α and β can be estimated by solving those two Equations.

Let there be M sites under consideration and μ_i be the proportion of accidents for site i , that is, $\mu_i = x_{ij} / n_i$, where, x_{ij} is the total numbers of target accidents of type j during the study period at site i and n_i is the total number of all types of accidents at site i during the same period. The mean proportion of target accident, j , is given by

$$\bar{\mu}_j = \frac{\sum_{i=1}^M \mu_{ij}}{M} \quad (3.38)$$

where, $\bar{\mu}_j$ is the mean proportion of target accident type j .

Similarly, the variance is given by

$$s^2 = \frac{1}{M-1} \left[\sum_{i=1}^M \left(\frac{x_i^2}{n_i^2} - \frac{x_i}{n_i} \right) - \frac{1}{M} \left(\sum_{i=1}^M \frac{x_i}{n_i} \right)^2 \right], \quad n \geq 2 \quad (3.39)$$

For a sufficiently large sample, the sample mean, $\bar{\mu}_j$, represents the expected value, $E(\mu_j)$ and the sample variance, s^2 , represents the population variance, $Var(\mu)$. Then the variance can also be expressed as

$$s^2 = \frac{\frac{\alpha^2}{\bar{\mu}} - \alpha^2}{\left(\frac{\alpha}{\bar{\mu}} \right)^2 \left(\frac{\alpha}{\bar{\mu}} + 1 \right)} \quad (3.40)$$

This can be further simplified as

$$s^2 = \frac{\frac{1}{\bar{\mu}} - 1}{\left(\frac{1}{\bar{\mu}}\right)^2 \left(\frac{\alpha}{\bar{\mu}} + 1\right)} \quad (3.41)$$

This gives

$$\alpha = \frac{\bar{\mu}^2 - \bar{\mu}^3 - s^2 \bar{\mu}}{s^2} \quad (3.42)$$

Then, β can be estimated as

$$\beta = \frac{\alpha}{\bar{\mu}} - \alpha \quad (3.43)$$

3.6.2 Maximum Likelihood Method

From Equation (3.29), the likelihood function can be written as

$$L_i = \binom{n_i}{x_i} \frac{B(\alpha + x_i, \beta + n_i - x_i)}{B(\alpha, \beta)} \quad (3.44)$$

It is more convenient to use the log-likelihood function which can be written as

$$\ln(L_i) = \ln \binom{n_i}{x_i} + \ln(B(\alpha + x_i, \beta + n_i - x_i)) - \ln(B(\alpha, \beta)) \quad (3.45)$$

Parameters α and β are such that the log-likelihood function is maximized. For this maximizing condition, the first term of right hand side of Equation (3.45) can be ignored as it is a constant. The objective function can be written as

$$\begin{aligned} \ln(L_i) &= \ln[\Gamma(\alpha + x_i)] + \ln[\Gamma(\beta + n_i - x_i)] - \ln[\Gamma(\alpha + \beta + n_i)] \\ &\quad - \ln[\Gamma(\alpha)] - \ln[\Gamma(\beta)] + \ln[\Gamma(\alpha + \beta)] \end{aligned} \quad (3.46)$$

where, α and β are strictly positive numbers.

The function given by (3.46) can be maximized using solver function of Microsoft Excel to get α and β .

3.6.3 Posterior Beta Distribution and Pattern Score

The median of the Beta prior distribution, μ_m is given by Equation (3.36). Once α and β are estimated, μ_m can be estimated using Microsoft Excel's worksheet function. The posterior parameters, α' and β' , can be calculated by using Equations (3.31) and (3.32). The pattern score can be calculated using Equation (3.35).

To summarize the above discussion, a stepwise procedure to estimate the pattern score used for ranking the sites is presented below.

- Divide the sites into logical groups. For example, 3-legged signalized ramp terminals, 3-legged stop controlled ramp terminals, 4-legged signalized terminals and 4-legged stop controlled ramp terminals, etc.
- Identify the different types of accidents. It depends on availability of data.
- Find the total number of accidents of each type during the study period in each site, x_i .
- Find the total number of all types of accidents in each site, n_i .
- Calculate the proportion, x_i/n_i for each site and for each type of accident.
- Calculate the mean of proportions for each accident type, $\bar{\mu}_j$ using Equation (3.38).
- Calculate the variance using Equation (3.39).
- Calculate α and β using Equations (3.42) and (3.43) respectively.
- Estimate the median of Beta prior distribution using Excel function as $(\mu_m = \text{betainv}(0.5, \alpha, \beta))$.
- Calculate the parameters of posterior Beta distribution as $\alpha' = \alpha + x_i$ and $\beta' = \beta + n_i - x_i$.
- Estimate the pattern score using Excel worksheet function as $P(\mu_i > \mu_m) = 1 - \text{betadist}(\mu_m, \alpha', \beta')$.
- Arrange the pattern score in descending order to rank the sites.

4. Data Collection and Preparation

Highways under the jurisdiction of Ministry of Transportation were divided into following five administrative regions.

- Central Region
- Eastern Region
- Southwest Region
- Northeast Region
- Northwest Region

There were a total of 482 interchanges in the province under the jurisdiction of MTO. Of these, 238 were in the Central Region, 120 in Eastern Region, 93 in Southwest region, 29 in Northeast region and 2 in Northwest region. The data used in this study were taken from Central, Southwest and Northeast regions, which had 360 interchanges, 2,444 ramps and 572 ramp terminals. Of these, 311 interchanges, 1,545 ramps and 380 ramp terminals had at least traffic flow and accident data available – the minimum data required for SPF development.

Previous SPFs were developed based on Central Region data from 1989 to 1996 (Bahar and Persaud, 2000). The majority of the Central region interchanges considered in this study was also considered in the earlier study. However, Simcoe County had been combined into Central Region's jurisdiction subsequent to the previous study. Also some new construction and reconfiguration of interchanges were found in Central Region after the previous study.

This study used data from 1997 to 2003. Various data compiled for the purpose of this study are discussed below under the following headings.

1. Geometric data
 - i. Interchange configurations
 - ii. Number of lanes on mainline
 - iii. Ramp lengths

- iv. Number of lanes on the ramps
 - v. Ramp configurations
 - vi. Configurations of exit ramps approaching the ramp terminals
2. Collision data
- i. Mainline collision data
 - ii. Ramp collision data
 - iii. Ramp terminal collision data by severity (for SPF calibration)
 - iv. Ramp terminal collision data by impact type (for screening for proportion)
3. Traffic volumes
- i. Mainline volumes
 - ii. Ramp volumes
 - iii. Turning movements at ramp terminals

4.1 Geometric Data

Most of the Central Region geometric data were taken from the previous study. These data were checked and verified for any changes that had taken place subsequent to the previous study. Simcoe County had been combined into Central Region's jurisdiction after the previous study. As a result, several interchanges and ramps of Highways 11 and 400 were added. Some of the interchanges were reconfigured. Where the interchanges were reconfigured, data prior to reconfiguration were ignored. Detailed discussion on various geometric data used in this study is presented in the following sections.

4.1.1 Interchange Configurations

Each interchange's location was identified by the unique identifier based on linear highway referencing system (LHRS). In this system, an interchange is assigned a 5-digit reference number followed by an offset distance from the start of the LHRS section to the interchange.

In the previous study, MTO's interchanges were classified into 11 different types as shown in Fig. 4.1. The same configurations were followed in this study.

Full Diamond

Partial Cloverleaf
(Parclo)

Parclo
(4-Quad)

Partial Loop
(Loop)

Loop
(4-Quad)

Full Cloverleaf

Trumpet

Service Road

Direct Link

**Any combination of two ramps
that is not identified as a typical
ramp according to other ten
categories**

(10)

**Freeway to Freeway
Interchanges**

(11)

Fig. 4.1 Interchange configurations (Source: MTO Report, Prioritization of Central Region Ramps and Interchanges, 2000)

Because of the paucity of the sample, interchanges were combined into 3 main groups, based on reasonable similarity in configurations as follows.

- Group 1: interchange types 1, 2, 4 and 8
- Group 2: interchange types 3, 5 and 6
- Group 3: interchange types 7, 9, 10 and 11

The influence length of each interchange was considered as one kilometer on either side of the interchange. However, where the interchanges were closely spaced (less than 2 kilometers), the influence length was equally divided.

4.1.2 Number of Lanes on the Mainline

The number of lanes refers to the total number of through lanes at the interchange in both the directions. Acceleration and deceleration lanes were not considered. Wherever available, the number of lanes data used in the previous study were also used in this study. The others were provided by the MTO staff from various sources including site visits. These data were not available for Southwest and Northeast Regions.

4.1.3 Ramp Configurations

A total of 2,444 ramps was identified in the three regions under study. In the previous study, ramps were classified into 16 types. The same classification was followed in this study. These configuration types are shown in Fig. 4.2 and Fig. 4.3. These ramps were grouped into five logical categories to have adequate sample sizes for statistical analysis. Ramp types in each group were as follows.

1. Flared ramps: ramp types 1, 4, 5 and 7. There were 797 ramps in this category. These ramps were further subdivided into on-ramps and off-ramps. There were 364 on ramps and 433 off-ramps.
2. Loop ramps: ramp types 2, 3 and 6. There were 398 ramps in this category. Of these, 277 were on-ramps and 121 were off-ramps.
3. Freeway to freeway ramps: these ramps were of ramp category 8 in MTO's classification and included those ramps that connect one freeway to the other freeway.

Full Diamond

Full Diamond Split

Parclo Loop

Parclo Loop Split

Free-Flow Loop

Free Flow Loop Split

Direct Connection Extension

Outer Connection

Outer Connection Split

Fig. 4.2 Ramp configurations page 1 of 2 (Source: MTO Report, Prioritization of Central Region Ramps and Interchanges, 2000)

Fig. 4.3 Ramp configurations page 2 of 2 (Source: MTO Report, Prioritization of Central Region Ramps and Interchanges, 2000)

4. Flared and loop combined: both flared and loop ramps were combined together in this group. There were 1,195 ramps in this group. Of these, 641 were on-ramps and 554 were off-ramps.

5. Others: this category included those ramps that did not fall into any of the above groups. Mainly, they were the split ramps. There were 224 ramps in this group. Of these 89 were on-ramps and 135 were off-ramps.

4.1.4 Ramp ID

Ramps associated with a given interchange were identified by the LHRS number of the interchange and a unique 2-digit ramp number. The first digit of the ramp ID identifies the origin direction and the second digit identifies the destination direction. The MTO's ramp numbering legend is provided in Fig. 4.4 and Fig. 4.5.

4.1.5 Ramp Length

Ramp length refers to the bullnose to bullnose distance of the ramp. These lengths were compiled from various sources. Wherever possible, the ramp lengths used in the previous study were used. The rest of the ramp lengths were determined by following three methods.

1. Using GIS map:

GIS maps were available from Central and Southwest Regions for scaling using GIS Arc Reader software. All the ramps of Southwest region and some of the Central Region ramps were scaled using this software.

2. Scaling the hard copy of the drawings:

Drawn to scale hard copies of drawings were obtained from the Northeast Region. These drawings were scaled using the digital scale master to find the ramp lengths.

INTERCHANGE DIAGRAM CONVENTION

Fig. 4.4 Interchange diagram convention page 1 of 2 (Source: Ministry of Transportation, Ontario)

INTERCHANGE DIAGRAM CONVENTION

MINISTRY OF TRANSPORTATION INTERCHANGE FILE

HWY. 401 IC 375	KEYPOINT NUMBER 47635/000
AT 404 - DON VALLEY PKWY.	M.T.O. DISTRICT TORONTO
	O.P.P. DISTRICT 5 AURORA

UPDATED 89 - 03 - 17

PROGRAM DEVELOPMENT & EVALUATION OFFICE

77 - 10 - 01

DATE

2 OF 2

Fig. 4.5 Interchange diagram convention page 2 of 2 (Source: Ministry of Transportation, Ontario)

3. **Scaling ortho photos:** Ortho photos were obtained for most of the ramps in the Simcoe County of Central Region. They were scaled using a digital scale master to get the ramp lengths. However, many of the photos were not very clear and method 2 was used to get the ramp lengths.

4.1.6 Number of Lanes on Ramps

Numbers of lanes on the ramps were obtained from MTO in Excel format but for some of the Northeast Region ramps, this was determined by scaling hard copy of drawings. Where the ramp widths were greater than or equal to 7 meters, the number of lanes was taken as 2 and where they were less than 7 meters, the number of lanes was taken as one. Most ramps had widths of 4.5 meters and 7.5 meters. Observed number of lane ranges from one to three.

4.1.7 Ramp Terminal Configurations

Ramp terminals were defined as the intersections of exit ramps with the arterial roads. A total of 572 ramp terminals were identified in the three regions. Schematic diagrams of interchanges were used to identify the configuration of ramp terminals. The configurations were

- Split ramp terminals
- Non-split ramp terminals

Split terminals had the right turning ramps that splits from the main ramp before meeting the arterial road and non-split ramps had a single ramp ending at the intersection. There were 426 ramp terminals with non-split ramps and 126 with split ramps.

Ramp terminals were further classified as

- Three-legged ramp terminals
- Four-legged ramp terminals

There were 409 three-legged ramp terminals and 163 four-legged ramp terminals

Another classification used was

- Signalized ramp terminals

- Stop controlled ramp terminals

246 of the ramp terminals were signalized and 307 were stop controlled. No data were available for 19 of them.

4.2 Collision data

All the collision data for the purpose of this study were obtained in soft copy in Excel format. Separate files were obtained for mainline, ramps and ramp terminal related collisions. For the mainline collisions, separate files were obtained for each highway. Detailed discussion of the collision data is presented in the following sections.

4.2.1 Mainline Collision Data

Mainline collision data from the three regions during the period 1997 to 2003 were obtained from MTO in soft copy in Excel format. They were classified by severity levels, namely, fatal, injury and property damage only. Collision locations for each highway were given to the nearest of 100 meter. Consistent with the previous study, an accident influence length of an interchange was considered to be one kilometer on either side of the interchange. Thus, the accidents occurring within one kilometer on either side of an interchange were assigned to that interchange. However, several of these interchanges under study were closely spaced and had influence lengths less than 2 kilometers. In such cases, as noted earlier, the spacing between the interchanges was equally divided for the purpose of assigning accidents to the interchanges.

To have a sufficient sample size for statistical analysis, mainline interchanges were grouped into three logical categories based on reasonable similarity in interchange configurations. Group 1 consisted of types 1, type 2, type 4 and type 8 interchanges. Collisions in each group were further divided into fatal and injury collision combined (FI) and PDO collisions. The summary of each group of collision data is presented in Tables 4.1(a) through 4.1 (c). A typical frequency histogram of fatal and injury collisions of group 1 interchanges is shown in Fig. 4.6. Likewise, a frequency histogram for PDO collisions of group 1 interchanges is shown in Fig. 4.7.

Table 4.1(a) Summary statistics of mainline group 1 collision data

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	136	4986	36.66	15.00	2	611	5560.37
Total PDO	136	16546	121.66	46.00	3	1304	42041.65

Table 4.1(b) Summary statistics of mainline group 2 collision data

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	105	10906	103.87	47.00	5	855	20852.04
Total PDO	105	39303	374.31	230.00	15	2474	203341.64

Table 4.1(c) Summary statistics of mainline group 3 collision data

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	70	3270	46.71	19.50	3	472	6982.21
Total PDO	70	12169	173.84	69.50	8	1625	94492.13

Fig. 4.6 Frequency histogram of fatal and injury collisions on mainline group 1 interchanges

Frequency Histogram of PDO Collisions
on Mainline Group 1

Fig. 4.7 Frequency histogram of PDO collisions on mainline group 1 interchanges

4.2.2 Ramp Collision Data

Ramp collision data were also made available in soft copy in Excel format. They were classified based on severity levels as fatal, injury and property damage only. These data consisted of ramp and ramp terminal related collisions. There was no separate record of ramp terminal related collisions. However, the database showed the road locations where the collision occurred. After studying the hard copy of accident reports and the collision database, it was decided that those collisions with road location coded as “intersection

related” and “at intersection” were ramp terminal related. These collisions were predominantly rear-end in type.

A separate ramp collision data file was obtained in Microsoft Excel format that contained only “intersection related” and “at intersection” collisions. Net ramp related collisions were calculated by deducting those collisions assumed to be ramp terminal related.

For modeling purposes, ramps were classified into five categories as flared, loop, freeway to freeway, flared and loop combined and others. Flared consisted of ramp types 1, 4, 5 and 7; loop ramps consisted of ramp types 2, 3 and 6; freeway to freeway consisted of type 8; flared and loop combined consisted of 1, 2, 3, 4, 5, 6, and 7 (see Figures 4.2 and 4.3). All other ramps, which were mainly the split ramps were classified as “others”. Each group was further categorized into on and off ramps. Summary statistics of the collision data associated with each of the above group are presented in Tables 4.2 (a) through 4.2(i). Frequency histograms of fatal and injury collisions and PDO collisions on flared off-ramps are shown in Fig. 4.8 and 4.9 respectively to illustrate the typical distribution patterns.

Table 4.2 (a) Summary statistics of collisions on flared on-ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	354	311	.88	0	0	13	2.2
Total PDO	354	1408	3.98	2	0	58	39.66

Table 4.2 (b) Summary statistics of collisions on flared off-ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	413	645	1.56	1	0	20	6.36
Total PDO	413	2645	6.4	3	0	65	69.86

Table 4.2 (c) Summary statistics of collisions on loop on-ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	270	180	.67	0	0	5	1.04
Total PDO	270	824	3.04	2	0	25	14.76

Table 4.2 (d) Summary statistics of collisions on loop off-ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	116	130	1.12	0	0	10	3.85
Total PDO	116	503	4.34	2	0	46	45.06

Table 4.2 (e) Summary statistics of collisions on freeway to freeway ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	124	409	3.3	1	0	33	29.48
Total PDO	124	1725	13.91	7	0	106	392.59

Table 4.2 (f) Summary statistics of collisions on flared and loop combined on-ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	624	491	0.79	0	0	13	1.71
Total PDO	624	2232	3.58	2	0	58	29.05

Table 4.2 (g) Summary statistics of collisions on flared and loop combined off-ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	529	775	1.47	1	0	20	5.84
Total PDO	529	3148	5.95	3	0	65	65.06

Table 4.2 (h) Summary statistics of collisions on 'others' on-ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	87	15	0.17	0	0	6	0.52
Total PDO	87	86	0.99	0	0	17	6.03

Table 4.2 (i) Summary statistics of collisions on 'others' off-ramps

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Accident	Min. Value	Max. Value	Variance
Total FI	134	37	0.28	0	0	5	0.56
Total PDO	134	169	1.26	0	0	15	5.91

Fig. 4.8 Frequency histogram of fatal and injury collisions on flared off-ramps

Frequency Histogram of PDO
Collisions on Flared Off-ramps

Fig. 4.9 Frequency histogram of PDO collisions on flared off-ramps

4.2.3 Ramp Terminal Collision Data

As mentioned in section 4.2.2, there were no separate data for ramp terminal related collisions. Ramp terminal related collisions were assumed to be those that occurred “at the intersection” or that were identified as “intersection related”. These data were obtained separately in Excel format. Ramp terminal collisions were extracted based on severity and initial impact location. The first set of data was used for SPF calibration and the second set for screening for high proportions of specific collision types.

Ramp terminals were categorized into four groups as follows.

- 3-legged signalized ramp terminals
- 3-legged stop-controlled ramp terminals

- 4-legged signalized ramp terminals
- 4-legged stop-controlled ramp terminals

4.2.3.1 Ramp Terminal Collision Data for SPF Calibration

Collision data for each ramp terminal type mentioned above were further categorized by severity. They were grouped as fatal and injury collisions and property damage only. Summary statistics of these data are presented in Tables 4.3 (a) through 4.3 (d).

Table 4.3 (a) Summary statistics of collisions at 3-legged signalized ramp terminals

Accident Type	No of Obs.	Total Accidents	Mean Accident	Median Value	Min. Value	Max. Value	Variance
Total FI	140	565	4.03	2	0	33	31.85
Total PDO	140	2669	19.06	14	0	84	361.35

Table 4.3 (b) Summary statistics of collisions at 3-legged stop-controlled ramp terminals

Accident Type	No of Obs.	Total Accidents	Mean Accidents	Median Value	Min. Value	Max. Value	Variance
Total FI	124	77	0.62	0	0	6	1.31
Total PDO	124	276	2.22	1	0	16	9.37

Table 4.3 (c) Summary statistics of collisions at 4-legged signalized ramp terminals

Accident Type	No of Obs.	Total Accidents	Mean Accidents	Median Value	Min. Value	Max. Value	Variance
Total FI	23	120	5.21	3	0	22	31.72
Total PDO	23	436	18.95	12	0	62	341.95

Table 4.3 (d) Summary statistics of collisions at 4-legged stop-controlled ramp terminals

Accident Type	No of Obs.	Total Accidents	Mean Accidents	Median Value	Min. Value	Max. Value	Variance
Total FI	23	120	5.21	3	0	22	31.72
Total PDO	23	436	18.95	12	0	62	341.95

Typical frequency histograms of fatal and injury collisions and PDO collisions at 3-legged signalized ramp terminals are presented in Fig. 4.10 and 4.11 respectively.

Fig. 4.10 Frequency histogram of fatal and injury collisions at 3-legged signalized ramp terminals

Frequency Histogram of PDO
Collisions at 3—legged Signalized Ramp Terminals

Fig. 4.11 Frequency histogram of PDO collisions at 3-legged signalized ramp terminals

4.2.3.2 Ramp Terminal Collision Data for Screening for High Proportions of Specific Accident Types

A separate set of data were available for screening for proportion in Excel format. These data were also of type “intersection related” or “at intersection” but were categorized by initial impact location instead of severities. There were a total of 9 initial impact types and these data were available for 492 ramp terminals.

The 9 impact types were

1. Rear end
2. Side swipe
3. Single motor vehicle
4. Turning movement
5. Angular
6. Approaching
7. Single motor vehicle unattended
8. Other impacts
9. Unknown impacts

The first 5 types were predominant in numbers. A summary of collision data for each ramp terminal type is presented in Tables 4.4 (a) through 4.4(d)

Table 4.4 (a) Summary of collision data at 3-legged signalized ramp terminals

Site type: 3-legged signalized ramp terminals

Total number of sites: 144

Total number of accidents: 3558

Impact type	No. of Accidents	Observed Proportion
Rear end	1370	0.385
Side swipe	381	0.107
Single motor vehicle	447	0.125
Turning movement	1110	0.3119
Angular	196	0.055
Approaching	9	0.003
Single unattended vehicle	9	0.003
Others	35	0.009
Unknown	1	0.000

Table 4.4 (b) Summary of collision data at 3-legged stop -controlled ramp terminals

Site type: 3-legged stop controlled ramp terminals

Total number of sites: 52

Total number of accidents: 325

Impact type	No. of Accidents	Observed Proportion
Rear end	92	0.283
Side swipe	19	0.058
Single motor vehicle	76	0.233
Turning movement	114	0.351
Angular	19	0.058
Approaching	2	0.006
Single unattended vehicle	1	0.003
Others	2	0.006
Unknown	0	0.000

Table 4.4 (c) Summary of collision data at 4-legged signalized ramp terminals

Site type: 4-legged signalized ramp terminals

Total number of sites: 65

Total number of accidents: 1877

Impact type	No. of Accidents	Observed Proportion
Rear end	836	0.445
Side swipe	175	0.093
Single motor vehicle	175	0.093
Turning movement	513	0.273
Angular	148	0.078
Approaching	1	0.000
Single unattended vehicle	7	0.004
Others	22	0.011
Unknown	0	0.000

Table 4.4 (d) Summary of collision data at 4-legged stop controlled ramp terminals

Site type: 4-legged stop controlled ramp terminals

Total number of sites: 27

Total number of accidents: 242

Impact type	No. of Accidents	Observed Proportion
Rear end	68	0.28
Side swipe	22	0.09
Single motor vehicle	27	0.111
Turning movement	41	0.169
Angular	83	0.343
Approaching	0	0.000
Single unattended vehicle	0	0.000
Others	1	0.004
Unknown	0	0.000

4.3 Traffic Volume

Traffic volume data for the mainline were made available in the Excel format while for ramp and ramp terminals these were in hard copy format. For the mainline, average annual daily traffic (AADT) from 1997 to 2002 was used but for ramps and ramp terminals, AADTs from 1997 to 2004 were used. Mainline volume data were taken only until 2002 as no data were available after that. In case of ramps and ramp terminals, 2004 data were used only at sites where there were no volume data at all.

4.3.1 Mainline Traffic Volume

The mainline AADT for the period of 1997 to 2002 were extracted from the provincial volume data file, which contained the AADTs of all locations under MTO's jurisdiction. Traffic volumes relevant to study locations were extracted by using the Microsoft Access query function. All the traffic volumes were in AADT. These traffic volumes referred to flows in both directions at the interchange locations.

Out of 360 interchanges, 316 had traffic volumes available. The missing volumes were either due to the fact that they were constructed after 2002 or because data were not recorded. The summary statistics of the mainline volume data are shown in Table 4.5. A very large range of volumes was observed, with a minimum value of 6,858 and maximum of 428,633.

Table 4.5 Summary of mainline traffic volume

No of Obs.	Mean AADT	Median AADT	Minimum AADT	Maximum Value
316	92,070	62,466	6,858	428,633

Mainline interchanges were grouped in three main categories based on closely matching configurations so that models could be developed at a disaggregate level. In group 1, there were 136 interchanges with mean AADT of 82,194, minimum AADT of 8,833 and maximum AADT of 383,833. Group 2 had total of 105 interchanges with mean, minimum and maximum AADTs of 135,374, 9,250 and 428,633 respectively. Group 3 had mean AADT of 85,157, minimum and maximum AADTs being 6,858 and 373,000 respectively. This showed that the range of AADTs was very high in each group.

4.3.2 Ramp Traffic Volume

All of the ramp traffic volumes were extracted and complied from the hard copy data obtained from the MTO Central Regional office. The ramp traffic volumes were either in the form of average daily traffic (ADT) or average weekday (AWD) traffic. In some cases, even the 8-hour turning movement counts were used to get the ramp volumes. ADT and AWD were converted in to AADT by using conversion factors developed by MTO. These factors varied, depending the location and time of the year. The AADTs used in the models were the average of all available AADTs during the study period.

About one third of the total ramps had no volume data at all. The majority of these ramps were the split ramps. As sufficient data were available for modeling purposes, these ramps were ignored.

For the purpose of modeling, ramps were grouped into 5 categories. Table 4.6 summarizes the AADT in each category..

Table 4.6 Summary of ramp traffic volume

Ramp type	No of ramps	Mean AADT	Min. AADT	Max. AADT
Flared	797	6199	63	62842
Loop	398	4787	40	33543
Freeway to freeway	126	12170	320	43692
Flared & loop	1195	5728	40	62842
Others	224	4368	13	24934

4.3.3 Ramp Terminal Traffic Volume

The traffic volumes used in ramp terminal models were taken from turning movement counts. 8-hour turning movement counts were obtained in hard copy for the period from 1997 to 2004. Hard copies of turning movement counts were used to calculate the approach volumes. Total approach volumes for cross roads and ramps were calculated separately. Ramp approach volume included the sum of the approach volumes from the ramp and the service roads where they exist. Cross road approach volume consisted of the sum of approach volumes from two approaches of arterial road. All the turning movements were in 8-hour counts (from 7 am to 11 am and 3 pm to 7 pm). 8-hour counts were converted into AADT using a scale factor of 2.3. This factor was estimated using continuous hourly counts on the mainline for a month at two locations. One was at Morning Side (VDS: 401DE1030DSR) of Highway 401 and the other was at Walker's line (VDS: WEWDE0320 DNR) of QEW.

For modeling purposes, ramp terminals were grouped into 4 categories. A summary of ramp terminal volumes in each group is presented in Table 4.7.

Table 4.7 Summary of ramp terminal traffic volume

Terminal type	No of ramp terminals	Avg. approach AADT		Min. approach AADT		Max. approach AADT	
		Ramp	X-road.	Ramp	X-road	Ramp	X-road
3-legged signalized	140	13641	39972	1148	7360	57590	116969
3-legged stop controlled	124	3261	13171	83	283	14552	65801
4-legged signalized	23	11351	37452	1971	8793	34677	76431
4-legged stop controlled	20	4026	10997	1394	4384	11495	31756

4.4 Chapter Summary

Various data sets available were combined using Microsoft Access' query function to get an appropriate format to be used in the SAS software, which was used for the modeling. Some of the variables at some locations were missing. Since a reasonable size of data set was available for modeling, those locations with missing data were ignored.

Frequency histogram plots indicated that the accident counts did not follow a normal distribution. Observations also indicated that the mean value of accidents in each model group was less than the variance. This indicated overdispersion in accident counts and therefore that the Poisson distribution was not a suitable assumption. Consistent with the assumptions made by other researchers, a negative binomial distribution was assumed for calibrating the SPFs.

5. Results and Discussions on SPF Calibration

The parameters of models specified in Chapter 3 were estimated using the GENMOD procedure of SAS software. Several modeling attempts were made to achieve the best possible models for mainline interchanges, ramps and ramp terminals. Attempts were also made to develop models at the maximum possible levels of disaggregation. The key variables used in the models were traffic flow (AADT), segment or ramp length and some dummy variables for geometric configurations. Including number of lanes in the models did not result an expected level of fit, likely because of the high degree of correlation between the AADT and number of lanes. So the number of lanes was dropped from the model after the initial attempt to include it.

The overall goodness of fit of models and significance of the estimated parameters were tested for each model. Goodness of fit was also checked by means of CURE plots.

5.1 Mainline Models

Three different types of mainline models were tried in the preliminary stage of modeling. They were of the following forms

$$Accident / year = a(AADT)^b e^{c(length)} \quad (5.1)$$

$$Accident / year = a(AADT)^b (length)^c \quad (5.2)$$

$$Accident / year / km = a(AADT)^b \quad (5.3)$$

In these models, lengths were in kilometers. An attempt had been made to include as many variables as possible in the model. Results showed that model (5.1) was the best fit of all. Analysis outputs of best fit models are shown in Table 5.1. Mean deviance and Chi-square ratio were used to test the overall goodness of fit of the models. The individual parameters were assessed by their P-value. Table 5.1 shows mean deviance and Chi-square ratio close to one indicating a good overall fit of models. P-values were well within the 5% level of

significance (equivalent to a 95% confidence level) in most of the models. However, these values are high for the influence length variable for fatal and injury collisions in both groups 1 and 2. As the overall goodness of fit indicators show a good fit, and since the sign of the coefficient was intuitively correct, this variable was not dropped from the models.

The Cumulative residuals were also plotted for each model as a supplementary test. Most of the CURE plots were satisfactory. The best and the poorest CURE plots are shown in Fig. 5.1 (a) and 5.1 (b) respectively. In 5.1(a), the cumulative residuals are within the ± 2 standard deviations for entire range of AADTs. But in 5.1 (b), the cumulative residuals go beyond the ± 2 standard deviations limit for certain range of AADTs. The former is an indication of good fit of model and later is an indication of poor fit. This might have occurred due to large range of AADT.

Fig. 5.1 (a) CURE Plot of Mainline Group 1 FI Collisions

Table 5.1 Analysis output of mainline models

Group	Collision type	Parameter	Estimates	Std. Error	P-value	Dev./DOF	Chi ² /DOF
1	FI	ln(a)	-6.9649	0.844	0.0001	1.06	2.47
		b	0.7697	0.0621	0.0001		
		c	0.0363	0.1854	0.84		
		ϕ	0.2904	0.04	-		
	PDO	ln(a)	-8.9941	0.651	0.0001	1.05	1.15
		b	1.0419	0.0503	0.0001		
		c	0.1931	0.1304	0.13		
		ϕ	0.1845	0.024	-		
2	FI	ln(a)	-11.365	0.708	0.0001	1.07	1.52
		b	1.1603	0.0511	0.0001		
		c	0.1576	0.1662	0.34		
		ϕ	0.1733	0.0264	-		
	PDO	ln(a)	-12.0379	0.6717	0.0001	1.05	1.00
		b	1.268	0.0497	0.0001		
		c	0.5749	0.1454	0.0001		
		ϕ	0.5161	0.022	-		
3	FI	ln(a)	-11.0256	0.9665	0.0001	1.097	1.4746
		b	1.0644	0.0737	0.0001		
		c	0.5304	0.1988	0.008		
		ϕ	0.3049	0.0563	-		
	PDO	ln(a)	-11.677	0.8751	0.0001	1.101	1.38
		b	1.2158	0.0677	0.0001		
		c	0.6621	0.1811	0.0003		
		ϕ	0.2865	0.049	-

Fig. 5.1 (b) CURE Plot of Mainline Group 2 FI Collisions

5.2 Ramp Models

Three different types of ramp models were tested during the modeling trials.

$$Accident / year = a(AADT)^b e^{c(length)} \quad (5.4)$$

$$Accident / year = a(AADT)^b (length)^c \quad (5.5)$$

$$Accident / year / km = a(AADT)^b \quad (5.6)$$

Of the three, model (5.4) was found to be the best fit. The analysis output of each model is presented in Table 5.2.

Table 5.2 Analysis output of ramp models

Ramp type	Collision	Parameter	Estimate	Std. Error	P-value	Dev/DOF	Chi ² /DOF
Flared on-ramps	FI	ln(a)	-8.9013	0.7744	0.0001	0.89	1.24
		b	0.7962	0.0883	0.0001		
		c	0.0001	0.0003	0.77		
		ϕ	0.6617	0.1508	-		
	PDO	ln(a)	-7.4134	0.5149	0.0001	1.08	1.412
		b	0.7535	0.0579	0.0001		
		c	0.0009	0.0003	0.0065		
		ϕ	0.8262	0.0920	-		
Flared off-ramps	FI	ln(a)	-9.1476	0.5910	0.0001	0.995	1.119
		b	0.8510	0.0662	0.0001		
		c	0.0004	0.0003	0.266		
		ϕ	0.6073	0.1034			
	PDO	ln(a)	-8.0417	0.3645	0.0001	1.081	1.037
		b	0.8911	0.0415	0.0001		
		c	0.0002	0.0002	0.29		
		ϕ	0.3796	0.0430			

Table 5.2 Analysis output of ramp models (continued)

Ramp type	Collision	Parameter	Estimate	Std. Error	P-value	Dev/DOF	Chi ² /DOF
Loop on-ramps	FI	ln(a)	-8.4405	0.8568	0.0001	0.91	0.994
		b	0.6741	0.0987	0.0001		
		c	0.0012	0.0004	0.003		
		ϕ	0.2939	0.1522	-		
	PDO	ln(a)	-6.0394	0.492	0.0001	1.09	1.66
		b	0.5630	0.0573	0.0001		
		c	0.0014	0.0003	0.0001		
		ϕ	0.4623	0.075	-		
Loop off-ramps	FI	ln(a)	-8.3723	1.2133	0.0001	0.88	1.19
		b	0.7002	0.1425	0.0001		
		c	0.0015	0.0007	0.0412		
		ϕ	1.2532	0.3874	-		
	PDO	ln(a)	-8.1072	0.7905	0.0001	1.092	0.92
		b	0.8478	0.0932	0.0001		
		c	0.001	0.0005	0.05		
		ϕ	0.6811	0.1427	-		
Freeway to freeway ramps	FI	ln(a)	-8.3446	0.9828	0.0001	1.10	1.23
		b	0.7742	0.1055	0.0001		
		c	0.0004	0.0002	0.0703		
		ϕ	0.9077	0.1872	-		
	PDO	ln(a)	-7.8696	0.7589	0.0001	1.12	1.14
		b	0.8694	0.0820	0.0001		
		c	0.0006	0.0002	0.009		
		ϕ	0.7466	0.111	-		

Table 5.2 Analysis output of ramp models (continued)

Ramp type	Collision	Parameter	Estimate	Std. Error	P-value	Dev/DOF	Chi ² /DOF
Flared and loop combined on-ramps	FI	ln(a)	-8.7880	0.5826	0.0001	0.883	1.145
		b	0.7561	0.0664	0.0001		
		c	0.0005	0.0003	0.07		
		ϕ	0.5665	0.1116	-		
	PDO	ln(a)	-6.8203	0.3589	0.0001	1.082	1.71
		b	0.6717	0.0410	0.0001		
		c	0.0011	0.0002	0.0001		
		ϕ	0.6863	0.0611	-		
Flared and loop combined off-ramps	FI	ln(a)	-8.9626	0.5258	0.0001	0.972	1.126
		b	0.8148	0.0592	0.0001		
		c	0.0006	0.0003	0.03		
		ϕ	0.7158	0.1058	-		
	PDO	ln(a)	-8.0689	0.3306	0.0001	1.08	1.027
		b	0.8847	0.0037	0.0001		
		c	0.0003	0.0002	0.06		
		ϕ	0.4357	0.0431	-		

Table 5.2 Analysis output of ramp models (continued)

Ramp type	Collision	Parameter	Estimate	Std. Error	P-value	Dev/DOF	Chi ² /DOF
Others on-ramps	FI	ln(a)	-12.8018	3.5616	0.0003	0.363	1.132
		b	1.0653	0.4019	0.008		
		c	0.0009	0.0048	0.8433		
		ϕ	2.6718	2.1477	-		
	PDO	ln(a)	-6.5062	1.142	0.0001	0.867	1.436
		b	0.4832	0.1376	0.0001		
		c	0.005	0.0025	0.044		
		ϕ	1.489	0.5102	-		
Others off-ramps	FI	ln(a)	-10.5913	2.1429	0.0001	0.513	0.835
		b	0.8505	0.2453	0.0005		
		c	0.0026	0.0047	0.58		
		ϕ	1.9855	1.04	-		
	PDO	ln(a)	-8.7222	1.259	0.0001	0.898	1.659
		b	0.7763	0.144	0.0001		
		c	0.0051	0.003	0.08		
		ϕ	1.403	0.365	-		

Looking at the Table 5.2, the mean deviance and Chi-square ratio are consistently close to one, indicating a good overall fit of the models. The P-values are greater than 0.05 in some of the models for some variables, specifically, in case of fatal and injury collisions on 'others' category of ramps. These ramps were the split ramps and fatal and injury accident counts on these ramps were predominantly zeros. That might be the reason for the poor P-value. If we look at the PDO collision models for the same category of ramps, the P-values for the same variable are at a satisfactory level. So, ramp length was not dropped from the model.

Cumulative residual plots also indicated a reasonable fit of the models. The best and the poorest CURE plots are shown in Fig. 5.2 (a) and Fig.5.2 (b) respectively. In 5.2(a), the cumulative residuals are all within ± 2 standard deviations for entire range of AADTs. This is an indication of good fit of model. In Fig. 5.2(b), the cumulative residuals go beyond ± 2 standard deviations for some small range of AADTs. As the majority of the cure plot is within the ± 2 standard deviation range, there is no ground to reject the model.

The new ramp models included ramp length as a variable. In the existing models, only freeway to freeway ramps had ramp length as a variable; all regular ramps models had only AADT as a model variable. There were no models developed for split type of ramps due to lack of sufficient data. New models could be developed for this category of ramps as enough data were available.

Fig. 5.2 (a) CURE plot of FI collisions on loop off-ramps
(An example of good CURE Plot)

Fig. 5.2 (b) CURE plot of PDO collisions on loop on-ramps
(An example of poor CURE plot)

5.3 Ramp Terminal Models

Four categories of models were tried using various variables. They were models for

1. Three-legged signalized ramp terminals
2. Four-legged signalized ramp terminals
3. Three-legged stop controlled ramp terminals
4. Four-legged stop controlled ramp terminals

Reasonably good models could be developed for categories 1 and 2 as given by Equation (5.7).

$$(Acc/year)_i = a(AADTramp_i)^b (AADTcross_i)^c e^{d(split)} \quad (5.7)$$

where,

AADTramp_i: approach AADT at the ramp approach and at the service road approach

AADT_{across}_i: approach AADT at the arterial road

split: a dummy variable, which had a value of 1 if the approach ramp was split and 0 if not

Model (5.7) could not be fitted satisfactorily for category 3 and 4. Therefore, these two categories were combined together and the model was formulated as in (5.8).

$$(Acc/year)_i = a(AADTtotal)_i^b e^{c(split)} \quad (5.8)$$

where,

AADTtotal: total AADT approaching the ramp terminal from all approaches.

split: dummy variable and had a value of 1 if the approach ramp was split and 0 otherwise.

The analysis output is summarized in Table 5.3.

Looking at the Table 5.3, the mean deviance and Chi-square ratio both are close to one for all the models. P-values are less than 0.05 for all the models, except in one case where it is 0.15. This is not too far from the acceptable limit of 0.05. This indicates that the parameter estimates were acceptable. Models were further tested by CURE plots. Figs. 5.3 (a) and 5.3 (b) show the CURE plots for PDO collisions at 3-legged signalized ramp terminals, one considering the ramp AADT and the other considering the cross road AADT. The oscillations of CURE plots are well within the ± 2 standard deviations indicating good fits of the models.

The effect of the dummy variable (whether the approach ramp is split or not) on accident frequency was negative, meaning that if other factors remaining the same, the introduction of split ramps at ramp terminals may reduce accident frequency.

Table 5.3 Analysis output of ramp terminal models

RT type	Collision type	Parameter	Estimate	Std. Error	P-value	Dev./DOF	Chi ² /DOF
3-Legged Signalized	FI	ln(a)	-12.7762	1.9129	0.0001	1.15	1.11
		b	0.6187	0.1776	0.0005		
		c	0.6114	0.1946	0.0017		
		d	-0.7555	0.1478	0.0009		
		ϕ	0.8132	0.1072			
	PDO	ln(a)	-11.5143	1.312	0.0001	1.14	1.29
		b	0.7360	0.1123	0.0001		
		c	0.5351	0.1181	0.0001		
		d	-0.7636	0.1465			
		ϕ	0.4257	0.0606			
4-Legged Signalized	FI	ln(a)	-17.1286	3.9417	0.0001	1.30	1.08
		b	0.7150	0.2558	0.005		
		c	0.9685	0.4299	0.02		
		d	-2.4316	1.0432	0.01		
		ϕ	0.1501	0.1235			
	PDO	ln(a)	-14.4269	4.1520	0.0001	1.44	1.06
		b	0.9566	0.2382	0.0001		
		c	0.6219	0.4321	0.15		
		d	-1.3896	0.4710	0.0032		
		ϕ	0.3328	0.1418			
3-Legged and 4-Legged Stop- Controlled	FI	ln(a)	-6.9588	1.9920	0.0005	0.82	0.89
		b	0.5028	0.2077	0.015		
		c	-1.1066	0.3405	0.0012		
		ϕ	1.173	0.4364			
	PDO	ln(a)	-6.7506	1.2659	0.0001	1.02	1.16
		b	0.6087	0.1319	0.0001		
		c	-1.0104	0.1976	0.0001		
		ϕ	0.5499	0.124

Fig. 5.3 (a) CURE Plot of PDO collisions at 3-legged signalized ramp terminals (against approach AADT of ramps)

Fig. 5.3 (b) CURE plot of PDO collisions at 3-legged signalized ramp terminals (against approach AADT of cross road)

6. Results of Screening for High Proportions of Specific Accident Types

This Chapter presents the results obtained from the method of screening for high proportions of specific accident types. Only ramp terminal related data were used for this purpose and it applies to ramp terminals only.

6.1 Parameter Estimation

Nine different impact types were available in the ramp terminal data set. They were

1. Rear end
2. Side swipe
3. Single motor vehicle
4. Turning movement
5. Angle
6. Approaching
7. Single motor vehicle unattended
8. Others
9. Impact unknown

Ramp terminals were divided into four categories for analysis as

1. Three-legged signalized ramp terminals
2. Four-legged signalized ramp terminals
3. Three-legged stop-controlled ramp terminals
4. Four-legged stop-controlled ramp terminal

The parameters α and β of Equation (3.24) were estimated for each ramp terminal type and for each impact category using the method of moment and the maximum likelihood method. The estimated parameters by both of the methods were quite close except for side swipe collisions. Table 6.1 shows a comparison of parameters estimated by the maximum likelihood method and by the method of moment for 3-legged signalized ramp terminals. As both the methods gave consistent estimates of parameters, parameters estimated by the simpler method of moment were used for further calculations.

Table 6.1 Comparison of parameter estimates for 3-legged signalized ramp terminals by the method of moment and by the maximum likelihood method

Collision type	α		β	
	ML Method	Moment Method	ML Method	Moment Method
Rear end	5.411	5.0015	9.282	8.6339
Side swipe	7.5	4.7917	61.15	36.1267
Single Motor vehicle	2.576	2.6411	15.15	13.6675
Turning Movement	4.620	3.5630	10.843	9.1100
Angle	2.47	2.6602	42.7	48.1129
Approaching	N/A	N/A	N/A	N/A
Single Motor Vehicle Unattended	N/A	0.1213	N/A	25.8599
Other Initial Impact	N/A	3.3647	N/A	287.5583
Initial Impact Unknown	N/A	N/A	N/A	N/A

Tables 6.2 through 6.5 summarize the estimated parameters and the corresponding mean proportions for different types of collisions.

Table 6.2 Parameter estimates, mean and variance for different types of collisions at 3-legged signalized ramp terminals

Collision type	α	β	μ	Variance
Rear end	5.0015	8.6339	0.3668	0.0159
Side swipe	4.7917	36.1267	0.1171	0.0025
Single Motor vehicle	2.6411	13.6675	0.1619	0.0078
Turning Movement	3.5630	9.1100	0.2811	0.0148
Angle	2.6602	48.1129	0.0524	0.0010
Single Motor Vehicle Unattended	0.1213	25.8599	0.0040	0.0002
Other Initial Impact	3.3647	287.5583	0.0116	0.0000

Table 6.3 Parameter estimates, mean and variance for different types of collisions at 4-legged signalized ramp terminals

Collision type	α	β	μ	Variance
Rear end	3.1045	4.4319	0.4119	0.0284
Side swipe	10.3601	109.4968	0.0864	0.0007
Single Motor vehicle	1.6522	11.8565	0.1223	0.0074
Turning Movement	3.8088	10.1454	0.2730	0.0133
Angle	0.7963	8.4597	0.0860	0.0077

Table 6.4 Parameter estimates, mean and variance for different types of collisions at 3-legged stop-controlled ramp terminals

Collision type	α	β	μ	Var
Rear end	1.2015	3.5861	0.2510	0.0325
Side swipe	0.6001	12.4320	0.0460	0.0031
Single Motor vehicle	1.4596	3.6778	0.2841	0.0331
Turning Movement	9.7804	19.1582	0.3380	0.0075

Table 6.5 Parameter estimates, mean and variance for different types of collisions at 4-legged stop-controlled ramp terminals

Collision type	α	β	μ	Var
Rear end	0.5055	1.0729	0.3202	0.0844
Side swipe	7.9306	161.1001	0.0469	0.0003
Single Motor vehicle	0.1462	0.4021	0.2667	0.1263
Turning Movement	0.3508	1.6768	0.1730	0.0473
Angle	0.4402	1.8535	0.1919	0.0471

For three-legged signalized ramp terminals, parameters for rear end, side swipe, single motor-vehicle, turning, angular, single-motor vehicle unattended and other categories of impact were estimated as shown in Table 6.2. Data were not available for ‘approaching’ and ‘initial impact unknown’ types. Likewise, for four-legged signalized ramp terminals, parameters were estimated for rear end, side swipe, single motor-vehicle, turning and angular types of collisions. Data were not sufficient to estimate parameters for other four types. The estimates are shown in Table 6.3. In three-legged stop-controlled ramp terminals, parameters for rear end, side swipe, single motor-vehicle, turning categories of impact were estimated as shown in Table 6.4. For four-legged stop-controlled ramp terminals, parameters for rear end, side swipe, single motor-vehicle, turning, angular were estimated and are shown in Table 6.5.

Significance of the estimated parameters was checked by comparing the observed proportions with the estimated proportion. Observed proportion was calculated as

$$\frac{\sum_M x_i}{\sum_M n_i}.$$

where,

x_i : no of target accidents at site i

n_i : total no of all accidents at site i

M : total no of sites

A comparison of observed and estimated proportions is presented in Table 6.6. The table shows that the observed and estimated proportions were very close except in the case of angular and single vehicle collisions at four-legged stop controlled ramp terminals.

Table 6.6 Comparison of observed and estimated proportions of collisions of different types

Collision Type	3-Leg Signalized		4-legged Signalized		3-legged Stop-controlled		4-legged Stop-controlled	
	Observed	Estimated	Observed	Estimated	Observed	Estimated	Observed	Estimated
Rear end	0.386	0.367	0.444	0.412	0.248	0.251	0.274	0.320
Side swipe	0.107	0.117	0.093	0.086	0.056	0.046	0.086	0.047
Single Motor vehicle	0.126	0.162	0.093	0.122	0.320	0.284	0.120	0.267
Turning Movement	0.311	0.281	0.273	0.273	0.313	0.338	0.184	0.173
Angle	0.055	0.052	0.079	0.086	0.049	N/A	0.331	0.192
Approaching	0.003	N/A	0.001	N/A	0.005	N/A	0.000	N/A
Single Motor Vehicle Unattended	0.003	0.004	0.005	N/A	0.002	N/A	0.000	N/A
Other Initial Impact	0.010	0.012	0.012	N/A	0.007	N/A	0.004	N/A
Initial Impact Unknown	0.000	N/A	0.000	N/A	0.000	N/A	0.000	N/A

Pattern scores were estimated based on Equation (3.35) for all collision types that had enough data for parameter estimation. For illustration purposes, the ten top ranked sites based on rear-end collisions at 4 different ramp terminal categories are presented in Tables 6.7 through 6.10.

The high pattern score for a particular type of collision means that the site is a candidate for detailed engineering study (DES) to identify treatments targeted at that specific collision type. For example, high pattern score of rear end collisions can indicate poor road surface condition, short cycle lengths, or poorly designed inter-green periods. More specific reasons can be identified after the DES, and treatments can be proposed accordingly.

Table 6.7 Ranking based on rear-end collisions at 3-legged signalized ramp terminals

Region	Hwy	LHRS	Offset	Ramp No.	Score	Rank
Central	401	47624	0	51	1.000	1
Southwest	401	47720	0	51	1.000	2
Central	401	47643	0	61	1.000	3
Central	401	47642	0	51	1.000	4
Central	400	46827	0	24	1.000	5
Central	401	47629	0	61	0.999	6
Central	401	47626	0	51	0.998	7
Central	403	48260	0	61	0.991	8
Central	401	47633	0	51	0.990	9
Central	401	47673	0	61	0.983	10

Table 6.8 Ranking based on rear-end collisions at 3-legged stop-controlled ramp terminals

Region	Hwy	LHRS	Offset	Ramp No.	Score	Rank
Central	1	10062	0	61	1.000	1
Southwest	7187	81083	0	64	0.999	2
Southwest	401	47910	0	61	0.976	3
Southwest	403	48380	0	51	0.974	4
Southwest	7	14590	0	24	0.953	5
Central	401	47577	0	51	0.953	6
Central	1	10070	0	51	0.924	7
Southwest	402	48225	0	61	0.875	8
Southwest	85	38182	0	34	0.849	9
Northeast	11	17085	0	24	0.821	10

Table 6.9 Ranking based on rear-end collisions at 4-legged signalized ramp terminals

Region	Hwy	LHRS	Offset	Ramp No.	Score	Rank
Central	404	48490	0	24	1.000	1
Central	401	47629	0	51	0.999	2
Southwest	8	15873	0	61	0.998	3
Central	401	47598	0	51	0.989	4
Central	401	47607	0	51	0.987	5
Central	401	47627	0	51	0.983	6
Central	406	48690	0	34	0.983	7
Central	400	46900	0	24	0.980	8
Central	401	47672	0	51	0.956	9
Central	406	48690	0	24	0.948	10

Table 6.10 Ranking based on rear-end collisions at 4-legged stop-controlled ramp terminals

Region	Hwy	LHRS	Offset	Ramp No.	Score	Rank
Central	401	47700	0	51	1.000	1
Central	58	32753	0	61	0.998	2
Central	400	46904	0	34	0.987	3
Central	11	16980	0	34	0.970	4
Southwest	7	14607	0	61	0.970	5
Central	58	32753	0	51	0.958	6
Central	58	32755	0	61	0.917	7
Central	1	10057	0	61	0.907	8
Central	400	46870	0	34	0.847	9
Central	401	47580	0	51	0.847	10

6.2 Comparison of Site Ranking Based on PSI Index Methods and Screening for High Proportions of Specific Accident Types

In order compare the site screening by PSI index methods and ‘screening for proportion’ method, ‘angle’ and ‘turning’ types of collisions at 3-legged signalized ramp terminals were chosen. This included accidents of all severities. To use the PSI index method, an SPF was developed for this collision type. The SPF was specified as follows.

$$Accident / year = a(AADT_ramp)^b (AADT_across)^c e^{d(split)} \quad (6.1)$$

where,

AADT ramp: AADT at ramp approach

AADT across: AADT at cross road

split: dummy variable that is 0 for ramp terminals without split ramp and 1 for with split ramps

The SAS analysis output of the SPF calibration is shown in Table 6.11.

Table 6.11 Analysis output of model for 3-legged signalized ramp terminals considering ‘turning’ and ‘angle collisions’ of all severities

Parameter	Estimate	Std. Error	P-value	Dev./DOF	Chi ² /DOF
ln(a)	-9.5549	1.7852	0.0001	1.16	1.11
b	0.3359	0.1522	0.027		
c	0.6425	0.1716	0.0002		
d	-1.222	0.2046	0.0001		
ϕ	0.6258	0.0981			

Looking at the goodness of fit criteria, mean deviance and Chi-square ratio are close to one, indicating a good overall fit of model. The P-values for each variable are less than 0.05, which indicate that the estimates are good at 95% confidence level.

The same data were used to estimate the parameters by the method of screening for high proportions of specific accident types. The parameters estimated by this method are shown in Table 6.12. This table also shows the comparison of estimated proportion and observed proportion. The two values are quite close.

Table 6.12 Parameters estimates by screening for high proportions of specific collision types method for ‘turning’ and ‘angle collisions’ of all severities

α	β	μ	Observed Prop.	Var.
3.868004	6.963712	0.3571	0.371	0.019

Pattern scores were estimated for each site. They were arranged in descending order, which showed the ranking of sites. Sites were further ranked based on PSI index method that is currently

being adopted by MTO and also by the method suggested by Hauer (2002). In the MTO's current methodology, the PSI index is defined as the difference between a site's expected number of collisions and those expected at sites with similar characteristics. But Hauer (2002) suggested that the PSI index should merely be the site's expected number of collisions.

The ten top ranked sites based on screening for high proportions of specific accident types were extracted. For these sites, the ranking was done by both the PSI index methods. Table 6.13 shows a comparison of rankings by the three methods.

A total of 128 sites were ranked. Table 6.13 shows that 7 out of 10 top ranked sites by the pattern score method were also in the top 10 by MTO's current PSI index method, although the actual rank was different in most cases. However, 4 out of 10 top ranked sites by the pattern score method appeared in the top 10 by Hauer's PSI method. Based on the site and accident types considered for comparison, the ranking by PSI index method currently adopted by MTO and that by pattern score method gave consistent results. Hence, the pattern score could be a possible alternative to the PSI index method for site screening, specifically when SPFs and/or AADTs are not available to undertake the PSI index method.

Table 6.13 Ranking comparison of sites based on three different methods

Total number of sites: 128

Site type: 3-legged signalized ramp terminals

Accident type: Turning and angle

Region	Hwy	LHRS	Offset	Location	Ramp	Pattern Score	Ranking by Pattern Score	Ranking by Current Method	Ranking by Hauer's Method
Central	401	47661	0	Weston Rd IC-357	61	0.999	1	1	2
Central	1	10125	0	Walkers line IC-105	51	0.999	2	20	21
Central	401	47633	0	Victoria Park Ave. IC-376	61	0.999	3	6	3
Central	401	47642	0	Bayview Ave IC-371	61	0.999	4	2	1
Central	410	49071	0	S.JCT Hwy 7 -Queen St IC	24	0.999	5	15	16
Central	401	47625	0	Neilson Rd IC-385	61	0.997	6	7	15
Central	401	47598	0	Hwy 12 IC-410 -Consumers Dr	64	0.997	7	14	24
Central	401	47610	0	Liverpool rd IC-397	61	0.997	8	8	18
Central	403	48266	0	Erin Mills Pkwy IC	61	0.994	9	10	11
Central	403	48273	0	Hwy 5 Dundas St.	34	0.981	10	7	7

7. Conclusions and Recommendations

Existing safety performance functions for mainline interchanges and ramps of Ontario freeways have been updated using data from 1997 to 2003. Updated models have been developed for three categories of mainline interchanges and five categories of ramps. Separate models were developed for fatal and injury (FI) collisions and for property damage only (PDO) collisions in each category. Safety performance functions for ramp terminals have also been developed; these did not exist previously. The updated models include some additional variables such as influence length in the mainline models and ramp length in ramp models.

The method of screening for high proportions of specific collision types has been illustrated using ramp terminal collision data. As part of this, parameter estimation and pattern score calculation based on method of moment and maximum likelihood method have been illustrated.

Collision site screening has been illustrated by the PSI index method currently adopted by MTO, by the method suggested by Hauer (2002), and by method of screening for high proportions of specific collision types for 3-legged signalized ramp terminals using 'angle' and 'turning' types of accidents of all severities. Comparison of ranked lists from the 3 methods shows that the PSI index based method currently used by MTO and the screening for high proportions of specific accident types method give consistent results. As the method of screening for high proportions of specific accident types does not require AADTs or SFPs (as the PSI index method does), it can be a possible alternative to PSI index method for screening based on specific accident types.

It is possible to include other variables such as turning radius and posted speed in the SPFs for mainline interchanges and ramps as they may be contributors to collisions in many cases. These variables could not be included in the models because unavailability of data. Evaluation of effect of these variables in safety estimation should be undertaken in further research. Also, data for several mainline freeways, ramps and ramp terminals were missing. Maintaining a complete database for all locations would be more meaningful in the modeling and screening efforts.

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